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Bridge Resource Program

Final Report
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16. Abstract
The mission of Rutgers University's Center for Advanced Infrastructure and Transportation (CAIT) Bridge Resource Program (BRP) is to provide bridge engineering support to the New Jersey Department of Transportation (NJDOT)'s Bridge Engineering and Structural Evaluation Unit. The program is a partnership between federal and state transportation agencies and Rutgers University, which provides technical and educational services to address infrastructure needs in New Jersey. CAIT supports the NJDOT by providing staff and resources to address the most pressing bridge engineering and training challenges in New Jersey (through advanced materials development, design enhancements, construction improvements, evaluation, monitoring, data mining, management enhancement and support, and bridge research).

The overarching goal of the Bridge Resource Program is to achieve more effective asset management. This includes consideration and potential adoption of next generation assessment approaches to augment current reliance on qualitative condition metrics with more quantitative performance metrics. Although conventional engineering terms are used in this proposal to describe program services, the proposed tasks within each service will be focused on providing decision making assistance for concept development.

BRP has provided opportunities to bring technologies to NJDOT, review existing practices, and propose the use of new construction techniques to improve asset management, design and construction practices. In addition, it has created a new channel of communication between CAIT and NJDOT that allows for the rapid deployment of innovative technologies. In the future, the BRP is envisioned to continue to identify opportunities for innovation. It is anticipated that new research topics will be borne out of the program. As research is completed, it can return back to the BRP for pilot testing and recommendation for deployment. The cycle of innovation, testing, implementation and need for further innovation can be perpetuated through the creation of similar resource programs.

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Executive Summary

The mission of Rutgers University's Center for Advanced Infrastructure and Transportation (CAIT) Bridge Resource Program (BRP) is to provide bridge engineering support to the New Jersey Department of Transportation (NJDOT)'s Bridge Engineering and Structural Evaluation Unit.

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The goal of the Bridge Resource Program is to achieve more effective asset management. This includes consideration and potential adoption of next generation assessment approaches to augment current reliance on qualitative condition metrics with more quantitative performance metrics. Although conventional engineering terms are used in this proposal to describe program services, the proposed tasks within each service will be focused on providing decision making assistance for concept development.

NJDOT is faced with significant challenges in addressing the state of good repair of their bridge asset system. Shrinking budgets and the need to "do more with less", has resulted in increasingly difficult decisions to repair or replace structurally deficient bridges. The state's current investing budget, $690 million in 2013 on bridge assets, follows a constrained model of asset management. With 6,452 bridges (over 20 feet long) in New Jersey, 2,584 state-owned bridges, and an average age of NJ bridges at 51 years, NJDOT is continually looking to innovate in order to meet their policy of maintaining an acceptability rate of 86% over the next 10 years. NJDOT was in need of a resource program that assists in advancing asset management practices, provides training in the use of advanced materials, technologies and construction techniques, identifies new technologies, and responds to unplanned, non-routine materials and construction issues.

BRP has provided opportunities to bring technologies to the department, review existing practices, and propose the use of new construction techniques to improve asset management, design and construction practices. In addition, it has created a new channel of communication between CAIT and NJDOT that allows for the rapid deployment of innovative technologies. In the future, the BRP is envisioned to continue to identify opportunities for innovation. It is anticipated that new research topics will be borne out of the program. As research is completed, it can return back to the BRP for pilot testing and recommendation for deployment. The cycle of innovation, testing, implementation and need for further innovation can be perpetuated through the creation of similar resource programs.

Introduction

The primary objective of the Rutgers Bridge Resource Program (BRP) is to utilize the extensive laboratory and field testing equipment and staff expertise in Bridge Engineering to assist the New Jersey Department of Transportation's Bridge Engineering and Structural Evaluation Unit in developing bridge management system strategies, innovative materials, improved bridge design tools, advanced laboratory and field data collection, bridge monitoring strategies, bridge inspection, non-destructive evaluation and innovative technologies/equipment aimed at enhancing the state's bridge inventory condition by optimizing available capital resources.

The program is divided into the following tasks, subtasks and deliverables:

Table 1 - Bridge Resource Program Tasks and Deliverables

Task	Subtask – Deliverable
1. Enhance the NJDOT's Structural Management Activities	1a. In collaboration with NJDOTs Bridge staff, review the recent FHWA audit and provide guidance to improve asset management. Also, evaluate the setup and procedures currently used by NJDOT and provide specific guidance in using Version 5.2.2 for deterioration modeling of bridge assets
	1b. Develop a method to populate the cost data fields in Pontis® from the available cost data in TRNS-Port™, and populate the cost data for one year's worth of Bridge Construction Projects.
	1c. Develop and provide a prioritization model of structurally deficient bridges through a qualitative risk assessment that explicitly recognizes vulnerability, hazard and consequence of failure.
2. Provide Technology Transfer	2a. Utilize a comprehensive multimodal NDE scanning of up to ten (10) bridges with HPC decks to determine the "seriousness" of the deck cracking observed in HPC decks.
	2b. Develop and implement an instrumentation plan to better understand the stresses developed and "felt" by the bridge during the placing and curing of an HPC deck for two (2) bridges. Provide a report to the Department on the findings.
	2c. Perform a comparative field survey on a select existing and new bridge structures and evaluate the durability of reinforced HPC concrete and its consequence on the long-term performance of bridge decks.
	2d. Perform a refined load rating of up to ten (10) bridges that have resulted in Overload Truck Permits being re-routed due to load carrying capacity limits resulting from "standard" structural ratings to determine if additional load carrying capacity exists in the selected bridges and to demonstrate the value of such advanced analytical approaches.

	2e. Provide a synthesis report on methods to mitigate construction deformations in steel superstructure of skewed bridges.
	2f. Review technical publications, journals and other resources including but not limited to FHWA, UTC and TRB to discover new technologies and construction techniques. Present result in technical memorandums.
3. On-Call Services	3a. Respond to non-routine and non-planned structural management, materials and technology issues that arise throughout the year.
	3b. Develop Standards for new materials and construction techniques as selected by NJDOT from the technical memorandums developed in the Technology Transfer Task.

Task Summary

Enhance the NJDOT's Structural Management Activities

The Bridge Resource Program is designed to provide the NJDOT Bridge Engineering and Structural Evaluation Unit technical support by working with the unit staff to enhance the capabilities of the bridge management system through a variety of upgrades to the software as well as on-site support. The team identified three initial asset management tasks that can be initiated to better assist the department in their bridge management needs. The following is a brief explanation of each subtask:

Bridge Management Software (BMS) deployment assistance

Background

Initially, the team intended to provide NJDOT with an audit of the existing BMS and development of a workplan. During the contract negotiation process, FHWA conducted an audit of NJDOTs BMS and provided action items. As a result, the team redeveloped the scope of this activity to better serve the department.

The team is using its expertise and familiarity with the AASHTOWare Bridge Management software (formerly Pontis) to evaluate the setup and procedures currently used by NJDOT and provide specific guidance using Version 5.2.2 for deterioration modeling of bridge assets. The following activities are underway under this task:

- Review the recent FHWA audit and develop a clear understanding for the direction provided, including how the following were addressed. The review should include action items and overall guidance on applying modeling and programming capabilities of the software in a manner that fits with the way NJDOT needs to work.
    - Documentation available for users
    - Types of users that need access to the system and their relevant permissions

- Software and overall IT configuration utilized by NJDOT
- Data collected by NJDOT on bridges and where it is stored
- Short, Intermediate, and Long-Term goals of NJDOT related to bridges
- Relevant NJDOT performance metrics for bridges

• Develop a guide document to be used by NJDOT for the deployment and use of the new BrM 5.2.2 software. It should be noted that these tasks are dependent on AASHTO's release of the BrM 5.2.2 software. The guide will be used to provide users with a detailed reference (step by step), and provide guidance on items such as selecting the proper deterioration models based on DOTs needs/goals. Specific guidance will be provided in the following areas of the new software:
  - Planning
  - Deterioration (Key Focus)
  - Risk
  - Multi-Objective Analysis
  - Lifecycle Costs
  - Project Models
  - Dashboards
  - Corridor Planning

The team will focus on guidance in developing the following:

- User-defined risk assessment types
- NJDOT-specified deterioration scales and formulas
- Setting NJDOT priority cost, assignment and programming of work
- Interface for external work accomplishments
- Alignment and integration with maintenance management systems
- Creating a multi-objective framework to valuate specific interventions for a deteriorating bridge
- Explain how utility can be used in terms of each sub-area
  - § Mobility
  - § Lifecycle Cost
  - § Condition
  - § Risk items
- Explain how NJDOT can review/revise work candidates as they contribute to mobility, lifecycle cost, condition, and risk weightings
- Step-by-Step guide to evaluating future condition at the detail and summary level
- Development of the deterioration model logic

Work performed

The team reviewed the FHWA audit report provided by NJDOT, which was performed to ensure that the BMS used in New Jersey performs the minimum functional requirements outlined by federal law and adequately support the new asset management plan requirements found in the MAP-21 legislation.

The team also met with NJDOT on December 5 and 6, 2013 as part of onsite meeting #1, dedicated to establishing baseline protocols for NJDOTs BMS as well as to review the findings of the FHWA audit report. Action items identified during the meeting provided the team with the direction for the work effort to be completed.

The team developed a report outlining how to incorporate software solutions, including deterioration modeling, into the improvements recommended by FHWA in the audit. This report contains the guidance on the use of the new software tools, specifically the deterioration modeling approach and procedures that can be used to better quantify the performance of New Jersey bridges. The purpose of this report will be to provide NJDOT with specific action items based on review of the audit and the team's knowledge and experience in the software arena. Recommendations are general in scope, with methodologies and procedures being the focus.

Cost Data Migration to PONTIS

Background

Presently, NJDOT has a comprehensive Bridge Management System (BMS) which prioritizes bridge replacement, rehabilitation, deck and superstructure replacement, painting, scour retrofit, and other maintenance repairs. The BMS system is used to develop recommended funding levels to improve the condition of 'State maintained' bridges.

NJDOT has identified a need to prioritize, maximize available bridge funding, and provide decision makers with key construction cost data associated with bridge repair/replacement. Currently, NJDOT stores project and asset-related costs in their Trns-port™ system. To provide this cost information to NJDOT's BMS for analysis, the team is translating and arranging data into a format that can be imported into Pontis®.

To accomplish this goal, cost information from NJDOT's Trns-port™ system is being extracted, combined with data from the Capital Program Management's Project Reporting System (PRS), and transformed into a format that NJDOT's Pontis® BMS can utilize. To facilitate this transfer of data, the team is working with NJDOT staff to define the data structure and content required for the Pontis® BMS. This detail will act as the blueprint in which the team will follow for the remainder of the project.

Once a full analysis has been performed, the team will develop the transformation process to make the connection between cost, projects, and bridge structures, in a format that is consistent with Pontis® needs. This transformation process will be a one-time data migration, but will be fully documented so that future data updates can be streamlined.

Work performed

The team obtained and preformed analysis on latest PRS, Trns-port & Pontis databases. The team completed the review and analysis of Data and prepared a Technical Memorandum. The team also held status update meetings with the department. Following the meetings, the team outlined a proposed ETL process and provided related documentation. The team provided NJDOT a list of assumptions and proposed actions for ETL process. The team provided a draft data set (PDI file) to NJDOT for testing purposes. Based on testing, it was made clear that the data set was incompatible with Structural Evaluation's needs. The team performed a second review of the ETL process, and reviewed the databases to determine if the data coming from the various databases could be used for the desired outcomes. The team provided recommendations and identified areas of potential future improvement relating to the cost data migration.

Risk-based prioritization

Background

The objective of this task is to demonstrate risk-based prioritization through the application to a sample of structurally deficient bridges. Risk-based prioritization has been successfully employed in many fields that range from nuclear power to drinking water policy, and since the 2007 collapse of the I-35W Bridge this approach has been gaining attention related to the allocation of bridge repair, retrofit and replacement resources.

Through this application NJDOT is being provided with a framework that can be potentially incorporated within Pontis®. While the framework being developed is qualitative in nature, it has distinct advantages over condition or sufficiency rating-based approaches in that (a) it explicitly recognizes key performance limit states, (b) directly addresses bridge hazards, vulnerabilities, and exposures, (c) incorporates the uncertainty associated with various assessment techniques and provides flexibility for their implementation, and (d) provides a means to capture (in a useable format) expert knowledge and heuristics from top bridge engineers.

Work performed

The team began the task by coordinating with NJDOT personnel to identify a sample of 100 structurally deficient bridges, shown in Table 2. For each of the sample bridges, the team reviewed at least the most recent two inspection reports as well as any design documentation available. Through this process a database that includes all of the requisite information needed to perform a risk-based prioritization on the sample of bridges was developed.

Table 2 - Structurally deficient bridges selected for risk-based prioritization

	Structure Number		Structure Number		Structure Number		Structure Number		Structure Number
1	114155	**21**	730152	**41**	1122150	**61**	1513153	**81**	2107156
2	202159	**22**	730192	**42**	1149160	**62**	1513154	**82**	2108155
3	203153	**23**	731154	**43**	1211152	**63**	1515150	**83**	2117160
4	206169	**24**	731156	**44**	1218154	**64**	1516152	**84**	2117161
5	206173	**25**	731157	**45**	1223153	**65**	1601162	**85**	2154160
6	209150	**26**	731160	**46**	1231168	**66**	1607161	**86**	3000163
7	214159	**27**	807152	**47**	1234173	**67**	1650161	**87**	3001151
8	220154	**28**	815150	**48**	1249165	**68**	1711156	**88**	3001176
9	221152	**29**	818151	**49**	1253164	**69**	1809150	**89**	917150
10	222153	**30**	821160	**50**	1304151	**70**	1809158	**90**	3000169
11	302151	**31**	823150	**51**	1308154	**71**	1817155	**91**	1409156
12	314151	**32**	905150	**52**	1309150	**72**	1817156	**92**	1105151
13	317155	**33**	906156	**53**	1310155	**73**	1904153	**93**	1850160
14	350162	**34**	907152	**54**	1337150	**74**	1922150	**94**	1850164
15	417158	**35**	908153	**55**	1409155	**75**	2003150	**95**	1850166
16	418163	**36**	1005151	**56**	1419169	**76**	2003166	**96**	2153161
17	510152	**37**	1007159	**57**	1426150	**77**	2101150	**97**	3000165
18	601152	**38**	1009150	**58**	1430153	**78**	2105152	**98**	3001159
19	722157	**39**	1014162	**59**	1502152	**79**	2105153	**99**	3001162
20	725171	**40**	1101163	**60**	1513152	**80**	2106151	**100**	3001175

Using the database developed, the team employed a series of different qualitative risk classification schemes and documented the sensitivity of the prioritization to these different schemes.

The culmination of this analysis is the calculation of Perceived Risk, which is defined as a function of Hazard, Vulnerability, Exposure, and an uncertainty premium. NJDOT elaborated that their approach to risk involved a review of reliability (a product of hazard and vulnerability) as a function of importance (exposure). Through this collaboration, the team is developing a report detailing the various prioritization schemes and the resulting outcomes. Following NJDOT review, the team will meet with NJDOT to discuss the findings and discuss (1) the usefulness of the various prioritization schemes, (2) areas were refinements are needed, and (3) the path forward (e.g. implementation within the BMS).

Technology Transfer

The Technology Transfer task was developed as an umbrella for a suite of activities and technologies that could be employed to respond to the department’s needs. The team envisioned their use as either independent or combined to form robust experiments. The activities included literature searches, technical reviews of technologies, nondestructive evaluation of bridge decks, refined load rating, bridge deck instrumentation during construction, comparative analyses of bridge decks and other technologies.

As part of the effort and as mentioned in the Methodology section, the team allocated testing of a number of bridge decks for evaluation using various techniques. Once NJDOT indicated their need to better understand the performance of High-Performance Concrete (HPC) in New Jersey and understand the effects of construction loading on highly skewed steel structures, the team developed a testing program to complement these needs. The following subsections describe the work performed to respond to the department's requests.

High Performance Concrete evaluation

Background

NJDOT began using HPC in the late 1990s and early 2000s. Throughout that period, NJDOT has had mixed performance with the material. The research bureau has undertaken a number of studies to better understand the material behavior, and recommend methods to improve performance. One hypothesis of the cause of deck cracking is self-desiccation and autogenous shrinkage in HPC. Nationwide, agencies have incorporated 3 to 14-day wet cure methods that begin immediately (up to 10 minutes after) final strike-off to provide curing HPC decks with sufficient moisture to prevent shrinkage cracking. While this has reduced the initial onset of plastic cracking, it has not resulted in conclusive proof that longer-term (28-365 day) cracking can be arrested using current practice.

NCHRP recently released a new synthesis report titled "High Performance Concrete specifications and practices for bridges", which documents a survey taken of state DOTs and is intended to help bridge owners, designers, contractors and material suppliers determine the appropriate specification requirements for HPC in bridges. In general, the report indicates that states vary in their means of specifying HPC and provides a list of changes in specifications and practices that have improved performance.

The program provided three subtasks to evaluate the performance of concrete in New Jersey:

1. Nondestructive evaluation (NDE) of bridge decks
2. Comparative durability analysis of concrete
3. Instrumentation and monitoring of bridge decks under construction

The main objectives of the first proposed technology, condition assessment using nondestructive evaluation (NDE) technologies are to characterize cracks in HPC bridge decks and to evaluate their performance. The crack characterization by NDE is being concentrated on the measurement of crack depth, while the condition assessment of HPC decks is being concentrated on the evaluation of consequences of cracking on bridge deck deterioration progression. The study was conducted on ten bridges with HPC decks.

A more objective condition assessment of bridge decks, than one relying solely on visual inspection, can be made by a complementary use of nondestructive evaluation (NDE) techniques. The condition assessment has three main components: assessment of corrosive environment and corrosion processes, concrete degradation assessment, and assessment with respect to deck delamination. The NDE technologies used in the assessment include: half-cell potential (HCP), electrical resistivity (ER), ultrasonic surface waves (USW), ground penetrating radar (GPR), and impact echo (IE) method. Each of the five techniques has its advantages and limitations. However, each of them can contribute to a more comprehensive assessment of the condition of a deck. In addition, since the data obtained from NDE surveys are quantitative, a more objective condition rating of bridge decks can be made. Different condition-rating schemes are being applied in the study.

The objective of the second proposed technology, instrumenting and monitoring bridge decks under construction, is to capture the in-situ, early age response displayed by high-performance concrete (HPC) decks due to the curing process. In particular, the goal is to capture and track the temperature profile, thermal strains (uniform and gradients), and shrinkage strains that occur during the curing process from initial casting through one month of operation (and longer if deemed necessary). Capturing the actual early age demands that HPC bridge decks are exposed to allows the identification (and potential ranking) of the causal effects related to early-age cracking. In addition, the quantification of in-situ early-age demands allows for the evaluation of various laboratory-scale specimens that could be used to further investigate the phenomenon and eventually underpin the development of a more robust HPC specification.

The objective of the third proposed technology, a comparative durability analysis of bridge decks, is to evaluate the benefits of using HPC for the construction of durable bridge decks exposed to de-icing salts, considering the impact of early-age cracking on long-term performance. The overall goal of the task is to characterize concrete on the basis of in-situ conditions, deck coring program, evaluation of new concrete deck construction, and using the data collected as input parameters in STADIUM® simulations to compare the service-life of HPC decks. The simulation program will include:

- The determination of representative exposure conditions on the existing HPC decks;
- Durability analysis of uncracked HPC decks, based on the concrete properties determined from the investigated structures;
- Durability analysis of *Class A* concrete decks, based on the properties of concrete mixtures with similar composition from the STADIUM® database.

Work performed

To perform this suite of experiments, the team collaborated with NJDOT to develop initial criteria that would be used in selecting bridge decks to be tested. Table 3 highlights the bridges selected as of this writing, along with the initial selection criteria. In some instances, the department identified two structures (5a and 5b) that were of particular interest and importance. In other instances, the department could not identify bridge decks that met the selection criteria (decks constructed in a salt-environment, which did not experience early age cracking). Also note, the first two entries are new bridge decks that are used as samples for the instrumentation and monitoring bridge decks, NDE and comparative durability analysis activities.

Table 3- Bridges selected for the study

Span	Struct. ID	Description	Initial Selection Criteria	Deck Cond. SI&A
1	0418151	Collings Ave over Route I-676 (SB)	New construction HPC (instrumented, durability, NDE)	(Reconstructed in 2013)
2	1601162	Route 3 over NJ Transit	New construction HPC with skew (instrumented, durability, NDE)	(Reconstructed in 2013)
3	0311150	Route 70 over Bisphams creek	2-5 yr old HPC (durability and NDE)	8 - Very Good
4	1234-509	Smith Street (CR 656) over State Route 440	2-5 yr old HPC (durability and NDE)	(Reconstructed in 2010)
5 & 6	0511156 - 0511157	RT 52 over Rainbow and Elbow Channels	2-5 yr old HPC (durability and NDE) – salt environment and early-age cracking	7 - Good
6	Not Chosen	Not Chosen	2-5 yr old HPC (durability and NDE) – salt environment/no early-age cracking	Not Chosen
7	1209155	Route 9 Edison (Northbound)	5-10 yr old HPC (durability and NDE)	7 - Good
8	1209156	Route 9 Edison (Southbound)	5-10 yr old HPC (durability and NDE Tested)	7 - Good
9	3100-001	Ocean City – Longport Bridge	5-10 yr old HPC (durability and NDE) – salt environment/early-age cracking	6 - Satisfactory
10	0327-166	Creek Road Over I-295	Class A concrete deck with condition rating at or under 5.	5 - Fair

Overall, 10 bridge decks, 8 existing and 2 new, were selected based on their exposure, geometry, age, composition and current condition for a comprehensive investigation. Internal degradation and potential corrosion activity measurements were performed on existing bridge decks by means of different NDE technologies (impact echo, electrical resistivity, ultrasonic surface waves, half-cell potential). Materials characterization testing was carried out on all 10 bridges and featured the determination of mechanical properties (compressive and tensile strength, modulus of elasticity), physical properties (shrinkage, thermal expansion, spacing factor), transport properties (diffusion coefficient, volume of permeable voids, permeability), chloride contamination, and petrographic examinations. Also, both new decks were instrumented to measure internal temperature and strains during the first months after casting. Generally, the NDE results and materials characterization indicated that the existing bridges were in good condition and showed little signs of widespread deterioration or corrosion activity. At the time of the investigation, the chloride contamination has rarely exceeded the critical value for corrosion initiation and cracks do not have a consistent influence on chloride diffusion. On new decks, calculations based on measured concrete properties and instrumentation data have shown that tensile stresses could exceed the tensile strength. Numerical calculations were performed to determine chloride exposure and compare the durability of decks built with uncracked HPC, cracked HPC (worst-case) and Class A concrete. As expected uncracked HPC decks exhibit the best durability. In some cases Class A concrete can perform as well or better than cracked HPC in the first decades after construction. This highlights the fact that HPC specifications for bridge decks must be driven by the need for accrued durability instead of relying on high mechanical properties. Improving the transport properties and reducing the cracking tendency would achieve this objective.

The results of the three analyses was compiled into a comprehensive report that provides specific guidance on improving the performance of HPC in New Jersey. The report outlined immediate and short-term actions to be investigated, as well as recommendations that were dismissed. Through this dialogue, CAIT provided NJDOT guidance and the department provided CAIT direction on future programs.

Refined load rating

Background

The objective of this task was to perform refined load ratings of eight (8) bridges through the integrated use sensing and simulation. While there are few (if any) bridges owned by the NJDOT that are currently posted, there are several bridges that force Overload Truck Permits to be re-routed due to load carrying capacity limits resulting from “standard” structural ratings. To examine the validity of these restrictions, the research team employed 3D finite element (FE) modeling, load testing, and model-experimental correlation to determine if additional load carrying capacity exists in the selected bridges. In addition to potentially alleviating constraints on the movement of overload vehicles, this task provided NJDOT personnel “best practices” capacity estimation techniques and demonstrated their value.

Work Performed

Table 4 provides a listing of the bridges selected for this study.

Table 4 – Bridges selected for the Refined Load Rating study

Span #	Structure #	Description	General Observation
SPAN 1	0118150	US Route 206 over Cedar Branch	Multi-span solid slab structure
SPAN 2	0324152	US Route 206 over Springers Brook	Single-span monolithic reinforced concrete T-beams
SPAN 3	1103152	US Route 1 over D&R Canal	Single-span solid slab with monolithic stiffening ribs on a severe skew
SPAN 4	1512152	NJ Route 72 over Mill Creek	Two-span continuous reinforced concrete three-sided culvert on a severe skew
SPAN 5	1516152	NJ Route 166 over Toms River	Two-span continuous monolithic reinforced concrete T-beams
SPAN 6	1701151	NJ Route 40 over West Branch Creek	Single-span hybrid structure consisting of fully-encased steel beams and prestressed adjacent box beam structure on a skew
SPAN 7	1703152	NJ Route 40 over Salem Creek	Single-span hybrid structure consisting of fully-encased steel beams and partially-encased steel beam structure
SPAN 8	1237155	NJ Route 18 over Raritan River	Hybrid Structure - Main span is a fracture critical, curved steel, two-girder bridge system on piers with skewed supports

In addition to the above spans, the team also modeled and instrumented the County Bridge #020023A, East Anderson Street Bridge; and modeled and instrumented the Route 3 Bridge over NJ Transit as part of the study of severely skewed steel structures under construction (next subtask). This effort provided NJDOT with a means of reducing the number of "bottleneck bridges" in the state highway corridor. The success of this task is evident by NJDOTs request to increase the number of bridges to be studied up to twenty (20) in the 2014 and 2015 programs.

<u>Construction effects on severely skewed steel structures</u>

Background

At the time of erection, steel girders in highly skewed bridges can deflect out-of-plumb due to differential deflections experienced at crossframe connections. The expectation of the girders is that upon concrete placement, the girders will deflect back to a plumb configuration. Erecting steel girders for severely skewed bridges require special consideration. AASHTO 6.7.4 requires for bridges with a skew greater than 20 degrees, that crossframes be installed normal to the main members. The National Steel Bridge Association (NSBA) Steel Bridge Design Handbook states that this practice results in large differential deflections between each end of the crossframes. It further suggests that special guidance should be provided to the fabricator and erector. NSBA indicates that crossframes and diaphragms tend to equalize deflections, further cautioning that designing the interior and exterior girders for different inertias and dead load deflections can result in significant differences in camber between girders. The amount of differential camber, which is attributed to the effect of the bridge skew, the design camber plus allowable fabrication variances can be on the order of 2 or 4 inches on highly skewed bridges in the crossframe lines closest to the support locations. The effect of this differential camber during construction needs to be considered by the designer since the differences will likely complicate girder fabrication and erection. However, according to available literature out of plane bending of girders may not be a long-term concern.

Work performed

NJDOT expressed interest in research focused on understanding the effects of highly skewed bridges, and developing recommendations for design, fabrication and erection of these structures. The team will review available literature on highly skewed bridges to identify the latest analyses and research as well as study contractors' means and methods for girder erection in these complex structures and fabricators diaphragm connection detailing to determine the impact that these variables may have on out-of-plane bending. A synthesis report will be provided outlining the results of the literature review, means and methods review, and discussions with fabricators and erectors.

In addition to a synthesis report, the team allocated (1) bridge from the refined load rating activity to be used in a parametric study of the construction effects on severely skewed steel superstructures. The overall goal of instrumenting and monitoring steel girder frame superstructures on bridges with severe skew is to capture the in-situ, construction-stage response displayed by steel girders during erection and concrete deck construction. In particular, the goal was to capture and track strains that occur during the erection and deck placement process from erection through initial casting (and longer if deemed necessary). Capturing the actual construction demands that steel girders are exposed to will allow the identification (and potential ranking) of the causal effects related to erection and construction.

The results of this effort yielded a comprehensive synthesis report that included (1) a thorough review of available research, (2) 3D modeling and instrumentation of a severely skewed steel structure under construction, and (3) guidance and recommendations on modifications to the NJDOT bridge design manual. In addition, the team met with industry experts including High Steel and Hirsch steel fabricators, to discuss the recommendations and solicit their comments. The result is a document that combines academic, industry and construction expertise in providing guidance.

Discover new technologies and construction techniques

Background

Traditionally, NJDOT relied on training provided by the Federal Highway and National Highway Institute. This training was provided on the basis of availability, which may not have responded to NJDOT's needs as they arose. BRP developed training courses customized to respond to NJDOTs needs as identified through the Technology Transfer activities.

Exposure to technologies may be manifested through a variety of activities. In reviewing the performance of HPC, the department requested that BRP staff review and provide guidance on the construction practice of internal curing of concrete. Through this effort, the team developed a synthesis report and a new standard specification for the internal curing of HPC bridge decks.

Work performed

ASCE Webinars

NJDOT requested the program provide structural engineering and analysis training via webinars provided by ASCE. BRP staff coordinated twenty (20) webinar sessions throughout the program year. The following is a listing of the webinar courses provided for NJDOT staff:

1. Friday, November 30, 2012, 12pm-1pm - Culvert Analysis Using FHWA HY8 Software
2. Wednesday, December 05, 2012, 11:30am-1pm EST – Preventing Bridge Damage During Earthquakes
3. Tuesday, December 11, 2012, 11:30am-1pm - LRFD Design of Ground Anchors & Anchored Wall Systems
4. Wednesday, December 12, 2012, 11:30am-1pm EST – Strengthening Structural Steel Beams
5. Monday, December 17, 2012, 11:30am-1pm EST - Verification of Computer Calculations by Approximate Methods
6. Thursday, December 20, 2012, 11:30am-1pm EST – Geosynthetic Reinforced Mechanically Stabilized Earth Walls
7. Monday, January 07 and 24, 2013, 11:30am-1pm EST – Load and Resistance Factor Design (LRFD) for Geotechnical Engineering Features (Two Part Series)
8. Thursday, January 10, 2013, 11:30am-12:30pm EST - Earthwork 101
9. Friday, January 11, 2013, 11:30am-1pm EST - Practical Design of Bolted and Welded Steel Connections
10. Tuesday, January 22, 2013, 12pm-1pm EST - Advanced Bridge Hydraulics with HEC-RAS
11. Friday, January 25, 2013, 11:30am-1pm EST - Underpinning & Strengthening of Foundations
12. Wednesday, February 06, 2013, 12pm-1pm EST – The Five Pieces of Equipment Every Bridge Inspector Should Have

13. Thursday, February 7, 2013, 11:30am-1pm - Energy Piles: Background & Geotechnical Engineering Concepts
14. Tuesday, February 12, 2013, 11:30am-1pm - LRFD for Geotechnical Engr. Features: Design & Construction of Driven Pile Foundations
15. Friday, February 15, 2013, 11:30am-1pm EST – Avoiding Failures of Retaining Walls
16. Friday, February 22, 2013, 11:30am-1:30pm - Design of Anchor Bolts
17. Monday, February 25, 2013, , 11:30am-1pm - Design of Concrete Embedments
18. Monday, March 4, 2013, 11:30am-1pm - LRFD for Geotechnical Engr. Features - Deep Foundations - Lateral Analysis
19. Monday, March 18, 2013, 11:30am-1pm EST – Corrective Work in Steel Structures
20. Thursday, March 28, 2013, 11:30am-1pm EST – Design for Extreme Event Loading

Internal Curing of HPC decks

Internal curing is a relatively new technique of casting concrete that includes the replacement of a portion of fine aggregate with an equivalent volume of pre-wetted fine aggregate. The result is a concrete mixture with a "reservoir" of moisture locked-in to be drawn upon once the concrete begins to shrink or self-desiccate.

New York State has taken initial steps to demonstrate the performance of internally Cured HPC (ICHPC). In the report, "Field Performance of Internally Cured Concrete Bridge Decks in New York State" (SP-290-7, Streeter et al), NYSDOT reports HPC-IC has shown improvements by reducing the cracking associated with concrete shrinkage. Seventeen (17) bridges were included in the study. The report concludes that ICHPC is a helpful tool that can be used to improve concrete properties, but it must be coupled with sound construction practices.

The BRP team reviewed the construction technique and developed a technical memo outlining the state-of-the-practice, as well as its applicability in improving the performance of HPC. The effort complements the team's review of HPC performance, and the team recommended that a standard specification be developed for future use in pilot testing of a design mix. The team met with NJDOT Bureau of Materials personnel, presented the team's findings and solicited comments. As a result, the team was requested to develop a specification, which was accomplished through the On-call services task.

On-Call Services

The intent of this on-call task is to rapidly respond to the State's needs beyond routine maintenance queries. The advanced forensics and Nondestructive Evaluation techniques being developed and refined at CAIT can be leveraged to provide advanced monitoring of NJ assets and to diagnose complex conditions that are undetectable using visual inspection practices. As a result, the NJDOT will be able to leverage advanced techniques, when needed, to perform highly specialized evaluations of state assets for planning maintenance strategies.

The BRP staff will respond to 90% of requests within one day and develop an appropriate work plan. For evaluation requests, BRP staff will review existing conditions, determine the appropriate method of evaluation, perform the needed evaluation and recommend improvements. Infrastructure Condition Monitoring Program (ICMP) will respond to NDE field evaluation upon NJDOT request within 3 days. For materials and technology review requests, BRP staff will review available resources such as FHWA, TRB and other publications, determine the viability of the materials and/or technology and provide a recommendation within 3 days.

In addition, the team developed a framework for NJDOT to select from technologies and/or construction techniques identified under task 2 for field application. BRP staff will develop technical standards, including construction inspection techniques, materials requirements, tolerances, and other key parameters sufficient in detail for inclusion in construction projects.

Development of these standards may include a number of ancillary activities, such as reviewing existing NJDOT standards, details and guides for conformance with the new technology or construction technique. For those standards, details and guides that may need revision, BRP staff will prepare recommended revisions to the standards/details that would allow for a cost-effective implementation of the new material or construction technique.

NJDOT may also request a review of specific standards, details and guides for updating based on new materials, construction techniques or improved details. BRP staff will coordinate the revisions with NJDOT staff and prepare documents reflecting the revisions.

The following subsections provide a summary of activities undertaken under the on-call task.

I-195 dump truck fire response and analysis

In response to a dump-truck fire under the I-195 Bridge over the NJ Turnpike, the team performed a rapid load testing of the structure to determine the remaining load capacity in terms of its ability to carry traffic prior to its demolition, scheduled approximately 8-weeks from the time of testing. The team performed testing on October 11, 2012, provided initial observations and recommendations to reopen the bridge on October 16, 2012; and provided a detailed report of the team’s findings on November 7, 2012.

Mass Concrete

NJDOT requested that Rutgers-CAIT perform a literature review on the state-of-the-art practice of mass concrete and use the findings to compare with the Thermal Control Plan for the Route 7 Wittpenn Bridge Pier 1W cap as well as the current mass concrete specifications included in the NJDOT 2007 Standard Specifications. The review focused on material composition, with description of each component’s contribution to heat of hydration. The team observed that the literature focused on two areas of concern, maximum temperature reached during curing and thermal differentials between the core and surface of the mass concrete element.

The literature has extensively documented the urgency of maintaining the maximum curing temperature below 160°F. The adverse effects associated with exceeding the maximum temperature threshold are severe, but not visible for months or years after construction. This threshold should never be exceeded.

The literature also documents damages resulting from exceeding temperature differential thresholds, which are more immediate and can be identified during construction. The thermal-induced cracking that results may be repaired through industry accepted means, from seals, coatings for hairline cracking, to more comprehensive repairs.

During early stages of curing, the concrete has not developed sufficient strength to resist excessive thermal gradients. Thus, form insulation and other methods to protect the concrete surface from dissipating heat greatly or reach excessively high peak temperatures reduces the likelihood of deleterious effects. The results of this literature review suggest that current research and industry agree that temperature thresholds are critical to mass concrete. Proper controls must be established in order to ensure well-performing concrete elements to be constructed.

Internal Curing of HPC

Following the team's presentation of internal curing of concrete, and coordination with the NJDOT Materials Bureau; the team prepared a standard specification for the internal curing of HPC in concrete decks. The team contacted industry experts including personnel from NE solite and the Expanded Shale, Clay and Slate Institute (ESCSI) to review the new specifications. The basis for the specifications was NYSDOTs specifications, which was successfully used on numerous recent bridge construction projects. The newly developed specification was submitted to NJDOT Materials Bureau for review, and comments were addressed. The specification was finalized and a recommendation to select potential pilot projects was submitted to NJDOT.

Conclusions

BRP has provided opportunities to bring technologies to the department, review existing practices, and propose the use of new construction techniques to improve asset management, design and construction practices. In addition, it has created a new channel of communication between CAIT and NJDOT that allows for the rapid deployment of innovative technologies. In the future, the BRP is envisioned to continue to identify opportunities for innovation. It is anticipated that new research topics will be borne out of the program. As research is completed, it can return back to the BRP for pilot testing and recommendation for deployment. The cycle of innovation, testing, implementation and need for further innovation can be perpetuated through the creation of similar resource programs.

APPENDIX 1A
GUIDANCE REPORT ON FHWA AUDIT AND DETERIORATION MODELING

RU435056-7

FHWA Audit Review and BrM Version 5.2.2 Guidance

Technical Memorandum
June 2014

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Disclaimer Statement

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16. Abstract

A framework for developing state specific solutions for the NJDOT Bridge Management System (BMS) is proposed by the Center for Advanced Infrastructure and Transportation (CAIT) Bridge Resource Program (BRP) team. Taking basis in the FHWA Program Review document dated January 18, 2013 and the related recommendations for the New Jersey's BMS, the BRP team herein provides guidance to NJDOT in the areas of development, standardization and implementation of processes and technologies for bridge assessment, maintenance, and rehabilitation. The identified areas of opportunity include advanced state-specific deterioration modeling of bridges, risk assessment for the bridge network, and technology based decision support for rehabilitation project prioritization.

This document provides recommendations over the course of CY 2014-2016 for integrating specific functionalities into the Bridge Management System. Longer term comprehensive approach to address the 2013 FHWA audit recommendations in the development of the BMS is presented. BrM software is expected to replace AASHTO Pontis software previously used by NJDOT. The outline of functionalities that are available in BrM version 5.2.1 and anticipated in versions 5.2.2 and 5.2.3 is presented.

17. Key Words Asset Management, Deterioration Modeling, Bridge Management System, assessment, maintenance, rehabilitation, risk assessment, risk based prioritization	18. Distribution Statement

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Table of Contents

1.0 Document Purpose

The purpose of this document is to provide a road map to address the FHWA audit report recommendations for the NJDOT Bridge Management System (BMS) using the AASHTO Pontis/BrM 5.X.X software. The recommendations in this document are based on the known functionality of the BrM software as of March 2014. It is to be used as a desk top guidance document by the NJDOT bridge evaluation team as the general direction on how to use BrM software in the future.

2.0 Introduction

This document is provided as part of the Center for Advanced Infrastructure and Transportation (CAIT), Bridge Resource Program (BRP) for the NJDOT CY 2013 program. In January 2013 the Federal Highway Administration (FHWA) performed a program review of the New Jersey Bridge Management System. Based on the findings of the review, the FHWA report identified opportunities for improvement and provided recommendations, which are summarized in section 3.0. In an effort to support NJDOT in developing a technology-centered BMS, and in implementing the FHWA recommendations, the BRP team proposes activities outlined in Section 3.1 and Section 8.0, and provides guidance in realization of the short- and long-term goals.

NJDOT has been a user of the older versions of the AASHTO software called Pontis, and plans to use the latest AASHTOWare Bridge Management software (BrM). This document provides guidance to NJDOT regarding the future use of the AASHTOWare Bridge Management software (BrM) in support of the recommendations based on the FHWA audit with an emphasis on deterioration modeling of bridge assets in BrM 5.2.2.

2.1 NJDOT Bridge Management System (BMS) Background

NJDOT’s current Bridge Management System (BMS) consists of two components:

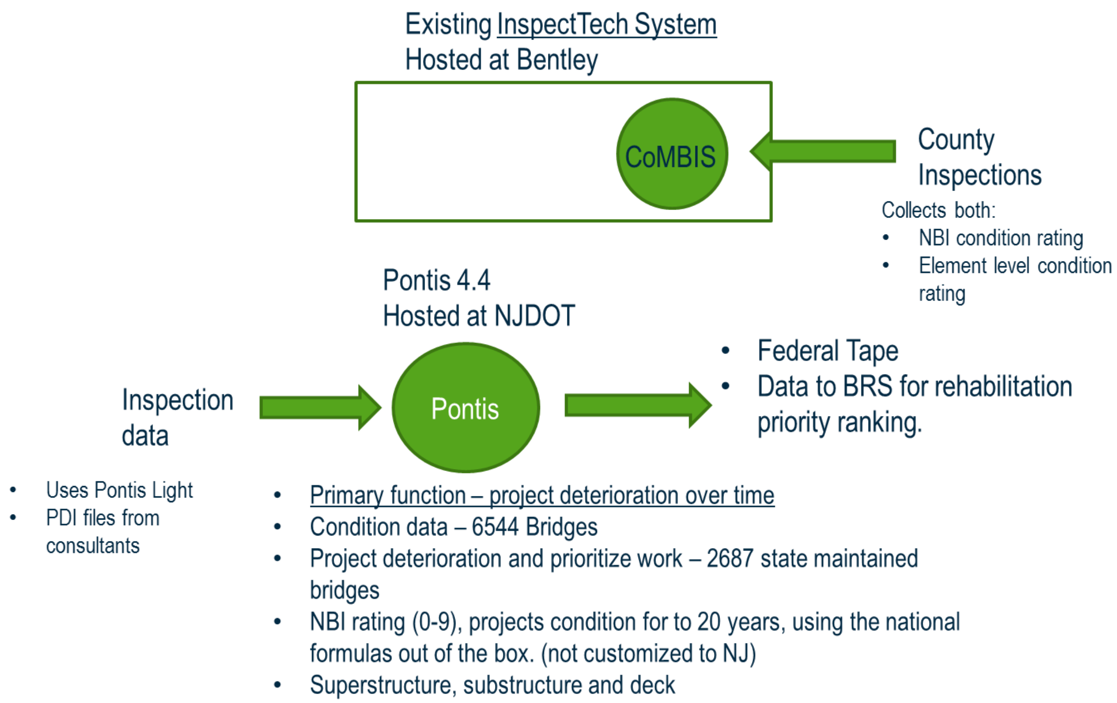
- The Pontis Bridge Management software – Version 4.4
- A manual Bridge Ranking System (BRS) – A spreadsheet based BRS which extracts bridge condition and non-condition data from BMS, applies a weighted formula to those factors and then calculates a total score.

The existing Pontis version 4.4 software provides:

- The storage of bridge inventory and condition data, not specific inspection reports. They store core element information. The collection of data is being populated from PDI files from an external source.
- Some state defined fields. No other customization.
- Pontis version 4.4 software features currently <u>not</u> used as stated in the audit report.
    - Deterioration modeling.
    - Costing out alternate scenarios
    - Recommendation of optimal programs and schedules
    - Performance curves for future bridge conditions developed for alternate funding level scenarios

2.2 Existing NJDOT Bridge Management System

The following diagram represents the current state of the NJDOT Bridge Management System (BMS) as of March 2014. Inspection data flows into the Pontis 4.4 system. The federal tape is created from the Pontis system. In addition, Pontis data is used for the manual Bridge Ranking System (BRS) for rehabilitation priority ranking. There is an existing InspectTech system that is hosted remotely that is used for county bridge inspections. It collects both NBI and element level condition ratings. The two systems are not connected in any way at this time.



2.3 Reconfiguration and Upgrade of the current Pontis 4.4 software system

The existing NJDOT Pontis version 4.4 system was upgraded and evaluated. The following represents a summary of that process and evaluation:

- The data export from the Pontis 4.4 system provided by NJDOT was upgraded to BrM version 5.2.1 with very few issues.
- The Pontis 4.4 database was upgraded to the latest BrM version 5.2.1 database.
- The existing Pontis 4.4 system captured inspection information using core elements for 8007 bridges.
- The existing Pontis 4.4 system is currently only being used for storing core element data for the bridges.
- An upgrade from the core elements in Pontis 4.4 to the NBE elements in BrM 5.2.1 can be applied to the Pontis 4.4 database, but was not performed at this time.

3.0 FHWA 2013 Program Review Summary

FHWA Program Review Report provided the following recommendations:

1. NJDOT, in consultation with Rutgers consulting staff assigned to the Bridge Resources Program, should assess the accuracy of Pontis deterioration algorithms and modify such in Pontis as necessary.
2. NJDOT should identify the investment level and performance associated with funded bridge rehabilitation projects in the capital program documents (TIP, STIP, MTP, SLRTP) as they compare with CIS target investment and performance levels. For example, if the CIS recommends a target investment level averaging $720 million annually to address a certain square footage of structurally deficient bridges, then the STIP should show the dollar value of actual bridge rehabilitation projects in the STIP and how those specific projects are anticipated to impact the targeted performance level associated with structurally deficient bridges.
3. NJDOT should begin entering data into Pontis regarding the programming/contracting of projects as they occur. Pontis performance curves used in the CIS cannot be accurate if the system does not know what improvements have already been funded.
4. NJDOT should establish and document formal procedures which govern the essential BMS tasks listed under 23 CFR.107.
5. Prepare a policy and procedure which documents the process by which bridge rehabilitation projects will be prioritized using the BRS and other factors. Projects are not initiated by selecting the next ranked bridge so a fuller description is needed to explain how decisions are made and who makes them.
6. Develop a decision log which provides a transparent record of what factors contributed to the prioritization of a project besides the BRS ranking.
7. NJDOT should either use Pontis cost/benefit modeling to guide decisions regarding preservation, rehabilitation, or replacement decisions, or alternately develop written policies and procedures based on objective criteria for making such decisions.
8. In conjunction with the Rutgers University's Bridge Resource Program, researchers should be asked to perform the following analysis:
    a. Determine whether the current Pontis network optimization logic should be customized to recommend an optimal selection of bridge rehabilitation projects to meet New Jersey performance goals or whether the Department should wait until the next Pontis upgrade includes such functionality.
    b. Evaluate the feasibility of integrating non-condition based rating factors found in the BRS into the Pontis optimization modeling process.
    c. Evaluate the feasibility of using the Agency Policy Goals to make treatment recommendations that are consistent with standard NJDOT contracting practices

3.1 Planned BRP Activities Related to FHWA Program Review Recommendations

There are key themes that have emerged from the FHWA review as it relates to BRP activities in the future. Those themes are as follows:

- Establish/develop a utility-based value for each bridge
    - The utility functionality provides information about the current state of the bridge, and is used to determine the relative importance of

particular elements or systems (e.g., deck, sub-structure) for a bridge structure. This capability is available in BrM version 5.2.1.

- Develop bridge deterioration modeling capabilities within the BMS. This functionality is planned for BrM version 5.2.2. This capability must also include the validation of historical inspection bridge inspection data specific to NJDOT.
    - Bridge Deterioration Model logic and analysis is a key feature in the BrM 5.2.2 release of the software. The accuracy of the Pontis deterioration algorithms in BrM version 5.2.2 as planned by the AASHTO task force recognized the need for the states to have the ability to use different deterioration scales and formulas to address their specific situation. It was also recognized that the Pontis 4.x deterioration models did not reflect actual deterioration in the field for changes from condition states 1 to 2. To address this issue, the task force recommended the use of the Weibull distribution analysis. The BrM 5.2.2 software will use a combination of both Weibull and Markovian models to improve its accuracy by adding a time factor component. In the older version of Pontis, the Markovian model was used exclusively. It was condition based only, and did not take age into consideration. It also showed faster deterioration rates in the earlier years. In addition, the new element structure with parent/child relationships was added to allow the addition of protective systems to the model.
- Enable Project and Capital Planning. This functionality is planned for BrM version 5.2.3
    - Capital and Project Planning are the key elements for 5.2.3. It is recommended that the existing Bridge Ranking System (BRS) functionality be developed in the BrM platform. The current BRS uses an external spreadsheet to rank bridges as follows:

NJDOT Bridge Rehabilitation Priority Ranking
Scoring Criteria

	Criteria	Weighting (W)	Scoring (S)
A	Average Daily Traffic (Item 29)	10%	0 to 30,000=0 30,001-60,000=.25 60,001 to 90,000=.5 90,001-120,000=1.0
B	Functional Class (Item 26)	5%	Interstate/Freeways (01,11,12)=1 Arterials (02,06,14,16)=.67 Collectors (07,08,17)=.33 Locals (09,19)=0
C	Deck (I_58)	5%	3 or 4=1 5 or 6=.5 >6=0
D	Sufficiency Rating	30%	(100-S.R)/100
E	Structurally Deficient	35%	Yes=1, No=0
F	Bypass Detour Length (I_19)	5%	00 to 01=0 2-4=.25 4-6=.5 6-9=.75 10 or more=1
G	Scour Critical	5%	Yes (Code 3 or less)=1 No=0
H	Fracture Critical (I_92A)	5%	Yes=1 No=0

Source: NJDOT Division of Structural Evaluation and Bridge Management

The following table shows the score and ranking of systems:

NJDOT Bridge Ranking System
2012

Priority Level	Criteria	# of Bridges	%
1	Score>350	40	1.5%
2	Interim Inspection Required	64	2.4%
3	175-349	246	9.1%
4	90-174	483	17.9%
5	0-89	1,858	69.0%
	Total	2,691	100.0%

Source: Compiled from NJDOT 2012 Priority Ranking List (Hanson)

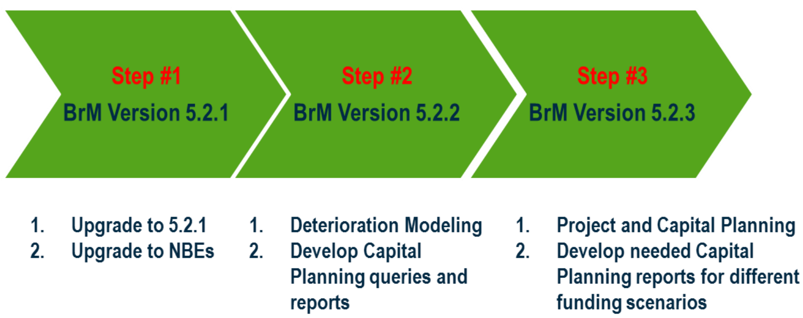
  - The BRP will investigate the integration of the current Bridge Ranking system into BrM 5.2.3.
- Implement a Risk Based Prioritization (RBP) methodology to enable project prioritization. Limited risk modeling capability is available in BrM version 5.2.2. A more robust capability will be available in BrM version 5.2.3.
  - User-defined risk assessment types are included with BrM version 5.2.2
  - Risks are a new feature in the 5.2.2 version of the software. Risks have not been fully defined with respect to how they will be used in BrM at this time. This will be done by the AASHTO task force in the future as 5.2.X is fully defined.

- The BRP will work with NJDOT on developing the module for implementing the Risk Based Prioritization framework suggested by Intelligent Infrastructure Systems (IIS) as part of the integrated solution.

- Develop a framework for Cost-benefit analysis. This functionality is expected in BrM version 5.2.3.

4.0 Recommendations for the Implementation of BrM version 5.2.X

The purpose of this section is to provide "overall direction" to NJDOT regarding the future use of the AASHTOWare Bridge Management software (BrM) version 5.2.X. The following diagram represents a roadmap for NJDOT for the use of BrM 5.2.X.



The following are BrM software recommendations:

- Load the released version of BrM 5.2.1 in a test environment.
- Formally request AASHTO for a quote for the needed Capital Planning reports and queries using service units.
- Formally request AASHTO for a quote to replace the BRS functionality using service units.
- Leverage the CAIT task related to a risk modeling with AASHTO as part of the future software.

4.1 CY 2014 recommendations:

The following is recommended for CY 2014:

- Focus on NJ DOT state-specific deterioration models. This task builds upon the CY 2013 CAIT project that provided NJ DOT with guidance on the use of the "out of the box" features in the 5.2.2 version of the AASHTOWare Bridge Management software. This includes the "out of the box" deterioration modeling functionality. Once this functionality has been defined in the new release, NJ DOT specific deterioration models can be developed via an expert elicitation interview process. In addition, historical inspection data on actual changes in element condition distributions over time will be captured. The deterioration model will take information from both of these sources and generate a model to represent how the condition of different elements change over time specific to NJ DOT. The following activities are recommended:

- Develop a plan for NJDOT specific deterioration model development based on BrM version 5.2.2
- Conduct expert elicitation interviews based on the new version of the software.
- Perform an historical inspection data elicitation that can be loaded into the new version of the software.
- Develop and validate NJDOT specific deterioration models.

- Upgrade to BrM version 5.2.1 System. The following information is focused on the system related issues when moving from Pontis 4.4 to BrM version 5.2.X.
  - 2011 elements vs. 2013 elements migration
    - Focus efforts on the 2013 element migrator once it is released in Q1 2014. There is little value in looking at the 2011 element migrator, since there is no effective migration path between the 2011 and 2013 elements. You must start with core elements and then migrate to the new bridge elements.
    - The 5.2.1 release of the software utilizes the 2013 elements, which was released in Q1 2014.
    - The current 5.1.2 and 5.1.3 versions of the software utilize the 2011 elements.
  - Element, Software and Database migrations
    - There is a significant difference between element and software migrations. The migrations can be done independent of each other. A software migration involves the use of a newer version of the BrM software, such as updating Pontis to 5.2.X. The software migration also requires the database to be migrated to match the software version. The element migration involves updating the core elements to the new 2013 elements. <u>The guidance for migration sequence is database, software then elements.</u>
    - The 5.2.X version still displays the core elements in a read only fashion. This allows NJDOT to use the old inspection data in a read only mode.
    - Guidance regarding the software migration is to create a test instance of the new software to get familiar with the new user interface (UI).
    - Guidance regarding the element migration is to pick an element transfer date, which may align with the current NJDOT inspection cycle or the CoMBIS upgrade. The Pontis software does not migrate the core elements. It’s done by a 3rd party software called the “element migrator”. It is recommended to run an “as is” migration of the NJDOT elements. The software comes with “best practice” migration routes out of the box. In addition, it is best to check the migrator rules. Note: The element migrator ignores any custom created NJDOT elements. It is recommended that NJDOT look at the 2013 migrator rules in the near

term. Once the rules are set, NJDOT can run the migrator on the old core data. This converts the core element data to NBE.

- BrM version 5.2.1 - short term guidance
  - NJDOT should review the new user interface (UI) in BrM 5.2.1, since is different from Pontis 4.4.
  - NJDOT should review the new functionality in BrM 5.2.1 – Utility curves, new elements, mapping (Google maps), filters and layouts, which is how data is brought back into BrM, and how data is then viewed, and finally bridge groupings.

4.2 CY 2015 Recommendations

The following is recommended for CY 2015:

- During CY 2014, develop NJDOT specific deterioration models based on historical bridge inspection data using a pre-released version of the 5.2.2 software. For CY 2015 implement this on the production release. The following activities are recommended:
  - Develop an implementation plan for the released version of BrM version 5.2.2.
  - Develop recommendations for the configuration of the software for bridge deterioration modeling based on families of bridge types based on the CY 2014 task.
  - Configure a test site for the BrM version 5.2.2 software for NJDOT specific deterioration models.
  - Develop upgrade scripts for 5.2.1 to 5.2.2 to configure the software to the recommended configuration.
  - Note: The production release of BrM 5.2.2 is expected in the first quarter of 2015. The actual upgrade to 5.2.2 can be done with AASHTO service units.
  - Develop a user's manual that can be used by the NJDOT Bridge structural evaluation staff.
  - Document the deterioration modeling software configuration for the BrM 5.2.2 software.

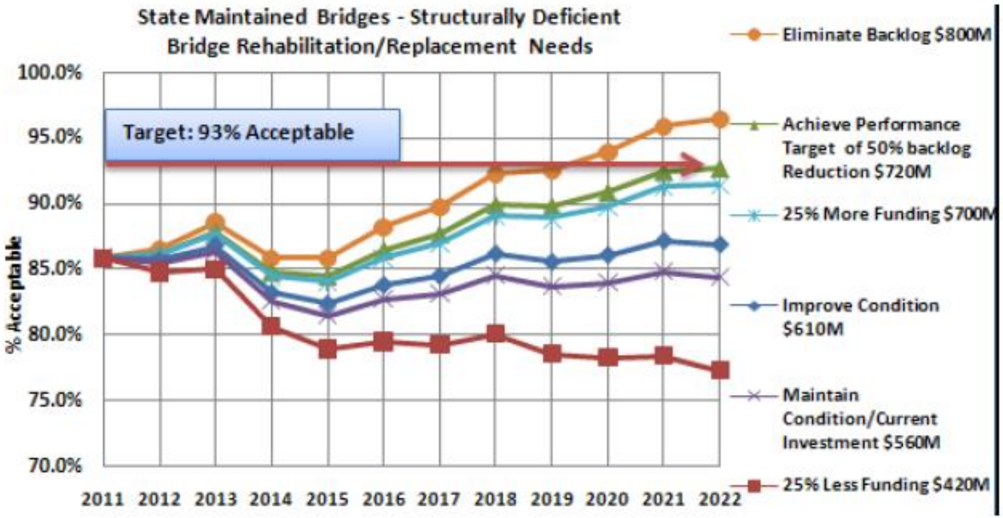
- Develop an integrated means of accomplishing risk based prioritization within the Department's BMS framework, using the IIS developed model. The BrM version 5.2.1 software provided the utility function as an initial step towards a framework to assess risk.

4.3 CY 2016 Recommendations

The following is recommended for CY 2016:

- Focus on Project and Capital Planning, since this functionality is planned for BrM 5.2.3.
- For Capital Planning, develop requirements for bridge reports and queries based on the information provided by NJDOT.

- Develop charts such as that shown below with different funding scenarios related to improvements (% acceptable and Deficient Bridge Deck Area).



- In support of capital planning, the following queries and reports can be provided through an AASHTO request for service units.

  - Prioritization of bridges based on condition of superstructure, deck, substructure and bridge sufficiency rating.

  - Total number of Priority 1, 2, 3 and ranking of specific bridge with respect to its Priority. (7th Priority 1 out of 88)

  - Total scoring to determine Priority rank.

  - Total number of:

    - Major Viaducts
    - Movable Bridges
    - Standard Bridges
    - Minor Bridges
    - Remaining service life for each type.
    - Percentage of each type in a State of Good Repair (SOGR)

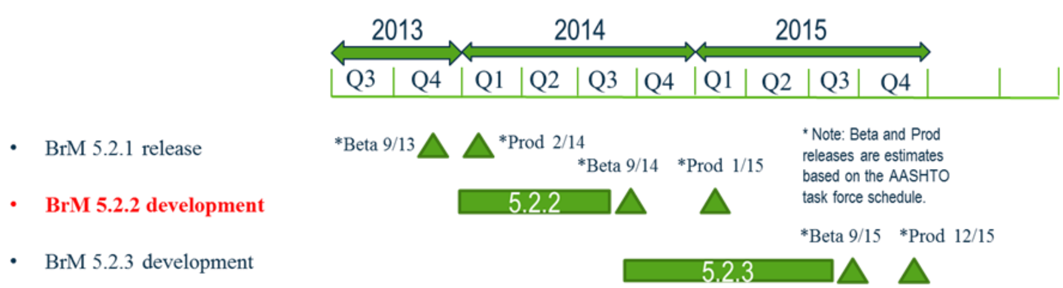
  - Track state bridges on the National Highway System (NHS).
    - Including number of structurally deficient bridges.
  - Percent of bridges in SOGR
  - Total deficient Bridge Deck Area
  - Area of Deficient Deck Area added each year.
  - Area of improved Deck Area completed each year.

- Number of bridges over a given length, including ownership/type. (Local, State, Toll Authority, NJ Transit, etc)
- Breakdown of bridges, by percent or total number, into structurally deficient, functionally obsolete, or neither.
- Breakdown of bridges by age and approximately when they will need rehabilitation/replacement.
- Percentage of bridge deck area in acceptable condition.
- Percentage of Total bridge deck area 75 years or older.
- Breakdown of bridge condition by owner (e.g, functionally obsolete state bridges, etc.)
- Number of State bridges in acceptable/unacceptable condition.
- Total bridge deck Area in acceptable/unacceptable condition.
- Break down of bridges by owner. (State, D&R Canal, DEP, NJ Transit.

5.0 AASHTO BrM Functionality Summary

The AASHTO software product has been rebranded from Pontis to BrM. The BrM family of software includes BrM 5.2.1, 5.2.2 and 5.2.3. The recommendations in this document are based on the expected beta release of BrM 5.2.2 in the September 2014 timeframe. The final production version is not expected until early Q1 2015. The intent of the guidance is to provide NJDOT a preview of what is expected in the BrM 5.2.2 release. The guidance provided in this document represents what is currently known about the expected functionality as of March 2014. As an example the deterioration modeling functionality has been prototyped, but has not been implemented into the software yet. This is also the case for the Project Planning functionality.

The BrM software product release plan is as follows:

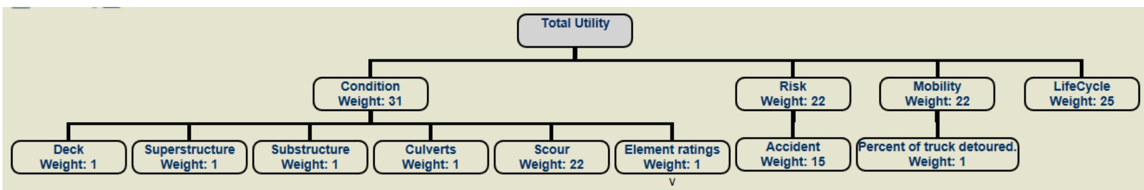


5.1 BrM 5.2.1

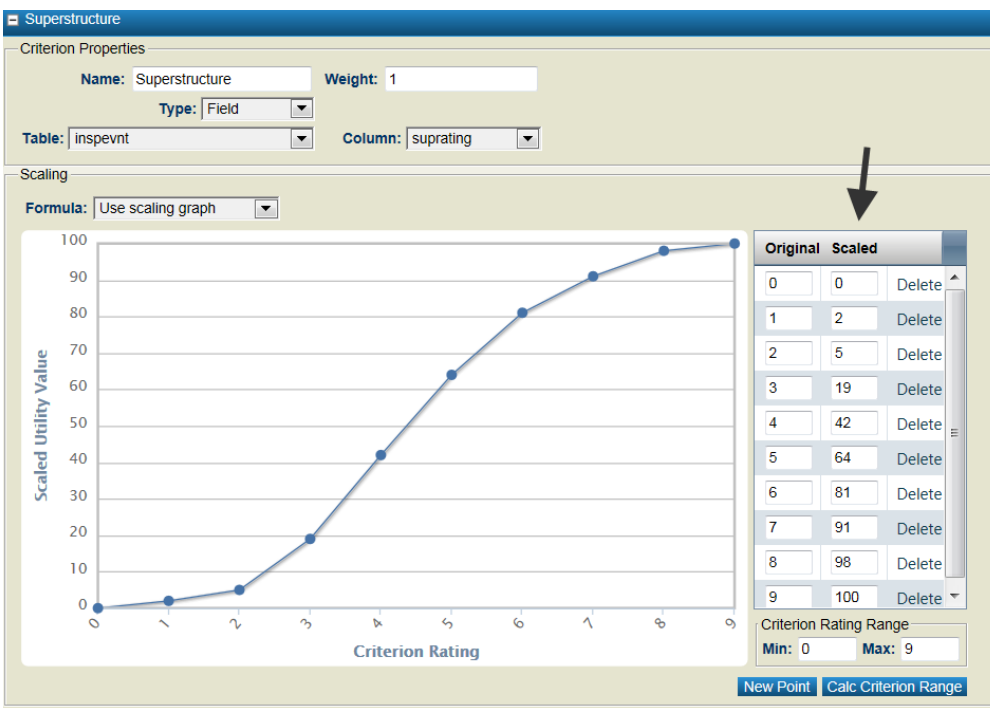
The BrM 5.2.1 is the current release of the software. It provides new and key functionality as follows:

- The Utility Function
    - The utility function is entirely new for 5.2.1. The utility function ranks what is most important to the bridge, such as, are the decks more important than the sub structure? In addition, NJDOT can define what is to be used to rate that utility. The Utility function comes with default out of the box weighting

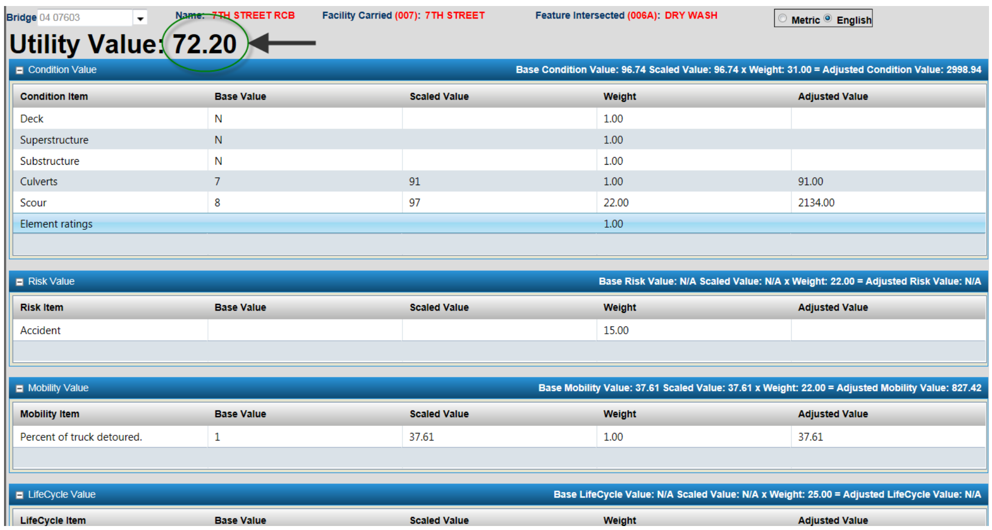
factors, but is configurable by the user. There are utility curves for the deck, superstructure, substructure, culverts and scour. There are default scaling curves for each item. Utility can be evaluated by condition, risk, mobility and lifecycle. Risk includes events such as scour and accidents. An example of mobility is the ability to support traffic, such as the % of trucks detoured. An example of lifecycle is how long the bridge will last and the associated rehabilitation projects that influence the life of the bridge. Ultimately the goal of the utility function is to analyze the overall utility value for a bridge. For project planning NJDOT could do fixes to move an asset from a 6 to an 8 to see the increase of the utility. It can also be used once deterioration modeling becomes available. Utility provides the current state of the bridge. The Deterioration modeling functionality will provide a “projected” utility value over time. The following diagram show the utility function and weighting factors configured out of the box:



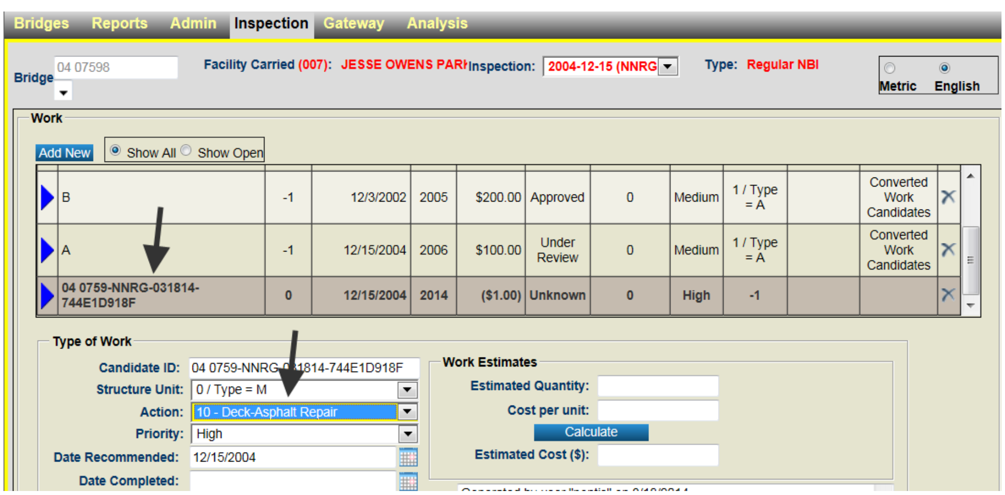
An example of how the Utility function is configured is shown below for “superstructures”. The scaled factors range from 0 to 100 and are user definable. The total utility function is weighted by these factors.



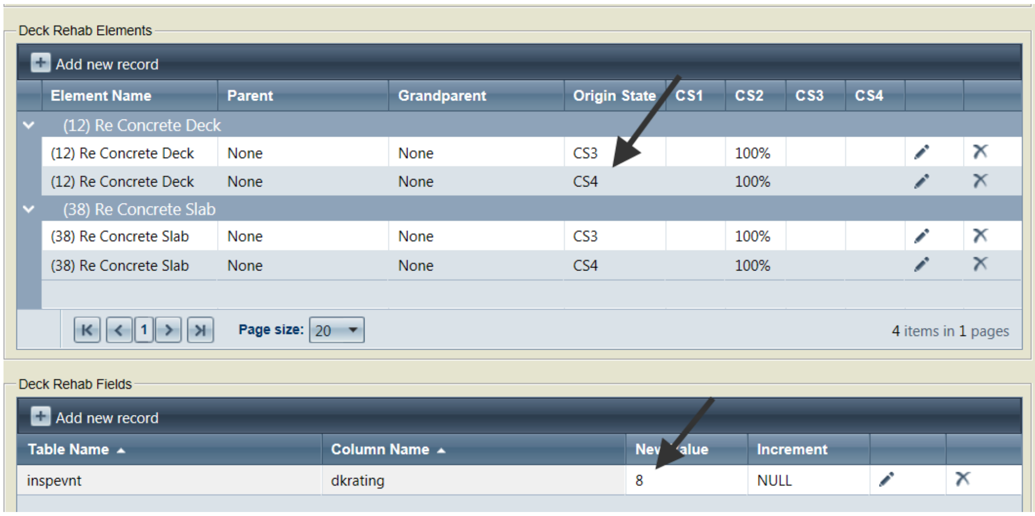
The Utility function is configured once. To determine the overall utility value of a bridge, the user selects a bridge and then goes to the Analysis module as shown below. The overall Utility value for this bridge is 72.20.



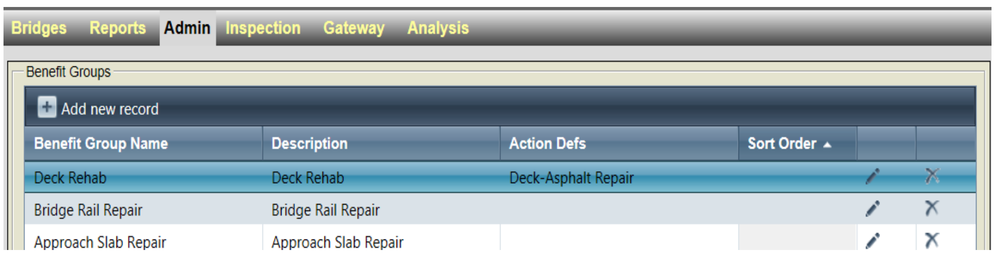
Once the Utility function is configured, a work candidate for a bridge can be added as shown below. Each work candidate has an associated action that will be taken. In the example below the action is “10-Deck – Asphalt Repair”. (Select Inspection => Work)



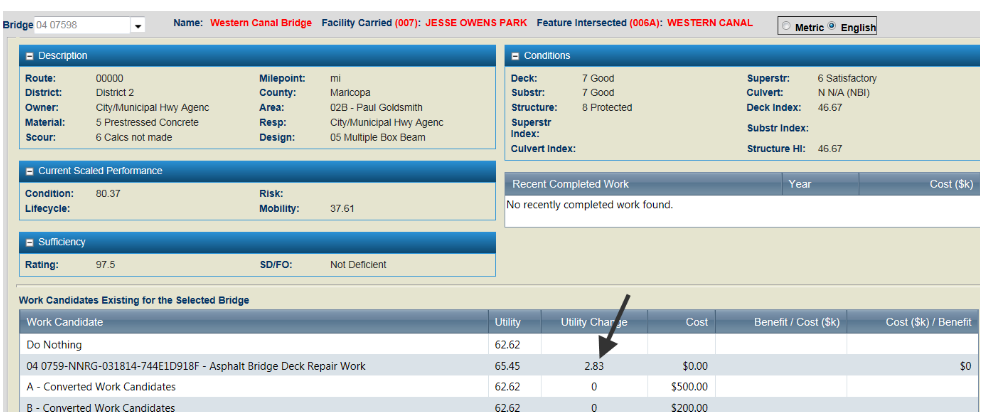
The following figure shows the impact of an action. In this example, the deck rehab results in the concrete deck moving from CS3 and CS4 to CS2. In addition, the deck rating moves to a rating of “8”.



Bridge repair actions need to be linked to a "benefit group". In this example the Benefit Group called "Deck Rehab" is linked to an action called "Deck-Asphalt Repair".



The next step in the process is to evaluate a proposed "work candidate" for a bridge to determine the impact of the action to be taken by calculating its new overall Utility value. The following diagram shows the increase in the Utility value based on this action using the preconfigured Utility weightings.



In summary, the Utility function is highly configurable to the needs of NJDOT.

5.2 BrM 5.2.2

- Bridge Deterioration modeling is the key functionality provided by this release. (See Appendix A for more information about the functionality)
- Project Planning
    - Project planning functionality is also new to BrM version 5.2.2. The planned functionality is as follows:
    - Create a Project from User Selected Bridges
        - One or multiple bridges can be identified on a selection screen to "add" to the project. Once the bridges are added to a project, the various items of work (inspector or system generated) on each bridge are presented (one bridge at a time) in descending utility cost ratio order for the user to select one or more actions for analysis.
        - Analysis includes a presentation of before and after utilities for all defined utility components, the impact of timing utilizing deterioration and life cycle cost analysis and forecasted condition metrics. The forecast future year performance for all defined utility functions is available. Analysis is done one bridge at a time for the selection of the "best" actions for an individual bridge.
        - Once the preferred bridges, action(s) and project timing have been determined, the project can be assigned to one of the defined funding programs. The user is presented information related to the remaining budget within the chosen year for the defined program. The project is then created with a status of "proposed" awaiting confirmation of the project and a status change to "programmed".
        - Programmed projects have top priority when evaluating network needs and priorities. If network program budget has remaining capacity after all defined “programmed” projects have been funded, then the software will consider "proposed" projects and finally individual or combinations of needs (Inspector and system generated) on single bridges within the database until the budget is exhausted. The user is able to establish a programming level budget that exceeds the actual funding available.
    - Create a Project from a Bridge Analysis Group (Corridor or Geographic Region Project)
        - Bridges are selected based on a bridge analysis group (geographical proximity, county, route milepost range, distance from a particular bridge or as otherwise defined by a bridge analysis group). The selection of individual bridges from within a defined bridge analysis group is supported.
    - Utilize System Generated Projects
        - “Conceptual” projects can be generated based on a set of user defined criteria. An agency can define queries that help it spot bridges that are candidates for types of repair it believes needs to be carried out. Then it can create a group of bridges using these queries.

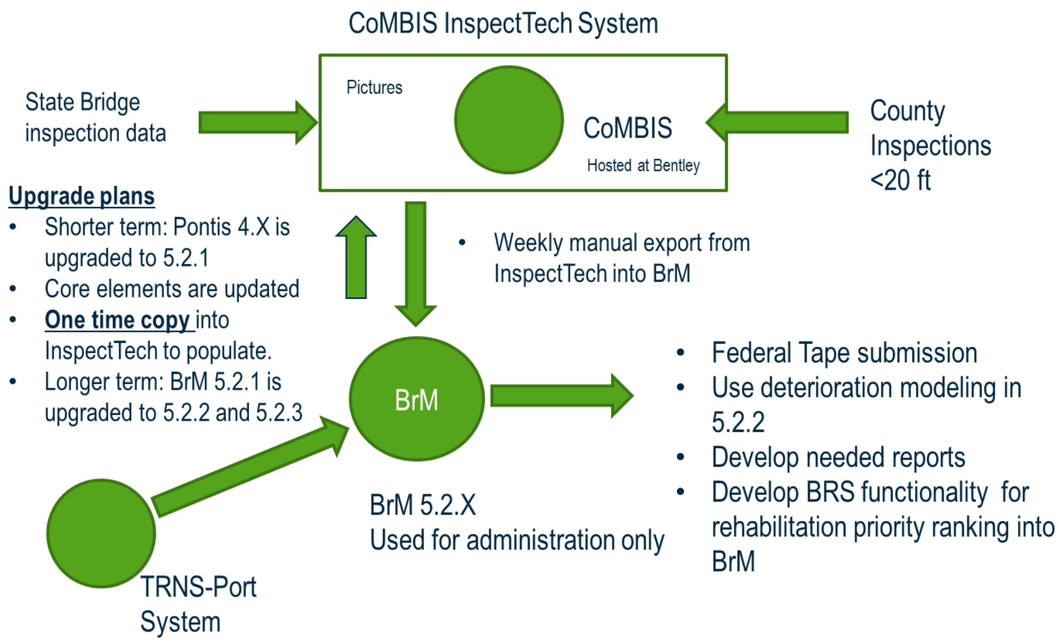
- Load Projects Developed Outside the Bridge Management Software
  - The software will support the import of project information that may have been developed outside the AASHTO Bridge Management software. Once imported, the projects would be ranked and analyzed as defined in previous use cases.
- Project Editing
  - The system also allows for the editing of projects including adding and removing bridges and work actions, combining and copying projects, and deleting projects.

5.3 BrM 5.2.3

- The functionality provided by 5.2.3 is expected to be project and capital planning.
- In BrM version 5.2.2 the Utility function, Bridge Deterioration Modeling and Project Planning set the stage to address the following features that will be addressed in BrM 5.2.3.
  - The BrM 5.2.2 bridge, project and program analysis will rely on both the deterioration modeling to provide for value over time of actions performed as well as the utility models and weights established on those factors of the most importance to the agency. Agencies with the same deterioration curves, but with different goals and thus utility functions, may yield different results on which bridge, project, and program work should be completed. With the vast amount of information and calculations conducted it will be critical to present this information in an understandable manner and also allow users to see inside how results are obtained. The detailed dashboards at the bridge, project, and program levels will convey a great deal of information in a way the user can easily understand. The dashboard will also be designed to show the effect over time and how adjusting various weights/objectives with interactive sliders may have varying effects on actions taken.

5.4 Proposed BMS concept of operations

The following is the proposed future state of the NJDOT Bridge Management System (BMS). The purpose of this diagram is to show the relationship of BrM 5.2.X to other systems. There are plans to develop a system called CoMBIS that uses the Bentley InspectTech software for both the state and county bridge inspections. All inspection data will be housed in that system. There will be a weekly manual export of State bridge inspection data to the BrM system. This data will be used for back end analysis purposes, such as deterioration modeling and project and capital planning. The federal tape will also be generated out of the BrM system.

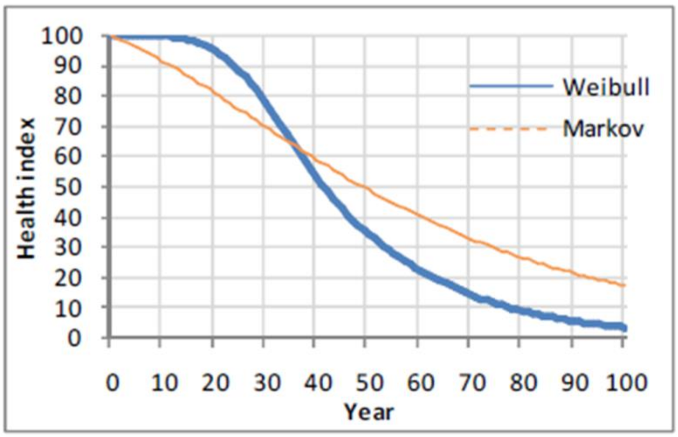


The BRP team will provide guidance and support to NJDOT in developing the BRS functionality using a phased approach. Distinct BRS modules will be developed based on the specific criteria recommended in the IIS Risk Based Prioritization process. Upon identifying relevant bridge performance limit states, the desirable criteria will be identified for implementation in the BRS modules. These modules will be employed and assessed to determine and document the effects of the applied criteria on the prioritization process.

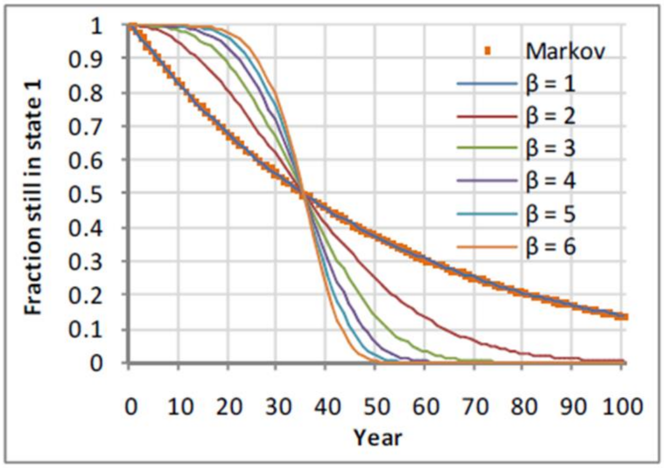
6.0 Appendix A – BrM 5.2.2 deterioration modeling basics

- **Deterioration Model Basics:**
  - BrM version 5.2.2
    - Combines Weibull with Markovian modeling.
    - It also considers the effect of other factors:
      - Protecting systems (paint, cathodic protection).
      - Environment.
      - Agency (e.g. traffic volume).
    - Weibull models the onset of deterioration.
    - The Markovian model uses:
      - Transition times: T_1 T_2 T_3
      - T_i – median # of years a unit stays in a Condition State (CS)
      - Expert elicitation is used to determine transition times. They are estimated for typical elements using a moderate environment and no protective systems.

Weibull is used to model the transition from Condition States (CS) 1 to 2. As can be seen from this diagram, Weibull provides a more conservative transition in the earlier years. This has been the experience reported by the DOTs.



- The Weibull model uses the following variables:
  - g – age
  - T_1 – CS1 transition time is provided by <u>user input</u>
  - α – scaling factor
  - β – shaping parameter is provided by <u>user input</u> as shown in the following figure. When beta is 1 this is equivalent to the Markovian results as shown below:



- Factors that affect element deterioration
  - Several factors can affect element deterioration as listed below.
  - $\boldsymbol{T_i = f \ * T_i}$
  - $\boldsymbol{f}$ – represents the effect of the following factors.
    - $\boldsymbol{f^E}$ - Environment factor
    - $\boldsymbol{f^F}$ - Formula factor – i.e. specific traffic volumes.
    - $\boldsymbol{f_e^M}$ - Protection factor of elements
  - An element may utilize several protective systems
    - 510 - Wearing Surfaces
    - 515 - Steel Protective Coating
    - 520 - Deck/Slab Protective System
  - 521 - Concrete Protective Coating
- pp_e^+ - a maximum protection parameter can be provided by user input to limit the effects of multiple systems.

– Environmental Factors - $\boldsymbol{f^E}$
- The environment factor represents the effect of the environment on the element's deterioration. Agencies typically define the following four environments: Benign (1), Low (2), Moderate (3) and Severe (4). Transition times of the elements are defined for typical environments. The environmental factors adjust the transition times of the element in the respective environment. i.e. for environments 1 and 2, the factors are going to be > 1, for environment 3 it is equal to 1 and for environment 4 it is going to be < 1.

- **Summary:**
  - BrM version 5.2.2 addresses areas of opportunity identified in the audit report. Bentley recommends the use of the new bridge deterioration functionality starting with the default parameters that represent the best practices as defined by the AASHTO task force. Once NJ DOT becomes familiar with those defaults, they can be modified to NJDOT's specific state requirements. Configuration will be required, based on expert elicitation from bridge engineers to establish parameters for NJDOTs specific situation.
  - Suggested BrM version 5.2.2 configuration methodology for Bridge Deterioration Modeling:

- The following process should be used by NJDOT to configure the deterioration models to their specific requirements. These updates are based on expert bridge engineer elicitation and the use of historical data.
    - Universal for all elements
        - Define the four (4) environmental factors. The following table illustrates typical values for each factor.

Environment Key	Environment factor
Benign	2
Low	1.5
Moderate	1
Severe	0.7

    - For Each Element Type (100+)
        - Define the Weibull Shaping Parameters
        - Define Transition times for T1, T2 and T3 for each element
        - The following table illustrates typical values:

Element key	Weibull shape param	Median years state 1	Median years state 2	Median years state 3	Element Name
12	1.3	7.21	21.00	7.43	Concrete Deck
13	1.3	11.97	24.26	13.35	Prestressed Concrete Deck
15	1.3	11.97	24.26	13.35	Prestressed Concrete Top Flange
16	1.3	11.97	24.26	13.35	Reinforced Concrete Top Flange
28	1.1	5.00	4.00	3.00	Steel Deck-Open Grid
29	1.1	5.00	3.00	3.00	Steel Deck-Concrete Filled Grid
30	1.1	7.00	6.00	5.00	Steel Deck-Corrugated/Orthotropic/Etc.
31	1.9	8.68	11.45	6.92	Timber Deck

    - For each Protective System (4)
        - Define the Weibull shaping parameters
        - Define the transition times for T1, T2 and T3 for each system
        - Define the Max protective parameter for each system
        - The following table provides typical data:

Element key	Weibull shape param	Max protect param	Median years state 1	Median years state 2	Median years state 3	Element name
510	1.0	1.41	4.00	3.00	2.00	Wearing Surfaces
515	1.8	1.52	6.00	4.00	2.00	Steel Protective Coating
520	1.0	1.39	4.00	3.00	2.00	Deck/Slab Prot Sys
521	1.0	1.16	2.00	2.00	1.00	Conc Prot Coating

    - Reduction Factor – used when multiple protective systems are being used. A single reduction factor is used for two, three, or four protective systems.

Nr. Of Prot. Systems	Reduction Factor
2	0.9
3	0.5
4	0.3

- Formula Factor for each element i.e. traffic volumes formula.
    - Note that there are no "protection formulas" in this table. They are still under review by AASHTO.

Element key	Short name	Health weight	Protection formula
12	Concrete Deck	6	
13	Prestressed Concrete Deck	NULL	
15	Prestressed Concrete Top Flange	6	
16	Reinforced Concrete Top Flange	NULL	
28	Steel Deck-Open Grid	6	
29	Steel Deck-Concrete Filled Grid	6	
30	Steel Deck-Corrugated/Orthotropic/Etc.	6	
31	Timber Deck	6	
38	Reinforced Concrete Slab	6	

In summary, BrM 5.2.2 provides functionality for bridge deterioration modeling that can be configured out of the box to NJDOT state specific requirements.

7.0 Appendix B – Audit Review of FHWA Recommendations

The Bridge Resource Program (BRP) team has reviewed the audit report and the associated relevant recommendations as listed above. The following recommendations are based on the expected beta release of BrM 5.2.2 in the September 2014 timeframe. The final production version is not expected until early Q1 2015. The intent of the guidance below is to provide NJDOT a preview of what is expected in the BrM 5.2.2 release in support of the audit recommendations. The guidance provided in this document represents what is currently known about the expected software functionality. This initial guidance also reflects only those areas that can be impacted by the software.

Areas of opportunity for the existing BMS based on the FHWA audit report:

- **Audit recommendation #1**.The accuracy of Pontis deterioration algorithms for NJ. Pontis allows different formulas, but this has not been used by New Jersey.
    - **Guidance**: The accuracy of the Pontis deterioration algorithms in BrM version 5.2.2 as planned by the AASHTO task force recognized the need for the states to have the ability to use different formulas and user defined parameters to address state specific conditions. It was also recognized that the current Pontis deterioration models did not reflect actual deterioration in the field for changes from condition states 1 to 3. To address this issue, the task force recommended the use of the Weibull model as part of the analysis. The BrM 5.2.2. software will use a combination of both Weibull and Markovian modeling to improve its accuracy by adding a time factor component. In the older version of Pontis, the Markovian model was used exclusively. In addition, the new element structure with parent/child relationships was added to allow the addition of protective systems to the model. NJDOT can use the new bridge deterioration functionality starting with the default parameters that represent the best practices as defined by the task force. Once NJDOT becomes familiar with those defaults, they can be modified to NJDOT's specific state requirements. Configuration will be required, based on efforts by the BRP team and expert elicitation from bridge engineers to establish parameters for NJDOTs specific situation.
- Bridge deterioration modeling at the superstructure, substructure and deck components, not the element level.
    - **Guidance:** Bridge deterioration models in 5.2.2 require the use of element level inspection. NJDOT needs to capture element level data to enable the use of the functionality. This requires the conversion of historical core data to NBE equivalents in order to preserve historical information required for bridge deterioration modeling. Detailed guidance will be provided in the 5.2.2 guidance document. Elements are mapped to a specific part of the bridge such as the superstructure, substructure, or deck. This will allow overall health indices to be created to monitor deck, superstructure, and substructure health.
- **Audit recommendation #8** Network optimization modeling. Does not optimize based on a given performance level. The curves must be generated manually using a trial and error process.

- **Guidance:** The team recommends the use of network optimization modeling in the future, since BrM version 5.2.2 will not have this functionality. Network optimization functionality will be provided as part of BrM version 5.2.3 available in second half of 2015. As part of 5.2.1, the utility function became available. It provides the ability to measure improvements on a structure based on specific actions. There are task force provided "best practice" utility curves that can be reviewed by NJDOT in the short term for use on a specific bridge to determine the benefits of a specific action to that bridge. This function does not factor in deterioration modeling today, but will in the future.

- **Audit recommendation #3**. Currently not capturing information about projects that are proposed or underway. The database only gets updated when a construction project has been completed and the associated bridge condition changed. Pontis usage for programmed/contracted projects.
  - **Guidance:** In version 5.2.1 various statuses have been added. A new field called "bridge status" includes unknown, inactive, closed, proposed and obsolete states. A "lifecycle field" was also added that includes unknown, service, design or pre-construction states. The specific use of this feature needs to be further discussed with NJDOT. Version 5.2.2 will significantly expand the Project capabilities as well.
- **Audit recommendation #8** Non-condition based rating factors found in the BRS.
  - **Guidance:** Non-condition based factors, such as average daily traffic are included in BrM. Those items listed on page 8 of the audit report are NBI items and part of the Pontis system.
- BRS is based solely on the current conditions of bridges and not how they will deteriorate over a given capital program period.
  - **Guidance:** Once bridge deterioration modeling is added to 5.2.2 deterioration of bridges over a given capital program period can be determined.
- Project programming based on lowest life cycle cost only.
  - **Guidance:** Project programming is planned for BrM version 5.2.3. It is recommended that NJDOT provide their requirements in more detail for use by the AASHTO task force when planning this release of software.
- **Audit recommendation #7** Cost/benefit modeling for preservation vs rehabilitation vs. replacement decisions.
  - **Guidance:** In BrM verson 5.2.1 actions can be defined for preservation, rehabilitation and replacement. Thus you can compare the cost/benefit ratio for each or any combination of scenarios. This analysis can be modeled today but not over a series of years.
- **Audit recommendation #8** Default logic used to identify improvements. Need a way to override Pontis default logic. The ability to use agency policy goals to make treatment recommendations. (i.e. such as reduce the overall level of non-structurally deficient bridges by 50%)
  - **Guidance:** In older versions of Pontis, there were limitations on agency customizations. In BrM version 5.2.1 and beyond, actions are completely editable and definable. While Pontis comes with pre-defined logic, all can be customized to fit the agency's needs. The example used was that 3 or 4 deck joints on a bridge were being

recommended by Pontis for repair. Joint rehabilitation, repair and replacement now have defaults that can be completely configured to the agency's needs. The multi-objective model also allows weights to be set based on an agency's relative priorities.

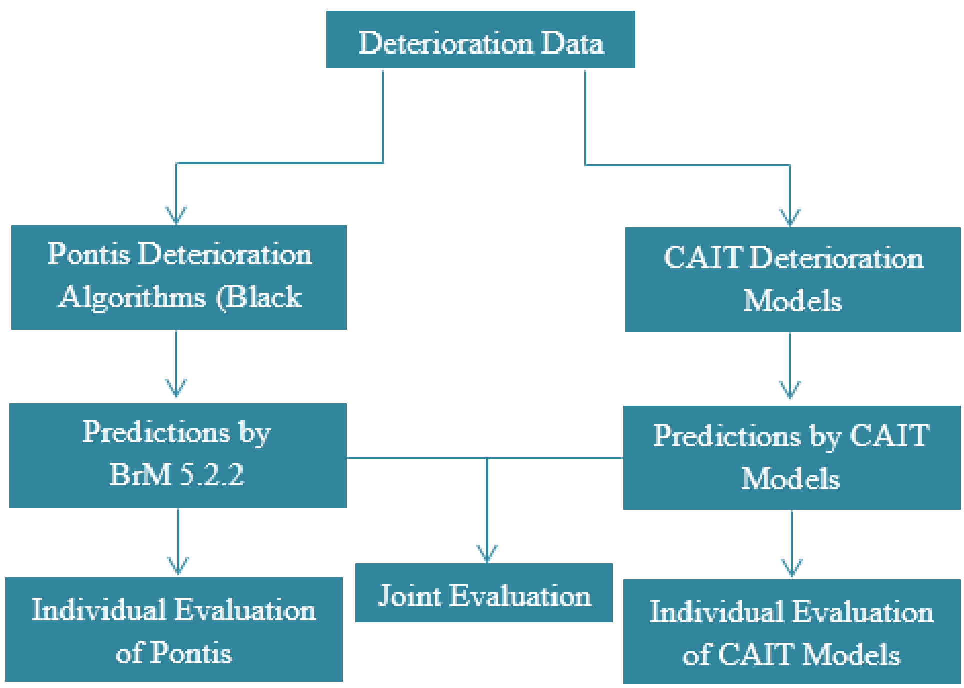
Areas of opportunity for the existing BRS based on **MAP-21**

- National Bridge Inventory (NBI) condition rating data as well as element level condition rating data.
    - Starting with BrM 5.2.1 the software has fully implemented the 2013 NBE elements, which are in full compliance with Map-21 requirements.
- MAP 21 requirement for a major asset (bridge) preservation component.
    - BrM 5.2.1 allows the user to apply preservation (protective systems) on NBE elements. In the future, BrM 5.2.2 will combine deterioration modeling and project planning to provide full and robust preservation capability which will be further expanded in BrM 5.2.3.
- Bridge related performance goals, lifecycle costs, and investment strategies.
    - Performance goals, lifecycle costs and investment strategies all require the analysis over the lifecycle of the bridge managed by the agency. This analysis similar to bridge preservation will not be available until version 5.2.3.
- Track actual performance against the goals set in the Asset Management Plan (AMP)
    - The team recommends that the goals from the NJDOT AMP be provided for further guidance.

8.0 Appendix C - Longer Term approach provided by the Bridge Resource Program (BRP)

The following is a suggested longer term comprehensive approach that will be provided by the Bridge Resource Program (BRP) team in support of addressing and implementing the audit recommendations in the future:

1. **NJDOT, in consultation with Rutgers consulting staff assigned to the Bridge Resources Program, should assess the accuracy of Pontis deterioration algorithms and modify such in Pontis as necessary.**
    - BRP team support: The Bridge Resource Program (BRP) team suggests joint meetings to discuss current NJDOT procedures to document deterioration and vision for the use of state specific deterioration modeling. Through this discussion, the team can help facilitate more suitable customization of BrM for NJDOT.
    - Furthermore, the team proposes the following supplemental procedure for the evaluation of the deterioration algorithms in the new BrM 5.2.2 module. For the protection of intellectual property, CAIT will evaluate the AASHTO deterioration algorithm in a black box environment. CAIT will in parallel develop a set of deterioration models for comparison purposes as shown in the following figure. A series of statistical tests will be performed for the evaluation process.



Based on this review, the team will develop state specific deterioration models that will be presented to NJDOT for their use.

2. **NJDOT should identify the investment level and performance associated with funded bridge rehabilitation projects in the capital program documents (TIP, STIP, MTP, SLRTP) as they compare with CIS target investment and performance levels. For example, if the CIS recommends a target investment level averaging $720 million annually to address to address a certain square footage of structurally deficient bridges, then the STIP should show the dollar value of actual bridge rehabilitation projects in the STIP and how those specific projects are anticipated to impact the targeted performance level associated with structurally deficient bridges**.
    - BRP team support: The team will collaborate with NJDOT to develop a qualitative strategy for establishing targets and classifying bridges based on factors beyond the BRS rankings, consistent with risk based target setting approach (MAP-21). In some cases the bridge condition data from the BMS is sufficient to adequately describe the risk associated with loss of functionality. For other bridges, economic and social outcomes of bridge deterioration or uncertainties in load demand versus overall structural capacity may be more detrimental factors. The team will meet and work with NJDOT to develop qualitative means of evaluating these factors, including location, importance, type of a bridge, and susceptibility of a bridge to loss of functionality due to natural or man-made hazards. The strategy will assign individual impact factors for influences such as location, importance, network benefit, community need, type of bridge, and damage scenarios. Using the individual impact factors, we would determine an overall bridge risk factor, and assign a bridge a qualitative risk measure such as low, medium, or high. Proposed rehabilitation activities, their effect on predicted risk, and tradeoffs related to maintaining or reducing risk level for a particular bridge would be specified. Data from previous Problem Statement submissions would be used to validate this impact factor approach for a wide range of bridges.

      The team will review published literature related to qualitative assessment methods, interview NJDOT representatives to capture engineering expertise in bridge evaluation, and use other relevant sources to develop this strategy. This effort is consistent with the recommendations, findings and guidance of BRP report 435056-6 titled, NJDOT Risk Based Prioritization.

3. **NJDOT should begin entering data into Pontis regarding the programming/contracting of projects as they occur. Pontis performance curves used in the CIS cannot be accurate if the system does not know what improvements have already been funded. Bridge funding performance curves are required by Capital Program Development (CPD) to develop NJDOT's Capital Investment Strategy (CIS). Structural Evaluation and Bridge Management (SEBM) staff uses Pontis to develop the performance curves, however current version of Pontis does not optimize based on a given performance level, therefore the SEBM staff manually derives these curves, using a trial and error method.**

- BRP team support: Concurrent with developing deterioration algorithms, the team will provide guidance on integrating inspection/monitoring data with prediction of time-dependent damage propagation into a performance-based framework to improve the decision-making process. With the aim to optimize bridge management activities, probabilistic models of the condition of structures could be used to explicitly represent structural assessments, deterioration processes, and needed maintenance interventions. Analytical modeling of structural behavior and of the processes that cause deterioration and affect safety and serviceability of the structure would enable us to probabilistically *quantify* structural safety of the bridge structure (in addition to the qualitative verbal descriptors available in element condition ratings). This analysis would inform the decisions for intervention based on minimizing the life-cycle cost, or maximizing the serviceable life of the structure, as long as the economic benefit of the chosen rehabilitation is justified.
- BRP team support: Performance-based asset management and preservation, a stated objective in NJDOT's BMS, corresponds to the requirements of MAP-21. Tracking of actual bridge performance against the performance goals, and analysis of the related costs and investment strategies over the life-cycle of the bridge, can inform performance curves generated with Pontis, BrM, or another model.

4. **NJDOT should establish and document formal procedures which govern the essential BMS tasks listed under 23 CFR.107**.

5. **Prepare a policy and procedure which documents the process by which bridge rehabilitation projects will be prioritized using the BRS and other factors. Projects are not initiated by selecting the next ranked bridge so a fuller description is needed to explain how decisions are made and who makes them.**
    - BRP team support: Based on condition ratings data in BMS (i.e., Pontis, BRS), and the relative risk associated with relevant bridge performance limit states, the BRP team will merge technology applications with NJDOT heuristics to formalize the process of assigning priority levels to prospective rehabilitation projects. NJDOT has identified performance measures (e.g., percentage of non-structurally deficient bridges) to inform decisions about multi-year funding needs for rehabilitation projects. The team will collaborate with NJDOT using the Risk Based Prioritization approach described in Report 435056-6 to develop a ranking tool that may be used to assist in the decision-making.

6. **Develop a decision log which provides a transparent record of what factors contributed to the prioritization of a project besides the BRS ranking.**
    - BRP team support: Transparent record of factors contributing to the prioritization of projects will require a move from subjective, visual measures for cataloguing bridge condition information, to an empirical assessment of the structural condition combined with capturing essential engineering heuristics. Working

meetings with NJDOT will allow the BRP team to formulate a database of case-studies and specific parameters essential in the decision-making process, which would be incorporated into the prioritization model. By using the framework developed in the BRP task on Risk Based Prioritization, an algorithm can be developed to automate the data entry and lookup process. While some data, such as flood mapping, weather/climate data, and other nonstructural factors may require manual entry, the process map flow would be outlined via the framework identified, and the focus can be shifted towards state-specific decision-making.

7. **NJDOT should either use Pontis cost/benefit modeling to guide decisions regarding preservation, rehabilitation, or replacement decisions, or alternately develop written policies and procedures based on objective criteria for making such decisions.**
    - o <u>BRP team support</u>: Optimal life-cycle planning of a structure based on cost-effectiveness ascertains that the structure is in a state at which the expected benefits of structural operation and rehabilitation are greater than the expected costs. The expected costs may include the initial cost of the structure, maintenance cost, and expected future losses resulting from structural deterioration or non-condition factors. Benefits can be quantified in terms of structural performance, operation costs, or regional economic impact. A probabilistic cost/benefit analysis may consider uncertainties in structural demands (e.g., external loads, environmental impacts), uncertainties in structural capacity, and also the correlation with other bridges in the network (e.g., closure of a bridge in the network resulting in increased traffic on other nearby bridges). BRP would examine these and other capabilities within the Pontis and BrM models, and provide guidance in developing procedures based on the NJDOT and FHWA criteria for cost-based decision making.

8. **In conjunction with the Rutgers University's Bridge Resource Program, researchers should be asked to perform the following analysis:**
    - o Determine whether the current Pontis network optimization logic should be customized to recommend an optimal selection of bridge rehabilitation projects to meet New Jersey performance goals or whether the Department should wait until the next Pontis upgrade includes such functionality.
        - ▪ See discussion point 1.

    - o Evaluate the feasibility of integrating non-condition based rating factors found in the BRS into the Pontis optimization modeling process.
        - ▪ See discussion points 2, 5 and 6.

    - o Evaluate the feasibility of using the Agency Policy Goals to make treatment recommendations that are consistent with standard NJDOT contracting practices.
        - ▪ <u>BRP team support</u>: The team anticipates that improvements resulting from developing state-specific deterioration models and incorporating a risk based prioritization approach will lead to significant improvements

in identifying needed actions. Following these accomplishments, the team can review the work performed by Illinois DOT and determine if a state-specific Agency Policy Goals module can be developed, which provides NJDOT with a state-specific solution for recommending actions.

APPENDIX 1B
COST DATA MIGRATION

Introduction

The New Jersey Department of Transportation (NJDOT) utilizes the Pontis software (version 4.4), which is a comprehensive Bridge Management System (BMS) which prioritizes bridge replacement, rehabilitation, deck and superstructure replacement, painting, scour retrofit and other maintenance repairs. Pontis has the technical capability to assist in identifying funding levels required to improve the condition of state maintained bridges.

NJDOT has identified a need to prioritize, maximize available bridge funding, and provide decision makers with key construction cost data associated with bridge repair and replacement. Currently, NJDOT stores project and asset-related costs in their Trns-port^{tm} system. To provide this cost information to NJDOT's BMS for analysis, related cost information from Trns-port^{tm} will be translated and arranged into a format that can be imported into Pontis.

This technical memorandum was developed as part of Task 1B, Activity 1 (Pontis Data Requirements Gathering) of the project and outlines the data requirements to store cost information within the Pontis system.

The Pontis database provided from NJDOT which will be used for the subsequent analysis and data transformation is from January of 2010. It should be noted that Pontis has been rebranded as *AASHTOWare Bridge Management* (BrM) and NJDOT plans to migrate to BrM (version 5.2.1) in the near future. While this is the case, guidance has been received that data and database structure changes that will occur are minimal and will not impact the overall analysis and data migration tasks.

Pontis Overview

Pontis is a comprehensive BMS that allows agencies to manage resource allocations related to existing infrastructure investments. Pontis supports the entire bridge management life cycle, including inventory, inspection, needs assessment and strategy development, and project and program development. These areas of management are organized into seven modules within Pontis:

- **Inspection Module** – Used to maintain inventory and inspection information about structures.
- **Project Planning Module** – Used to assist with project development and tracking.
- **Programming Module** – Used to manage structure improvement policies and standards.
- **Preservation Module** – Used to develop and run models to support in the development of long-term preservation policies.
- **Results Module** – Used to view reports on predicted network costs and performance associated with different scenarios.
- **Gateway Module** – Used to import and export data between Pontis and other systems.
- **Configuration Module** – Used to customize Pontis settings.

Historically, NJDOT has primarily used Pontis to maintain an asset inventory of existing structures and track information related to bridge inspections (Inspection Module). While Pontis functionality allows for the tracking of specific cost information relating to bridge repair/replacement projects, NJDOT currently manages this information within the Trns-port^{tm} system. It is the goal of this project to translate cost information stored within Trns-port^{tm} into a format that can be utilized within Pontis.

Data Review

All information pertaining to construction and maintenance project related costs within Pontis are managed in the **Project Planning Module**. When projects are completed, the actual work completed and associated costs are recorded using this module to maintain a history of work for each structure. Since this is the case, the database tables associated with this module will be the target of the incoming historical Trns-port^{tm} data. A thorough review of Pontis revealed that the following data points are required at minimum to store cost information within the system:

- **Structure ID** – A unique reference to an individual structure. All costs must associate with a structure or an element of a structure.
- **Project ID** – A unique reference to a project in which a structure was repaired or replaced. Each project defined in Pontis must be assigned to a program.
- **Program ID** - A unique reference to a program, which are used to organize projects. All projects must be associated to a program within Pontis.
- **Funding Source** - Identifies the primary funding source associated with a cost.
- **Action** - The type of work performed (e.g. replaced deck or repaired joint).
- **Cost** –Cost associated with the project action. Costs (referred to as *Work Items* in Pontis) can be associated with an entire structure or particular elements of a structure. Work Items must be associated with a project.

While these data points represent the minimum required information to adequately associate and store cost data within Pontis, it is recognized that NJDOT needs may warrant the inclusion of additional data fields. This will be explored further during Task 1B, Activity 2 (Trns-port^{tm} system evaluation) and Task 1B, Activity 3 (NJDOT PRS evaluation) of the project.

The required data points are maintained in the following tables within Pontis (full field descriptions can be found in Appendix A):

1. **BRIDGE (Bridges)**

   Contains NBI and other information related to structures, including physical, administrative and operational characteristics. This table contains one record for each structure, and serves as the primary source for NBI data reporting. While it is assumed that no new bridges will be added during the migration of Trns-port^{tm} data, this table maintains a reference to all individual structures. All imported information must relate to an existing record in this table.

2. **PROJECTS (Projects)**

   Contains data on past, current, and proposed future projects. Additional details on the work items for each project are contained in the PRJ_WITEMS table. It is assumed that only information pertaining to past and current projects will be migrated during the import process.

3. **PRJ_PROGRAMS (Programs)**

   Contains details on programs, used to organize projects. Each project defined in Pontis must be assigned to a program.

4. **PRJ_WITEMS (Work Items)**

Contains detail (including cost information) on each work item of each project. A work item consists of a specific action on a specific structure or element of a structure.

5. **PRJ_FUNDSRC (Funding Sources)**

   Contains definitions of funding sources used in tracking costs for programs, projects, and work items. While it is assumed that no new funding sources will be added during the migration of Trns-port[tm] data, all work items must have an associated funding source.

6. **ACTYPDFS (Action Type Definitions)**

   This is a definition table that contains all the action types that can occur for structures and elements. No data will be added to this table. The full ACTYPDFS table can be found in Appendix B.

Data Summary

The table below provides the specific breakdown of where data points are stored within the Pontis system and the number of records in each related table.

Pontis Table	Data Point Stored	Record Count	Comment
BRIDGES	Structures	2521	2521 Bridge Structures identified
PROJECTS	Projects	796	796 Projects identified
PRJ_PROGRAMS	Programs	5	
PRJ_WITEMS	Work Items	2123	
	Action	-	See ACTYPDFS below.
	Costs	-	While costs are stored in this table, indirect costs are stored at the project level (PROJECTS table).
PRJ_FUNDSCR	Funding Sources	2	Currently only ‘State’ and ‘Federal’ funding sources exist.
ACTYPDFS	Action (definitions)	21	Actions can occur at the structure level or element level. The full ACTYPDFS table can be found in Appendix B.

Note: Items in **Bold** are required fields.

1. BRIDGE

Field	Description
brkey	Bridge key. Primary structure identifier in Pontis.
bridge_id	Agency bridge identification number. Used to identify structures on most screens and reports.
struct_num	NBI structure number (NBI Item 8).
strucname	Agency structure name. Non-NBI field.
featint	Feature intersected by the structure. NBI Item 6A.
fhwa_regn	FHWA Region. Third digit of NBI Item 1.
district	Agency district in which the structure lies. NBI Item 2.
county	County or parish. NBI Item 3.
facility	Facility carried by the structure. NBI Item 7.
location	Location of structure. NBI Item 9.
custodian	Maintenance responsibility. NBI Item 21.
owner	Owner of the structure. NBI Item 22.
adminarea	Administrative area or geographic stratification for the structure. Non-NBI field.
bridgegroup	Agency-defined group for the bridge. Intended for use in grouping together bridges for inspection purposes. Non-NBI field.
nstatecode	Neighbor state code for structures crossing a state border. First two digits of NBI Item 98.
n_fhwa_reg	Neighbor FHWA Region for structures crossing a state border. Third digit of NBI Item 98.
bb_pct	Percent of the deck area of the structure for which the neighbor state is responsible for funding, for structures crossing a state border. Last two digits of NBI Item 98.
bb_brdgeid	Neighbor structure number for structures crossing a state border. Should be coded with the structure number for the structure used by the neighbor state. NBI Item 99.
propwork	Proposed work description. NBI Item 75A.
workby	Description of whether proposed work is to be performed by a contractor or agency work forces. NBI Item 75B.
nbiimpcost	Cost of structure construction portion of proposed work. NBI Item 94.
nbirwcost	Cost of roadway improvement portion of proposed work. NBI Item 95.
nbitotcost	Total cost of proposed work. NBI Item 96.
nbiyrcost	Year of improvement cost estimate. NBI Item 97.
yearbuilt	Year structure was built. NBI Item 27.
yearrecon	Year structure was last reconstructed. NBI Item 106.
histsign	Historical significance indicator. NBI Item 37.
designload	Live load for which the structure was designed. NBI Item 31.
servtypon	Type of service on bridge. NBI Item 42A.
servtypund	Type of service under bridge. NBI Item 42B.
sumlanes	Sum of the number of lanes under the structure. NBI Item 28B when the inventory route is on the structure.
mainspans	Number of spans in main unit. NBI Item 45.
appspans	Number of approach spans. NBI Item 46.

Field	Description
maxspan	Length of maximum span. NBI Item 48.
length	Structure length. NBI Item 49.
deck_area	Structure deck area. Non-NBI field used as the basis for area-based replacement and improvement cost estimates.
bridgemed	Structure median indicator. NBI Item 33.
skew	Structure skew angle. NBI Item 34.
materialmain	Kind of material and/or design for the main span. NBI Item 43A.
designmain	Type of design and/or construction for the main span. NBI Item 43B.
materialappr	Kind of material and/or design for the approach span. NBI Item 44A.
designappr	Type of design and/or construction for the approach span. NBI Item 44B.
dkstructyp	Deck structure type. NBI Item 107.
dkmembtype	Deck membrane type. NBI Item 108B.
dksurftype	Deck wearing surface type. NBI Item 108A.
dkprotect	Deck Protection. NBI Item 108C.
deckwidth	Deck width, out-to-out. NBI Item 52.
lftcurbsw	Left curb or sidewalk width. NBI Item 50A.
rtcurbsw	Right curb or sidewalk width. NBI Item 50B.
strflared	Structure flared indicator. NBI Item 35.
refvuc	Reference feature for minimum vertical underclearance measurement, NBI Item 54A.
refhuc	Reference feature for minimum lateral underclearance measurement. NBI Item 55A
hclrurt	Minimum lateral underclearance on right side of structure. NBI Item 55B.
hclrult	Minimum lateral underclearance on left side of structure. NBI Item 56.
lftbrnavcl	Minimum navigation vertical clearance, vertical lift bridge. NBI Item 116.
navcntrol	Navigational control. NBI Item 38.
navhc	Navigation horizontal clearance. NBI Item 40.
navvc	Navigation vertical clearance. NBI Item 39.
paralstruc	Parallel structure designation. NBI Item 101.
tempstruc	Temporary structure designation. NBI Item 103.
nbislen	Specifies whether the structure meets the National Bridge Inventory length criterion of 6 meters. NBI Item 112.
latitude	Latitude. NBI Item 16.
longitude	Longitude. NBI Item 17.
vclrover	Minimum vertical clearance over structure roadway. NBI Item 53.
vclrunder	Minimum vertical underclearance. NBI Item 54B.
placecode	Place code. NBI Item 4.
implen	Length of proposed structure improvement. NBI Item 76.
fips_state	FIPS state code. First two digits of NBI Item 1.
tot_length	Total length of structure, including approach roadways. Always greater than or equal to structure length.
nextinspid	User key for the planned next inspector.
crewhrs	Number of crew hours required for a regular inspection for the structure.

Field	Description
flaggerhrs	Number of flagger hours required for a regular inspection for the structure.
helperhrs	Number of helper hours required for a regular inspection for the structure.
snooperhrs	Number of snooper hours required for a regular inspection for the structure.
spcrewhrs	Number of special crew hours required for a regular inspection for the structure.
spequiphrs	Number of special equipment hours required for a regular inspection for the structure.
on_off_sys	Specifies whether the structure is on or off the agency system. Typically based on the value for either structure ownership (NBI Item 22) or custodian (NBI Item 21). Used for determining applicable policies and costs, and for reporting results.
ratingdate	Date load rating calculation was made. Non-NBI field.
rater_ini	Initials of load rater/engineer responsible for performing the load rating. Non-NBI field.
orload	Operating rating load. NBI Item 64.
ortype	Method used to determine operating rating. NBI Item 63.
irload	Inventory rating load. NBI Item 66.
irtype	Method used to determine inventory rating. NBI Item 65.
posting	Bridge posting status. NBI Item 70.
req_op_rat	Load rating review. Indicates whether review of the load ratings is recommended.
def_op_rat	Indicates whether the functional improvement policy will be applied to the bridge during program simulation.
fc_detail	Fracture critical detail on structure. Non-NBI field.
altorload	Alternate operating load rating. Optional non-NBI field to hold a load rating by some alternative method aside from the one used in the NBI load rating fields.
altormeth	Alternate operating rating method. Optional non-NBI field to indicate the method used in developing the alternate operating load rating fields.
altirload	Alternate inventory load rating. Optional non-NBI field to hold a load rating by some alternative method aside from the one used in the NBI load rating fields.
altirmeth	Alternate inventory rating method. Optional non-NBI field to indicate the method used in developing the alternate operating load rating fields.
otherload	Other load rating. Optional non-NBI field to provide for a separate type of load rating other than inventory or operating load ratings.
truck1or	Operating rating for truck type 1. Optional non-NBI field to provide for a load rating specific to a particular type of truck.
truck2or	Operating rating for truck type 2. Optional non-NBI field to provide for a load rating specific to a particular type of truck.
truck3or	Operating rating for truck type 3. Optional non-NBI field to provide for a load rating specific to a particular type of truck.
truck1ir	Inventory rating for truck type 1. Optional non-NBI field to provide for a load rating specific to a particular type of truck.

Field	Description
truck2ir	Inventory rating for truck type 2. Optional non-NBI field to provide for a load rating specific to a particular type of truck.
truck3ir	Inventory rating for truck type 3. Optional non-NBI field to provide for a load rating specific to a particular type of truck.
srstatus	Tracks whether sufficiency rating (SR) needs to be recalculated. This field is set to 1 when a new roadway or inspection is created (and in response to other circumstances that may trigger a need to recalculate SR), and set to 0 when SR is recalculated.
userkey1	Agency-defined field 1.
userkey2	Agency-defined field 2.
userkey3	Agency-defined field 3.
userkey4	Agency-defined field 4.
userkey5	Agency-defined field 5.
userkey6	Agency-defined field 6.
userkey7	Agency-defined field 7.
userkey8	Agency-defined field 8.
userkey9	Agency-defined field 9.
userkey10	Agency-defined field 10.
userkey11	Agency-defined field 11.
userkey12	Agency-defined field 12.
userkey13	Agency-defined field 13.
userkey14	Agency-defined field 14.
userkey15	Agency-defined field 15.
btrigger	Flag for triggering formula calculation. When set to 1, all applicable fields for the structure are updated during formula recalculation. Otherwise, they are recalculated only if the result field contains a missing value code other than Not Applicable.
traceflag	Trace flag. Indicates whether the structure is traced in the log file during program simulation.
createdatetime	Date and time the record was created.
createuserkey	Key value for the user that created the record.
modtime	Date and time the record was last modified.
userkey	Key value for the user that last modified the record.
docrefkey	Reference key for multi-media documents related to the structure. Reserved for future use.
notes	Structure notes.

2. PROJECTS

Field	Description
projkey	Project key. Primary identifier for projects.
progkey	Program key. Foreign key to the prj_programs table.
project_id	Agency project identifier
projname	Project name.
district	Primary agency district for the project.
proj_acttype	Primary action type for the project.
progyear	Year for which the project is programmed.

projenddate	Project end date.
proj_status	Project status
proj_review_status	Project review status.
proj_reviewed_by	Specifies the userkey of the user that review the project.
indirectben	Indirect project benefit.
indirectcost	Indirect project cost.
scen_treat	Scenario treatment. Specifies how the project should be modeled during program simulation.
routenum	Route number for the project. Intended for agency use in identifying the project.
beginkmpost	Start kilometer point for the project. Intended for agency use in identifying the project.
endkmpost	End kilometer point for the project. Intended for agency use in identifying the project.
avghindex	Health index for the most recent inspection for the structures included in the project.
avgsuffrate	Average sufficiency rating for the most recent inspection for the structures included in the project (Averaged based on deck area).
agencyrank	Agency project rank. Intended for use by agencies that wish to rank projects using an agency-defined ranking formula.
programrank	Program rank. Rank of the project in the program for a specified year or overall years, in the order determined using the ranking screen
contractor	Project contractor. Intended for use in project tracking.
contract_id	Project contractor identifier. Intended for use in project tracking.
estcost	Estimated project cost. Intended for use in project tracking.
contractcost	Contract cost. Intended for use in project tracking.
finalcost	Final project cost. Intended for use in project tracking.
agcyprojkey1	Agency-defined field 1.
agcyprojkey2	Agency-defined field 2.
agcyprojkey3	Agency-defined field 3.
agcyprojkey4	Agency-defined field 4.
agcyprojkey5	Agency-defined field 5.
agcyprojkey6	Agency-defined field 6.
agcyprojkey7	Agency-defined field 7.
agcyprojkey8	Agency-defined field 8.
agcyprojkey9	Agency-defined field 9.
agcyprojkey10	Agency-defined field 10.
createdatetime	Date and time the record was created.
createuserkey	Key value for the user that created the record.
modtime	Date and time the record was last modified.
userkey	Key value for the user that last modified the record.
docrefkey	Reference key for multi-media documents related to the project. Reserved for future use.
notes	Project notes.

3. PRJ_PROGRAMS

Field	Description
progkey	Program key. Primary identifier for programs.
prog_id	Agency program identifier.
progname	Program name.
progobjective	Program objective.

Field	Description
progtype	Program type.
progstatus	Program status.
progstartyr	Program start year.
progendyr	Program end year.
createdatetime	Date and time the record was created.
createuserkey	Key value for the user that created the record.
modtime	Date and time the record was last modified.
userkey	Key value for the user that last modified the record.
docrefkey	Reference key for multi-media documents related to the program. Reserved for future use.
notes	Program notes.

4. PRJ_WITEMS

Field	Description
witemkey	Key of the work item to which the candidate is assigned. Foreign key to the prj_witems table.
witem_id	Agency work item identifier.
projkey	Project key. Foreign key to the projects table.
brkey	Bridge key. Primary structure identifier in Pontis. Foreign key to the bridge and structure_unit tables.
strunitkey	Structure unit key. Specifies the structure unit to which the work item applies, where applicable. Foreign key to the structure_unit table.
objkind	Kind of object. Valid values are 0 (bridge), 1 (element), 2 (element category) and 3 (element type).
objcode	Code for the applicable object. Must be interpreted based on the objkind field. For example, if the objkind value is 1, then the code is an element key.
actkind	Kind of action. Valid values are 1 (action type), 2 (action category) and 3 (flexible action).
actcode	Code for the applicable action. Must be interpreted based on the actkind field. For example, if the actkind value is 1, then the code is an action type.
ykey	Year.
fskey	Funding source key. Identifies the primary funding source for the work item. Foreign key to the prj_fundsrc table.
flag_whole	Specifies whether an item applies to a part of the structure or the entire structure. A value of 0 indicates that it applies to part of the structure. A value of 1 indicates it applies to the entire structure.
agency_status	Agency status.
agency_priority	Agency priority.
workrecdate	Date work item was completed.
workassignment	Work assignment. Specifies how the work is assigned such as to agency work forces or to a contractor.
witemsource	Work item key. Primary identifier for work items in Pontis.
cost	Work item cost.
benefit	Work item benefit.
lockcost	Specifies whether the cost of the work item is locked. If the cost is locked, then the Pontis models will use the specified cost for modeling. Otherwise, Pontis will recalculate the cost.
lockben	Specifies whether the benefit of the work item is locked. If the benefit is locked, then the Pontis models will use the specified benefit for modeling. Otherwise, Pontis will recalculate the benefit.

Field	Description
quantity	Quantity to which the work candidate applies. For element work candidates the units are the units applicable to the element. For improvement and replacement work candidates, the units are square meters of deck area.
state1	Specifies whether the work item applies to the portion of the element in state 1, for element work items.
state2	Specifies whether the work item applies to the portion of the element in state 2, for element work items.
state3	Specifies whether the work item applies to the portion of the element in state 3, for element work items.
state4	Specifies whether the work item applies to the portion of the element in state 4, for element work items.
state5	Specifies whether the work item applies to the portion of the element in state 5, for element work items.
flexcode	INSERTED BY DATADICT HOUSEKEEPING ROUTINE (ver. 2000) 10/19/2000 19:18:04
createdatetime	Date and time the record was created.
createuserkey	Key value for the user that created the record.
modtime	Date and time the record was last modified.
userkey	Key value for the user that last modified the record.
docrefkey	Reference key for multi-media documents related to the work item. Reserved for future use.
notes	Work item notes.

5. PRJ_FUNDSRC

Field	Description
fskey	Funding source key. Primary identifier for funding sources.
fs_name	Funding source name.
fs_type	Funding source type.
fs_desc	Funding source description.
createdatetime	Date and time the record was created.
createuserkey	Key value for the user that created the record.
modtime	Date and time the record was last modified.
userkey	Key value for the user that last modified the record.
notes	Funding source notes.

6. ACTYPDFS

Field	Description
atypcat	Action category. Refers to action categories listed in the paramtrs table.
atypeelig	Federal eligibility flag. Indicates whether the specified action type is eligible for federal funding.
atypelong	Action long name.
atypenum	Action type number. Can be recoded by the user.
atypeshort	Action short name.
paircode	Metric/English unit pair for the specified action type. Foreign key to the metric_english table.

ACTYPDFS Table

TKEY	ATYPENUM	ATYPESHORT	ATYPELONG	ATYPCAT	ATYPEELIG	PAIRCODE
-1	-1	Missing	Missing Value	0	0	-1
00	0	Do Nothing	Do Nothing	0	0	-1
11	11	Replace	Replace Structure	1	1	20
12	12	Repl Super	Repl Superstructure	1	1	20
13	13	Remove	Remove Structure	6	_	-1
21	21	Widen	Widen Structure	2	1	20
22	22	Raise	Raise Structure	2	1	20
23	23	Strengthen	Strengthen Structure	2	1	20
24	24	Scour	Scour Remediation	2	1	20
25	25	Seismic	Seismic Retrofit	2	1	20
26	26	Fatigue	Fatigue Remediation	2	0	6
31	31	Repl Elem	Replace Element	3	1	-1
32	32	Ovly Deck	Overlay Deck/Slab	3	1	-1
33	33	Rehab Elem	Element Rehabilitation	3	0	-1
34	34	Repl Paint	Replace Paint System	7	1	-1
35	35	Repl Joint	Replace Joint	3	0	-1
40	40	Pr Maint	Routine/Preventative	4	0	-1
41	41	Min Repair	Element Repair	4	0	-1
43	43	Part Paint	Zone/ Partial paint	7	0	-1
50	50	Crib	Temporary Cribbing	5	0	-1
60	60	Other	Uncategorized Action	6	0	-1

Introduction

The New Jersey Department of Transportation (NJDOT) has a comprehensive Bridge Management System (BMS) which prioritizes bridge replacement, rehabilitation, deck and superstructure replacement, painting, scour retrofit and other maintenance repairs. The BMS system is used to develop recommended funding levels to improve the condition of state maintained bridges.

NJDOT has identified a need to prioritize, maximize available bridge funding, and provide decision makers with key construction cost data associated with bridge repair and replacement. Currently, NJDOT stores project and asset-related costs in their Trns-port system. To provide this cost information to NJDOT's BMS for analysis, related cost information from Trns-port will be translated and arranged into a format that can be imported into Pontis.

This technical memorandum was developed as part of Task 1B, Activity 2 (Trns-port Evaluation) of the project and outlines the findings related to the review of Trns-port data.

The Trns-port data received was provided by the New Jersey Office of Information Technology (NJOIT) in October 2013. Additional detail about the data received can be found under the *Data Review* section within the memo.

Trns-port Overview

Trns-port (recently rebranded as *AASHTOWare Project*) allows agencies to manage project information from contract through construction. Trns-port supports the entire construction life cycle, including cost estimation, proposal preparation, letting bids, construction and material management, and data collection.

Data Review

At the commencement of the data review process, Baker worked with NJOIT (the current database administrator of the Trns-port system) to determine the relevant data points required from the database. Through this effort and at the recommendation of NJOIT, it was determine the best course of action was to generate a single report from the Trns-port system that contained only pertinent data items related to structures. The resulting report contained the following data fields:

Field	Descriptions
Cont_ID	Unique contract/project identifier.
Project Description	General project description to which costs relate.
Spec Yr.	Year of project initiation.
Bridge Structure #	Structure ID which relates to the Pontis structure number.
Let Date	Date line item cost was incurred.
LN Item#	Line item number of the construction material within a given contract/project.
Item Descr	Line item description of the construction material within a given contract/project.
Units	The unit of measurement for the given line item.
Unit Price	The unit price for the given line item.
Contract QTY	Total units of the given line item needed for the project.
QTY to Date	Total units of given line item acquired to date for the project.
Item Total Cost	Calculated field (Unit Price X Contract QTY).

Field	Descriptions
Road/Bridge	Identifies whether line item cost is associated with a bridge (B) or road (R)
Proj Total Cost	Total project cost (unique to a project).

Since only costs related to bridges were necessary, the following parameters were applied to the report:

- Include only projects associated with bridge (Road/Bridge = B)
- Include only items where associated costs are included (Unit Price <> 0)

NJOIT provided a sample PDF report (see Appendix A) to Baker on 9/18/2013. After an agreement was reached on the required data points and structure, the complete report (containing 14,599 records) was provided to Baker on 10/1/2013 in a comma-delineated format (.CSV) and imported into a SQL database for analysis.

Findings

The items below present identified areas of deficiencies in the Trns-port data reviewed:

1. Projects are missing bridge structure identifiers.

   During the review of the Pontis data structure (see *Tech Memo #1 – Pontis Data Structure and Review*)*,* it was determined that a reference to a unique bridge identifier must be maintained in order to track related project costs to an individual structure. Analysis performed on the Trns-port data revealed that 42 of the 73 unique projects within the data provided contained no reference to a bridge structure (identified as *"No Bridge ID on Record"*). In order to correctly associate cost information appropriately in Pontis, bridge-related cost items within Trns-port must have an associated bridge identifier that can be linked to the Pontis system.

   Recommended Actions:

   - Use the NJDOT Project Reporting System (PRS) data to link projects to bridges, when the data is available.
   - Associate at least one structure to each bridge-related project within Trns-port.

2. Direct and indirect project costs are not identified.

   Pontis has functionality to track costs at the project level (indirect costs) and individual unit cost which are applied at the element level of a structure (direct costs). Indirect costs are associated with mobilization, traffic control and administration while direct costs define actual unit cost information applied at an element-level (e.g. concrete footing unit cost) for a given action.

   Line items represent individual unit costs within Trns-port. In order to separate direct and indirect costs, each of the 863 unique line items within the Trns-port data set reviewed would need to be differentiated appropriately. While it is realized that defining direct and indirect cost would yield more accurate cost summaries, this additional level of detail may not align with NJDOT's needs.

   Recommended Action:

   - Determine the benefit of tracking direct and indirect cost separately.
   - Categorize each of the 863 unique line items as a direct or an indirect cost.

3. Cost actions need to be defined.

Analysis performed during Task 1 (Pontis Data Review) revealed that all work items must have an associated project action within the Pontis system. An action is a work activity that occurs at the bridge or element level (e.g. 'Rehab Deck/Slab' or 'Replace Structure'). Actions are broken into types and categories within Pontis (a full list of all actions can be found in Appendix B of *Tech Memo #1 – Pontis Data Structure and Review*).

Since business rules built within the Pontis system require that all cost items have a related action, associated actions must be linked to data from the Trns-port system. It is reasonable that actions can be applied at the project level within Trns-port. Take the following project for example:

CONT_ID	PROJECT DESCRIPTION	# LINE ITEMS
11124	ROUTE 322 OVER BIG DITCH BRIDGE REPLACEMENT	97

Since the description suggests that this is a 'bridge replacement' project, the cost action of each of the related 97 line items can be designated with ''Replace Structure' when transferred to Pontis. Each of the 73 uniquely identified projects would need to be handled on a case-by-case basis with input required from NJDOT.

To have a full understanding of the Trns-port Project Descriptions information and make an effective match, this effort will require coordination with subject matter experts, such as the NJDOT Trns-port administrator and/or Baker bridge engineers.

Recommended Action:

- Match the related Pontis action (as described in Appendix B of *Tech Memo #1 – Pontis Data Structure and Review*) to the most appropriate Trns-port project.

4. The level at which cost information should be tracked must be defined.

Pontis functionality allows for the tracking of inventory, inspection and cost information at a structure level or an element level. An element is an individual component that together with other elements constitutes the structure. While it is recognized that tracking information at the element level would result in more precise analysis within certain Pontis modules, this additional level of detail may not be warranted.

Assuming that maintaining a finer level of detail (at the element level) is desired, the following would need to be performed:

- Review Pontis to assure that data exists at the element level for each structure;
- Differentiate 'structure level' projects and 'element level' projects within Trns-port; Example:

CONT_ID	PROJECT DESCRIPTION	LEVEL
11124	ROUTE 322 OVER BIG DITCH BRIDGE REPLACEMENT	STRUCTURE
12136	COLLINGS AVE (CR-630) OVER I-676 SB BRIDGE DECK REPLACEMENT	ELEMENT (DECK)

- Relate each line item cost to specific elements.

NJDOT should carefully consider the level at which project cost information is recorded, as maintaining a lower level of data will increase the complexity of data transfer to Pontis.

Recommended Action:

- Define level at which project cost information should be recorded.

5. Costs must be allocated for projects involving multiple bridges.

During the review of Trns-port data, it was revealed that numerous projects included multiple bridges which were not defined. Below is an example of various projects within the Trns-port data provided that appear to identify with multiple bridges:

CONT_ID	PROJECT DESCRIPTION
11401	MAINTENANCE BRIDGE PAINTING CONT. 2011-2,FHWA, RTS:19&80
12438	MAINTENANCE CONCRETE STRUCTURAL REPAIR CENTRAL CONT NO. 2013
12441	MAINTENANCE MOVABLE BRIDGE & TUNNEL REPAIR CONTRACT 2013

No bridges were related to the project in each of the above examples (see Finding 1). However, based on the project description, it can be reasoned that each project relates to multiple bridges. As this is the case, to correctly translate cost data into Pontis, project costs must be allocated across all bridges to which the project pertains. The following options could be considered:

1. Divide costs equally among involved bridges;
2. Proportionally allocate costs based on the element-level inventory unit replaced;
   a. *Example: Bridge A and Bridge B were repaired as part of a project, which cost $10,000. Bridge A had 100 sqft. of decking repaired, while B had 150 sqft. of decking repaired. Since a total of 250 sqft was replaced and 40% of the total decking replaced involved Bridge A, 40% of the project cost ($4,000) would be associated appropriately.*
3. Divide indirect cost equally among involved bridges; proportionally allocate direct costs based on the element-level inventory unit replaced.

While these don't represent all possible options, NJDOT should consider a solution that can be integrated into a repeatable process. Note, due to the potential complexity and dependence of option 2 to other variables, logic to distribute cost in an "intelligent" manor may have to be simplified for this effort since it has potential to be complex.

Recommended Action:

- Define how costs related to projects involving multiple bridges should be assigned, either proportionally or through some level of logic using a different bridge metric (deck size, etc.).

6. No funding source or program information exists in the data.

To adequately assign cost information in Pontis, funding source and program information is required. An evaluation on the Trns-port data received revealed that these data items were missing entirely. To overcome this deficiency, NJDOT can consider the following options:

- Associate existing and/or known funding sources/programs to each project via PRS;
- Assign a new 'catch-all' data identifier, for items that cannot be associated to existing programs;
  - *Example: All imported Trns-port cost information could be identified with a program of 'Trns-port Cost Import'.*

Recommended Action:

- Define how funding sources and programs should be assigned.

7. <u>The contract number within Trns-port does not correlate with the UPC code in PRS.</u>

   In order to correctly associate projects, bridges and costs, a distinct link between the Trns-port and PRS systems must be identified. A review of the data revealed no direct association between the contract number within Trns-port and the UPC code, which NJDOT's standard project identifier within PRS. This relationship must be identified so cost information can be arranged and transformed accurately into Pontis.

   Recommended Action:

   - Provide a crosswalk between the Trns-port contract number and PRS UPC code.

<u>Summary of Action Items for NJDOT Review:</u>

Based on the findings of the data review, a face-to-face meeting with NJDOT is recommended to discuss the following critical path items which required action:

1. Link projects to bridges within PRS where applicable *(Finding 1).*
2. Define Pontis cost actions for each project *(Finding 3).*
3. Provide a crosswalk between the Trns-port contract number and PRS UPC code *(Finding 7).*

The items below represent that require NJDOT review and response:

1. Define/Provide the relationship between the Trns-port "Contract ID" and the NJDOT standard UPC number in the NJDOT PRS system. This relationship will be critical for assigning bridge structures to cost items where structure numbers are not provided from Trns-port.
2. Determine the benefit of tracking direct and indirect cost separately *(Finding 2).*

   <u>Recommendation:</u>
   Differentiate direct and indirect cost items by categorizing each unique line item within Trns-port. These items appear to be easily distinguishable and would result in better cost analyses within Pontis.
3. Determine the level at which cost information should be tracked within Pontis *(Finding 4).*

   <u>Recommendation:</u>
   Cost should be tracked consistently at a structure level rather than element level, as smaller cost units can easily be aggregated and applied within Pontis.
4. Define how costs related to projects involving multiple bridges should be assigned *(Finding 5).*

   <u>Recommendation:</u>
   Proportionally allocate all costs (direct and indirect) based on bridge span length, as this is a standard baseline data item recorded and maintained in Pontis.
5. Define a program and funding source for all projects *(Finding 6).*

   <u>Recommendation:</u>
   Attach a funding source and a program to all projects within PRS.

APPENDIX A – NJOIT SAMPLE TRNS-PORT REPORT

09/18/2013

New Jersey Department of Transportation
Contract Summary Information For Justin Furch

Contract ID

06406 **Project Description:** BPC 2006-4 (This is the contract/project description))))) **Spec Year:** 2001

Item No.	Item Description	Units	Unit Price	Contract Qty	Qty to Date	Total Cost
0011	EXTRA ILLUMINATED FLASHING ARROWS, 4' X 8'	DAY	0.10	50.00	23.00	2.30
0021	NEAR-WHITE BLAST CLEANING AND PAINTING	LS	80,000.00	1.00	1.00	80,000.00
0024	NEAR-WHITE BLAST CLEANING AND PAINTING	LS	225,000.00	1.00	0.90	202,500.00
0025	NEAR-WHITE BLAST CLEANING AND PAINTING	LS	225,000.00	1.00	1.00	225,000.00
9009	HOT MIX ASPHALT SURFACE COURSE MIX I-4 H	T	25.00	100.00	10.00	250.00

09158 **Project Description:** READVERTISEMENT OF RT 72 MANAHAWKIN BAY BRIDGE DECK **Spec Year:** 2007

Item No.	Item Description	Units	Unit Price	Contract Qty	Qty to Date	Total Cost
0007	FIELD OFFICE TYPE A SET UP	U	30,000.00	1.00	1.00	30,000.00
0008	FIELD OFFICE TYPE A MAINTENANCE	MO	1,200.00	14.00	2.00	2,400.00
0015	OIL ONLY EMERGENCY SPILL KIT, TYPE 1	U	800.00	2.00	2.00	1,600.00
0023	TRAFFIC CONTROL TRUCK WITH MOUNTED CRASH CUSHION	U	7,200.00	4.00	0.50	3,600.00

10001 **Project Description:** ROUTE 55 **Spec Year:** 2007

Item No.	Item Description	Units	Unit Price	Contract Qty	Qty to Date	Total Cost

09/18/2013

New Jersey Department of Transportation
Contract Summary Information For Justin Furch

Contract ID

10001 **Project Description:** ROUTE 55 **Spec Year:** 2007

Item No.	Item Description	Units	Unit Price	Contract Qty	Qty to Date	Total Cost
0001	DENSE-GRADED AGGREGATE BASE COURSE, VARIABLE THICKNESS	CY	5.00	740.00	740.00	3,700.00
0002	CONSTRUCTION LAYOUT	LS	4.00	1.00	1.00	4.00
0003	ASPHALT PRICE ADJUSTMENT	LS	2.00	1.00	1.00	2.00
0004	FUEL PRICE ADJUSTMENT	LS	3.00	1.00	1.00	3.00
0005	SUBBASE	CY	6.00	4,456.00	4,456.00	26,736.00
0006	TACK COAT	GAL	7.00	43,100.00	42,150.00	295,050.00
0007	PRIME COAT	GAL	9.00	450.00	450.00	4,050.00
0008	MATERIALS FIELD LABORATORY MAINTENANCE	MO	5.00	5.00	5.00	25.00
0009	EXCAVATION, UNCLASSIFIED	CY	2.00	1,145.00	1,145.00	2,290.00
0010	I-10 SOIL AGGREGATE	CY	1.00	1,000.00	1,000.00	1,000.00
0011	HOT MIX ASPHALT 19 M 64 BASE COURSE	T	12.00	1,500.00	1,500.00	18,000.00
0012	HOT MIX ASPHALT 12.5 H 64 INTERMEDIATE COURSE	T	13.00	500.00	500.00	6,500.00
0013	HOT MIX ASPHALT 12.5 H 64 SURFACE COURSE	T	14.00	640.00	640.00	8,960.00
0015	LANE OCCUPANCY CHARGES	DOLL	16.00	1.00	1.00	16.00
0016	SUBSTRUCTURE CONCRETE REPAIR	SF	1.00	242.00	242.00	242.00
0017	STRUCTURAL STEEL REPAIR, TYPE 1	LB	71.00	7,600.00	7,600.00	539,600.00

RU435056-8

NJDOT Cost Data Migration

Technical Memorandum
June, 2014

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16. Abstract
This technical memorandum was developed as part of Activity 3, Task 1B (NJDOT PRS Evaluation) of the project and outlines the findings related to the review of PRS and Trns-port data. This memorandum also outlines recommendations and consideration for future improvement regarding the availability and use of cost data associated with structures managed by NJDOT.

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Introduction

The New Jersey Department of Transportation (NJDOT) has a comprehensive Bridge Management System (BMS) which prioritizes bridge replacement, rehabilitation, deck and superstructure replacement, painting, scour retrofit and other maintenance repairs. The BMS system is used to develop recommended funding levels to improve the condition of state maintained bridges.

NJDOT has identified a need to prioritize available bridge funding and provide decision makers with key construction cost data associated with bridge repair and replacement. Currently, NJDOT stores asset-related costs for capital projects in the Project Reporting System (PRS) and stores actual construction cost data within Trns-port. To provide this cost information to NJDOT's BMS for analysis, cost information from Trns-port will be combined with related project information from PRS and arranged into a format that can be imported into Pontis.

This technical memorandum was developed as part of Activity 3, Task 1B (NJDOT PRS Evaluation) of the project and outlines the findings related to the review of PRS and Trns-port data. This memorandum also outlines recommendations and consideration for future improvement regarding the use of cost data within the Pontis system.

PRS data provided by the Division of Capital Program Support was received April 29, 2014. Additional detail about the data received can be found under the *Data Review* section within the memo.

PRS Overview

PRS is an application which was developed internally by NJDOT and is used to track the delivery and progress of capital projects from concept development through the final design phase. PRS is used to produce budget estimates, track and record project data and funding, and create various reports. The system is administered by the Division of Capital Program Support within the Bureau of Program Systems Management.

Data Review

At the commencement of the data review process, Baker worked with the Division of Capital Program Support to determine the appropriate data points required from the database. Through this collaboration, the following data points were deemed relevant:

- UPC #
- Bridge Structure #
- Project Name/Description
- Concept Development Cost (actual)
- Preliminary Design Cost (actual)
- Final Design Cost (actual)
- Construction Award Data

PRS information was received from NJDOT on April, 29, 2014 in the form of a Microsoft Access database. The database contained two data tables:

- **dbo_v4PRS_Bridge_ProjInfo** (438 records)

Field	Description
UPC	Universal Project Code. A six-digit project identifier. The first two digits represent the fiscal year the project was created. The last four digits are the next available sequential numbers.

Field	Description
TITLE	Name of the capital project.
SRI	NJDOT's Standard Route Identifier used to differentiate routes. Indicates the route on which the project is located.
AltSRI	Secondary route on which the project is located.
StartMp	Starting mile point location of the project along the SRI.
EndMp	Ending mile point location of the project along the SRI.
AltStartMp	Starting mile point location of the project along the secondary SRI.
AltEndMp	Ending mile point location of the project along the secondary SRI.
ScheduledAward	Anticipated date the project will be awarded during the bidding phase.

- **dbo_v4PRS_TblStructures** (915 records)

Field	Description
UPC	Universal Project Code. A six-digit project identifier. The first two digits represent the fiscal year the project was created. The last four digits are the next available sequential numbers.
Structure_Number	7-digit bridge structure identifier.

Findings

The items below present reflect areas of deficiencies in the PRS data reviewed:

1. No cost was provided.

   While it is assumed that PRS contains information relating to engineering, utility and ROW costs, this information was not provided within the submitted dataset. Ideally this information would be combined with actual construction cost data from the Trns-port system to allow for more accurate analysis and better cost projections within Pontis.

   Recommended Actions:

   - Work with NJOIT to create methods to extract cost information from the PRS system. Furthermore, if available, NJDOT should consider how these costs should be categorized within Pontis and utilized within the system.

Recommendations for Future Improvements

Throughout Task 1B, there were several identified areas of potential future improvements for data migration into Pontis. These items, recommendations and associated options for improvement are provided below:

1. Incorporate Preventive Maintenance Costs

   At the onset of the task, it was determined that the focus of the effort would be capturing costs related to capital projects as opposed to extracting maintenance and preservation project costs. While maintenance tasks are usually smaller in cost and scope then capital projects, capturing these costs would provide decision makers with more complete information and facilitate the prioritization of available bridge funding.

   During the task, it was revealed that maintenance costs are organized and tracked in the Highway Maintenance Management System (HMMS), which was not reviewed as part of this assignment. NJDOT should investigate and review HMMS to determine what information is contained within the system relating to structures, and determine the feasibility of utilizing that data within the Pontis system. At minimum, the following data points would need to exist within HMMS to align with the Pontis data model:

- Bridge structure number
- Type of maintenance activity performed
- Cost
- Funding source

2. Determine Required Bridge/Element Relationships within Pontis

Having available element information within Pontis is a prerequisite to tracking costs at the element level. To this end, a query was performed across the population of bridges within the database which were classified as NJDOT owned to assure element information existed for each accordingly. A summary of this analysis is provided below:

Bridge/Element Relationship Review	Number	Percent
Total Number of Unduplicated Bridges	**8,013**	**-**
NJDOT Owned Bridges	**3,662**	**100%%**
Associated with >= 1 Element(s)	3,532	96.5%
No Associated Element	130	3.5%

In total, 130 bridges categorized as NJDOT owned had no available element information stored within Pontis. While the majority of these bridges appear to be non-highway carrying structures (pedestrian bridges and railroad bridges over roadways), an element-level inventory would be required if NJDOT determines that it is viable to track costs at a more detailed level for these types of structures. Appendix A - *NJDOT Owned Bridge Missing Elements* contains a full listing of these 130 bridges.

3. Add Cost Action Categorization in Trns-port

During the review of the Trns-port system (refer to *Technical Memorandum #2 – Trns-port Data Review* developed as part of Activity 2 of this task), it was found that cost actions are not defined. Analysis performed during Activity 1 of the assignment (Pontis Data Review) revealed that all project work items must have an identifiable cost action within the Pontis system. Since cost actions are not tracked within Trns-port, each of 305 projects exported from the system needed to be assigned an action manually. During the activity, each project was grouped into one of the following actions:

- Bridge Replacement
- Bridge Rehab
- Bridge Painting
- Superstructure Replacement
- Deck Repair
- Deck Replacement

While preferably cost action definitions would exist in the Trns-port system, NJDOT should determine the feasibility of integrating this categorization. This assessment should include the following:

- Determine if the Trns-port data model can be modified to incorporate the new cost action definitions.

- Determine level of effort required to categorize cost actions within Trns-port.

If this analysis reveals that it would not be viable to integrate cost actions into Trns-port, an effort could be initiated to incorporate actions into PRS. However, since not all Trns-port projects are within PRS, manual cost action assignment would still need to be performed.

4. Add Direct/Indirect Cost Categorization within Trns-port

During the task, an effort was made to match each individual standard line item[1] within Trns-port to Pontis bridge elements. The first step involved a review of all 2,384[2] unique line items that were provided within the Trns-port database to determine which items directly relate to bridges. During this review process, all line items were classified into one of the following categories:

- **Direct** – Items that appear directly related to the infrastructure of a bridge or a bridge project.
- **Bridge Indirect** – Items related to a bridge project, but not necessarily the structure of a bridge. Examples of items in this category include costs for mobilization, traffic control and administration.
- **Indirect** – Items which do not relate to a bridge project or infrastructure of a bridge.

The result of the categorization is listed below (see Appendix B – *Line Item Categorization* for full classification detail).

Trns-port Standard Line Item Review	Number	Percent
Total Number of Unduplicated Line Items	**2,384**	**100%**
Direct	**404**	**16.9%**
Bridge Indirect	**172**	**7.2%**
Indirect	**1,808**	**75.9%**

While ideally this cost classification would exist in the Trns-port system, NJDOT should determine the feasibility of integrating this categorization. This assessment should include the following:

- Determine if the Trns-port data model can be modified to incorporate the new cost categorization.
- Determine level of effort required to make cost categorizations within Trns-port.

If this assessment determines that it would not be feasible to incorporate costs categorization in Trns-port, utilizing the categorization that was developed during this assignment would be an alternative option.

5. Create One-To-One Relationship Between Line Items and Elements

After the categorization of direct/indirect items was complete, an effort was made to align each of the 404 direct items to at least one of the current Pontis elements (see Appendix C – *Unused Elements*)[3]. As part of this process, each direct line item was reviewed and matched to an

[1] 'Line items' as referenced in this document is synonymous with 'pay items' as used within the *NJDOT Standard Specification for Road and Bridge Construction (2007).*

[2] This number may not be representative of all the standard line items available within the Trns-port system, since only a subset of the database was received as part of the task. Only the unique line items received within the dataset provided were reviewed as part of this analysis.

[3] While the Pontis database contains 161 unique elements, NJDOT only utilizes 138 of these elements within the system. An investigation may be warranted to determine the practicality of keeping these unused elements within the system.

associated element where possible based on the appropriate record descriptions. Line Item/Element matches were classified into one of the following categories:

- **Matched to Element(s) –** Line item matched to one or more element based on description provided.

- **No Matching Element –** No logical corresponding element exists based descriptions provided.

- **Insufficient Description –** Line item description is too vague or too broad to be matched with an element.

A summary of the matching across the 404 direct line items is provided below:

Line Item/Element Matching	Number	Percent
Direct Line Items	**404**	**100%**
Matched to Element(s)	**274**	**67.8%**
Matched to One (1) Element	59	14.6%
Matched to More than One (1) Element	215	53.2%
Not Matched	**130**	**32.2%**
No Matching Element	115	28.5%
Insufficient Description	15	3.7%

The full listing of the 215 line items matched to more than element is provided in Appendix D – *Line Items Matched to >1 Element*. A specific example of this scenario is provided below:

Standard Line Item	Equivalent Elements
504055P - CONCRETE BEAM	105 - Reinforced Concrete Closed Webs/Box Girder
	110 - Reinforced Conc Open Girder/Beam
	116 - Reinforced Conc Stringer
	155 - Reinforced Conc Floor Beam
	171 - Concrete-Encased Steel Stringer
	174 - Floor Beam - Concrete Encased Steel

In this case, the standard line item '*Concrete Beam*' could be related to six (6) different elements. In order to migrate cost information into Points at element level, a one-to-one relationship must be established between line items and elements. In the current state as a one-to-many relationship, there is insufficient information as to how line item cost and quantities should be allocated across the different elements when they exist on one bridge structure.

6. <u>Create New Agency-Defined Elements</u>

During the line item/element matching process, 130 line items could not be associated with any one element (115 being a result of no matching element while 15 were due to an insufficient description). The table below shows representative examples of unmatched line items (see Appendix E – *Unmatched Line Items* for full list):

Standard Line Item	Equivalent Elements
613005P - NOISE BARRIER, BRIDGE	None
509102P - PICKET FENCE, STEEL, BRIDGE, 6' 3" HIGH	None
652293P - 12" STEEL SEWER PIPE, BRIDGE	None

While these line items all relate directly to a bridge, they are generally ancillary structure items which have no current element equivalence. A possible solution would be to develop and create new independent agency-specific elements, as this functionality exists within the current BMS. If NJDOT decides to undertake such action, a feasible study may be warranted that take the following considerations into account upon the identification of a new potential element:

- How to define baseline element inventory
- How to define inspection and condition assessment guidelines
- Perform cost/benefit analysis of tracking and managing new element

While any study should not be limited to the outlined items above, it important that NJDOT develop guidelines that are consistently applied when considering the addition of a new element. Any guidelines developed should be incorporated into NJDOT's current *Pontis Coding Guide*[4]*.*

7. Align Unit Measurements across Trns-port and Pontis

Each of the 274 line items which matched to elements were researched to determine unit compatibility, which would be a prerequisite to data migration. The matching process resulted in a total of 2,306 unique line item/element combinations due to the one-to-many relationship between line items and elements (see Recommendation 5). Each combination was reviewed to determine the unit compatibility and categorized as such:

- **Matching Units –** Line item/element combinations that directly align or can be aligned using a simple unit conversion.

- **Unmatched Units** – Combinations that have a unit type mismatch which cannot be aligned using unit conversions.

- **No Unit Information –** Line item/element combinations that were matched, but which no element unit information was available. These relate to 23 unused elements within Pontis (see Appendix C – *Unused Elements*).

A summary of the unit compatibility analysis is provided in the table below:

Line Item/Element Unit Compatibility	Number	Percent
Line Item/Element Combinations	**2,306**	**100%**
Matching Units	629	27.3%
Unmatched Units	1,393	60.4%
No Unit Information	284	12.3%

The matrix on the next page provides a count of all line item/element unit combinations categorized across each existing unit group. A detailed breakdown of combinations with matching units can be found in Appendix F – *Matched Unit Detail*, while the unmatched unit breakdown is provided in *Appendix G – Unmatched Unit Detail*.

[4] Current version is located online here: http://www.state.nj.us/transportation/eng/structeval/pdf/PontisCodingGuide.pdf (accessed June 2014)

		Element Unit			
		Unit (ea.)	Meter (m.)	Square Meter (sq. m.)	No Unit
Line Item Unit	Cubic Foot (CF)	12	40	28	16
	Cubic Yard (CY)	34	101	122	33
	Lump Sum (LS)	60	108	3	19
	Linear Foot (LF)	143	238	65	35
	Square Foot (SF)	53	164	217	82
	Square Yard (SY)	30	34	79	21
	Ton (T)	0	16	0	2
	Unit (U)	95	57	21	28
	Pound (LB)	63	180	46	48
	Hour (Hour)	13	0	0	0

Matching Units (629) **Unmatched Units (1,393)** **No Unit Information (284)**

Potential options for reconciling unit mismatches could be:

- **Modify line item units within Trns-port** – This process would involve modifying current line item units within Trns-port to match those within Pontis. Should NJDOT pursue this option, significant changes may be required to the *NJDOT Standard Specification for Road and Bridge Construction (2007)*.

- **Modify element units within Pontis** – Pontis functionality allows for the conversion of element units within the *Configuration* module of the application. Unit modifications, however, would likely require adjustments to current inventory and inspection procedures.

- **Ignore element unit quantities within Pontis** – There may be instances where unit conversions are impractical or impossible without additional inventory. Take for example the following line item/element unit mismatches:

Line Item	Line Item Unit	Element	Element Unit
507021P - CONCRETE BRIDGE DECK	Cubic Yard	12 - Concrete Deck - Bare	Sq. Meter

In the example above, a conversion could be possible but would require more information regarding the depth of the deck to make the proper translation (a volume unit (CY) to area unit (sq. m.)). This method would require a more detailed inventory of the concrete bridge deck (a depth measurement to calculate volume).

Line Item	Line Item Unit	Element	Element Unit
02001MB088 - BRIDGE DECK CRACK SEALING	Linear Feet	12 - Concrete Deck - Bare	Sq. Meter

In this example, there is no logical unit conversion between the units (linear unit (LF) to an area unit (sq. m)). It is possible within the Pontis framework to import only costs without quantities at the element level.

8. Ensure Bridge Association with Relevant Data Across NJDOT Management Systems

Within Pontis, a reference to a unique bridge identifier must be maintained in order to track related project costs to an individual structure. As this is the case, all projects within the Trns-port and PRS systems should be associated with the standard seven (7) digit bridge structure identifier as appropriate. While associating structure identifiers to projects within Trns-port has become standard practice at NJDOT recently, several projects needed to be linked with bridges on a manual basis during the task. To facilitate the import of cost information into Pontis in the future, NJDOT should establish measures to ensure that bridges are related to data in all relevant management systems including, but not limited to the following:

- Trns-port
- PRS
- HMMS
- Financial Systems

APPENDICES

APPENDIX A – NJDOT Owned Bridges With No Elements

BRKEY	STRUCTURE NAME
0170150	WINSLOW INDUST. TRK/CEDAR LAKE CK
0202157	WASHINGTON TER PED BR./US RTS 1+9 & 46
0206160	FORREST AV PED BRIDGE OVER RT 4
0206162	PED BRIDGE AT GRAND AVE OVER NJ RT 4
0206170	LINCOLN AVENUE PED BR OVER RT4
0206178	PHELPS RD PED BR OVER RT4
0206191	BERGEN MALL PEDESTRIAN BRIDGE OVER NJ RT 4
0210152	PEDESTRIAN BRIDGE OVER US 9W
0212152	NJ TRANSIT BRGN CO LN / RT 17 NB
0216162	PEDESTRIAN BR. OVER RT 17
0217161	PEDESTRIAN BR-PROSPECT AVE/RT 17
0217166	RACE TRACK RD PED BRG OVER RT 17
0220151	PED BR AT BANK ST/US46 & ROOSEVELT AV
0220156	PEDESTRIAN BR AT KEASLER AVE/US RT 46
0220163	6TH ST PED BR OVER US46
0304153	RAMP K OVER BR.OF N.BR.PENNSAUKEN CR.
0316156	PEDESTRIAN BR AT RIVERTON ROAD/US 130
0401151	SOUTHERN BRANCH OVER ROUTE 30
0406155	RODMAN AVE PED BR/RT 30
0408157	PEDESTRIAN BRIDGE OVER ROUTE 38
0422151	GARFIELD AVE PEDESTRIAN BRIDGE/US 130
0422152	FEDERAL ST PED. BRIDGE/RT 130
0422157	MERCHANTVILLE AVE PED BR/RT 130
0428156	GRENLOCH SECONDARY RR OVER I-295
0430161	BORDENTOWN SECONDARY/MLK BLVD & RIVERLINE
0430163	BORDENTOWN SECONDARY/ELEVENTH ST.
0730158	NJ TRANSIT MORRISTOWN LINE/I-280
0832166	RELOC CONRAIL TRACK OVER RT 55
0870150	WINSLOW INDUST TRACK/EGG HARBOR
0870151	WINSLOW IND. TRK/HOSPITALITY LAKE
0902156	US 1+9 Ramp I over Ramps B, C, D
0906161	US 1&9T Ramp B over US 1&9 and Ramp C
0907150	PEDESTRIAN AT 7TH OVER RT.3 WESTBOUND
0907151	PED BR AT 7 TH OVER RT 3 EB & WOOD AV
0914151	WORTHINGTON PUMP SPUR/FRANK_S CRK
0915150	GREENVILLE BR&LVMLRR OVER RT 440
0917151	PEDESTRIAN BRIDGE OVER RT 495
0921150	CONSTABLE HOOK IT(CONRAIL)/ RT 440 (FORMER RT.169)
0951167	CENTRAL&HOBOKEN AVE/BERGEN CO BRANCH - TUNNEL
1106152	PEDESTRIAN BR AT S.HERMITAGE AV/NJ 29
1107151	EASTFIELD AVE PED BR / RT 29
1114X01	US 130 & NJ 33 OVER UNNAMED STREAM
1119151	ROUTE 31 OVER SEMINARY CREEK
1120156	CONRAIL NEW YORK BRANCH OVER I-95
1130157	South Pedestrian Bridge at Rt. 29 Tunnel
1130158	NJ 29 TUNNEL
1131162	NORTH PEDESTRIAN BRIDGE AT NJ RT 29
1136179	PEDESTRIAN BRIDGE OVER ROUTE 295
1202157	CONNECTOR A OVER US ROUTE 1
1204150	Greenway Trail Pedestrian Bridge over Route 1
1209150	CONRAIL PAMBY- SPNFLD BR/US9 &GSP
1211160	ST. THOMAS CH. PED BR. OVER RT 18
1229151	PEDESTRIAN BRIDGE OVER RT 172

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BRKEY	STRUCTURE NAME
1230157	CONRAIL BONHAMPTON BR SPUR/I-287
1232156	PEDESTRIAN BRIDGE/ NJ 440 & LOCAL RDS
1234163	CONRAIL RARIT BRANCH/RT.440
1234164	RARITAN BR./RAMPS GL,GK,GNK,GN&GM
1234165	CONRAIL RARITAN BRANCH/SMITH ST.
1234176	PAMBOY-SPLNFLD BR./RT440 & RAMPGT
1237156	DECK(SPORTS COMPLEX) OVER RT. 18
1237163	MULTI USE PATH OVER RIVER ROAD (CR 609)
1309151	PEDESTRIAN BRIDGE OVER RT 34
1324159	PEDESTRIAN BRIDGE OVER-FREHLD LN/NJ 18
1328152	FORDHAM RD PED BRIDGE OVER RT18
1333177	CR 547 OVER OLD RIVER BED
1405157	PED BG OVER RTE 23
1413178	PEDESTRIAN BR /RAMP _K_ TO I-80WB
1417154	CONRAIL HIGH BRIDGE OVER ROUTE US 206
1417162	HIGH BRIDGE BR./US ROUTE 206 RAMP A
1419176	NJ TRANSIT MORRISTOWN LN OVER 287
1420173	BOONTON LINE (TRANSIT) OVER I-287)
1428154	PED BR (PREV. ABANDONED SUSSEX RR) OVER 206 CONN
1470150	EAST WYE SCRANTON BR/CENTER ST.
1470153	SCRANTON BRANCH OVER MUSCONETCONG RIV
1604172	PEDESTRIAN BRIDGE OVER ROUTE 23
1607169	ST. PHILLIPS DR PEDTN BR OVER US46
1609151	NJ TRNST BOONTON LN/I80 & RMPS EI
1609152	NJT (BOONTON LINE) / SINGAC BROOK
1610154	TAFT AVE PEDESTRIAN BRIDGE / I-80
1612152	ETHEL AVE PED BRIDGE / NJ RT 208
1615153	CANNONBALL TRAIL PED BR OVER I-287
1803160	Pedestrian Bridge over US 22
1812162	CONRAIL MIDDLE BROOK BR OVER 287
1815175	NJ TRANSIT GLADSTONE BR OVER 287
1911155	SCRANTON BRANCH OVER US RT 206
1970150	SCRANTON BRANCH OVER ABANDONED CTY602
1970152	SCRANTON BRANCH OVER BROOK @ (49.30)
1970155	SRANTON BRANCH OVER CO RT 607
1970156	SCRANTON BRANCH OVER ROSEVILLE ROAD
1970157	SCRANTON BRANCH OVER ROSEVILLE RD
1970158	SCRANTON BRANCH OVER BROOK AT 52.86
1970160	SCRANTON BRANCH OVER CO RTE 517
1970162	SCRANTON BRANCH OVER AIRPORT RD (603)
1970163	SCRANTON BRANCH OVER PEQUEST RIVER
1970164	SCRANTON BRANCH OVER PEQUEST ROAD.
2004158	PEDESTRIAN BRIDGE OVER US ROUTE 22
2011168	CONRAIL IRVINGTON BR RR OVER RT78
2011169	CONRAIL IRVINGTON SPUR OVER I-78
2012156	LEHIGH VALLEY MAIN LINE OVER RT82
2015155	SIRT RR OVER I278 RAMP & US1&9 SB
2018154	NJ TRANSIT MORRISTN LN OVER RT 24
2070150	LEHIGH VALLEY LINE OVER SIRT RR.
2070151	SIRT RR OVER KLEINEIDERS RUN
2070152	SIRT RR OVER MARTINS RUN CREEK
2070153	SIRT RR LINE OVER ELIZABETH AVE
2070154	SIRT RR LINE OVER AMTRAC CORRIDOR

APPENDIX A – NJDOT Owned Bridges With No Elements

BRKEY	STRUCTURE NAME
2070155	SIRT RR OVER LINDEN AVENUE.
2070156	SIRT RR LINE OVER PEACH GARDEN CRK.
2070157	ADJACENT SIRT LINE OVER PEACH GARDEN
2070158	SIRT OVER EXXON PLANT SUBWAY
2070159	SIRT RR OVER SIMMONS SIDETRACK
2070160	SIRT RR OVER PSCT (ABANDONED LINE)
2070161	SIRT RR OVER TURNPIKE SB RAMP
2070162	SIRT RR LINE OVER TURNPIKE
2070163	STATEN ISLAND RAPID TRST/JM,BY&AMBY
2070164	STATEN ISLAND RAPID TRST/CHEM COAST $
2071150	RAHWAY VALLEY LINE(ABND)/RAHWAY RIVER
2071151	RAHWAY VALY LINE/SHUNPIK
2071153	RAHWAY VALLEY LINE(ABAND)/ASHWOOD AVE
2071154	RAHWAY VALLEY LINE(ABAND.)/MORRIS AVE
2112154	LEHIGH MAIN LINE(CONRAIL)/I-78
2170152	SCRANTON BRANCH OVER RAMSEY RD (661)
2170165	SCRANTON BRANCH OVER BROOK AT RRMP67.
2170166	SCRANTON BRANCH OVER CO RTE 603
2170167	SCRANTON BRANCH OVER BROOK ADJ RT 603
2170168	SCRANTON BRANCH OVER CO RTE 658
2170169	SCRANTON BRANCH OVER BROOK AT KILL RD
2170170	SCRANTON BRANCH OVER KILL ROAD
2170171	SCRANTON BRH OVER STATION RD ;PAULINS
M06343R	Windsor Road over NJTPK (I-95)

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
019961C21E	PERFORMANCE BOND AND PAYMENT BOND	Indirect	020016V46G	8" PVC SANITARY SEWER PIPE	Indirect	156015M	NUCLEAR DENSITY GAUGE	Indirect
019961E21C	CONSTRUCTION LAYOUT	BRIndirect	020017C94E	LIGHTING STANDARD ASSEMBLIES,	Indirect	156018M	FLEXURAL BEAM TESTING EQUIPMENT	Indirect
019962A15E	COMPOSITE SOIL SAMPLING AND ANALYSES	Indirect	02001MB070	BRIDGE HEADER REPAIR	Direct	156021M	CONCRETE COMPRESSION TESTING EQUIPMENT	BRIndirect
019964D02J	CORE SAMPLES, HOT MIX ASPHALT	Indirect	02001MB077	REPAIR CONCRETE CURB	Direct	157003M	CONSTRUCTION LAYOUT	BRIndirect
019964D21E	ASPHALT PRICE ADJUSTMENT	Indirect	02001MB084	NEAR-WHITE BLAST CLEANING AND PAINTING	Direct	157004M	CONSTRUCTION LAYOUT	BRIndirect
019966C15X	RESET MANHOLES, SANITARY SEWER, USING NE	Indirect	02001MB085	FLOODLIGHTS FOR NIGHTTIME OPERATION	BRIndirect	157006M	MONUMENT	Indirect
019966C81C	INLETS, TYPE B MODIFIED	Indirect	02001MB086	DECK CORROSION INHIBITOR	Direct	157008M	NOAA MONUMENT REMOVAL AND REPLACEMENT	Indirect
019966N60H	GATES, CHAIN-LINK FENCE, PVC-COATED STEE	Indirect	02001MB087	DECK JOINT RESEAL (SILICON)	Direct	157009M	MONUMENT BOX	Indirect
019966Q32Z	EXTRA ILLUMINATED FLASHING ARROWS,	Indirect	02001MB088	BRIDGE DECK CRACK SEALING	Direct	157021M	STRAIN GAUGE	Indirect
019966U11I	CRASH CUSHIONS, INERTIAL BARRIER SYSTEM,	Indirect	02001MB089	DECK JOINT REPAIR	Direct	157030P	GEOPHYSICAL SURVEY	Indirect
019966U67J	CRASH CUSHIONS, QUADGUARD, 7 BAYS,	Indirect	02001MB090	DECK JOINT RESEAL (RUBBER ASPHALT)	Direct	157031P	MARINAAND CHANNEL SURVEY	Indirect
01996NSLS	NON-STANDARD ITEM UNIT LS	Indirect	02001MB091	FORCE ACCOUNT LABOR, EQUIPMENT AND MATER	Indirect	158003M	CAUTION FENCE	BRIndirect
020011C21	TEST ITEM	Indirect	02001MG001	FORCE ACCOUNT, MATERIALS	Indirect	158006M	SILT FENCE	Indirect
020011C21E	PERFORMANCE BOND AND PAYMENT BOND	Indirect	02001MG004	OFFICE EQUIPMENT	Indirect	158009M	HEAVY DUTY SILT FENCE, ORANGE	Indirect
020011E21C	CONSTRUCTION LAYOUT	BRIndirect	02001NSLF	NON-STANDARD ITEM UNIT LF	Indirect	158012M	HEAVY DUTY SILT FENCE, BLACK	Indirect
020011E21E	FIELD OFFICE TYPE A SET-UP	Indirect	02001NSLS	NON-STANDARD ITEM UNIT LS	Indirect	158015M	HAYBALE	Indirect
020011E21F	FIELD OFFICE TYPE A MAINTENANCE	BRIndirect	02001NSU	NON-STANDARD ITEM UNIT U	Indirect	158018M	HAYBALE CHECK DAM WITH TEMPORARY STONE OUTLET	Indirect
020011E31E	FIELD OFFICE TYPE B SET-UP	Indirect	104003M	VALUE ENGINEERING	Indirect	158021M	TEMPORARY STONE CHECK DAM	Indirect
020011E31F	FIELD OFFICE TYPE B MAINTENANCE	BRIndirect	105003P	TEMPORARY SUPPORT, UTILITY	Indirect	158024M	TEMPORARY SLOPE DRAIN	Indirect
020011E31G	TELEPHONE SERVICE	BRIndirect	108003M	LANE OCCUPANCY CHARGES	Indirect	158026M	SILT SACK INLET PROTECTION	Indirect
020011G13I	OWNER'S AND CONTRACTOR'S PROTECTIVE	Indirect	108006M	INCENTIVE	Indirect	158027M	INLET FILTER TYPE 1	Indirect
020011G22I	RAILROAD PROTECTIVE LIABILITY INSURANCE	Indirect	108009M	DISINCENTIVE	Indirect	158030M	INLET FILTER TYPE 2, 2' X 4'	Indirect
020011G34I	POLLUTION LIABILITY INSURANCE	Indirect	109006M	FORCE ACCOUNT	Indirect	158033M	INLET FILTER TYPE 2, 4' X 4'	Indirect
020011H21C	PROGRESS SCHEDULE	Indirect	109009M	DELAY DAMAGES	Indirect	158036M	TEMPORARY INLET SEDIMENT TRAP	Indirect
020011H21D	MOBILIZATION	BRIndirect	151003M	PERFORMANCE BOND AND PAYMENT BOND	Indirect	158041M	TURBIDITY DAM	Indirect
020012A11H	DEMOLITION OF BUILDING	Indirect	151006M	PERFORMANCE BOND AND PAYMENT BOND	Indirect	158042M	FLOATING TURBIDITY BARRIER, TYPE 1	Indirect
020012A21C	CLEARING SITE	Direct	152003P	OWNER'S AND CONTRACTOR'S PROTECTIVE LIABILITY INSURANCE	Indirect	158045M	FLOATING TURBIDITY BARRIER, TYPE 2	Indirect
020012A36H	REMOVAL OF ASBESTOS	Indirect	152004P	OWNER'S AND CONTRACTOR'S PROTECTIVE LIAB	Indirect	158048M	FLOATING TURBIDITY BARRIER, TYPE 3	Indirect
020012A60C	CLEARING SITE, TANK REMOVAL	Indirect	152005P	TRACK PROTECTION	Indirect	158051M	DEWATERING BASIN	Indirect
020012D03A	BORROW EXCAVATION, ZONE 3	Indirect	152006P	RAILROAD PROTECTIVE LIABILITY INSURANCE	Indirect	158052P	STREAM DIVERSION SYSTEM	Indirect
020012G04F	PIPE BEDDING, CLASS A	Indirect	152007P	RAILROAD MONITORING	Indirect	158053M	RECHARGE BASIN	Indirect
020012G10G	BROKEN STONE OR WASHED GRAVEL	Indirect	152009P	POLLUTION LIABILITY INSURANCE	Indirect	158054M	SEDIMENT CONTROL BAG	Indirect
020012G23E	ROCK EXCAVATION, SUBSURFACE STRUCTURES	Indirect	152012P	RAILROAD PROTECTIVE LIABILITY INSURANCE	Indirect	158055M	SEDIMENT CONTROL BAG	Indirect
020012L06A	HEAVY DUTY SILT FENCE	Indirect	152015P	POLLUTION LIABILITY INSURANCE	Indirect	158057M	SEDIMENT CONTROL TANK	Indirect
020012L23J	CONCRETE WASHOUT FACILITY	Indirect	153003P	PROGRESS SCHEDULE	Indirect	158059M	CONSTRUCTION DRIVEWAY, WOOD MATS	Indirect
020012L25J	OIL ONLY EMERGENCY SPILL KIT	Indirect	153006P	PROGRESS SCHEDULE UPDATE	Indirect	158060M	CONSTRUCTION DRIVEWAY	Indirect
020013F20E	CONCRETE BASE COURSE, REINFORCED, 8" THI	Indirect	153007M	PROGRESS SCHEDULE UPDATE DAMAGES	Indirect	158061P	STABILIZED ACCESS ROAD	Indirect
020014D14B	HOT MIX ASPHALT SURFACE COURSE MIX I-4 H	Indirect	153009P	BAR CHART PROGRESS SCHEDULE AND UPDATES	Indirect	158063P	CONCRETE WASHOUT SYSTEM	Indirect
020015N63E	TESTING, IF AND WHERE DIRECTED	BRIndirect	153012P	TRAINEES	Indirect	158065P	NOISE CONTROL	Indirect
020015R23C	REPAIR OF CONCRETE DECK, TYPE B	Direct	153013P	TRAINING REIMBURSEMENT	Indirect	158066M	ABSORBENT BOOM	Indirect
020015R25C	REPAIR OF CONCRETE DECK, TYPE C	Direct	154003P	MOBILIZATION	BRIndirect	158067M	BIORETENTION TRENCH	Indirect
020016N24I	GATES, CHAIN-LINK FENCE, 8' WIDE	Indirect	155003M	FIELD OFFICE TYPE A SET UP	BRIndirect	158068P	BIORETENTION SYSTEM	Indirect
020016N24T	TEMPORARY CHAIN-LINK FENCE, 8' HIGH	BRIndirect	155006M	FIELD OFFICE TYPE B SET UP	BRIndirect	158069M	OIL-WATER SEPARATOR	Indirect
020016Q06T	TRAFFIC DIRECTORS, FLAGGERS	Indirect	155009M	FIELD OFFICE TYPE C SET UP	BRIndirect	158072M	OIL ONLY EMERGENCY SPILL KIT, TYPE 1	Indirect
020016Q10F	CONSTRUCTION SIGNS	BRIndirect	155012M	FIELD OFFICE TYPE D SET UP	BRIndirect	158084M	EROSION CONTROL SEDIMENT REMOVAL	Indirect
020016Q20K	TRAFFIC CONTROL TRUCKS WITH MOUNTED	BRIndirect	155015M	FIELD OFFICE TYPE E SET UP	BRIndirect	158087M	TEMPORARY RIPRAP	Direct
020016Q21D	DRUMS	Indirect	155018M	FIELD OFFICE TYPE F SET UP	BRIndirect	158088M	INFILTRATION SAND LAYER, 6" THICK	Indirect
020016Q22B	BREAKAWAY BARRICADES	Indirect	155019M	FIELD OFFICE TYPE G SET UP	BRIndirect	158094M	SLUICE GATE	Indirect
020016Q24I	ILLUMINATED FLASHING ARROWS, 4' X 8'	Indirect	155021M	FIELD OFFICE TYPE A MAINTENANCE	BRIndirect	158097P	SAND BED, 12" THICK	Indirect
020016Q30A	EMERGENCY TOWING SERVICE	Indirect	155024M	FIELD OFFICE TYPE B MAINTENANCE	BRIndirect	158101P	MONITORING FOR TURTLES	Indirect
020016Q31D	TRAFFIC CONES	BRIndirect	155027M	FIELD OFFICE TYPE C MAINTENANCE	BRIndirect	158110P	NEST BOX	Indirect
020016Q32Z	EXTRA ILLUMINATED FLASHING ARROWS, 4' X 8'	Indirect	155030M	FIELD OFFICE TYPE D MAINTENANCE	BRIndirect	159003M	BREAKAWAY BARRICADE	Indirect
020016Q36E	CONSTRUCTION IDENTIFICATION SIGNS, 6' X 12'	Indirect	155033M	FIELD OFFICE TYPE E MAINTENANCE	BRIndirect	159004M	TRAFFIC CONTROL	BRIndirect
020016Q60E	VARIABLE MESSAGE SIGNS	Indirect	155036M	FIELD OFFICE TYPE F MAINTENANCE	BRIndirect	159005M	TRAFFIC CONTROL	BRIndirect
020016R11C	TRAFFIC STRIPES	Indirect	155037M	FIELD OFFICE TYPE G MAINTENANCE	BRIndirect	159006M	DRUM	Indirect
020016R17D	TRAFFIC MARKINGS, SYMBOLS, PREFORMED TAP	Indirect	155039M	TELEPHONE SERVICE	BRIndirect	159007M	EMERGENCY TRAFFIC CONTROL	Indirect
020016R22C	TRAFFIC STRIPES, LONG LIFE, EPOXY RESIN	Indirect	155042M	TELEPHONE SERVICE	BRIndirect	159009M	TRAFFIC CONE	BRIndirect
020016R22D	TRAFFIC MARKINGS, LINES, LONG LIFE,	Indirect	156003M	MATERIALS FIELD LABORATORY SET-UP	Indirect	159011M	BOLLARDS	Indirect
020016R26D	TRAFFIC MARKINGS, SYMBOLS, LONG LIFE,	Indirect	156006M	MATERIALS FIELD LABORATORY MAINTENANCE	Indirect	159012M	CONSTRUCTION SIGNS	BRIndirect
020016V02B	UTILITY DISCONNECT	Indirect	156009M	CURING FACILITY SET-UP	BRIndirect	159014M	CONSTRUCTION IDENTIFICATION SIGN, 6'6" X 7'	BRIndirect
020016V10C	WATER SERVICE CONNECTIONS	Indirect	156012M	CURING FACILITY MAINTENANCE	BRIndirect	159015M	CONSTRUCTION IDENTIFICATION SIGN, 4' X 8'	BRIndirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
159018M	CONSTRUCTION IDENTIFICATION SIGN, 6' X 12'	BRIndirect	159242M	TEMPORARY CRASH CUSHION, LOW MAINTENANCE COMPRESSIVE BARRIER	BRIndirect	203044P	GEOFOAM	Indirect
159020P	CONSTRUCTION BARRIER CURB, BRIDGE	Direct	160003M	FUEL PRICE ADJUSTMENT	Indirect	203050M	CONTROLLED LOW STRENGTH MATERIAL	Indirect
159021P	CONSTRUCTION BARRIER CURB	Indirect	160004M	FUEL PRICE ADJUSTMENT	Indirect	203053M	LIGHTWEIGHT FLOWABLE FILL	Indirect
159022P	CONSTRUCTION HALF BARRIER CURB	Indirect	160006M	ASPHALT PRICE ADJUSTMENT	Indirect	203054M	FLOWABLE CONCRETE FILL	Indirect
159023P	CONSTRUCTION BARRIER CURB, LEFT IN PLACE	Indirect	160007M	ASPHALT PRICE ADJUSTMENT	Indirect	203056P	LIGHTWEIGHT SOIL AGGREGATE	Indirect
159024M	FLASHING ARROW BOARD, 2' X 4'	BRIndirect	161003P	FINAL CLEANUP	Indirect	203066P	SLOPE INCLINOMETER CASING	Indirect
159025M	EXTRA ILLUMINATED FLASHING ARROW BOARD, 4' X 8'	Indirect	162003M	CONDITION SURVEY	Direct	203072P	SETTLEMENT PLATFORM	Indirect
159027M	FLASHING ARROW BOARD, 4' X 8'	BRIndirect	162005P	VIBRATION MONITORING	Direct	203074P	EARTH PRESSURE CELLS	Indirect
159028M	FLOODLIGHTS FOR NIGHTTIME OPERATIONS	BRIndirect	162010P	UTILITY SERVICE CONSTRUCTION	Indirect	203081P	DEFORMATION MONITORING POINT	Indirect
159029M	PORTABLE VARIABLE MESSAGE SIGN W/REMOTE COMMUNICATION	BRIndirect	201003P	CLEARING SITE	Direct	203103P	CLAY LINER	Indirect
159030M	PORTABLE VARIABLE MESSAGE SIGN	BRIndirect	201006P	CLEARING SITE, BRIDGE (___)	Direct	203110P	COLUMN SUPPORTED EMBANKMENT SYSTEM (CSES)	Indirect
159031M	VARIABLE MESSAGE SIGN ASSEMBLY	Indirect	201009P	CLEARING SITE, STRUCTURE (___)	Direct	203111M	DEMONSTRATION STATIC LOAD TEST	Direct
159032M	PORTABLE TRAILER MOUNTED CCTV CAMERA ASSEMBLY	BRIndirect	201012P	CLEARING SITE, TANK REMOVAL	Indirect	205003P	GEOSYNTHETIC EMBANKMENT REINFORCEMENT	Indirect
159033M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 7 MODULES	BRIndirect	201013P	RECONSTRUCTION OF GAS STATION FILLER PAD	Indirect	301006P	SUBBASE	Indirect
159036M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 8 MODULES	BRIndirect	201015M	DISPOSING AND RECYCLING OF CONTAMINATED SOIL - TANKS	Indirect	302012P	SOIL AGGREGATE BASE COURSE, 6" THICK	Indirect
159039M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 9 MODULES	BRIndirect	201018M	MONITORING WELL	Indirect	302028P	SOIL AGGREGATE SURFACE COURSE, TYPE A	Indirect
159042M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 10 MODULES	BRIndirect	201019M	SEALING OF MONITORING WELLS	Indirect	302029P	SOIL AGGREGATE BASE COURSE, TYPE B	Indirect
159045M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 11 MODULES	BRIndirect	201020M	RESET MONITORING WELL BOX	Indirect	302033P	DENSE-GRADED AGGREGATE BASE COURSE, 4" THICK	Indirect
159048M	TEMPORARY CRASH CUSHION, INERTIAL BARRIE	BRIndirect	201021M	POST EXCAVATION SOIL SAMPLING AND ANALYSES	Indirect	302036P	DENSE-GRADED AGGREGATE BASE COURSE, 6" THICK	Indirect
159051M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 13 MODULES	BRIndirect	201024M	COMPOSITE SOIL SAMPLING AND ANALYSES	Indirect	302039P	DENSE-GRADED AGGREGATE BASE COURSE, 7" THICK	Indirect
159054M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 14 MODULES	BRIndirect	201027M	GROUND WATER SAMPLING AND ANALYSES	Indirect	302042P	DENSE-GRADED AGGREGATE BASE COURSE, 8" THICK	Indirect
159057M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 15 MODULES	BRIndirect	201030M	SEALING OF ABANDONED WELL	Indirect	302045P	DENSE-GRADED AGGREGATE BASE COURSE, 10" THICK	Indirect
159058M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM	BRIndirect	201033M	DEMOLITION (___), PARCEL (___)	Indirect	302048P	DENSE-GRADED AGGREGATE BASE COURSE, 12" THICK	Indirect
159059M	TEMPORARY CRASH CUSHION, INTERTIAL BARRIER SYSTEM	BRIndirect	201036M	REMOVAL OF ASBESTOS (___), PARCEL (___)	Indirect	302050P	DENSE-GRADED AGGREGATE BASE COURSE, 15" THICK	Indirect
159060M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 16 MODULES	BRIndirect	201037P	ASBESTOS REMOVAL, BRIDGE NO. ___	Direct	302051P	DENSE-GRADED AGGREGATE BASE COURSE, VARIABLE THICKNESS	Indirect
159061M	TEMPORARY CRASH CUSHION, INERTIAL BARRIER SYSTEM, 17	BRIndirect	201039P	TEMPORARY SHIELDING	Direct	302053P	COARSE GRADE AGGREGATE, SIZE NO. 4	Indirect
159062M	TEMPORARY CRASH CUSHION, QUADGUARD, 2 BAYS X 24" WIDE	Indirect	201050P	HYDRO-DEMOLITION	Indirect	302060P	COARSE AGGREGATE, SIZE NO. 57	Indirect
159063M	TEMPORARY CRASH CUSHION, QUADGUARD 3 BAYS X 24" WIDE	BRIndirect	201060P	SALVAGE OF OPERATOR'S HOUSE	Indirect	303003M	ASPHALT-STABILIZED DRAINAGE COURSE	Indirect
159066M	TEMPORARY CRASH CUSHION, QUADGUARD 4 BAYS X 24" WIDE	BRIndirect	201061P	SALVAGE AND REFURBISH SAFETY GATES	Indirect	303005P	SEPARATION AND FILTRATION GEOTEXTILE	Indirect
159069M	TEMPORARY CRASH CUSHION, QUADGUARD 5 BAYS X 24" WIDE	BRIndirect	202003P	STRIPPING	Indirect	303006M	ASPHALT-STABILIZED DRAINAGE COURSE, MODIFIED	Indirect
159072M	TEMPORARY CRASH CUSHION, QUADGUARD 6 BAYS X 24" WIDE	BRIndirect	202006M	EXCAVATION, TEST PIT	Indirect	303008P	COARSE AGGREGATE STORAGE BED	Indirect
159075M	TEMPORARY CRASH CUSHION, QUADGUARD 7 BAYS X 24" WIDE	BRIndirect	202009P	EXCAVATION, UNCLASSIFIED	Indirect	304002P	CONCRETE BASE COURSE, 6" THICK	Indirect
159078M	TEMPORARY CRASH CUSHION, QUADGUARD 8 BAYS X 24" WIDE	BRIndirect	202013M	ROCK DOWEL	Indirect	304003P	CONCRETE BASE COURSE, 8" THICK	Indirect
159081M	TEMPORARY CRASH CUSHION, QUADGUARD 9 BAYS X 24" WIDE	Indirect	202014M	ROCK DOWEL PULL TEST	Indirect	304006P	CONCRETE BASE COURSE, 9" THICK	Indirect
159105M	TEMPORARY CRASH CUSHION, N.E.A.T.	BRIndirect	202015P	EXCAVATION, REGULATED MATERIAL	Indirect	304008P	CONCRETE BASE COURSE, 10.5" THICK	Indirect
159107M	TEMPORARY CRASH CUSHION, COMPRESSIVE BARRIER, TYPE ___,	BRIndirect	202018P	EXCAVATION, ACID PRODUCING SOIL	Indirect	304009P	CONCRETE BASE COURSE, 10" THICK	Indirect
159108M	TRAFFIC CONTROL TRUCK WITH MOUNTED CRASH CUSHION	BRIndirect	202019M	REMOVAL AND DISPOSAL OF TRANSITE PIPE	Indirect	304010P	CONCRETE BASE COURSE, 11" THICK	Indirect
159110M	TEMPORARY BEAM GUIDE RAIL	BRIndirect	202020P	REMOVAL OF EXISTING DRAIN PIPE	Indirect	304012P	CONCRETE BASE COURSE, 12" THICK	Indirect
159111M	CHANNELIZING GUIDE POST	Indirect	202021P	REMOVAL OF PAVEMENT	Indirect	304030P	REINFORCED CONCRETE GRADE SLAB	Indirect
159114M	REMOVABLE BLACK LINE MASKING TAPE, 6"	Indirect	202024M	DISPOSAL OF REGULATED MATERIAL	Indirect	305003P	RUBBLIZATION	Indirect
159117M	REMOVABLE BLACK LINE MASKING TAPE, 8"	Indirect	202027M	DISPOSAL OF REGULATED MATERIAL, HAZARDOUS	Indirect	305027P	COARSE AGGREGATE, SIZE NO. 2	Indirect
159120M	TEMPORARY PAVEMENT MARKING TAPE, 4"	BRIndirect	202030M	SOIL SAMPLING AND ANALYSES, REGULATED	Indirect	306003P	FULL DEPTH RECLAMATION, CEMENT	Indirect
159123M	TEMPORARY PAVEMENT MARKING TAPE, 6"	BRIndirect	202033M	SOIL SAMPLING AND ANALYSES, ACID PRODUCING SOIL	Indirect	306004M	PORTLAND CEMENT	Indirect
159126M	TEMPORARY TRAFFIC STRIPES, 4"	BRIndirect	202036P	ACID PRODUCING SOIL REMEDIATION	Indirect	401009P	HMA MILLING, 3" OR LESS	Indirect
159129M	TEMPORARY TRAFFIC STRIPES, 6"	BRIndirect	202039M	DISPOSAL OF ACID PRODUCING SOIL	Indirect	401012P	HMA MILLING, MORE THAN 3" TO 6"	Indirect
159132M	TEMPORARY PAVEMENT MARKINGS	BRIndirect	202040M	CONCRETE WALL PROTECTION DURING CONSTRUCTION	Indirect	401014P	HMA MILLING, MORE THAN 6" TO 9"	Indirect
159135M	TEMPORARY PAVEMENT MARKERS	Indirect	202047M	EXCAVATION, ROCK SCALING	Indirect	401015P	CONCRETE MILLING	Indirect
159137P	TEMPORARY STAIRS	Indirect	202050M	SUBSOIL SCARIFICATION	Indirect	401017P	MICRO-MILLING	Indirect
159138M	HMA PATCH	Indirect	202060M	FRAC TANK	Indirect	401018P	HMA PROFILE MILLING	Indirect
159141M	TRAFFIC DIRECTOR, FLAGGER	BRIndirect	202061M	DISPOSAL OF GROUND WATER	Indirect	401021M	HOT MIX ASPHALT PAVEMENT REPAIR	Indirect
159144M	EMERGENCY TOWING SERVICE	Indirect	202062M	GROUND WATER SAMPLING AND ANALYSES, FRAC TANK	Indirect	401022M	GEOTEXTILE, PAVING FABRIC	Indirect
159146M	EMERGENCY TOWING SERVICE, ON CALL, HEAVY DUTY	Indirect	203003P	BREAKING PAVEMENT	Indirect	401024M	SEALING OF CRACKS IN HOT MIX ASPHALT SURFACE COURSE	Indirect
159147M	EMERGENCY TOWING SERVICE, ON CALL, LIGHT DUTY	Indirect	203005P	I-5 SOIL AGGREGATE	Indirect	401027M	POLYMERIZED JOINT ADHESIVE	Direct
159148M	EMERGENCY TOWING SERVICE, ON SITE, LIGHT DUTY	Indirect	203006P	I-7 SOIL AGGREGATE	Indirect	401030M	TACK COAT	Indirect
159149M	SHUTTLE SERVICE	Indirect	203009P	I-9 SOIL AGGREGATE	Indirect	401033M	TACK COAT 64-22	Indirect
159163M	REAL-TIME WORK ZONE TRAFFIC SYSTEM	BRIndirect	203012P	I-10 SOIL AGGREGATE	Indirect	401034M	TACK COAT 76-22	Indirect
159170M	SCHOOL BUS SERVICE	BRIndirect	203018P	I-13 SOIL AGGREGATE	Indirect	401036M	PRIME COAT	Indirect
159175P	CONSTRUCTION BARRIER CURB, MOVABLE SYSTEM	Indirect	203021P	I-14 SOIL AGGREGATE	Indirect	401042M	HOT MIX ASPHALT 9.5 M 64 SURFACE COURSE	Indirect
159190M	SNOW REMOVAL	Indirect	203040M	GEOTEXTILE	Indirect	401048M	HOT MIX ASPHALT 9.5 M 76 SURFACE COURSE	Indirect
159200M	TEMPORARY CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 2, WIDTH	BRIndirect	203041P	GEOTEXTILE, ROADWAY STABILIZATION	Indirect	401051M	HOT MIX ASPHALT 9.5 H 76 SURFACE COURSE	Indirect
159212M	TEMPORARY CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 3, WIDTH	BRIndirect	203043P	GEOGRID REINFORCEMENT	Indirect	401052M	HOT MIX ASPHALT 9.5M76 SURFACE COURSE HIGH RAP	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
401054M	HOT MIX ASPHALT 12.5 M 64 SURFACE COURSE	Indirect	453007M	FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS A	Indirect	502350M	CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	Direct
401055M	HOT MIX ASPHALT 12.5 M 64 SURFACE COURSE HIGH RAP	Indirect	453009M	FULL DEPTH REINFORCED CONCRETE PAVEMENT REPAIR	Indirect	502351M	CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	Direct
401057M	HOT MIX ASPHALT 12.5 H 64 SURFACE COURSE	Indirect	453011M	FULL DEPTH CONCRETE PAVEMENT REPAIR, LMVES	Indirect	502352M	FIBERGLASS PIPE PILE, FURNISHED, 12" DIAMETER	Direct
401060M	HOT MIX ASPHALT 12.5 M 76 SURFACE COURSE	Indirect	454003M	RETROFIT DOWEL BAR	Indirect	502353M	FIBERGLASS PIPE PILE, DRIVEN, 12" DIAMETER	Direct
401063M	HOT MIX ASPHALT 12.5 H 76 SURFACE COURSE	Indirect	455003P	DIAMOND GRINDING EXISTING CONCRETE PAVEMENT	Indirect	503003P	FURNISHING DRILLED SHAFT EQUIPMENT	BRIndirect
401066M	HOT MIX ASPHALT 9.5 M 64 INTERMEDIATE COURSE	Indirect	456003M	SEALING EXISTING JOINTS IN CONCRETE PAVEMENT	Indirect	503006M	DEMONSTRATION DRILLED SHAFT	BRIndirect
401072M	HOT MIX ASPHALT 12.5 M 64 INTERMEDIATE COURSE	Indirect	456006M	CLEANING AND SEALING JOINTS AND CRACKS IN CONCRETE	Indirect	503009M	LOAD TEST	BRIndirect
401075M	HOT MIX ASPHALT 12.5 H 64 INTERMEDIATE COURSE	Indirect	501003P	TEMPORARY SHEETING	Direct	503012M	CROSSHOLE SONIC LOGGING	BRIndirect
401076M	HOT MIX ASPHALT 12.5M64 INTERMEDIATE COURSE HIGH RAP	Indirect	501006P	PERMANENT SHEETING	Direct	503014M	SHAFT INSPECTION DEVICE	BRIndirect
401078M	HOT MIX ASPHALT 12.5 M 76 INTERMEDIATE C	Indirect	501007P	PERMANENT SHEETING, FIBERGLASS	Direct	503015M	SHAFT CORING	BRIndirect
401084M	HOT MIX ASPHALT 19 M 64 INTERMEDIATE COURSE	Indirect	501008P	SHEET PILE WALL	Direct	503017M	DRILLED SHAFT IN SOIL, 30" DIAMETER	Direct
401087M	HOT MIX ASPHALT 19 H 64 INTERMEDIATE COURSE	Indirect	501009P	TEMPORARY COFFERDAM	BRIndirect	503018M	DRILLED SHAFT IN SOIL 36" DIAMETER	Direct
401090M	HOT MIX ASPHALT 19 M 76 INTERMEDIATE COURSE	Indirect	501012P	PERMANENT COFFERDAM	Direct	503019M	DRILLED SHAFT IN SOIL, 42" DIAMETER	Direct
401093M	HOT MIX ASPHALT 19 H 76 INTERMEDIATE COURSE	Indirect	501018P	STAGE LINE EMBANKMENT SUPPORT SYSTEM	Indirect	503021M	DRILLED SHAFT IN SOIL 48" DIAMETER	Direct
401096M	HOT MIX ASPHALT 19 M 64 BASE COURSE	Indirect	501021P	PIER EXCAVATION SUPPORT SYSTEM	BRIndirect	503024M	DRILLED SHAFT IN SOIL 54" DIAMETER	Direct
401099M	HOT MIX ASPHALT 25 M 64 BASE COURSE	Indirect	502002P	FURNISHING EQUIPMENT FOR DRILLING PILES	BRIndirect	503030M	DRILLED SHAFT IN SOIL 72" DIAMETER	Direct
401103M	HOT MIX ASPHALT 25 M 76 BASE COURSE	Indirect	502003P	FURNISHING EQUIPMENT FOR DRIVING PILES	BRIndirect	503031M	DRILLED SHAFT IN SOIL, 96" DIAMETER	Direct
401104M	HOT MIX ASPHALT 19 M 76 BASE COURSE	Indirect	502006M	PREBORED HOLE	Direct	503033M	DRILLED SHAFT IN ROCK 36" DIAMETER	Direct
401105M	SAWING AND SEALING JOINTS IN HOT MIX ASPHALT OVERLAY	Indirect	502009M	TEST PILE, FURNISHED	Direct	503035M	DRILLED SHAFT IN ROCK, 42" DIAMETER	Direct
401108M	CORE SAMPLES, HOT MIX ASPHALT	Indirect	502012M	TEST PILE, DRIVEN	Direct	503036M	DRILLED SHAFT IN ROCK 48" DIAMETER	Direct
401112M	BRIDGE DECK WATERPROOF SURFACE COURSE	Direct	502015M	STATIC PILE LOAD TEST	Direct	503046M	DRILLED SHAFT IN ROCK, 90" DIAMETER	Direct
401115M	HMA AIR VOID QUALITY ADJUSTMENT	Indirect	502018M	DYNAMIC PILE LOAD TEST	Direct	503048M	OBSTRUCTION	BRIndirect
401118M	HMA RIDEABILITY QUALITY ADJUSTMENT	Indirect	502021M	CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	Direct	503051M	TOMOGRAPHY	BRIndirect
401121M	HMA THICKNESS QUALITY ADJUSTMENT	Indirect	502024M	CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	Direct	503055M	DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION	Direct
401125M	WARM MIX ASPHALT 9.5M64 SURFACE COURSE	Indirect	502027M	CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	Direct	503060M	PERMANENT STEEL CASING, 96" DIAMETER	Direct
401127M	WARM MIX ASPHALT 9.5M76 SURFACE COURSE	Indirect	502036M	CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	Direct	504003P	REINFORCEMENT STEEL	Direct
401130M	WARM MIX ASPHALT 12.5M64 SURFACE COURSE	Indirect	502045M	CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	Direct	504006P	REINFORCEMENT STEEL, EPOXY-COATED	Direct
401132M	WARM MIX ASPHALT 12.5M76 SURFACE COURSE	Indirect	502048M	CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	Direct	504008P	REINFORCEMENT STEEL, STAINLESS STEEL	Direct
402009M	MODIFIED OPEN-GRADED 12.5 MM FRICTION COURSE	Indirect	502051M	CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	Direct	504009P	REINFORCEMENT STEEL, GALVANIZED	Direct
404003M	STONE MATRIX ASPHALT 9.5 MM SURFACE COURSE	Indirect	502060M	CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	Direct	504010P	DRILL AND GROUT REINFORCEMENT STEEL	Direct
404004M	STONE MATRIX ASPHALT, RUBBER, 9.5 MM SURFACE COURSE	Indirect	502067P	RESTRIKE CAST-IN-PLACE CONCRETE PILE, 12" DIAMETER	Direct	504012P	CONCRETE CULVERT, STRUCTURES	Direct
404006M	STONE MATRIX ASPHALT 12.5 MM SURFACE COURSE	Indirect	502152M	PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER	Direct	504014P	ROCK ANCHOR	BRIndirect
404010M	WARM STONE MATRIX ASPHALT 12.5MM SURFACE COURSE	Indirect	502155M	PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER	Direct	504015P	CONCRETE FOOTING	Direct
405003P	UNDERLAYER PREPARATION	Indirect	502157M	PRESTRESSED CONCRETE PILES, INSTALLED	Direct	504018P	CONCRETE WING WALL	Direct
405006P	CONCRETE SURFACE COURSE, 8" THICK	Indirect	502165M	STEEL H-PILE, FURNISHED, HP 12 X 53	Direct	504024P	CONCRETE ABUTMENT WALL	Direct
405009P	CONCRETE SURFACE COURSE, 9" THICK	Indirect	502168M	STEEL H-PILE, FURNISHED, HP 12 X 74	Direct	504025P	MODIFICATION OF EXISTING ABUTMENTS	Direct
405012P	CONCRETE SURFACE COURSE, 10" THICK	Indirect	502171M	STEEL H-PILE, FURNISHED, HP 14 X 73	Direct	504026P	CONCRETE PIER COLUMN AND CAP, HPC	Direct
405015P	CONCRETE SURFACE COURSE, 12" THICK	Indirect	502173M	STEEL H-PILE, FURNISHED, HP 14 X 102	Direct	504027P	CONCRETE PIER COLUMN AND CAP	Direct
405018M	CONTRACTION JOINT ASSEMBLY	Direct	502174M	STEEL H-PILE, FURNISHED, HP 14 X 117	Direct	504028P	PIER CAP RECONSTRUCTION	Direct
405021M	EXPANSION JOINT ASSEMBLY	Direct	502183M	STEEL H-PILE, DRIVEN, HP 12 X 53	Direct	504029P	CONCRETE SEAL	Direct
405024M	CORE SAMPLES, CONCRETE	Indirect	502186M	STEEL H-PILE, DRIVEN, HP 12 X 74	Direct	504030P	CONCRETE PIER SHAFT	Direct
405029P	CONCRETE SURFACE COURSE, REINFORCED, 8" THICK	Indirect	502189M	STEEL H-PILE, DRIVEN, HP 14 X 73	Direct	504031P	MODIFICATION OF EXISTING PIERS	Direct
405030P	CONCRETE SURFACE COURSE, REINFORCED, 18" THICK	Indirect	502191M	STEEL H-PILE, DRIVEN, HP 14 X 102	Direct	504032P	CONCRETE DIAPHRAGM, HPC	Direct
406005M	HIGH PERFORMANCE THIN OVERLAY	Indirect	502192M	STEEL H-PILE, DRIVEN, HP 14 X 117	Direct	504033P	CONCRETE PEDESTRIAN BRIDGE	Direct
407003M	HIGH PERFORMANCE THIN OVERLAY	Indirect	502201M	SPLICE CAST-IN-PLACE PILE	Direct	504036P	EPOXY WATERPROOFING	Direct
409003P	BINDER RICH INTERMEDIATE COURSE, 4.75MM	Indirect	502202M	SPLICE PRESTRESSED CONCRETE PILE	Direct	504037P	SPRAY APPLIED WATERPROOFING MEMBRANE	Direct
409004M	BINDER RICH INTERMEDIATE COURSE, 4.75MM	Indirect	502204M	SPLICE STEEL H-PILE	Direct	504038P	MEMBRANE WATERPROOFING	Direct
409005M	ASPHALT RUBBER GAP GRADED SURFACE COURSE	Indirect	502205M	SPLICE CONCRETE FILLED STEEL PIPE PILE	Direct	504040P	CONCRETE SURFACE TREATMENT	Direct
409006M	ASPHALT RUBBER GAP GRADED INTERMEDIATE COURSE	Indirect	502207M	PILE SHOE	Direct	504046P	PAINTING OF CONCRETE SURFACE	Direct
421003M	MICRO-SURFACING AGGREGATE, TYPE II	Indirect	502208M	CONCRETE-FILLED STEEL PIPE PILE, FURNISH	Direct	504047P	CONCRETE STAIN AND ANTI-GRAFFITI TREATME	Direct
421004M	MICRO SURFACING AGGREGATE, TYPE III RUT-FILLING	Indirect	502209M	CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	Direct	504053P	CONCRETE PYLON	Direct
421005M	MICRO-SURFACING EMULSION	Indirect	502300P	VIBRATION MONITORING	BRIndirect	504055P	CONCRETE BEAM	Direct
421010M	SLURRY SEAL AGGREGATE, TYPE II	Indirect	502301P	VIBRATION AND SETTLEMENT MONITORING	BRIndirect	504064P	STONE VENEER	Direct
421011M	SLURRY SEAL EMULSION	Indirect	502307M	RESTRIKE WITH PDA AND CAPWAP ANALYSIS	BRIndirect	504065P	BRICK VENEER	Direct
422003M	FOG SEAL SURFACE TREATMENT	Indirect	502310M	CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	Direct	504067P	FORMLINER	Direct
451006M	SLAB STABILIZATION, POLYURETHANE GROUT	Indirect	502325M	STEEL KING PILES, W44X290, DRIVEN	Direct	504073P	CAST STONE CAP	Direct
452003M	PARTIAL DEPTH CONCRETE REPAIR	Indirect	502326M	STEEL KING PILES, W44X290, FURNISHED	Direct	504075P	ARCHITECTURAL CAST STONE	Direct
452004M	PARTIAL DEPTH CONCRETE REPAIR, HOT APPLIED SYNTHETIC	Indirect	502330M	STEEL KING PILES, W40X167 DRIVEN	Direct	504080P	CONCRETE SPILLWAY	Direct
453003M	FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS B	Indirect	502331M	STEEL KING PILES, W40X167 FURNISHED	Direct	505004P	PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	Direct
453005M	FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	Direct	502340M	STEEL KING PILES, HZ 1180M D-24, DRIVEN	Direct	505006P	PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	Direct
453006M	FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	Direct	502341M	STEEL KING PILES, HZ 1180M D-24, FURNISHED	Direct	505009P	PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	Direct

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
505011P	PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	Direct	507030P	CONCRETE BRIDGE SIDEWALK	Direct	509042P	CHAIN-LINK FENCE, PVC-COATED STEEL, BRIDGE, 6' 3" HIGH	Direct
505015P	PRESTRESSED CONCRETE BOX BEAM, (TYPE BI-36), 36" X 27"	Direct	507031P	CONCRETE CRACK SEAL, SAFETY WALK	Direct	509051P	CHAIN-LINK FENCE, GALVANIZED STEEL, BRID	Direct
505027P	PRESTRESSED CONCRETE BOX BEAM, (TYPE BI-48), 48" X 27"	Direct	507032P	CONCRETE BRIDGE SIDEWALK, HES	Direct	509057P	CHAIN-LINK FENCE, GALVANIZED STEEL, BRIDGE, 6' 3" HIGH	Direct
505030P	PRESTRESSED CONCRETE BOX BEAM, (TYPE BII-48), 48" X 33"	Direct	507033P	CONCRETE BRIDGE SIDEWALK, HPC	Direct	509058P	CHAIN-LINK FENCE, TYPE I, ZINC-COATED STEEL, BRIDGE, 6' 3" H	Direct
505039P	PRESTRESSED CONCRETE SLAB BEAM, (TYPE SII-36), 36" X 15"	Direct	507034P	CONCRETE BARRIER CURB	Direct	509065P	CHAIN-LINK FENCE, TYPE I, ZINC-COATED, STEEL, BRIDGE, 13'-0"	Direct
505045P	PRESTRESSED CONCRETE SLAB BEAM, (TYPE SIV-36), 36" X 21"	Direct	507036P	CONCRETE BRIDGE PARAPET	Direct	509078P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 3"	Direct
505048P	PRESTRESSED CONCRETE SLAB BEAM, (TYPE SII-48), 48" X 15"	Direct	507037P	BARRIER PARAPET MODIFICATIONS	Direct	509079P	CHAIN-LINK FENCE, TYPE II, ALUMINUM-COATED STEEL, BRIDGE, 6'	Direct
505054P	PRESTRESSED CONCRETE SLAB BEAM, (TYPE SIV-48), 48" X 21"	Direct	507038P	CONCRETE BRIDGE PARAPET WITH MOMENT SLAB, HPC	Direct	509083P	CHAIN-LINK FENCE, TYPE IV	Direct
505055P	PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	Direct	507039P	CONCRETE BRIDGE PARAPET, HPC	Direct	509084P	CHAIN-LINK FENCE, PVC-COATED STEEL, BRIDGE, 6' 3" HIGH, CURV	Direct
505057P	PRECAST CONCRETE CULVERT	Direct	507040P	CONCRETE BRIDGE PARAPET, HES	Direct	509085P	PICKET FENCE, STEEL, BRIDGE, 6' 3" HIGH, CURVED TOP	Direct
505058P	PRECAST CONCRETE CULVERT MODIFICATIONS	Direct	507042P	4-BAR OPEN STEEL PARAPET	Direct	509086P	PICKET FENCE, STEEL, BRIDGE, 4' 0" HIGH	Direct
505060P	PRECAST CONCRETE ARCH STRUCTURE	Direct	507046M	15" BY 32" CONCRETE BARRIER CURB, BRIDGE, HPC	Direct	509096P	CHAIN-LINK FENCE, GALVANIZED STEEL, BRIDGE, 6' 3" HIGH,	Direct
505061P	PREFABRICATED SUBSTRUCTURE UNITS	Direct	507048M	24" BY 32" CONCRETE BARRIER CURB, BRIDGE	Direct	509097P	CHAIN-LINK FENCE,TYPE I,ZINC-COATED STEEL,BRIDGE,6' 3" HIGH,	Direct
505063P	PREFABRICATED SUPERSTRUCTURE UNITS	Direct	507050M	CONCRETE SLEEPER SLAB	Direct	509100P	ORNAMENTAL RAILING	Direct
505064P	PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	Direct	507051P	CONCRETE BRIDGE APPROACH	Direct	509101P	PIPE RAIL	Direct
505072P	GIRDER JACKING	Direct	507052M	CONCRETE MOMENT SLAB	Direct	509102P	PICKET FENCE, STEEL, BRIDGE, 6' 3" HIGH	Direct
505075P	CONCRETE STEPS, PRECAST CONCRETE	Indirect	507053P	BEAM JACKING	BRIndirect	509111P	RELOCATE CHAIN-LINK FENCE, TYPE I, ZINC-COATED STEEL, BRIDGE	Direct
505084P	PRECAST PIER	Direct	507054P	CONCRETE BRIDGE RELIEF SLAB, HPC	Direct	509120P	FISH LADDER	Direct
505088P	PRECAST PARAPET PANEL	Direct	507055P	FIVE BAR OPEN STEEL PARAPET	Direct	509123P	FIBERGLASS REINFORCED PLASTIC GRATING	Direct
505090P	PRESTRESSED CONCRETE DECK PANELS, HPC	Direct	507056P	CONCRETE MEDIAN BARRIER, HES	Direct	509127P	CHAIN-LINK FENCE, TYPE IV, PVC-COATED STEEL, BRIDGE, 6' 3" H	Direct
505091P	PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	Direct	507058P	CONCRETE MEDIAN SLAB, HPC	Direct	509131P	METAL MEDIAN BARRIER	Direct
505094P	PRECAST CONCRETE STRUCTURE	Direct	507059P	CONCRETE MEDIAN BARRIER, HPC	Direct	509132P	METAL HALF MEDIAN BARRIER	Direct
505096P	TEMPORARY PRECAST CONCRETE SLAB	BRIndirect	507060P	BEAM JACKING	BRIndirect	510011P	TIMBER WALL MODIFICATION	BRIndirect
506003P	STRUCTURAL STEEL	Direct	507062M	CAST-IN-PLACE EXODERMIC BRIDGE DECK SYSTEM, HPC	Direct	511006P	STEEL SHEET PILING	Direct
506004M	STRUCTURAL STEEL	Direct	507065P	CONCRETE CAST-IN-PLACE SLABS	Direct	511012M	COMPOSITE PILE, ___ INCH DIAMETER	Direct
506005P	STRUCTURAL BEARING ASSEMBLY, SEISMIC	Direct	507066P	PRECAST CONCRETE BRIDGE APPROACH	Direct	511015P	FIBERGLASS REINFORCED PLASTIC LUMBER	Direct
506006P	REINFORCED ELASTOMERIC BEARING ASSEMBLY	Direct	507067P	CONCRETE BALUSTRADE	Direct	511019M	TIDE CLEARANCE GAUGE	Direct
506008P	RESET BEARING	Direct	507070M	BRIDGE DECK WATERPROOF SURFACE COURSE	Direct	511020P	FENDER SYSTEM	Direct
506009M	STRUCTURAL BEARING ASSEMBLY	Direct	507073M	DIAMOND GRINDING, CONCRETE DECK SURFACE	Direct	511023P	FALL PROTECTION SYSTEM	Direct
506010M	HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	Direct	507095P	PREFORMED JOINT FILLER ASSEMBLY	Direct	511025M	TIE-ROD SYSTEM	Direct
506011M	ANCHOR BOLT INSTALLATION	Direct	507096P	SLIDING PLATE EXPANSION JOINT ASSEMBLY	Direct	512003M	CANTILEVER SIGN SUPPORT, STRUCTURE NO. ___	Direct
506012P	SHEAR CONNECTOR	Direct	507101P	CONCRETE CLOSURE POUR	Direct	512004P	RELOCATE CANTILEVER SIGN SUPPORT, STRUCTURE NO. ___	Direct
506015P	SHEAR CONNECTOR, GALVANIZED	Direct	507102M	LATEX MODIFIED CONCRETE OVERLAY	Direct	512006M	BRIDGE MOUNTED SIGN SUPPORT, STRUCTURE NO. ___	Direct
506016P	GIRDER JACKING	Direct	507103P	RAPID SETTING LATEX MODIFIED CONCRETE OVERLAY	Direct	512007P	REMOVE/REINSTALL EXISTING BRIDGE MOUNTED SIGN	Direct
506018P	STEEL PEDESTRIAN BRIDGE	Direct	507123P	CONCRETE BRIDGE DECK, UHPC	Direct	512009M	BUTTERFLY SIGN SUPPORT, STRUCTURE	Direct
506021P	STEEL GRID FLOORING	Direct	508003M	INLET FRAME AND GRATE	Direct	512012M	OVERHEAD SIGN SUPPORT, STRUCTURE NO. ___	Direct
506024P	MECHANICAL CONNECTOR	Direct	508004M	NEW SCUPPER IN EXISTING DECK	Direct	513003P	RETAINING WALL, LOCATION NO. ___	Direct
506040P	STEEL REPAIR, TYPE ___	Direct	508005M	CLEAN EXISTING SCUPPERS AND PIPES	Direct	513006P	RETAINING WALL, CAST-IN-PLACE, LOCATION NO. ___	Direct
506041P	STRUCTURAL STEEL REPAIR, TYPE 1	Direct	508006M	SCUPPER	Direct	513007P	STAGE LINE MSE RETAINING WALL	Direct
506042P	STRUCTURAL STEEL REPAIR, TYPE 2	Direct	508007M	SCUPPER RESET	Direct	513008P	RETAINING SYSTEM	Direct
506070P	TEMPORARY BEAM	BRIndirect	508008P	6" STEEL ALLOY PIPE	Direct	513009M	COARSE AGGREGATE LAYER	Direct
507001M	CONCRETE RIDEABILITY QUALITY ADJUSTMENT	BRIndirect	508009P	8" STEEL ALLOY PIPE	Direct	513015P	LANDSCAPE RETAINING WALL	Direct
507002P	ELASTOMERIC CONCRETE BRIDGE JOINT SYSTEM	Direct	508012P	10" STEEL ALLOY PIPE	Direct	513022P	CONCRETE COPING	Direct
507003P	1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	Direct	508017P	STANDPIPE	Direct	514003P	TEMPORARY STRUCTURE, ONE-WAY	BRIndirect
507004P	1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	Direct	508018M	STANDPIPE	Direct	514006P	TEMPORARY STRUCTURE, TWO-WAY	BRIndirect
507006P	2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	Direct	508020P	MANHOLE ON STRUCTURE	Direct	514008P	TEMPORARY STRUCTURE, WALKWAY	BRIndirect
507007P	2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	Direct	508900P	FIBERGLASS DRAIN PIPE	Direct	514009P	TEMPORARY STRUCTURE, PEDESTRIAN BRIDGE	BRIndirect
507008P	4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	Direct	508902P	_____" FIBERGLASS PIPE	Direct	514012P	TEMPORARY WALL	Indirect
507009P	4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	Direct	509003P	BRIDGE RAILING (1 RAIL, ALUMINUM)	Direct	514014P	REMOVAL OF TEMPORARY SUPPORT	BRIndirect
507014P	NEOPRENE STRIP SEAL GLAND	Direct	509006P	BRIDGE RAILING (2 RAIL, ALUMINUM)	Direct	514015P	TEMPORARY SUPPORT	BRIndirect
507015P	STRIP SEAL EXPANSION JOINT ASSEMBLY	Direct	509007P	ALUMINIUM RAILING, BRIDGE, 5'-6" HIGH	Direct	514016P	TEMPORARY SUPPORTS	BRIndirect
507016P	FINGER JOINT EXPANSION DAM	Direct	509008P	ALUMINIUM RAILING, BRIDGE, 7'-0" HIGH	Direct	514019P	TEMPORARY SIDEWALK	BRIndirect
507018P	MODULAR EXPANSION JOINT ASSEMBLY	Direct	509009P	BRIDGE RAILING (1 RAIL, STEEL)	Direct	514021P	CONSTRUCTION ACCESS	BRIndirect
507020P	ASPHALTIC BRIDGE JOINT SYSTEM	Direct	509010P	STEEL BRIDGE RAILIN, TWO-RAIL	Direct	515005M	STONE POST	BRIndirect
507021P	CONCRETE BRIDGE DECK	Direct	509011P	STEEL BRIDGE RAILING, THREE-RAIL	Direct	516003P	PRECAST EXODERMIC BRIDGE DECK SYSTEM	Direct
507022P	CONCRETE BRIDGE SEATS, HES	Direct	509012P	BRIDGE RAILING (2 RAIL, STEEL)	Direct	516004P	PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	Direct
507023P	CONCRETE BRIDGE APPROACH, HES	Direct	509013P	2-BAR STEEL BRIDGE RAILING	Direct	517003P	HYBRID-COMPOSITE BEAMS, FURNISHING AND TESTING	Direct
507024P	CONCRETE BRIDGE DECK, HPC	Direct	509024P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 4' 0" HIGH	Direct	517006P	HYBRID-COMPOSITE BEAMS, ERECTING	Direct
507025P	CONCRETE BRIDGE DECK, HES	Direct	509030P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 0" HIGH	Direct	518009P	TEMPORARY JACKING SYSTEM	BRIndirect
507027M	DATE PANEL	Direct	509033P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 3" HIGH	Direct	518014P	SPAN LOCK	BRIndirect
507028M	ENCASEMENT CONCRETE	Direct	509039P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 8' 6" HIGH	Direct	518015P	SPAN BALANCE	BRIndirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
518016P	SPAN DRIVE MACHINERY REHABILITATION	BRIndirect	555042P	SUPPORT AND PROTECTION OF EXPOSED ELECTRICAL WIRE	BRIndirect	601192P	12" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518017P	BARRIER GATE FOUNDATION	BRIndirect	555043P	SUPPORT AND PROTECTION OF FLEXIBLE CONDUIT	BRIndirect	601194P	15" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518019P	BARRIER GATE PLATFORM	BRIndirect	556006P	FURNISH EQUIPMENT FOR SUPERSTRUCTURE REMOVAL AND ERECTION	BRIndirect	601196P	18" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518022P	WARNING GATE PLATFORM	BRIndiect	556012P	PRESSURE WASHING AND FINISHING OF SUBSTRUCTURE CONCRETE	BRIndirect	601200P	24" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518030P	CONTROL DESK MODIFICATIONS	Indirect	557004M	STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	Direct	601204P	30" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518043P	GROUNDING AND BONDING SYSTEM	Indirect	557007P	FLOORBEAM REPAIR, __ VIADUCT, TYPE FB1	Direct	601206P	36" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518053M	MOTOR DISCONNECT SWITCH	Indirect	557008M	FLOORBEAM REPAIR, __ VIADUCT, TYPE FB2, IF AND WHERE DIRECTE	Direct	601208P	42" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518065P	TEMPORARY BRIDGE OPERATION	BRIndiect	557012P	BRACING REPLACEMENT, TYPE BR1	Direct	601210P	48" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518066M	TEMPORARY BRIDGE OPERATION	BRIndiect	557013P	BRACING REPLACEMENT, TYPE BR2	Direct	601214P	60" REINFORCED CONCRETE PIPE, CLASS V	Indirect
518113P	OPERATOR HOUSE EXPANSION	BRIndiect	557014P	BRACING REPLACEMENT, TYPE BR3	Direct	601243P	54" REINFORCED CONCRETE PIPE, USING ALTERNATE METHODS	Indirect
520003P	PERMANENT GROUND ANCHOR	Direct	557018P	TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR1	Direct	601244P	42" REINFORCED CONCRETE PIPE, USING ALTERNATE METHODS	Indirect
520006P	GROUND ANCHOR PERFORMANCE LOAD TEST	BRIndiect	557019P	TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR2	Direct	601245P	60" REINFORCED CONCRETE PIPE, USING ALTERNATE METHODS	Indirect
551001M	DECK EDGE STABILIZATION	Direct	557020P	TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR3	Direct	601248P	15" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551002M	CONCRETE DECK CRACK REPAIR	Direct	557021M	TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR4	Direct	601249P	6" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551003M	REPAIR OF CONCRETE DECK, TYPE A	Direct	557022M	TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR5, IF AND WHERE DIRECT	Direct	601250P	18" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551006M	REPAIR OF CONCRETE DECK, TYPE B	Direct	558003P	DECK SLURRY OVERLAY SYSTEM	Direct	601251P	42" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551007M	REPAIR OF CONCRETE DECK, TYPE B1	Direct	558005P	RIVET REPLACEMENT	Direct	601252P	21" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551009M	REPAIR OF CONCRETE DECK, TYPE C	Direct	558010P	OPERATOR'S AND GATE HOUSE CARPENTRY	BRIndirect	601253P	48" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551012P	SCARIFICATION	BRIndirect	559003P	SUBSTRUCTURE CONCRETE REPAIR	Direct	601254P	24" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551014P	DECK JOINT RESEAL	Direct	559005P	ABUTMENT POCKET	BRIndirect	601255P	54" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551015M	DECK JOINT RECONSTRUCTION	Direct	562003P	OPERATOR'S AND GATE HOUSE FINISHES	BRIndirect	601256P	27" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551017M	CONCRETE OVERLAY, HPC	Direct	601003M	VIDEO INSPECTION OF PIPE	Indirect	601258P	30" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551018M	CRACK SPANNING MEMBRANE	Direct	601014P	15" CORRUGATED ALUMINUM ALLOY PIPE	Indirect	601259P	36" HIGH DENSITY POLYETHYLENE PIPE	Indirect
551019M	POLYESTER POLYMER CONCRETE OVERLAY	BRIndirect	601016P	18" CORRUGATED ALUMINUM ALLOY PIPE	Indirect	601262M	15" CORRUGATED ALUMINUM ALLOY END SECTION	Indirect
551021M	HEADER RECONSTRUCTION	Direct	601020P	24" CORRUGATED ALUMINUM ALLOY PIPE	Indirect	601332M	12" CORRUGATED METAL END SECTION	Indirect
551022M	HEADER RECONSTRUCTION	Direct	601022P	27" CORRUGATED ALUMINUM ALLOY PIPE	Indirect	601334M	15" CORRUGATED METAL END SECTION	Indirect
551027M	HEADER RECONSTRUCTION, VESLMC	BRIndirect	601028P	36" CORRUGATED ALUMINUM ALLOY PIPE	Indirect	601340M	24" CORRUGATED METAL END SECTION	Indirect
551030M	CURB RECONSTRUCTION, BRIDGE	Direct	601047P	8" CORRUGATED STEEL PIPE	Indirect	601368M	12" REINFORCED CONCRETE END SECTION	Indirect
551032M	CONCRETE BRIDGE DECK, PPC	BRIndirect	601048P	12" CORRUGATED STEEL PIPE	Indirect	601370M	15" REINFORCED CONCRETE END SECTION	Indirect
551033M	CONCRETE BRIDGE SIDEWALK REPAIR	Direct	601050P	15" CORRUGATED STEEL PIPE	Indirect	601372M	18" REINFORCED CONCRETE END SECTION	Indirect
551035M	MEDIAN RECONSTRUCTION, BRIDGE	BRIndirect	601086P	15" CORRUGATED METAL PIPE	Indirect	601374M	21" REINFORCED CONCRETE END SECTION	Indirect
551045M	PARAPET MODIFICATIONS	Direct	601088P	18" CORRUGATED METAL PIPE	Indirect	601376M	24" REINFORCED CONCRETE END SECTION	Indirect
551070M	MISCELLANEOUS CONCRETE	Direct	601092P	24" CORRUGATED METAL PIPE	Indirect	601380M	30" REINFORCED CONCRETE END SECTION	Indirect
552003M	PRESSURE INJECTION, CONCRETE CRACKS	Direct	601096P	30" CORRUGATED METAL PIPE	Indirect	601382M	36" REINFORCED CONCRETE END SECTION	Indirect
553003M	PNEUMATICALLY APPLIED MORTAR	Indirect	601106P	54" CORRUGATED METAL PIPE	Indirect	601404P	SUBBASE OUTLET DRAIN	Indirect
554003P	POLLUTION CONTROL SYSTEM	BRIndirect	601120P	12" REINFORCED CONCRETE PIPE	Indirect	601405P	BASIN UNDERDRAIN	Indirect
554006P	HAND/POWER TOOL CLEANING AND PAINTING	Indirect	601122P	15" REINFORCED CONCRETE PIPE	Indirect	601407P	6" PERFORATED HIGH DENSITY POLYETHYLENE PIPE	Indirect
554009P	NEAR-WHITE BLAST CLEANING AND PAINTING	Direct	601124P	18" REINFORCED CONCRETE PIPE	Indirect	601408P	6" CORRUGATED STEEL UNDERDRAIN PIPE	Indirect
554010P	NEAR-WHITE BLAST CLEANING AND PAINTING,	Direct	601126P	21" REINFORCED CONCRETE PIPE	Indirect	601410P	8" CORRUGATED STEEL UNDERDRAIN PIPE	Direct
554012P	TESTING, IF AND WHERE DIRECTED	BRIndirect	601128P	24" REINFORCED CONCRETE PIPE	Indirect	601412P	6" CORRUGATED ALUMINUM ALLOY UNDERDRAIN PIPE	Indirect
554016P	CONCRETE ENCASEMENT REMOVAL AND PAINTING	Direct	601130P	27" REINFORCED CONCRETE PIPE	Indirect	601413P	12" PERFORATED HIGH DENSITY POLYETHYLENE PIPE	Indirect
554017P	CONCRETE ENCASEMENT REMOVAL AND PAINTING (BEAMS)	Direct	601132P	30" REINFORCED CONCRETE PIPE	Indirect	601415P	UNDERDRAIN, TYPE Y	Indirect
554018P	CONCRETE ENCASEMENT REMOVAL AND PAINTING (EXPANSION	Direct	601133P	33" REINFORCED CONCRETE PIPE	Indirect	601416P	UNDERDRAIN, TYPE F	Indirect
554019P	CONCRETE ENCASEMENT REMOVAL AND PAINTING	Direct	601134P	36" REINFORCED CONCRETE PIPE	Indirect	601417P	UNDERDRAIN, TYPE X	Indirect
554027P	CLEANING AND PAINTING OF BEARING	Direct	601135P	36" PERFORATED REINFORCED CONCRETE PIPE	Indirect	601562M	14" X 23" REINFORCED CONCRETE END SECTION	Indirect
554040P	SOUND ABATEMENT	BRIndirect	601136P	42" REINFORCED CONCRETE PIPE	Indirect	601570M	27" X 42" REINFORCED CONCRETE END SECTION	Indirect
555003M	SUBSTRUCTURE CONCRETE REPAIR	Direct	601137P	42" PERFORATED REINFORCED CONCRETE PIPE	Indirect	601578M	38" X 60" REINFORCED CONCRETE END SECTION	Indirect
555006M	BRIDGE DECK WATERPROOF SURFACE COURSE	Direct	601138P	48" REINFORCED CONCRETE PIPE	Indirect	601582M	48" X 76" REINFORCED CONCRETE END SECTION	Indirect
555008P	CULVERT REPAIR, TYPE ___	Direct	601140P	54" REINFORCED CONCRETE PIPE	Indirect	601596P	14" X 23" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555009M	CONCRETE SPALL REPAIR, TYPE 1	Direct	601142P	60" REINFORCED CONCRETE PIPE	Indirect	601598P	19" X 30" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555010M	REPAIR OF CONCRETE, TYPE E	Direct	601144P	66" REINFORCED CONCRETE PIPE	Indirect	601600P	22" X 34" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555011M	REPAIR OF CONCRETE, TYPE D	Direct	601146P	72" REINFORCED CONCRETE PIPE	Indirect	601602P	24" X 38" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555012M	CONCRETE SPALL REPAIR, TYPE 2	Direct	601148P	78" REINFORCED CONCRETE PIPE	Indirect	601604P	27" X 42" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555013M	CONCRETE SPALL REPAIR	Direct	601150P	84" REINFORCED CONCRETE PIPE	Indirect	601606P	29" X 45" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555015M	SUPERSTRUCTURE CONCRETE REPAIR	Direct	601152P	90" REINFORCED CONCRETE PIPE	Indirect	601608P	32" X 49" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555020M	SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	Direct	601154P	96" REINFORCED CONCRETE PIPE	Indirect	601610P	34" X 53" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555023P	PIER RECONSTRUCTION	Direct	601156P	12" REINFORCED CONCRETE PIPE, CLASS IV	Indirect	601612P	38" X 60" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555025P	RETROFIT STRIP SEAL JOINT SYSTEM	Direct	601158P	15" REINFORCED CONCRETE PIPE, CLASS IV	Indirect	601614P	43" X 68" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555035M	MASONRY REPOINTING	Direct	601160P	18" REINFORCED CONCRETE PIPE, CLASS IV	Indirect	601616P	48" X 76" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - III	Indirect
555040P	SUPPORT AND PROTECTION OF RIGID CONDUIT	Direct	601164P	24" REINFORCED CONCRETE PIPE, CLASS IV	Indirect	601630P	14" X 23" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect
555041P	SUPPORT AND PROTECTION OF TRAFFIC SIGNAL HEADS	BRIndirect	601168P	30" REINFORCED CONCRETE PIPE, CLASS IV	Indirect	601632P	19" X 30" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
601634P	22" X 34" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect	602051M	INLET, TYPE B-2 MODIFIED	Indirect	602235M	SPECIAL DRAINAGE STRUCTURE	Indirect
601636P	24" X 38" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect	602052M	INLET, TYPE Y	Indirect	602270M	SET INLET TYPE X, CASTING	Indirect
601638P	27" X 42" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect	602053M	MANHOLE, TYPE MH-1	Indirect	602271M	SET INLET TYPE Y, CASTING	Indirect
601640P	29" X 45" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - IV	Indirect	602054M	MANHOLE, 4' DIAMETER	Indirect	602280M	INLET, TYPE BX	Indirect
601650P	48" X 76" REINFORCED CONCRETE ELLIPTICAL	Indirect	602055M	MANHOLES	Indirect	602281M	INLET, TYPE CX	Indirect
601663P	19" X 30" REINFORCED CONCRETE ELLIPTICAL PIPE, CLSS HE - V	Indirect	602056M	MANHOLE	Indirect	602282M	INLET, TYPE EX	Indirect
601664M	VIDEO INSPECTION OF PIPE	Indirect	602057M	MANHOLE, 5' DIAMETER	Indirect	602290M	INLET, NON-STANDARD	Indirect
601665P	29"X45" REINFORCED CONCRETE ELLIPTICAL PIPE, CLASS HE V	Indirect	602058M	MANHOLE, 8' DIAMETER	Indirect	602291M	18" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601668M	CLEANING EXISTING PIPE, 10" DIAMETER	Indirect	602060M	MANHOLE, 6' DIAMETER	Indirect	602292M	24" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601670M	CLEANING EXISTING PIPE, 12" TO 24" DIAMETER	Indirect	602066M	INLET, TYPE B, USING EXISTING CASTING	Indirect	602293M	30" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601672M	CLEANING EXISTING PIPE, OVER 24" TO 48" DIAMETER	Indirect	602093M	MANHOLE, USING EXISTING CASTING	Indirect	602294M	36" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601674M	CLEANING EXISTING PIPE, OVER 48" TO 72" DIAMETER	Indirect	602095M	MANHOLE, 7' DIAMETER	Indirect	602295M	42" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601676M	CLEANING EXISTING PIPE, OVER 72" TO 96" DIAMETER	Indirect	602096M	INLET CONVERTED TO MANHOLE	Indirect	602296M	54" TIDE CONTROL CHECK VALVE, INLINE	Indirect
601678M	CLEANING EXISTING PIPE, 102" DIAMETER	Indirect	602099M	RESET EXISTING CASTING	Indirect	602297M	18" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601679M	8" DUCTILE IRON PIPE	Indirect	602100M	INLET, TYPE DOUBLE B2R	Indirect	602298M	30" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601680M	16" DUCTILE IRON PIPE	Indirect	602102M	SET INLET TYPE A, CASTING	Indirect	602299M	36" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601681M	18" DUCTILE IRON PIPE	Indirect	602105M	SET INLET TYPE B, CASTING	Indirect	602300M	42" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601682M	24" DUCTILE IRON PIPE	Indirect	602108M	SET INLET TYPE E, CASTING	Indirect	602301M	48" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601683M	30" DUCTILE IRON PIPE	Indirect	602109M	SET INLET TYPE D-1, CASTING	Indirect	602302M	54" TIDE CONTROL CHECK VALVE, SLIPON	Indirect
601684M	12" DUCTILE IRON PIPE	Indirect	602111M	SET INLET TYPE ES, CASTING	Indirect	602306M	30" TIDEFLEX CHECK VALVE	Indirect
601685M	14" DUCTILE IRON PIPE	Indirect	602114M	SET MANHOLE CASTING	Indirect	602310M	42" TIDEFLEX CHECK VALVE	Indirect
601686M	CURED-IN-PLACE PIPE, __" X __"	Indirect	602115M	COMPOSITE ADJUSTMENT RISERS	Indirect	602311M	48" TIDEFLEX CHECK VALVE	Indirect
601698M	CURED-IN-PLACE PIPE, 42"	Indirect	602117M	SET SQUARE FRAMED MANHOLE CASTING, CIRCULAR COVER	Indirect	602320M	MANHOLE, TYPE NS-1	Indirect
601700M	CURED-IN-PLACE PIPE, 54"	Indirect	602120M	RECONSTRUCTED INLET, TYPE A, USING EXIST	Indirect	603003P	CONCRETE SLOPE GUTTER, 4" THICK	Indirect
601703P	UNDERDRAIN, TYPE F MODIFIED	Indirect	602123M	RECONSTRUCTED INLET, TYPE B, USING EXISTING CASTING	Indirect	603009P	CONCRETE SLOPE GUTTER, 8" THICK	Indirect
601705P	42" HIGH DENSITY POLYETHYLENE PIPE	Indirect	602129M	RECONSTRUCTED INLET, TYPE E, USING EXISTING CASTING	Indirect	603012P	CONCRETE SLOPE PROTECTION, 4" THICK	Indirect
601708P	60" HIGH DENSITY POLYETHYLENE PIPE	Indirect	602138M	RECONSTRUCTED INLET, TYPE D-1, USING EXISTING CASTING	Indirect	603015P	CONCRETE SLOPE PROTECTION, REINFORCED, 4	Indirect
601712P	CONCRETE ENCASEMENT	Indirect	602148M	RECONSTRUCTED INLET, TYPE___, EXIST CAST	Indirect	603016P	STONE SLOPE PROTECTION	Direct
601720P	6" POLYVINYL CHLORIDE PIPE	Indirect	602149M	RECONSTRUCTED INLET, TYPE___, USING NEW CASTING	Indirect	603017P	RIP RAP STONE SLOPE PROTECTION, 12" THICK (D50=6")	Indirect
601722P	8" POLYVINYL CHLORIDE PIPE	Indirect	602150M	RECONSTRUCTED INLET, TYPE A, USING NEW CASTING	Indirect	603018P	RIPRAP STONE SLOPE PROTECTION, 16" THICK (D50=8")	Indirect
601741M	36" DUCTILE IRON PIPE	Indirect	602153M	RECONSTRUCTED INLET, TYPE B, USING NEW CASTING	Indirect	603021P	RIPRAP STONE SLOPE PROTECTION, 18" THICK (D50=9")	Indirect
601742M	42" DUCTILE IRON PIPE	Indirect	602154M	RECONSTRUCTED INLET, TYPE B MODIFIED, USING NEW CASTING	Indirect	603024P	RIPRAP STONE SLOPE PROTECTION, 24" THICK (D50=12")	Indirect
601743M	48" DUCTILE IRON PIPE	Indirect	602155M	RECONSTRUCTED INLET, TYPE B MODIFIED, USING EXISTING CASTING	Indirect	603033P	RIPRAP STONE SLOPE PROTECTION, 36" THICK (D50=18")	Indirect
601760P	PIPE BEDDING	Indirect	602159M	RECONSTRUCTED INLET, TYPE E, USING NEW CASTING	Indirect	603036P	RIPRAP STONE CHANNEL PROTECTION, 12" THICK (D50=6")	Indirect
601770M	15" CORRUGATED METAL SLOTTED DRAIN PIPE, VARIABLE HEIGHT	Indirect	602160M	RECONSTRUCTED INLET, TYPE ES, USING NEW CASTING	Indirect	603039P	RIPRAP STONE CHANNEL PROTECTION, 16" THICK (D50=8")	Indirect
602002P	PRECAST CONCRETE CULVERT CHANNEL, TYPE 2	Indirect	602168M	RECONSTRUCTED INLET, TYPE D-1, USING NEW CASTING	Indirect	603041P	RIPRAP STONE CHANNEL PROTECTION, 18" THICK (D50=6")	Indirect
602004P	PRECAST CONCRETE CULVERT CHANNEL, TYPE 1	Indirect	602171M	RECONSTRUCTED INLET, TYPE D-2, USING NEW	Indirect	603042P	RIPRAP STONE CHANNEL PROTECTION, 18" THICK (D50=9")	Indirect
602005P	STONE HEADWALL	Indirect	602178M	RECONSTRUCTED INLET, TYPE S-___, USING NEW CASTING	Indirect	603045P	RIPRAP STONE CHANNEL PROTECTION, 20" THICK (D50=10"	Indirect
602006P	CONCRETE HEADWALL	Indirect	602180M	RECONSTRUCTED MANHOLE, USING EXISTING CASTING	Indirect	603048P	RIPRAP STONE CHANNEL PROTECTION, 24" THICK (D50=12")	Indirect
602007M	TIDAL CHECK VALVE	Indirect	602183M	RECONSTRUCTED MANHOLE, USING NEW CASTING	Indirect	603049P	RIPRAP STONE CHANNEL PROTECTION, 27" THICH (D50=9")	Indirect
602008M	FLOW CONTROL STRUCTURE	Indirect	602186M	EXTENSION FRAME FOR EXISTING INLET, TYPE A	Indirect	603054P	RIPRAP STONE CHANNEL PROTECTION, 36" THI	Indirect
602009M	INLET, TYPE A	Indirect	602189M	EXTENSION FRAME FOR EXISTING INLET, TYPE B	Indirect	603055P	RIPRAP STONE CHANNEL PROTECTION, 42" THICK (D50=21")	Indirect
602011M	SPECIAL INLET, TYPE B	Indirect	602192M	EXTENSION FRAME FOR EXISTING INLET, TYPE C	Indirect	603056P	RIPRAP STONE CHANNEL PROTECTION, 48" THICK (D50=12")	Indirect
602012M	INLET, TYPE B	Indirect	602193M	EXTENSION FRAME FOR EXISTING INLET, TYPE D	Indirect	603057P	ROCK BACKFILL	Indirect
602013M	INLET, TYPE DOUBLE B	Indirect	602195M	EXTENSION FRAME FOR EXISTING INLET, TYPE	Indirect	603059P	ROCKFALL DRAPE NET PROTECTION	Indirect
602018M	INLET, TYPE E	Indirect	602198M	EXTENSION RING FOR EXISTING MANHOLE, 4' DIAMETER	Indirect	603060P	WIRE MESH SLOPE PROTECTION	Indirect
602019M	INLET, TYPE DOUBLE E	Indirect	602201M	EXTENSION RING FOR EXISTING MANHOLE, 5' DIAMETER	Indirect	603061M	MESH PIN	Indirect
602021M	INLET, TYPE ES	Indirect	602204M	EXTENSION RING FOR EXISTING MANHOLE, 6' DIAMETER	Indirect	603066P	SHOTCRETE	Indirect
602024M	INLET, TYPE B-1	Indirect	602207M	MANHOLE COVER	Indirect	603067P	SHOTCRETE, 6" THICK	Indirect
602027M	INLET, TYPE B-2	Indirect	602210M	BICYCLE SAFE GRATE	Indirect	603068P	SHOTCRETE, 4" THICK	Indirect
602028M	INLET, TYPE DOUBLE B-2	Indirect	602213M	CURB PIECE	Indirect	603069P	BROKEN STONE BACKFILL	Indirect
602029M	INLET, TYPE B-3	Indirect	602214M	INLET FACE PLATE	Indirect	603073P	ARTICULATED CONCRETE BLOCK MATTING, OPEN CELL	Indirect
602030M	INLET, TYPE D-1	Indirect	602215M	CAPPING EXISTING DRAINAGE STRUCTURES	Direct	603076M	EARTH ANCHOR FOR ARTICULATED CONCRETE BLOCK MATTING	Indirect
602033M	INLET, TYPE D-2	Indirect	602216M	CLEANING DRAINAGE STRUCTURE	Indirect	603079M	ROCK ANCHOR FOR ARTICULATED CONCRETE BLOCK MATTING	BRIndirect
602034M	INLET, TYPE D MODIFIED	Indirect	602217M	WATER QUALITY TREATMENT STRUCTURE	Indirect	603083P	ARTICULATED CONCRETE BLOCK MATTING, 6",	Indirect
602036M	INLET, TYPE E-1	Indirect	602219P	STORMWATER PUMPING STATION	Indirect	603100M	RIPRAP STONE SCOUR PROTECTION (D50=6")	Indirect
602039M	INLET, TYPE E-2	Indirect	602220M	TRENCH DRAIN	Indirect	603100P	RIPRAP STONE SCOUR PROTECTION (D50=6")	Indirect
602042M	INLET, TYPE A MODIFIED	Indirect	602223M	OUTLET CONTROL STRUCTURE	Indirect	603103P	RIPRAP STONE SCOUR PROTECTION (D50=12")	Indirect
602045M	INLET, TYPE B MODIFIED	Indirect	602226M	JUNCTION CHAMBER	Indirect	603109P	SCOUR COUNTERMEASURE	Indirect
602047M	INLET, TYPE B1R	Indirect	602229M	MANUFACTURED TREATMENT DEVICE	Indirect	603111M	OUTFALL SCOUR HOLE	Indirect
602048M	INLET, TYPE B-1 MODIFIED	Indirect	602230M	MANUFACTURED TREATMENT DEVICE NO. ___	Indirect	603112P	RIPRAP EMBANKMENT (D50=12")	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
603125M	ROCK DRAIN	Indirect	606009P	HOT MIX ASPHALT SIDEWALK, 5 1/2" THICK	Indirect	607042P	9" X 4" CONCRETE VERTICAL CURB, DOWELLED	Indirect
603130P	FAULT ZONE TREATMENT	Indirect	606012P	CONCRETE SIDEWALK, 4" THICK	Indirect	607045P	9" X 6" CONCRETE VERTICAL CURB, DOWELLED	Indirect
604003P	GABION WALL	Indirect	606013P	TINTED CONCRETE SIDEWALK, 4" THICK	Indirect	607051P	9" X 10" CONCRETE VERTICAL CURB, DOWELLE	Indirect
604006P	GABION MATTRESS	Indirect	606016P	CONCRETE SIDEWALK, 4" THICK, EXPOSED AGGREGATE FINISH	Indirect	607054P	12" X 3" CONCRETE SLOPING CURB, DOWELLED	Indirect
604009M	GABION WALL RECONSTRUCTION	Indirect	606017P	PERVIOUS CONCRETE SIDEWALK	Indirect	607055P	_"X_" CONCRETE SLOPING CURB, DOWELLED	Indirect
605009P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, 6' HIGH	Indirect	606018P	CONCRETE SIDEWALK, 6" THICK	Indirect	607060P	24" X VARIABLE HEIGHT CONCRETE BARRIER CURB	Indirect
605010P	POST AND RAIL FENCE	Indirect	606019P	CONCRETE SIDEWALK, 4" THICK, TEXTURED	Indirect	607063P	15" X VARIABLE HEIGHT CONCRETE BARRIER CURB, DOWELLED	Indirect
605015P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, 8' HIGH	Indirect	606021P	CONCRETE SIDEWALK, 8" THICK	Indirect	607066P	24" X VARIABLE HEIGHT CONCRETE BARRIER C	Indirect
605018P	CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, 10' HIGH	Indirect	606023P	PRECAST CONCRETE PAVERS	Indirect	607067P	24 1/2" X VARIABLE HEIGHT CONCRETE BARRIER CURB	Indirect
605021P	CHAIN-LINK FENCE, PVC-COATED STEEL, 4' HIGH	Indirect	606024P	CONCRETE SIDEWALK, REINFORCED, 6" THICK	Indirect	607069P	9" X VARIABLE HEIGHT CONCRETE VERTICAL CURB	Indirect
605024P	CHAIN-LINK FENCE, PVC-COATED STEEL, 5' HIGH	Indirect	606027P	CONCRETE SIDEWALK, REINFORCED, 8" THICK	Indirect	607070P	12" X VARIABLE HEIGHT CONCRETE SLOPING CURB, DOWELLED	Indirect
605027P	CHAIN-LINK FENCE, PVC-COATED STEEL, 6' HIGH	Indirect	606028P	RESET PRECAST CONCRETE PAVERS	Indirect	607072P	9" X VARIABLE HEIGHT CONCRETE VERTICAL CURB, DOWELLED	Indirect
605033P	CHAIN-LINK FENCE, PVC-COATED STEEL, 8' HIGH	Indirect	606029P	BRICK PAVERS	Indirect	607073P	VARIABLE WIDTH X 41" CONCRETE BARRIER CURB	Indirect
605036P	CHAIN-LINK FENCE, PVC-COATED STEEL, 10' HIGH	Indirect	606030P	HOT MIX ASPHALT DRIVEWAY, 1 1/2" THICK	Indirect	607074P	VARIABLE WIDTH X VARIABLE HEIGHT CONCRETE BARRIER CURB	Indirect
605039P	CHAIN-LINK FENCE, 4' HIGH	Indirect	606033P	HOT MIX ASPHALT DRIVEWAY, 2" THICK	Indirect	607075P	GRANITE CURB	Indirect
605045P	CHAIN-LINK FENCE, 6' HIGH	Indirect	606036P	HOT MIX ASPHALT DRIVEWAY, 4" THICK	Indirect	607076P	BELGIAN BLOCK CURB	Indirect
605048P	CHAIN-LINK FENCE, 7' HIGH	Indirect	606039P	HOT MIX ASPHALT DRIVEWAY, 6" THICK	Indirect	607077P	RESET BELGIAN BLOCK CURB	Indirect
605051P	CHAIN-LINK FENCE, 8' HIGH	Indirect	606042P	HOT MIX ASPHALT DRIVEWAY, VARIABLE THICKNESS	Indirect	607078P	RESET GRANITE CURB	Indirect
605060M	GATE, CHAIN-LINK FENCE, ALUMINUM COATED STEEL, 4' WIDE	Indirect	606043P	STONE OR GRAVEL DRIVEWAY, 4" THICK	Indirect	607081P	9" X 4" HOT MIX ASPHALT CURB	Indirect
605093M	GATE, CHAIN-LINK FENCE, ALUMINUM COATED STEEL, 30' WIDE	Indirect	606045P	CONCRETE DRIVEWAY, 4" THICK	Indirect	607084P	9" X 6" HOT MIX ASPHALT CURB	Indirect
605099M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 4' WIDE	Indirect	606051P	CONCRETE DRIVEWAY, 6" THICK	Indirect	607087P	9" X 8" HOT MIX ASPHALT CURB	Indirect
605102M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 6' WIDE	Indirect	606054P	CONCRETE DRIVEWAY, 8" THICK	Indirect	607089M	PRECAST CONCRETE WHEEL STOP	Indirect
605108M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 10' WIDE	Indirect	606057P	CONCRETE DRIVEWAY, REINFORCED, 6" THICK	Indirect	607090P	CONCRETE TRANSITION SECTION	Indirect
605111M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 12' WIDE	Indirect	606060P	CONCRETE DRIVEWAY, REINFORCED, 8" THICK	Indirect	607100P	CONCRETE BARRIER PYLON	Indirect
605117M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 15' WIDE	Indirect	606075P	CONCRETE ISLAND, 4" THICK	Indirect	608003P	NONVEGETATIVE SURFACE, HOT MIX ASPHALT	Indirect
605126M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 20' WIDE	Indirect	606076P	WHITE CONCRETE ISLAND, 4" THICK	Indirect	608004P	NONVEGETATIVE SURFACE, POROUS HOT MIX ASPHALT, 4" THICK	Indirect
605129M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 24' WIDE	Indirect	606078P	CONCRETE ISLAND, 6" THICK	Indirect	608005P	NONVEGETATIVE SURFACE, POROUS HOT MIX ASPHALT, 6" THICK	Indirect
605132M	GATE, CHAIN-LINK FENCE, PVC-COATED STEEL, 30' WIDE	Indirect	606081P	CONCRETE ISLAND, 8" THICK	Indirect	608012P	NONVEGETATIVE SURFACE, POLYESTER MATTING	Indirect
605147M	GATE, CHAIN-LINK FENCE, 10' WIDE	Indirect	606084P	DETECTABLE WARNING SURFACE	Indirect	608015P	NONVEGETATIVE SURFACE, BROKEN STONE	Indirect
605150M	GATE, CHAIN-LINK FENCE, 12' WIDE	Indirect	606090P	PUBLIC SIDEWALK CURB RAMP DELINEATION	Indirect	608017P	NONVEGETATIVE SURFACE, POROUS RESIN BOUND AGGREGATE, 2"	Indirect
605153M	GATE, CHAIN-LINK FENCE, 14' WIDE	Indirect	606092P	IMPRINTED CROSSWALK	Indirect	609003M	BEAM GUIDE RAIL	Indirect
605159M	GATE, CHAIN-LINK FENCE, 16' WIDE	Indirect	606093P	IMPRINT RESIN ISLAND	Indirect	609004M	BEAM GUIDE RAIL, BRIDGE	Direct
605165M	GATE, CHAIN-LINK FENCE, 20' WIDE	Indirect	606094P	GRANITE PAVING	Indirect	609005M	BEAM GUIDE RAIL, DUAL-FACED, ATTACHED TO CONCRETE PAD	Indirect
605168M	GATE, CHAIN-LINK FENCE, 24' WIDE	Indirect	606095P	TURF PAVERS	Indirect	609006M	BEAM GUIDE RAIL, DUAL-FACED	Indirect
605171M	GATE, CHAIN-LINK FENCE, 30' WIDE	Indirect	606097P	TURF PAVERS, CONCRETE	Indirect	609009M	MODIFIED THRIE BEAM GUIDE RAIL	Indirect
605174M	REPAIRING CHAIN-LINK FENCE	Indirect	606109P	CONCRETE STEPS, REINFORCED	Indirect	609010M	MODIFIED THRIE BEAM GUIDE RAIL BLOCKOUT	Indirect
605175M	BARRIER CURB MOUNTED FENCE	Indirect	606110P	STONE STEPS	Indirect	609012M	MODIFIED THRIE BEAM GUIDE RAIL, DUAL FACED	Indirect
605177P	TEMPORARY CHAIN-LINK FENCE, 4' HIGH	BRIndirect	606130P	HANDICAP RAMP, WOOD	Indirect	609015M	THRIE BEAM GUIDE RAIL, BRIDGE	Direct
605183P	TEMPORARY CHAIN-LINK FENCE, 6' HIGH	BRIndirect	606145P	RESET SIDEWALK	Indirect	609020M	BEAM GUIDE RAIL ATTACHMENT CD-609-8.5	Indirect
605189P	TEMPORARY CHAIN-LINK FENCE, 8' HIGH	BRIndirect	607003P	15" X 41" CONCRETE BARRIER CURB	Indirect	609021M	RUB RAIL	Indirect
605192P	TEMPORARY CHAIN-LINK FENCE, 10' HIGH	BRIndirect	607005P	17 3/8" X 53" CONCRETE BARRIER CURB	Indirect	609024M	FLARED GUIDE RAIL TERMINAL	Indirect
605193P	HANDRAIL	Indirect	607006P	24" X 32" CONCRETE BARRIER CURB	Indirect	609025M	BURIED GUIDE RAIL TERMINAL	Indirect
605194P	TEMPORARY PEDESTRIAN HANDRAIL	Indirect	607007P	15"X54" CONCRETE BARRIER CURB	Indirect	609027M	TANGENT GUIDE RAIL TERMINAL	Indirect
605195P	WOOD STOCKADE FENCE	Indirect	607008P	38" X 79" CONCRETE BARRIER CURB	Indirect	609028M	TANGENT GUIDE RAIL TERMINAL-ET200, TL-2, 25' LONG	Indirect
605196M	GATE, WOOD STOCKADE FENCE, 12' WIDE	Indirect	607012P	24" X 41" CONCRETE BARRIER CURB	Indirect	609030M	TELESCOPING GUIDE RAIL END TERMINAL	Indirect
605197P	WOOD STOCKADE FENCE, TEMPORARY	Indirect	607016P	REINFORCED CONCRETE BARRIER CURB	Indirect	609033M	CONTROLLED RELEASE TERMINAL	Indirect
605198M	GATE, WOOD STOCKADE FENCE, 42" WIDE	Indirect	607017P	CONCRETE CURB (TYPE 1)	Indirect	609034M	EXTRUDER TERMINAL, POWDER COATED	Indirect
605202P	ROCK CATCH FENCE END TERMINAL ANCHORAGE	Indirect	607018P	9" X 16" CONCRETE VERTICAL CURB	Indirect	609036M	CONTROLLED RELEASE TERMINAL ANCHORAGE	Indirect
605203P	ROCK CATCH FENCE	Indirect	607020P	9"X18" WHITE CONCRETE VERTICAL CURB	Indirect	609039M	BEAM GUIDE RAIL ANCHORAGE	Indirect
605205P	DEER FENCE, TEMPORARY	Indirect	607021P	9" X 18" CONCRETE VERTICAL CURB	Indirect	609042M	BEAM GUIDE RAIL POST	Indirect
605207P	VINYL FENCE	Indirect	607023P	24"X16" CONCRETE VERTICAL CURB	Indirect	609045M	BEAM GUIDE RAIL POST, 7' LONG	Indirect
605209P	ORNAMENTAL FENCE	Indirect	607024P	9" X 20" CONCRETE VERTICAL CURB	Indirect	609048M	BEAM GUIDE RAIL POST, 8' LONG	Indirect
605210P	RESTE WROUGHT IRON FENCE	Indirect	607026P	12"X16" CONCRETE VERTICAL CURB	Indirect	609051M	BEAM GUIDE RAIL POST, 10' LONG	Indirect
605212P	RESET FENCE	Indirect	607027P	9" X 22" CONCRETE VERTICAL CURB	Indirect	609054M	BEAM GUIDE RAIL ELEMENT	Indirect
605220P	ROCKFALL PROTECTION FENCE, 15 FT HIGH	Indirect	607029P	21" X 44" CONCRETE BARRIER CURB	Indirect	609057M	MODIFIED THRIE BEAM GUIDE RAIL ELEMENT	Indirect
605222P	ROCKFALL PROTECTION FENCE, 20 FT HIGH	Indirect	607030P	12" X 13" CONCRETE SLOPING CURB	Direct	609060M	BEAM GUIDE RAIL BLOCKOUT	Indirect
605223P	ROCKFALL PROTECTION FENCE, 30 FT HIGH	Indirect	607033P	15" X 35" CONCRETE BARRIER CURB, DOWELLED	Indirect	609063M	RESET BEAM GUIDE RAIL WITH EXISTING POSTS	Indirect
605230P	PRIVACY SLATS	Indirect	607035P	19" X 32" CONCRET BARRIER CURB, DOWELLED	Indirect	609066M	RESET BEAM GUIDE RAIL, DUAL-FACED, WITH EXISTING POSTS	Indirect
605240P	SPLIT RAIL FENCE, ___' HIGH	Indirect	607036P	24" X 32" CONCRETE BARRIER CURB, DOWELLED	Indirect	609072M	RESET MODIFIED THRIE BEAM GUIDE RAIL, DUAL-FACED, WITH	Indirect
606003P	HOT MIX ASPHALT SIDEWALK, 2" THICK	Indirect	607039P	24" X 35" CONCRETE BARRIER CURB, DOWELLED	Indirect	609075M	REMOVAL OF BEAM GUIDE RAIL	Indirect
606006P	HOT MIX ASPHALT SIDEWALK, 5" THICK	Indirect	607040P	CONCRETE BARRIER CURB WITH MOMENT SLAB	Indirect	609078M	FLEXIBLE DELINEATORS, GUIDE RAIL MOUNTED	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
609081M	WOOD BEAM RAIL, WEATHERING STEEL POSTS	Indirect	612030P	OVERHEAD STREET NAME SIGNS	Indirect	651293P	4" POLYVINYL CHLORIDE WATER PIPE	Indirect
610003M	TRAFFIC STRIPES, LONG LIFE, EPOXY RESIN 4"	Indirect	612032P	CUSTOM SIGN	Indirect	651295P	6" POLYVINYL CHLORIDE WATER PIPE	Indirect
610006M	TRAFFIC STRIPES, LONG LIFE, EPOXY RESIN 6"	Indirect	612033P	SPECIALIZED SIGN	Indirect	651296P	8" POLYVINYL CHLORIDE WATER PIPE	Indirect
610007M	TRAFFIC STRIPES, LONG LIFE, EPOXY RESIN, 8"	Indirect	612034M	RELOCATE SPECIALIZED SIGN	Indirect	651300P	12" POLYVINYL CHLORIDE WATER PIPE	Indirect
610008M	TRAFFIC MARKINGS, SYMBOLS, LONG LIFE, THERMOPLASTIC	Indirect	612041P	BRIDGE VERTICAL UNDERCLEARANCE SIGN	Direct	651421P	12" DUCTILE IRON WATER PIPE	Indirect
610009M	TRAFFIC MARKINGS, THERMOPLASTIC	Indirect	612043P	CAST IRON SIGN AND POST RESTORATION	Indirect	651427P	12" STEEL CASING	Indirect
610010M	TRAFFIC STRIPES, HIGH PERFORMANCE WET REFLECTIVE TAPE	Indirect	613002P	NOISE BARRIER, FOUNDATION	Indirect	651428P	16" STEEL CASING	Indirect
610011M	TRAFFIC MARKINGS, LINES, LONG LIFE, THERMOPLASTIC	Indirect	613004P	NOISE BARRIER, ROADWAY	Indirect	651430P	20" STEEL CASING	Indirect
610012M	RPM, MONO-DIRECTIONAL, WHITE LENS	Indirect	613005P	NOISE BARRIER, BRIDGE	Direct	651515P	12" WATER PIPE, TRENCHLESS CONSTRUCTION	Indirect
610015M	RPM, BI-DIRECTIONAL, WHITE LENS	Indirect	613006P	REMOVE AND REINSTALL NOISE BARRIER	Indirect	651520P	24" DUCTILE IRON WATER PIPE, CLASS 350	Indirect
610018M	RPM, MONO-DIRECTIONAL, AMBER LENS	Indirect	613010P	NOISE BARRIER FOUNDATION	Direct	651521P	30" DUCTILE IRON WATER PIPE, CLASS 350	Indirect
610021M	RPM, BI-DIRECTIONAL, AMBER LENS	Indirect	613011P	NOISE BARRIER, INTERIM	Indirect	651522P	36" DUCTILE IRON WATER PIPE, CLASS 350	Indirect
610024M	REMOVAL OF RPM	Indirect	613012P	SOUND ABSORPTIVE COATING	Indirect	651523P	24" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	Direct
610027M	REMOVAL AND REPLACEMENT OF RPM LENS	Indirect	613015P	NOISE BARRIER TEST POSTS AND PANELS	Indirect	651524P	30" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	Direct
610030M	FLEXIBLE DELINEATOR, GROUND MOUNTED	Indirect	614003M	TRASH RECEPTACLE, DECORATIVE	Indirect	651525P	36" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	Direct
610031M	FLEXIBLE DELINEATORS, BARRIER CURB MOUNTED	Indirect	615006P	CONTROL BUILDING	Indirect	652002P	6" DUCTILE IRON SEWER PIPE	Indirect
610033M	RUMBLE STRIP	Indirect	618003M	BENCH	Indirect	652003P	12" DUCTILE IRON SEWER PIPE	Indirect
610034M	STRIPED RUMBLE STRIP	Indirect	651001P	WATER UTILITY RELOCATION	Indirect	652004P	8" DUCTILE IRON SEWER PIPE	Indirect
610035M	SPEED TABLE	Indirect	651003P	4" DUCTILE IRON WATER PIPE, CLASS 50	Indirect	652005P	10" DUCTILE IRON SEWER PIPE	Indirect
610036M	REMOVAL OF TRAFFIC STRIPES	Indirect	651006P	6" DUCTILE IRON WATER PIPE, CLASS 50	Indirect	652006P	15" DUCTILE IRON SEWER PIPE	Indirect
610039M	REMOVAL OF TRAFFIC MARKINGS	Indirect	651009P	8" DUCTILE IRON WATER PIPE, CLASS 50	Indirect	652008P	14" DUCTILE IRON SEWER PIPE	Indirect
610045M	BOLLARD	Indirect	651015P	12" DUCTILE IRON WATER PIPE, CLASS 50	Indirect	652009P	18" DUCTILE IRON SEWER PIPE	Indirect
611012M	CRASH CUSHION, INERTIAL BARRIER SYSTEM, 10 MODULES	Indirect	651051P	4" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652010P	18" DUCTILE IRON SEWER PIPE WITH 36" STEEL CASING PIPE	Indirect
611015M	CRASH CUSHION, INERTIAL BARRIER SYSTEM, 11 MODULES	Indirect	651054P	6" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652018P	27" DUCTILE IRON SEWER PIPE	Indirect
611024M	CRASH CUSHION, INERTIAL BARRIER SYSTEM, 14 MODULES	Indirect	651055P	6" DUCTILE IRON WATER PIPE, CLASS 54	Indirect	652021P	30" DUCTILE IRON SEWER PIPE	Indirect
611063M	CRASH CUSHION, QUADGUARD, 2 BAYS, 24" WIDE	Indirect	651056P	6" DUCTILE IRON WATER PIPE, CLASS 53	Indirect	652027P	36" DUCTILE IRON SEWER PIPE	Indirect
611066M	CRASH CUSHION, QUADGUARD, 3 BAYS, 24" WIDE	Indirect	651057P	8" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652144P	36" REINFORCED CONCRETE SEWER PIPE, CLASS IV	Indirect
611069M	CRASH CUSHION, QUADGUARD, 4 BAYS, 24" WIDE	Indirect	651058P	8" DUCTILE IRON WATER PIPE, CLASS 54	Indirect	652177P	12" REINFORCED CONCRETE SEWER PIPE, CLAS	Indirect
611072M	CRASH CUSHION, QUADGUARD, 5 BAYS, 24" WIDE	Indirect	651059P	8" DUCTILE IRON WATER PIPE, CLASS 53	Indirect	652232P	4" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611075M	CRASH CUSHION, QUADGUARD, 6 BAYS, 24" WIDE	Indirect	651060P	10" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652233P	6" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611078M	CRASH CUSHION, QUADGUARD, 7 BAYS, 24" WI	Indirect	651063P	12" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652234P	12" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611102M	CRASH CUSHION, QUADGUARD, 3 BAYS, 30" WIDE	Indirect	651064P	12" DUCTILE IRON WATER PIPE, CLASS 54	Indirect	652235P	10" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611105M	CRASH CUSHION, QUADGUARD, 4 BAYS, 30" WIDE	Indirect	651069P	16" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652236P	8" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611108M	CRASH CUSHION, QUADGUARD, 5 BAYS, 30" WIDE	Indirect	651070P	20" DUCTILE IRON WATER PIPE, CLASS 53	Indirect	652237P	15" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611111M	CRASH CUSHION, QUADGUARD, 6 BAYS, 30" WIDE	Indirect	651071P	20" DUCTILE IRON WATER PIPE, CLASS 52	Indirect	652238P	16" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611114M	CRASH CUSHION, QUADGUARD, 7 BAYS, 30" WIDE	Indirect	651074P	24" DUCTILE IRON WATER PIPE, CLASS 54	Indirect	652239P	14" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611117M	CRASH CUSHION, QUADGUARD, 8 BAYS, 30" WIDE	Indirect	651145P	8" DUCTILE IRON WATER PIPE BRIDGE, CLASS 54	Indirect	652240P	18" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611147M	CRASH CUSHION, QUADGUARD, 6 BAYS, 36" WIDE	Indirect	651146P	12" DUCTILE IRON WATER PIPE BRIDGE, CLASS 54	Indirect	652242P	20" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611156M	CRASH CUSHION, QUADGUARD, 9 BAYS, 36" WIDE	Indirect	651177P	8" DUCTILE IRON WATER PIPE BRIDGE, CLASS 52	Indirect	652244P	12" POLYVINYL CHLORIDE SEWER PIPE, GRAVITY	Indirect
611168M	CRASH CUSHION, QUADGUARD, 3 BAYS, 70" WIDE	Indirect	651243M	WATER SERVICE CONNECTION	Indirect	652245P	12" POLYVINYL CHLORIDE SEWER PIPE, FORCE MAIN	Indirect
611177M	CRASH CUSHION, QUADGUARD, 6 BAYS, 70" WIDE	Indirect	651244M	EXTEND WATER SERVICE	Indirect	652249P	27" POLYVINYL CHLORIDE SEWER PIPE	Indirect
611198M	CRASH CUSHION, QUADGUARD, 3 BAYS, 90" WIDE	Indirect	651245P	WATER SERVICE PIPE	Indirect	652293P	12" STEEL SEWER PIPE, BRIDGE	Direct
611207M	CRASH CUSHION, QUADGUARD, 6 BAYS, 90" WIDE	Indirect	651246M	FIRE HYDRANT	Indirect	652417M	SANITARY SEWER SERVICE CONNECTION	Indirect
611210M	CRASH CUSHION, QUADGUARD, 7 BAYS, 90" WIDE	Indirect	651249M	RELOCATE FIRE HYDRANT	Indirect	652418M	SANITARY SEWER BY-PASS PUMPING	Indirect
611240M	CRASH CUSHION, COMPRESSIVE BARRIER, TYPE ___, WIDTH ___	Indirect	651252M	RESET FIRE HYDRANT	Indirect	652419M	SANITARY SEWER CLEANOUT	Indirect
611312M	CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 3, WIDTH NARROW	Indirect	651253M	RESET WATER MANHOLE	Indirect	652420M	MANHOLE, SANITARY SEWER	Indirect
611315M	CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 3, WIDTH MEDIUM	Indirect	651255M	RESET WATER VALVE BOX	Indirect	652421M	MANHOLE, DOGHOUSE, SANITARY SEWER, 4' DIAMETER	Indirect
611318M	CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 3, WIDTH WIDE	Indirect	651256M	WATER SERVICE VAULT	Indirect	652423M	MANHOLE, SANITARY SEWER, USING EXISTING CASTING	Indirect
611321M	CRASH CUSHION, COMPRESSIVE BARRIER, TYPE 3, WIDTH X-WIDE	Indirect	651258M	WATER AS-BUILT PLAN	Indirect	652424M	MANHOLE, SANITARY SEWER, SHALLOW	Indirect
611336M	CRASH CUSHION, LOW MAINTENANCE COMPRESSIVE BARRIER, TYPE 2,	Indirect	651261M	INSERTION VALVES AND BOXES	Indirect	652426M	RECONSTRUCTED MANHOLE, SANITARY SEWER, USING EXISTING CAS	Indirect
611348M	CRASH CUSHION, LOW MAINTENANCE, COMPRESS	Indirect	651264M	WET TAP	Indirect	652429M	RECONSTRUCTED MANHOLE, SANITARY SEWER, USING NEW CASTIN(	Indirect
612003P	REGULATORY AND WARNING SIGN	Indirect	651267M	LINE STOP AND TIE-IN	Indirect	652432M	RESET MANHOLE, SANITARY SEWER, USING EXISTING CASTING	Indirect
612004P	PEDESTRIAN CROSSING SIGN WITH WARNING BEACON	Indirect	651268M	6" VALVES AND BOXES	Indirect	652435M	RESET MANHOLE, SANITARY SEWER, USING NEW CASTING	Indirect
612006P	GUIDE SIGN, TYPE GA, STEEL "U" POST SUPPORTS	Indirect	651269M	8" VALVES AND BOXES	Indirect	652438M	VIDEO INSPECTION OF SEWER	Indirect
612009P	GUIDE SIGN, TYPE GA, BREAKAWAY SUPPORTS	Indirect	651270M	GATE VALVES AND BOXES	Indirect	652441M	SEWER AS-BUILT PLAN	Indirect
612010P	RELOCATE GUIDE SIGN, TYPE GA, NEW BREAKAWAY SUPPORTS	Indirect	651271M	10" VALVES AND BOXES	Indirect	652451M	RESET SANITARY SEWER VENT	Indirect
612013P	GUIDE SIGN, TYPE GA, TIMBER SUPPORTS	Indirect	651272M	12" VALVES AND BOXES	Indirect	652452P	SANITARY SEWER PUMP STATION	Indirect
612015P	GUIDE SIGN PANEL, TYPE GO	Indirect	651273M	BUTTERFLY VALVES AND BOXES	Indirect	652466P	12" SANITARY SEWER MAIN	Indirect
612018P	GUIDE SIGN PANEL, TYPE GOX	Indirect	651274M	16" VALVES AND BOXES	Indirect	652467P	3" SANITARY FORCE MAIN	Indirect
612021M	RELOCATE SIGN	Indirect	651277M	2" BLOWOFF VALVE	Indirect	652469P	10" SANITARY FORCE MAIN	Indirect
612024M	RESET SIGN USING EXISTING POSTS	Indirect	651285P	WATER METER PIT	Indirect	652506M	108" SEWER REHABILITATION, ALTERNATIVE D	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
652550M	CLEANING ACCESS ASSEMBLY	Indirect	701012P	1 1/2" RIGID METALLIC CONDUIT	Indirect	701157M	FOUNDATION, TYPE 3M	Indirect
652555M	AUTOMATIC AIR RELEASE VALVE, SANITARY	Indirect	701013P	1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	Direct	701159M	FOUNDATION, TYPE 1M-MC	Indirect
652563P	AIR RELEASE VALVE CHAMBER	Indirect	701015P	2" RIGID METALLIC CONDUIT	Indirect	701162M	FOUNDATION, TYPE 2M-MC	Indirect
652566P	MANHOLE, SANITARY SEWER TYPE A	Indirect	701016P	2" RIGID METALLIC CONDUIT, PVC COATED	Indirect	701165M	CABLE RACK	Indirect
652570P	12" SEWER PIPE, TRENCHLESS CONSTRUCTION	Indirect	701018P	2 1/2" RIGID METALLIC CONDUIT	Indirect	701168M	METER CABINET, TYPE T	Indirect
653001P	GAS UTILITY RELOCATION	Indirect	701019P	2" RIGID METALLIC CONDUIT ON STRUCTURE	Direct	701171M	METER CABINET, TYPE TL	Indirect
653003P	2" GAS MAIN	Indirect	701020P	3" RIGID METALLIC CONDUIT, PVC COATED	Indirect	701174M	METER CABINET, TYPE 1M	Indirect
653006P	4" GAS MAIN	Indirect	701021P	3" RIGID METALLIC CONDUIT	Indirect	701175M	METER CABINET, TYPE 1M MODIFIED	Indirect
653009P	6" GAS MAIN	Indirect	701022P	3" RIGID METALLIC CONDUIT ON STRUCTURE	Direct	701177M	METER CABINET, TYPE 1M-MC	Indirect
653012P	8" GAS MAIN	Indirect	701024P	4" RIGID METALLIC CONDUIT	Indirect	701178M	METER CABINET, TYPE 2M	Indirect
653015M	GAS MAIN, TIE-IN ASSISTANCE	Indirect	701025P	4" RIGID METALLIC CONDUIT, PVC COATED	Indirect	701179M	METER CABINET, TYPE 3M	Indirect
653018P	12" GAS MAIN	Indirect	701026P	1" RIGID NONMETALLIC CONDUIT	Indirect	701180M	METER CABINET, TYPE 2M-MC	Indirect
653024P	16" GAS MAIN	Indirect	701027P	2" RIGID NONMETALLIC CONDUIT	Indirect	701183M	METER CABINET, TYPE L	Indirect
653048P	6" GAS MAIN, BRIDGE	Direct	701029P	RIGID NONMETALLIC MULTIDUCT CONDUIT	Indirect	701185M	METER CABINET, TYPE ITS	Indirect
653051P	8" GAS MAIN, BRIDGE	Direct	701030P	3" RIGID NONMETALLIC CONDUIT	Indirect	701187M	MODIFY METER CABINET	Indirect
653057P	12" GAS MAIN, BRIDGE	Direct	701031P	4" SPLIT STEEL CONDUIT	Indirect	701189P	GROUND WIRE, NO. 6 AWG	Indirect
653081M	GAS SERVICE CONNECTION	Indirect	701032P	6" SPLIT STEEL CONDUIT	Indirect	701192P	GROUND WIRE, NO. 8 AWG	Indirect
653084M	RESET GAS VALVE BOX	Indirect	701033P	4" RIGID NONMETALLIC CONDUIT	Indirect	701193P	GROUND WIRE, NO. 10 AWG	Indirect
653087P	GAS AS-BUILT PLAN	Indirect	701035P	6" RIGID METALLIC CONDUIT	Indirect	701195P	MULTIPLE LIGHTING WIRE, NO. 2 AWG	Indirect
653096M	GAS EXPANSION CHAMBER	Indirect	701036P	1 1/4" FLEXIBLE METALLIC CONDUIT	Indirect	701196P	MULTIPLE LIGHTING WIRE, NO. 4 AWG	Indirect
654003P	ELECTRICAL CONDUIT	Indirect	701039P	1 1/2" FLEXIBLE METALLIC CONDUIT	Indirect	701198P	MULTIPLE LIGHTING WIRE, NO. 6 AWG	Indirect
654004P	ELECTRICAL SERVICE	Indirect	701042P	2" FLEXIBLE METALLIC CONDUIT	Indirect	701201P	MULTIPLE LIGHTING WIRE, NO. 8 AWG	Indirect
654007P	ELECTRICAL UTILITY RELOCATION, _________	Indirect	701043P	3" FLEXIBLE METALLIC CONDUIT	Indirect	701204P	MULTIPLE LIGHTING WIRE, NO. 10 AWG	Indirect
654008P	RELOCATE UNDERGROUND ELECTRIC SERVICE	Indirect	701046P	1 1/2" RIGID NONMETALLIC CONDUIT	Indirect	701207P	SERVICE WIRE, NO. 1/0 AWG	Indirect
654010M	UTILITY POLE SHEATH	Indirect	701051P	3 - 1 1/4" FLEXIBLE NONMETALLIC CONDUIT	Indirect	701210P	SERVICE WIRE, NO. 2 AWG	Indirect
654011P	RELOCATE EXISTING SERVICE LINES	Indirect	701063P	3 - 1 1/2" FLEXIBLE NONMETALLIC CONDUIT	Indirect	701213P	SERVICE WIRE, NO. 6 AWG	Indirect
654012P	CONCRETE ENCASED DUCT BANK	Indirect	701069P	1 - 2" FLEXIBLE NONMETALLIC CONDUIT	Indirect	701216P	SERVICE WIRE, NO. 8 AWG	Indirect
654013P	CONCRETE ENCASED DUCT BANK, 2 DUCTS	Indirect	701091P	FIBERGLASS CONDUIT	Indirect	701217P	SERVICE WIRE, 350KCMIL	Indirect
654016P	CONCRETE ENCASED DUCT BANK, 4 DUCTS	Indirect	701093M	38" JUNCTION BOX	Indirect	701218P	SERVICE WIRE, 250 KCMIL	Indirect
654018P	CONCRETE ENCASED DUCT BANK	Indirect	701094M	JUNCTION BOX	Indirect	701220P	SERVICE WIRE, NO. 4/0 AWG	Indirect
654027M	ELECTRIC RISER	Indirect	701095M	JUNCTION BOX, PVC PLASTIC	Indirect	701230M	TRANSFORMER	Indirect
654029M	RESET ELECTRIC MANHOLE	Indirect	701096M	10" X 36" JUNCTION BOX	Indirect	701231M	STEP DOWN TRANSFORMER	Indirect
654030M	ELECTRICAL MANHOLE	Indirect	701099M	17" X 30" JUNCTION BOX	Indirect	701238M	DUAL AIRWAY OBSTRUCTION LIGHT	Indirect
654031M	ELECTRICAL MANHOLE, THREE WAY	Indirect	701102M	18" X 36" JUNCTION BOX	Indirect	701240P	MULTIPLE LIGHTING WIRE, NO. 1/0 AWG	Indirect
655002P	CONCRETE ENCASED TELECOMMUNICATION CONDUIT	Indirect	701103M	EXTENSION OF JUNCTION BOX	Indirect	701241P	MULTIPLE LIGHTING WIRE, NO. 2/0 AWG	Indirect
655003P	TELECOMMUNICATION CONDUIT	Indirect	701105M	8" X 8" X 6" METAL JUNCTION BOX	Indirect	701253M	FOUNDATION, TYPE A	Indirect
655004M	TELECOMMUNICATION RISER	Indirect	701107M	TRANSITE JUNCTION BOX REMOVAL	Indirect	701280M	FOUNDATION DECORATIVE LIGHT STANDARD	Indirect
655007P	TELEPHONE UTILITY RELOCATION	Indirect	701108M	12" X 10" X 8" METAL JUNCTION BOX	Indirect	701320P	#2 COPPER CONDUCTOR	Indirect
655010P	RECONSTRUCT TELEPHONE MANHOLE	Indirect	701109M	12" X 12" X 6" METAL JUNCTION BOX	Indirect	701322P	#4/0 COPPER CONDUCTOR	Indirect
655012M	TELECOMMUNICATION MANHOLE	Indirect	701111M	12" X 20" X 18" METAL JUNCTION BOX	Indirect	701324P	#6 COPPER CONDUCTOR	Indirect
655013P	RESET TELECOMMUNICATIONS MANHOLE	Indirect	701112M	18" X 8" X 6" METAL JUNCTION BOX	Indirect	701325P	#8 COPPER CONDUCTOR	Indirect
655015P	TELEPHONE WIRE	Indirect	701113M	24" X 24" X 8" METAL JUNCTION BOX	Indirect	701330P	#10 COPPER CONDUCTOR	Indirect
656006M	CABLE HANDHOLE	Indirect	701114M	12" X 20" X 36" METAL JUNCTION BOX	Indirect	701331P	#12 COPPER CONDUCTOR	Indirect
656009M	CABLE RISER	Indirect	701116M	STAINLESS STEEL JUNCTION BOX	Indirect	701334P	#14 COPPER CONDUCTOR	Indirect
656012M	CABLE MANHOLE	Indirect	701117M	JUNCTION BOX FOUNDATION	Indirect	701343P	GROUND WIRE, NO. 2/0 AWG	Indirect
656015P	CABLE CONDUIT	Indirect	701118P	JUNCTION BOX MODIFICATIONS	Indirect	701350P	ELECTRICAL EQUIPMENT RELOCATION	Indirect
656018P	CONCRETE ENCASED CABLE CONDUIT	Indirect	701120M	JUNCTION BOX FRAME AND COVER	Indirect	701375P	MODIFY EXISTING LOAD CENTER	Indirect
656020P	1.5" QUAD DUCT CONDUIT, BRIDGE	Indirect	701123M	FOUNDATION, TYPE SFT	Indirect	702003M	CONTROLLER, 2 PHASE	Indirect
656021P	1.5" QUAD DUCT CONDUIT	Indirect	701126M	FOUNDATION, TYPE MCF	Indirect	702009M	CONTROLLER, 8 PHASE	Indirect
657003P	TEMPORARY SUPPORT, UTILITY	Indirect	701128M	FOUNDATION, TYPE P-MC MODIFIED	Indirect	702012M	TRAFFIC SIGNAL STANDARD, ALUMINUM	Indirect
657005P	UTILITY PIPE SUPPORT	Indirect	701129M	FOUNDATION, TYPE P	Indirect	702013M	TRAFFIC SIGNAL STANDARD, ALUMINUM, DECORATIVE	Indirect
657006P	PIPELINE INSTALLED VIA HORIZONTAL DIRECTIONAL DRILLING	Indirect	701132M	FOUNDATION, TYPE P-MC	Indirect	702015M	TRAFFIC SIGNAL STANDARD, STEEL	Indirect
657009P	16" STEEL CASING	Indirect	701135M	FOUNDATION, TYPE SPF	Indirect	702016M	TRAFFIC SIGNAL STANDARD, STEEL, DECORATIVE	Indirect
658003P	BUS SHELTER	Indirect	701138M	FOUNDATION, TYPE STF	Indirect	702018M	PEDESTRIAN SIGNAL STANDARD	Indirect
659012M	RAILROAD GATE FOUNDATION	Indirect	701141M	FOUNDATION, TYPE SFX	Indirect	702019M	PEDESTRIAN SIGNAL STANDARD, DECORATIVE	Indirect
701003P	1/2" RIGID METALLIC CONDUIT	Indirect	701143M	FOUNDATION, TYPE SFT MODIFIED	Indirect	702021M	TRAFFIC SIGNAL MAST ARM, ALUMINUM	Indirect
701006P	3/4" RIGID METALLIC CONDUIT	Indirect	701144M	FOUNDATION, TYPE SFK	Indirect	702022M	TRAFFIC SIGNAL MAST ARM, ALUMINUM, DECORATIVE	Indirect
701008P	1 1/4" RIGID METALLIC CONDUIT	Indirect	701147M	FOUNDATION, TYPE SSF	Indirect	702023M	TRAFFIC SIGNAL MAST ARM, STEEL, DECORATIVE	Indirect
701009P	1" RIGID METALLIC CONDUIT	Indirect	701150M	FOUNDATION, TYPE SSF-A	Indirect	702024M	TRAFFIC SIGNAL MAST ARM, STEEL	Indirect
701010P	1" RIGID METALLIC CONDUIT, PVC COATED	Indirect	701153M	FOUNDATION, TYPE 1M	Indirect	702027P	TRAFFIC SIGNAL CABLE, 2 CONDUCTOR	Indirect
701011P	1 1/2" RIGID METALLIC CONDUIT, PVC COATED	Indirect	701156M	FOUNDATION, TYPE 2M	Indirect	702030P	TRAFFIC SIGNAL CABLE, 5 CONDUCTOR	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
702032P	TRAFFIC SIGNAL CABLE, 7 CONDUCTOR	Indirect	704011M	METER CABINET ITS	Indirect	704180M	DMS SIGN WITH CONTROLLER INSTALL	Indirect
702033P	TRAFFIC SIGNAL CABLE, 10 CONDUCTOR	Indirect	704012P	COMMUNICATION CABLE	Indirect	704182M	CONTROLLER CABINET TYPE P-TMS	Indirect
702036M	TRAFFIC SIGNAL HEAD	Indirect	704014M	FOUNDATION, ITS	Indirect	704183M	CONTROLLER, DMS	Indirect
702037M	GEOMETRICALLY PROGRAMMED LOUVER	Indirect	704015M	FOUNDATION ITS TYPE A	Indirect	704184M	CONTROLLER, DMS INSTALL	Indirect
702039M	PEDESTRIAN SIGNAL HEAD	Indirect	704021M	FOUNDATION ITS TYPE C	Indirect	704185M	CONTROLLER MODIFICATIONS, DMS	Indirect
702041M	PUSH BUTTON ASSEMBILIES, TYPE APS	Indirect	704022M	FOUNDATION ITS TYPE C-MC	Indirect	704186M	CONTROLLER, WIM	Indirect
702042M	PUSH BUTTON	Indirect	704023M	FOUNDATION ITS TYPE D-MC	Indirect	704189M	WIM ROADWAY DEVICES 1 LANES	Indirect
702043M	PUSH BUTTON SIGN	Indirect	704024M	FOUNDATION ITS TYPE D	Indirect	704192M	WIM ROADWAY DEVICES 2 LANES	Indirect
702044P	IMAGE DETECTOR CABLE	Indirect	704027M	CONTROLLER, ITS	Indirect	704198M	WIM ROADWAY DEVICES 4 LANES	Indirect
702045M	IMAGE DETECTOR	Indirect	704028M	CONTROLLER MODIFICATIONS	Indirect	704200M	WIM ROADWAY DEVICES 6 LANES	Indirect
702046M	RADAR DETECTOR	Indirect	704029M	COMMUNICATION HUB MODIFICATIONS	Indirect	704201M	CONTROLLER, TVS	Indirect
702048M	LOOP DETECTOR	Indirect	704030M	COMMUNICATION HUB	Indirect	704203M	TVS ROADWAY LOOPS __ LANES	Indirect
702049M	LOOP DETECTOR	Indirect	704031M	MINI-HUB CABINET	Indirect	704207M	TVS ROADWAY DEVICES 2 LANES	Indirect
702051P	LOOP DETECTOR CABLE	Indirect	704033P	CONTROL CENTER SYSTEM, LOCATION NO. ___	Indirect	704210M	TVS ROADWAY DEVICES 3 LANES	Indirect
702054M	TEMPORARY TRAFFIC SIGNAL SYSTEM, LOCATION NO. ___	BRIndirect	704034P	MODIFY CONTROL CENTER SYSTEM	Indirect	704214M	TMS ROADSIDE DEVICES	Indirect
702057M	INTERIM TRAFFIC SIGNAL SYSTEM, LOCATION NO. ___	Indirect	704035M	FOUNDATION CSS	Indirect	704215M	FIBER CROSSCONNECT CABINET	Indirect
702059M	GPS UNIT	Indirect	704036M	FOUNDATION CSS TYPE A	Indirect	704216M	EQUIPMENT CABINET	Indirect
702060M	CONTROLLER TURN-ON	Indirect	704047M	CAMERA (OTHER THAN STANDARD)	Indirect	704217M	ETHERNET SWITCH	Indirect
702061M	MASTER CONTROLLER	Indirect	704048M	CAMERA STANDARD TYPE A	Indirect	704218M	TVS AUTOMATIC TRAFFIC RECORDER	Indirect
702062M	APS CONTROL UNIT	Indirect	704049M	VIRTUAL WEIGH STATION CIRCUIT BOARD	Indirect	704220M	MEDIA CONVERTER	Indirect
702067M	STROBE BEACON LIGHT	Indirect	704050M	VIRTUAL WEIGH STATION CAMERA	Indirect	704223M	STATIC I.P. COMMUNICATION SYSTEM	Indirect
702069M	RAILROAD WARNING DEVICE	Indirect	704051M	CAMERA STANDARD TYPE B	Indirect	704227M	VIRTUAL WEIGH STATION WIRELESS ROUTER	Indirect
702072M	TRAFFIC SIGNAL ASSEMBLY, TYPE 20S-1	Indirect	704054M	CAMERA STANDARD TYPE C	Indirect	704228M	ROUTER	Indirect
702078M	TRAFFIC SIGNAL ASSEMBLY, TYPE 30MA-2	Indirect	704057M	CAMERA STANDARD TYPE D	Indirect	704229M	ISP SERVICE	Indirect
702079M	SIGN ASSEMBLY "A"	Indirect	704059M	CAMERA STANDARD	Indirect	704232M	SURGE SUPPRESSION	Indirect
702080M	SPREAD SPECTRUM RADIO	Indirect	704060M	CAMERA	Indirect	704235M	WIRELESS LINK	Indirect
702081M	SPREAD SPECTRUM RADIO CABLE	Indirect	704063M	CONTROLLER, CAMERA	Indirect	704238M	PATCH PANEL	Indirect
702082M	SPREAD SPECTRUM RADIO ANTENNA	Indirect	704065P	FIBER OPTIC CABLE	Indirect	704243M	WIRELESS ANTENNA	Indirect
702090M	FIBER OPTIC BLANK-OUT SIGN	Indirect	704066P	FIBER OPTIC CABLE TYPE A	Indirect	704246P	ITS INTEGRATION	Indirect
702100M	UNINTERRUPTIBLE POWER SOURCE UNIT WITH CONTROLLER	Indirect	704067P	REINSTALL FIBER OPTIC CABLE	Indirect	704247M	FURNISH AND INSTALL ___	Indirect
702101M	UNINTERRUPTIBLE POWER SUPPLY	Indirect	704069P	FIBER OPTIC CABLE TYPE B	Indirect	704260P	ITS MODIFICATION TO TOC	Indirect
702103P	OPTICAL EMERGENCY PRE-EMPTION SYSTEM	Indirect	704070M	CONSTRUCTION WEBCAM	Indirect	704270M	RESEALING OF LOOPS AND SENSORS	Indirect
702104P	FLASHING WARNING SIGN	Indirect	704075P	FIBER OPTIC CABLE TYPE D	Indirect	704275M	RECURRING ELECTRICAL SERVICE CHARGES--ITS DEVICES	Indirect
702105M	DYNAMIC RED SIGNAL AHEAD SIGN ASSEMBLY	Indirect	704078P	FIBER OPTIC CABLE TYPE E	Indirect	704276M	RECURRING COMMUNICATIONS SERVICE CHARGES--ITS DEVICES	Indirect
702106P	TRAFFIC SIGNAL REIMBURSEMENT	Indirect	704079M	FIBER OPTIC SPLICE	Indirect	704277M	RECURRING ELECTRICAL SERVICE CHARGES, ITS DEVICES	Indirect
702109M	TRAFFIC SIGNAL ASSEMBLY, CLAMP MOUNTED	Indirect	704081P	FIBER OPTIC CABLE TYPE F	Indirect	704278M	RECURRING COMMUNICATIONS SERVICE CHARGES, ITS DEVICES	Indirect
702110M	BC TRAFFIC SIGNAL POLE FOUNDATION, 12" DIAMETER	Indirect	704084M	CONTROLLER, CTSS	Indirect	704280M	CPVMSRC	Indirect
703003M	LIGHTING STANDARD ALUMINUM	Indirect	704085M	FIBER OPTIC SIGN	Indirect	704285M	DIRECTIONAL AMPLIFIER SYSTEM, LOCATION ___	Indirect
703004M	LIGHTING STANDARD CONCRETE	Indirect	704087M	CTSS CONTROLLER UNIT	Indirect	706005P	REMOVAL, RELOCATION, AND INSTALLATION OF MISCELLANEOUS	Indirect
703006M	LIGHTING STANDARD STEEL	Indirect	704088M	FIBER MARKOUT	Indirect	706006M	REMOVAL, RELOCATION, INSTALL MISC ELEC	Indirect
703010M	LIGHTING STANDARD DECORATIVE	Indirect	704090M	CONTROLLER, CTSS TURN ON	Indirect	706007P	GENERATOR LOAD BANK	Indirect
703011M	RELOCATE LIGHTING STANDARD	Indirect	704091M	ADAPTIVE SIGNAL PROCESSOR, CTSS	Indirect	706009P	MOTOR CONTROL CENTER, AUTOMATIC TRANSFER SWITCH,	Indirect
703012M	LIGHTING MAST ARM ALUMINUM	Indirect	704092M	ADAPTIVE IMAGE DETECTOR, CTSS	Indirect	706013M	PANELBOARD	Indirect
703015M	LIGHTING MAST ARM STEEL	Indirect	704093M	FOUNDATION TTS TYPE A	Indirect	706019M	BARRIER GATE	Direct
703016M	LIGHTING MAST ARM DECORATIVE	Indirect	704106M	CONTROLLER MODIFICATIONS, CAMERA	Indirect	706022M	BRAKE REPLACEMENT	Direct
703018M	LUMINAIRE	Indirect	704108M	CONTROLLER, TTS	Indirect	706024M	TRAFFIC SIGNAL	Indirect
703019M	LUMINAIRE DECORATIVE	Indirect	704109M	CONTROLLER MODIFICATIONS, TTS	Indirect	750003P	FLASHER AND SOUNDER	Indirect
703020M	TUNNEL LUMINAIRE	Indirect	704111M	TTS DETECTOR TYPE A	Indirect	750015P	PROGRAMMABLE LOGIC CONTROLLER SYSTEM	Indirect
703021M	SIGN LIGHTING, STRUCTURE NO. ___	Direct	704114M	TTS DETECTOR TYPE B	Indirect	750018P	FLUX VECTOR DRIVE AND CABINET	Indirect
703024M	UNDERDECK LIGHTING TYPE W	Direct	704117M	TTS DETECTOR TYPE C	Indirect	750021P	MOTOR BRAKES	Indirect
703027M	UNDERDECK LIGHTING TYPE P	Direct	704118M	TTS DETECTOR	Indirect	750024P	MACHINERY BRAKES	Indirect
703032P	VIADUCT LIGHTING SYSTEM	Indirect	704119M	BLUETOOTH READER SOLAR/WIRELESS ASSEMBLY	Indirect	750028P	DATA LOGGER	Indirect
703033P	TEMPORARY HIGHWAY LIGHTING SYSTEM	BRIndirect	704120M	WEATHER STATION	Indirect	750030P	VECTOR DUTY MOTOR	Indirect
703034P	NAVIGATIONAL LIGHTING SYSTEM	Indirect	704123M	WEATHER STATION ROADWAY DEVICES 1 LANES	Indirect	750031P	MOTOR CONTROL CENTER	Indirect
704002M	ITS CONDUIT, TYPE A	Indirect	704135M	WEATHER STATION ROADWAY DEVICES 5 LANES	Indirect	750034P	CHANNEL FLOOD LIGHTS	BRIndirect
704003M	JUNCTION BOX ITS TYPE A	Indirect	704157M	FOUNDATION DMS GROUND MOUNTED	Indirect	750036P	TRAFFIC WARNING GATE	Indirect
704004M	JUNCTION BOX ITS REMOVAL	Indirect	704169M	DMS STANDARD GROUND MOUNTED	Indirect	750038P	DRAW BRIDGE BARRIER GATE	Direct
704006M	JUNCTION BOX ITS TYPE B	Indirect	704170M	DMS SIGN, ONE YEAR WARRANTY	Indirect	750043P	CONTROL DESK	Indirect
704007M	JUNCTION BOX ITS RELOCATION	Indirect	704171M	DMS SIGN	Indirect	750048P	DEMOLITION AND REMOVAL	Indirect
704009M	JUNCTION BOX ITS TYPE C	Indirect	704172M	DMS SIGN REMOVAL	Indirect	750050P	ELECTRICAL WORK	Indirect
704010M	JUNCTION BOX ITS TYPE D	Indirect	704177M	DMS SIGN INSTALL	Indirect	750052P	GROUNDING AND BONDING SYSTEM	Indirect

Line Item	Description	Type	Line Item	Description	Type	Line Item	Description	Type
750060P	TRAINING	Indirect	810003M	MOWING	Indirect	903006M	MISCELLANEOUS CONCRETE	Indirect
750062P	OPERATION AND MAINTENANCE MANUALS	Indirect	810006M	MOWING	Indirect	999999P	NO ITEM	Indirect
750064P	TESTING	BRIndirect	811003M	LARGE DECIDUOUS TREE, 3-3 1/2" CALIPER, B&B	Indirect	MMA001M	INSPECTION OF UNITS	Indirect
801003M	SELECTIVE THINNING	Indirect	811004M	LARGE DECIDUOUS TREE, 2 1/2-3" CALIPER, B&B	Indirect	MMA002M	CARTRIDGE, TYPE 80, REPLACE	Indirect
801006M	SELECTIVE THINNING	Indirect	811006M	LARGE DECIDUOUS TREE, 2-2 1/2" CALIPER, B&B	Indirect	MMA003M	CARTRIDGE, TYPE 81, REPLACE	Indirect
801009M	SELECTIVE CLEARING	Indirect	811015M	LARGE DECIDUOUS TREE, SEEDLING 18-24" HIGH, POT OR CONTAINER	Indirect	MMA004M	CARTRIDGE, TYPE 82, REPLACE	Indirect
801012M	SELECTIVE CLEARING	Indirect	811019M	LARGE DECIDUOUS TREE, 8-10' HIGH, MULTI-STEM, B&B	Indirect	MMA005M	SAFETY FLEX BELT, REPLACE	Indirect
802003M	TRIMMING EXISTING TREE, OVER 6" TO 12" DIAMETER	Indirect	811020M	DECIDUOUS TREE, 6-8' HIGH, #7 CONTAINER	Indirect	MMA006M	LEG SUPPORT, SAFETY FLEX BELT, REPLACE	Indirect
802006M	TRIMMING EXISTING TREE, OVER 12" TO 18" DIAMETER	Indirect	811021M	SMALL DECIDUOUS TREE, 3-3 1/2" CALIPER, B&B	Indirect	MMA007M	FENDER PANEL WITH HARDWARE, REPLACE	Indirect
802009M	TRIMMING EXISTING TREE, OVER 18" TO 24" DIAMETER	Indirect	811022M	SMALL DECIDUOUS TREE, 2 1/2-3" CALIPER, B&B	Indirect	MMA008M	FENDER PANEL WITHOUT HARDWARE, REPLACE	Indirect
802012M	TRIMMING EXISTING TREE, OVER 24" TO 30" DIAMETER	Indirect	811024M	SMALL DECIDUOUS TREE, 2-2 1/2" CALIPER, B&B	Indirect	MMA009M	STABILIZER, FENDER PANEL, REPLACE	Indirect
802015M	TRIMMING EXISTING TREE, OVER 30" TO 36" DIAMETER	Indirect	811027M	SMALL DECIDUOUS TREE, 1 1/4-1 1/2" CALIPER, B&B	Indirect	MMA010M	HINGE, FENDER PANEL, REPLACE	Indirect
802018M	TRIMMING EXISTING TREE, OVER 36" DIAMETER	Indirect	811028M	SMALL DECIDUOUS TREE, 3' HIGH, #2 CONTAINER	Indirect	MMA011M	END FRAME, 1' AND 1.5' WIDE, REPLACE	Indirect
802021M	TREE REMOVAL, OVER 6" TO 12" DIAMETER	Indirect	811029M	SMALL DECIDUOUS TREE, 6-7' HIGH B&B	Indirect	MMA012M	END FRAME, 2' AND 3' WIDE, REPLACE	Indirect
802024M	TREE REMOVAL, OVER 12" TO 18" DIAMETER	Indirect	811030M	SMALL DECIDUOUS TREE, 5-6' HIGH, B&B	Indirect	MMA013M	CENTER PANEL, 1.4' AND 2.2' WIDE, REPLACE	Indirect
802027M	TREE REMOVAL, OVER 18" TO 24" DIAMETER	Indirect	811031M	SMALL DECIDUOUS TREE, 7-8' HIGH, B&B	Indirect	MMA014M	CENTER PANEL, 3' AND 4' WIDE, REPLACE	Indirect
802030M	TREE REMOVAL, OVER 24" TO 30" DIAMETER	Indirect	811032M	SMALL DECIDUOUS TREE, 8-10' HIGH, B&B	Indirect	MMA015M	CABLE GUIDE, ADJUSTABLE, REPLACE	Indirect
802033M	TREE REMOVAL, OVER 30" TO 36" DIAMETER	Indirect	811033M	EVERGREEN TREE, 9-10' HIGH, B&B	Indirect	MMA016M	TUBE SPACER, 0.9', 1.5' AND 1.75' LONG, REPLACE	Indirect
802036M	TREE REMOVAL, OVER 36" DIAMETER	Indirect	811036M	EVERGREEN TREE, 8-9' HIGH, B&B	Indirect	MMA017M	TUBE SPACER, 2.2', 2.6' AND 3' LONG, REPLACE	Indirect
802037M	CERTIFIED TREE EXPERT	Indirect	811037M	EVERGREEN TREE, 7-8' HIGH B&B	Indirect	MMA018M	BRACKET, CARTRIDGE SUPPORT, REPLACE	Indirect
803006M	PREPARATION OF EXISTING SOIL	Indirect	811039M	EVERGREEN TREE, 6-7' HIGH, B&B	Indirect	MMA019M	BACKUP, STEEL (NARROW), REPLACE	Indirect
804006P	TOPSOILING, 4" THICK	Indirect	811042M	EVERGREEN TREE, 5-6' HIGH, B&B	Indirect	MMA020M	BACKUP, STEEL (MEDIUM), REPLACE	Indirect
804009P	TOPSOILING, 6" THICK	Indirect	811045M	EVERGREEN TREE, 4-5' HIGH, B&B	Indirect	MMA021M	BACKUP, STEEL (WIDE), REPLACE	Indirect
804010P	TOPSOILING, WETLAND, 6" THICK	Indirect	811048M	EVERGREEN TREE, 3-4' HIGH, B&B	Indirect	MMA022M	ADAPTOR, HORIZONTAL BRACE (NARROW), REPLACE	Indirect
804012P	TOPSOILING, 8" THICK	Indirect	811055M	DECIDUOUS TREE, 5'-8' HIGH, #7 CONTAINER	Indirect	MMA023M	ADAPTOR, HORIZONTAL BRACE (MEDIUM), REPLACE	Indirect
804013P	TOPSOILING, 12" THICK	Indirect	811056M	DECIDUOUS SHRUB, 4-5' HIGH, B&B	Indirect	MMA024M	ADAPTOR, HORIZONTAL BRACE (WIDE), REPLACE	Indirect
804014P	TOPSOILING	Indirect	811057M	DECIDUOUS SHRUB, 3-4' HIGH, B&B	Indirect	MMA025M	FOOT, BACKUP STEEL (NARROW), REPLACE	Indirect
804015P	BORROW TOPSOIL	Indirect	811060M	DECIDUOUS SHRUB, 24-30" HIGH, B&B	Indirect	MMA026M	FOOT, BACKUP STEEL (MEDIUM), REPLACE	Indirect
804019P	PLANTING SOIL BED, 24" THICK	Indirect	811061M	DECIDUOUS SHRUB, 30-36" HIGH, B&B	Indirect	MMA027M	FOOT, BACKUP STEEL (WIDE), REPLACE	Indirect
804020P	PLANTING SOIL BED, 36" THICK	Indirect	811062M	DECIDUOUS SHRUB, 18-24" HIGH, #1 CONTAINER	Indirect	MMA028M	ADAPTOR, DIAGONAL BRACE, REPLACE	Indirect
805003M	TURF REPAIR STRIP	Indirect	811063M	DECIDUOUS SHRUB, 18-24" HIGH, #3 CONTAINER	Indirect	MMA029M	BRACE, DIAGONAL, REPLACE	Indirect
805010M	REGRADE BERM	Indirect	811066M	DECIDUOUS SHRUB, 15-18" HIGH, #2 CONTAINER	Indirect	MMA030M	FOOT, DIAGONAL BRACE, REPLACE	Indirect
806003P	FERTILIZING AND SEEDING, TYPE A	Indirect	811067M	DECIDUOUS SHRUB, 24-36" HIGH, CONTAINER	Indirect	MMA031M	SHIM, RESTRAINING CABLE, REPLACE	Indirect
806006P	FERTILIZING AND SEEDING, TYPE A-3	Indirect	811069M	EVERGREEN SHRUB, 36-42" HIGH, B&B	Indirect	MMA032M	GROMMET, CABLE WEAR PLATE, REPLACE	Indirect
806009P	FERTILIZING AND SEEDING, TYPE A-4	Indirect	811072M	EVERGREEN SHRUB, 30-36" HIGH, B&B	Indirect	MMA033M	RESTRAINING CABLE ASSEMBLY, REPLACE	Indirect
806012P	FERTILIZING AND SEEDING, TYPE B	Indirect	811075M	EVERGREEN SHRUB, 24-30" HIGH, B&B	Indirect	MMA034M	SECONDARY CABLE ASSEMBLY, REPLACE	Indirect
806015P	FERTILIZING AND SEEDING, TYPE D	Indirect	811078M	EVERGREEN SHRUB, 18-24" HIGH, #3 CONTAINER	Indirect	MMA035M	PULL-OUT CABLE ASSEMBLY, REPLACE	Indirect
806018P	FERTILIZING AND SEEDING, TYPE F	Indirect	811087M	EVERGREEN SHRUB, 30-36" SPREAD, B&B	Indirect	MMA036M	FRONT CABLE ANCHOR, REPLACE	Indirect
806019P	FERTILIZING AND SEEDING, TYPE M	Indirect	811093M	EVERGREEN SHRUB, 18-24" SPREAD, #3 CONTAINER	Indirect	MMA037M	VINYL CELL, 2.9' LONG, YELLOW, REPLACE	Indirect
806020P	FERTILIZING AND SEEDING, TYPE R	Indirect	811097M	EVERGREEN SHRUB, 4-5' HIGH, B&B	Indirect	MMA038M	EXPANSION TYPE CONCRETE ANCHOR BOLT 3/4" X 7.5", REPLACE	Indirect
806024P	FERTILIZING AND SEEDING, TYPE W	Indirect	811099M	GROUND COVER OR VINE, #1 CONTAINER	Indirect	MMA039M	BRACKET, CARTRIDGE UPPER SUPPORT, REPLACE	Indirect
806027P	FERTILIZING AND SEEDING, TYPE PINELANDS	Indirect	811102M	GROUND COVER OR VINE, 4" SQUARE OR ROUND POT	Indirect	MMA040M	BRACKET, CARTRIDGE LOWER SUPPORT, REPLACE	Indirect
806028P	FERTILIZING AND SEEDING, BASIN SLOPE	Indirect	811108M	GROUND COVER OR VINE, 2" PLUG	Indirect	MMA041M	COVER, NOSE, REPLACE	Indirect
806029P	FERTILIZING AND SEEDING, BASIN BOTTOM	Indirect	811110P	COIR FASCINE	Indirect	MMA042M	NOSE, LEG ASSEMBLY, REPLACE PIN, ANCHOR, REPLACE	Indirect
806030P	WILDFLOWER SEEDING	Indirect	811111M	PERENNIAL, #1 CONTAINER	Indirect	MMA043M	MUSHROOM BOLT ASSEMBLY, REPLACE	Indirect
807003M	TOPSOIL STABILIZATION, TYPE 1 MAT	Indirect	811114M	PERENNIAL, #SP5 CONTAINER	Indirect	MMA044M	THRIE BEAM GUIDE RAIL ELEMENT, STANDARD 13' LONG, REPLACE	Indirect
807006M	TOPSOIL STABILIZATION, TYPE 2 MAT	Indirect	811117M	PERENNIAL, #SP4 CONTAINER	Indirect	MMA045M	THRIE BEAM GUIDE RAIL ELEMENT, STANDARD 4' LONG, REPLACE	Indirect
807009M	TOPSOIL STABILIZATION, TYPE 3 MAT	Indirect	811120M	PERENNIAL, 2" PLUG	Indirect	MMA046M	RESTRAINING CABLE LESS THAN 25' LONG, REPLACE	Indirect
807012M	TOPSOIL STABILIZATION, TYPE 4 MAT	Indirect	811123M	BULB	Indirect	MMA047M	RESTRAINING CABLE, GREATER THAN 25' LONG, REPLACE	Indirect
807015M	EROSION CONTROL MATTING	Indirect	811138M	PLANT ESTABLISHMENT PERIOD	Indirect	MMA048M	STEEL BACKUP 2' WIDE WITH DIAPHRAGM, REPLACE	Indirect
807018M	TURF REINFORCEMENT MATTING	Indirect	811140P	TREE MAINTENANCE	Indirect	MMA049M	STEEL BACKUP 2.5' WIDE WITH DIAPHRAGM, REPLACE	Indirect
807021M	WASHED RIVERJACK STONE WITH WEED BARRIER	Indirect	811150M	LANDSCAPE ACCENT WALL	Indirect	MMA050M	STEEL BACKUP 3' WIDE WITH DIAPHRAGM, REPLACE	Indirect
807024M	LANDSCAPE WEED BARRIER	Indirect	811170P	IRRIGATION SYSTEM	Indirect	MMA051M	SIDE STEEL BACKUP PANEL, REPLACE	Indirect
808003P	SODDING	Indirect	811171P	RESET IRRIGATION SYSTEM	Indirect	MMA052M	SIDE STEEL TRANSITION PANEL ASSEMBLY, REPLACE	Indirect
809003M	STRAW MULCHING	Indirect	811175P	RECONSTRUCT PLANTING BED	Indirect	MMA053M	TRANSITION PANEL SUPPORT BRACKET, REPLACE	Indirect
809006M	FIBER MULCHING	Indirect	812005P	LANDSCAPE TIMBER EDGING	Indirect	MMA054M	TRANSITION PANEL ASSEMBLY 7' LONG, REPLACE	Indirect
809009M	STONE MULCHING	Indirect	812006P	STEEL EDGING	Indirect	MMA055M	TRANSITION PANEL ASSEMBLY, W-THRIE BEAM 7.2' LONG, REPLACE	Indirect
809012M	GRAVEL MULCHING	Indirect	851003M	DECORATIVE WALL	Indirect	MMA056M	END SHOE, THRIE BEAM 2.5' LONG, REPLACE	Indirect
809015M	SHREDDED HARDWOOD BARK MULCHING	Indirect	851004P	DECORATIVE COLUMN	Indirect	MMA057M	EXTENSION PANEL, 2 DEGREES 5' LONG, REPLACE	Indirect
809018M	WOOD MULCHING	Indirect	900001P	SITEMANAGER SUPPLEMENTAL MATERIALS TESTING	Indirect	MMA058M	EXTENSION PANEL, THRIE BEAM 5' LONG, REPLACE	Indirect
809021M	WOOD MULCHING, 2" THICK	Indirect	903003M	CONCRETE STRENGTH QUALITY ADJUSTMENT	BRIndirect	MMA059M	END SHOE, W-BEAM 2.5' LONG, REPLACE	Indirect

Line Item	Description	Type		Line Item	Description	Type	Line Item	Description	Type
MMA060M	THRIE BEAM DEFLECTOR BACKUP, REPLACE	Indirect		MMA156M	WOOD POSTS FOR TELESCOPING GUIDE RAIL END TERMINALS, REPLACE	Indirect	MMB092M	CREW COORDINATOR	Indirect
MMA061M	THRIE BEAM SIDE BACKUP PANEL, REPLACE	Indirect		MMA157M	RAIL ELEMENT FOR TELESCOPING GUIDE RAIL END TERMINALS,	Indirect	MMB094M	AERIAL LIFT 85' BOOM EXTENSION	Indirect
MMA062M	CABLE GUIDE ASSEMBLY, REPLACE	Indirect		MMA158M	PIN, ANCHOR, REPLACE	Indirect	MMB107M	TOWABLE LIGHT TOWER	Indirect
MMA063M	THRIE BEAM DIAPHRAGM ASSEMBLY, REPLACE	Indirect		MMA159M	SLOTTED GUIDE RAIL TERMINALS, POWDER COATED	Indirect	MMB108M	TRAFFIC DIRECTORS FLAGGERS	Indirect
MMA064M	CARTRIDGE ASSEMBLY, TYPE 51, REPLACE	Indirect		MMA160M	BEAM GUIDE RAIL, POWDER COATED	Indirect	MMB109M	MANPOWER ACCESS MACHINE - TRUCK MOUNTED	Indirect
MMA065M	CHAIN RAIL ASSEMBLY (9' LONG OR LESS), REPLACE	Indirect		MMA162M	BEAM GUIDE RAIL POSTS 10' LONG, POWDER COATED	Indirect	MMB110M	MANPOWER ACCESS MACHINE - TRUCK MOUNTED (WD)	Indirect
MMA066M	CHAIN RAIL ASSEMBLY (GREATER THAN 9'), REPLACE	Indirect		MMA163M	X-TENSION GUIDE RAIL END TERMINAL	Indirect	MMB130M	QUALITY MANAGEMENT SYSTEM (QMS)	Indirect
MMA067M	CABLE ANCHOR, REPLACE	Indirect		MMA164M	DIAPHRAGM ASSEMBLY,	Indirect	MMB132M	RAILROAD SERVICES	Indirect
MMA068M	CABLE ANCHOR (STUD END), REPLACE	Indirect		MMA165M	YELLOW NOSE ASSEMBLY,	Indirect	MMB135M	POWER TOOL CLEANING AND PAINTING	Indirect
MMA069M	EXPANSION TYPE CONCRETE ANCHOR BOLT 0.5" X 6", REPLACE	Indirect		MMA166M	FENDER PANEL ASSEMBLY, QUAD BEAM	Indirect	MMB136M	MILLING, VARIABLE DEPTH	Indirect
MMA070M	TURNBUCKLE, 0.04" DIAMETER X 6", REPLACE	Indirect		MMA167M	TRANSITION PANEL,	Indirect	MMB142M	MANPOWERED ACCESS MACHINE	Indirect
MMA071M	TURNBUCKLE, 0.05" DIAMETER X 9", REPLACE	Indirect		MMA168M	TENSION STRUT BACKUP,	Indirect	MMB147M	ELECTRICAL BOXES, CONDUIT AND WIRING	Indirect
MMA072M	CHEVRON, 2.5' X 2.5', REPLACE	Indirect		MMA169M	MOUNTING HARDWARE FOR CONCRETE BACKUP	Indirect	MMB148M	MOVABLE BRIDGE MONITORING SYSTEM	BRIndirect
MMA073M	CARTRIDGE ASSEMBLY, TYPE I, REPLACE	Indirect		MMA170M	TRACC 350,	Indirect	MMB149M	REAL-TIME SURVEILLANCE, WARRANTY AND PART MANAGEMENT	Indirect
MMA074M	CARTRIDGE ASSEMBLY, TYPE II, REPLACE	Indirect		MMA171M	SCI70GM IMPACT ATTENUATOR	Indirect	MMB150M	ELECTRICAL TESTING AND MEASUREMENTS	Indirect
MMA078M	YELLOW NOSE ASSEMBLY, 2.5', REPLACE	Indirect		MMA172M	SCI100GM IMPACT ATTENUATOR	Indirect	MMB151M	TRAINING AND MANUALS	Indirect
MMA082M	END SHOE, QUAD BEAM, REPLACE	Indirect		MMB001M	SOLID WASTE DISPOSAL	Indirect	MMB155M	INSTALLATION OF MONITORING SYSTEM	BRIndirect
MMA090M	CARTRIDGE SUPPORT BRACKET FOR TENSION STRUT BACKUP, REPLACE	Indirect		MMB002M	REPAIR CATEGORY "A", CONCRETE	Direct	MMB157M	SENSORS, DETECTORS, AND MONITORING DEVICES	Indirect
MMA091M	CARTRIDGE SUPPORT BRACKET FOR DIAPHRAGM, REPLACE	Indirect		MMB004M	REPAIR CATEGORY "A", TIMBER	Direct	MMB159M	PROGRAMMING MOVABLE BRIDGE MONITORING SYSTEM	BRIndirect
MMA092M	SIDE PANEL FOR STEEL AND CONCRETE BACKUPS, REPLACE	Indirect		MMB005M	REPAIR CATEGORY "A" (WD), TIMBER	Direct	MMB160M	ELECTRICAL CONDUIT	Indirect
MMA093M	MUSHROOM WASHER ASSEMBLY, REPLACE	Indirect		MMB006M	REPAIR CATEGORY "B", TIMBER	Direct	MMB161M	OPERATION AND MAINTENANCE OF BRIDGE DURING CONSTRUCTIOI	BRIndirect
MMA094M	HINGE PLATE, FENDER PANEL FOR 7.5', REPLACE	Indirect		MMB007M	REPAIR CATEGORY "B" (WD), TIMBER	Direct	MMB162M	HVAC	Indirect
MMA095M	MONORAIL GUIDE, REPLACE	Indirect		MMB008M	REPAIR CATEGORY "C", TIMBER	Direct	MMB163M	MPT FOR SHOULDER OR LANE CLOSURE ON	Indirect
MMA100M	350.4 COVER, REPLACE	Indirect		MMB009M	REPAIR CATEGORY "C" (WD), TIMBER	Direct	MMB164M	MPT FOR SHOULDER OR LANE CLOSURE NOT ON INTERSTATE HIGHW	Indirect
MMA101M	350.6 COVER, REPLACE	Indirect		MMB010M	REPAIR CATEGORY "D", TIMBER	Direct	MMB165M	BUCKET VAN WITH OPERATOR - 30'	Indirect
MMA102M	350.9 COVER, REPLACE	Indirect		MMB011M	REPAIR CATEGORY "D" (WD), TIMBER	Direct	MMB166M	CHERRY PICKER WITH OPERATOR - 35'	Indirect
MMA103M	350.4 NEW BACKUP SINGLE CABLE, REPLACE	Indirect		MMB020M	DIVING CREW, TIMBER	Direct	MMB167M	CHERRY PICKER WITH OPERATOR - 60'	Indirect
MMA104M	350.6 NEW BACKUP SINGLE CABLE, REPLACE	Indirect		MMB021M	DIVING CREW (WD), TIMBER	Direct	MMB170M	ENGINE-GENERATOR REPLACEMENT	Indirect
MMA105M	350.9 NEW BACKUP SINGLE CABLE, REPLACE	Indirect		MMB022M	REPAIR CATEGORY "A", MOVABLE	Direct	MMB171M	AUXILIARY DRIVE REPLACEMENT	Indirect
MMA106M	350.4 CABLE SYSTEM FOR SIDE, REPLACE	Indirect		MMB024M	REPAIR CATEGORY "B", MOVABLE	Direct	MMB172M	LANE CLOSURES	Indirect
MMA107M	350.6 CABLE SYSTEM FOR SIDE, REPLACE	Indirect		MMB026M	WELDING CREW	Indirect	MMB173M	SOLID WASTE DISPOSAL	Indirect
MMA108M	350.9 CABLE SYSTEM FOR SIDE, REPLACE	Indirect		MMB028M	IRON WORKER	Indirect	MMB174M	BRIDGE & MONITORING SYSTEM INTEGRATION	BRIndirect
MMA109M	350.4 BOTTOM CABLE, REPLACE	Indirect		MMB029M	IRON WORKER (WD)	Indirect	MMB175M	FIVE MAN SIGN CREW	Indirect
MMA110M	350.6 BOTTOM CABLE, REPLACE	Indirect		MMB030M	SYSTEMS TECHNICIAN, MOVABLE	Indirect	MMB176M	THREE MAN SIGN CREW	Indirect
MMA111M	350.9 BOTTOM CABLE, REPLACE	Indirect		MMB034M	PLUMBER, MOVABLE	Indirect	MMB177M	BUCKET VAN WITH OPERATOR - 30'	Indirect
MMA112M	SIDE MOUNT CABLE ANCHOR, REPLACE	Indirect		MMB035M	PLUMBER (WD), MOVABLE	Indirect	MMB178M	CHERRY PICKER WITH OPERATOR - 60'	Indirect
MMA113M	SIDE DELINEATORS, REPLACE	Indirect		MMB038M	HELPER	Indirect	MMB179M	50' TO 60' CRANE STINGER, 8 TON WITH OPERATOR	Indirect
MMA114M	2-CABLE RETAINER ASSEMBLY, REPLACE	Indirect		MMB039M	HELPER (WD)	Indirect	MMB180M	CHERRY PICKER WITH OPERATOR - 35'	Indirect
MMA115M	4-CABLE RETAINER ASSEMBLY, REPLACE	Indirect		MMB044M	60' UNDERBRIDGE INSPECTION UNIT	BRIndirect	MMB181M	REPAIR CATEGORY "A"	Direct
MMA116M	CABLE ANCHOR COVER PLATE, REPLACE	Indirect		MMB059M	SHOP FABRICATION	Indirect	MMB182M	REPAIR CATEGORY "B"	Direct
MMA117M	FRONT REFLECTIVE SHIELD, REPLACE	Indirect		MMB060M	LABOR CREW, STEEL	Indirect	MMB183M	REPAIR CATEGORY "C"	Direct
MMA118M	RETAINER PLATE, REPLACE	Indirect		MMB062M	SITE SURVEY AND ENGINEERING	BRIndirect	MMB184M	REPAIR CATEGORY "D"	Direct
MMA119M	3" X 3" TUBING, REPLACE	Indirect		MMB067M	STRUCTURAL STEEL REPAIR	Direct	MMB185M	DIVING CREW	BRIndirect
MMA120M	GALVANIZED STABILIZER BAR, REPLACE	Indirect		MMB070M	BRIDGE HEADER REPAIR	Direct	MMB186M	LABOR CREW	Indirect
MMA121M	GALVANIZED FRONT TUBE, REPLACE	Indirect		MMB071M	CONCRETE SPALL REPAIR, TYPE 1	Direct	MMB187M	STEEL SURFACE PREPARATION AND COATING	Indirect
MMA122M	4" X 4" GALVANIZED WASHERS, REPLACE	Indirect		MMB072M	CONCRETE SPALL REPAIR, TYPE 2	Direct	MMD001M	MOBILIZATION PER SITE, NORTH	Indirect
MMA123M	U-BOLTS, REPLACE	Indirect		MMB073M	APPROACH SIDEWALK REPAIR	Indirect	MMD004M	FLOOD LIGHTS FOR NIGHTTIME OPERATIONS	Indirect
MMA124M	U-BOLT PLATE, REPLACE	Indirect		MMB074M	MICROPAVING JOINTS	Indirect	MMD006M	VARIABLE MESSAGE SIGN	Indirect
MMA125M	ANCHOR BOLTS FOR CONCRETE, 60 COUNTS, REPLACE	Indirect		MMB075M	DECK JOINT RECONSTRUCTION	Direct	MMD007M	DISCHARGE PUMP	Indirect
MMA126M	ANCHOR BOLTS FOR ASPHALT, 60 COUNTS, REPLACE	Indirect		MMB076M	CONCRETE BALUSTRADE REPAIR	Direct	MMD014M	CLEANING EXISTING DRAINAGE STRUCTURE	BRIndirect
MMA127M	TRANSITION PLATE, REPLACE	Indirect		MMB077M	REPAIR CONCRETE CURB	Direct	MMD015M	CLEANING OF BOX CULVERTS	Indirect
MMA128M	REPLACEMENT CYLINDERS	Indirect		MMB080M	BORROW EXCAVATION, SELECTED MATERIAL	Indirect	MMD016M	BITUMINOUS CONCRETE PATCH, VARIOUS THICK	Indirect
MMA129M	C-CHANNEL STAKES FOR ASPHALT, 12 COUNTS, REPLACE	Indirect		MMB081M	REPAIR OF SLOPE PROTECTION	Indirect	MMD021M	RIPRAP STONE SLOPE PROTECTION, 6" THICK	Indirect
MMA130M	CRASH CUSHIONS, REACT 350 SYSTEM, 4 CYLINDERS, REMOVE &	Indirect		MMB084M	NEAR-WHITE BLAST CLEANING AND PAINTING	Direct	MMD022M	PIPE REPAIRS	Indirect
MMA131M	CRASH CUSHIONS, REACT 350 SYSTEM, 6 CYLINDERS, REMOVE &	Indirect		MMB085M	FLOODLIGHTS FOR NIGHTTIME OPERATION	BRIndirect	MMD024M	REPLACE PIPE	Indirect
MMA132M	CRASH CUSHIONS, REACT 350 SYSTEM, 9 CYLINDERS, REMOVE &	Indirect		MMB086M	DECK CORROSION INHIBITOR	Direct	MMD029M	MINOR REPAIR OF STRUCTURES, LESS THAN 6' IN DEPTH	BRIndirect
MMA151M	CRASH CUSHIONS, QUADGUARD, 3 BAYS, 2.5'	Indirect		MMB087M	DECK JOINT RESEAL (SILICON)	Direct	MMD030M	MINOR REPAIR OF STRUCTURES, GREATER THAN 6' IN DEPTH	BRIndirect
MMA152M	CRASH CUSHIONS, QUADGUARD, 5 BAYS, 2.5' WIDE REMOVE AND CONS	Indirect		MMB088M	BRIDGE DECK CRACK SEALING	Direct	MMD033M	TREE REMOVAL	Indirect
MMA153M	CRASH CUSHIONS, QUADGUARD, 7 BAYS, 2.5' WIDE, REMOVE AND CON	Indirect		MMB089M	DECK JOINT REPAIR	Direct	MMD035M	CLEARING SITE	Direct
MMA154M	CRASH CUSHIONS, QUADGUARD, 8 BAYS, 2.5' WIDE REMOVE AND CONS	Indirect		MMB090M	DECK JOINT RESEAL (RUBBER ASPHALT)	Direct	MMD037M	DISPOSAL OF LITTER/DEBRIS	Indirect
MMA155M	SLEEVE FOR TELESCOPING GUIDE RAIL END TERMINALS, REPLACE	Indirect		MMB091M	FORCE ACCOUNT, LABOR, EQUIPMENT AND MATERIALS	BRIndirect	MMD038M	RECYCLE/REUSE OF SWEEPINGS	Indirect

Line Item	Description	Type		Line Item	Description	Type		Line Item	Description	Type
MMD039M	DISPOSAL OF TRASH AND BULKY WASTE	Indirect		MME109M	CRANE	Indirect		MMR004M	MOBILIZATION	BRIndirect
MMD041M	REUSE/RECYCLE OF SOIL/SEDIMENTS & MATERIALS	Indirect		MME110M	REPLACEMENT TRAFFIC SIGNAL CABLE 2/C	Indirect		MMR005M	SPECIAL MOBILIZATION	Indirect
MMD042M	RETROFIT COVER PLATE FOR INLET CURB PIECE	Indirect		MME111M	REPLACEMENT TRAFFIC SIGNAL CABLE 5/C	Indirect		MMR006M	EMERGENCY MOBILIZATION	Indirect
MMD045M	TRENCH REPAIR	Indirect		MME112M	REPLACEMENT TRAFFIC SIGNAL CABLE 7/C	Indirect		MMR008M	MILLING 2" AVERAGE DEPTH (UNDER 2000 S.Y.)	Indirect
MMD046M	TRENCHLESS PIPE REPAIR	Indirect		MME113M	REPLACEMENT TRAFFIC SIGNAL CABLE 10/C	Indirect		MMR009M	MILLING 2" AVERAGE DEPTH (2001 TO 8000 S.Y.)	Indirect
MMD047M	CLEAN EXISTING TRASH RACK	Indirect		MME114M	TRAFFIC SIGNAL STANDARD, ALUMINUM	Indirect		MMR010M	MILLING 2" AVERAGE DEPTH (OVER 8000 S.Y.)	Indirect
MMD048M	DISCHARGE PIPE	Indirect		MME115M	TRAFFIC SIGNAL STANDARD, STEEL	Indirect		MMR011M	EARTH EXCAVATION FOR TEST PITS	Indirect
MMD049M	REPAIR OF STRUCTURE	Direct		MME116M	PEDESTAL SIGNAL STANDARD	Indirect		MMR012M	REGRADE BERM FOR BEAM GUIDE RAIL	Indirect
MMD050M	SAND BED, VARIABLE THICKNESS	Indirect		MME117M	TRAFFIC SIGNAL MAST ARM, ALUMINUM	Indirect		MMR033M	ADIEM ENERGY ABSORBING END TREATMENT	Indirect
MMD051M	PLANTING SOIL BED, VARIABLE THICKNESS	Indirect		MME118M	TRAFFIC SIGNAL MAST ARM, STEEL	Indirect		MMR034M	REMOVAL OF SIDEWALK	Indirect
MMD052M	CLEANING MANUFACTURED TREATMENT DEVICE	Indirect		MME119M	TRAFFIC SIGNAL HEAD	Indirect		MMR035M	CABLE GUIDE WIRE	Indirect
MMD053M	Emergency Repair Work - Extra Work Items	Indirect		MME120M	PEDESTRIAN SIGNAL HEAD	Indirect		MMR036M	CABLE GUIDE WIRE ANCHORAGES	Indirect
MME002M	CONDUIT CLEARING	Indirect		MME121M	PUSH BUTTON	Indirect		MMR037M	TRI-BEAM GUIDE RAIL ELEMENT	Indirect
MME003M	RWIS SENSOR INSTALLATION	Indirect		MME122M	FOUNDATION, TYPE PBI	Indirect		MMR038M	BEAM GUIDE RAIL POSTS, EXCLUDING SPACERS	Indirect
MME004M	RWIS PROBE INSTALLATION	Indirect		MME123M	LIGHTING LOAD CENTER	Indirect		MMR039M	BEAM GUIDE RAIL POSTS, ADDITIONAL LENGTH	Indirect
MME008M	FOUNDATION REMOVAL, ELECTRICAL	Indirect		MME124M	VARIOUS SIZE LIGHTING CONDUIT, NON-METAL	Indirect		MMR040M	DRILLING OR CORING FOR BEAM GUIDE RAIL POSTS, NON-BRIDGE	Indirect
MME009M	TRANSITE JUNCTION BOX REMOVAL	Indirect		MME125M	SERVICE WIRE	Indirect		MMR041M	SAWCUTTING FOR BEAM GUIDE RAIL POSTS, NON-BRIDGE	Indirect
MME010M	TROUBLESHOOTING	Indirect		MME126M	MULTIPLE LIGHTING WIRE	Indirect		MMR042M	BEAM GUIDE RAIL INSTALLATION IN ROCK	Indirect
MME011M	BASE DOOR, TYPE T/C	Indirect		MME127M	UNDERDECK LIGHTING ASSEMBLIES INSTALLATI	Direct		MMR043M	RESET BEAM GUIDE RAIL WITH NEW POSTS	Indirect
MME012M	BASE DOOR, TYPE K	Indirect		MME128M	LIGHTING ASSEMBLIES REMOVAL	Indirect		MMR044M	ATTACHMENT TO STRUCTURE, BALUSTRADE, DETAIL NO.CD-609-10.1	Direct
MME013M	BASE DOOR, TYPE P	Indirect		MME129M	HIGH MAST TOWER ASSEMBLIES	Indirect		MMR045M	ATTACHMENT TO STRUCTURE, DETAIL NO. CD-609-10.2	Direct
MME015M	PEDESTRIAN LED INSTALLATION	Indirect		MME130M	WIRE REMOVAL	Indirect		MMR046M	ATTACHMENT TO STRUCTURE, DETAIL NO. CD-609-10.2, W/	Direct
MME016M	LIGHTING ASSEMBLIES INSTALLATION, TYPE N	Indirect		MME139M	ADA Push Button Installation	Indirect		MMR047M	ATTACHMENT TO STRUCTURE, TYPE A	Direct
MME017M	HIGHWAY LIGHTING BALLAST INSTALLATION	Indirect		MME140M	12 RAG Revised Price	Indirect		MMR048M	ATTACHMENT TO STRUCTURE, TYPE B	Direct
MME018M	HIGHWAY LIGHTING BALLAST INSTALLATION, TYPE N	Indirect		MME141M	8 RAG Revised Price	Indirect		MMR049M	EXTRA SPACER BLOCKS	Indirect
MME024M	HIGHWAY LIGHTING RELAMPING	Indirect		MME142M	Rt 295 Camera Installation	Indirect		MMR050M	REMOVAL OF WIRE ROPE GUARD FENCE	Indirect
MME025M	HIGHWAY LIGHTING RELAMPING, TYPE N	Indirect		MMG001M	FORCE ACCOUNT, MATERIALS	Indirect		MMR051M	REMOVAL OF DUAL FACED BEAM GUIDE RAIL	Indirect
MME026M	BASE DOOR, TYPE L	Indirect		MMG002M	FORCE ACCOUNT, LABOR	Indirect		MMR052M	REMOVAL OF MEDIAN BREAKAWAY CABLE TERMINAL	Indirect
MME027M	PEC REPLACEMENT	Indirect		MMG005M	CELLULAR PHONE SERVICE	Indirect		MMR053M	REMOVAL OF BREAKAWAY CABLE TERMINAL - INCLUDING CONCRETE	Indirect
MME051M	SIGN LIGHTING RELAMPING	Indirect		MMG006M	E-Z PASS	Indirect		MMR054M	WOOD POSTS FOR EXTRUDER TERMINALS	Indirect
MME052M	SIGN LIGHTING RELAMPING, TYPE N	Indirect		MMG007M	FIELD OFFICE EQUIPMENT	BRIndirect		MMR055M	BUFFER END PIECES FOR END TREATMENTS	Indirect
MME055M	SIGN LUMINAIRE INSTALLATION	Indirect		MMG008M	TRAFFIC CONTROL TRUCK WITH MOUNTED CRASH CUSHION	BRIndirect		MMR056M	WOODEN POSTS FOR END TREATMENTS	Indirect
MME056M	SIGN LUMINAIRE INSTALLATION, TYPE N	Indirect		MMG009M	E-Z PASS	Indirect		MMR057M	RAIL ELEMENTS FOR END TREATMENTS	Indirect
MME068M	LED TYPE R-ARROW-R	Indirect		MMG010M	TELEPHONE SERVICE	BRIndirect		MMR058M	REFLECTIVE SHEETING FOR END TREATMENTS	Indirect
MME069M	LED TYPE A-ARROW-R	Indirect		MMI002M	ELECTRICIAN	Indirect		MMR060M	FLASHING ARROW BOARD, 4' X 8'	BRIndirect
MME070M	LED TYPE G-ARROW-R	Indirect		MMI003M	BOLLARD	Indirect		MMR061M	PORTABLE VARIABLE MESSAGE SIGN	BRIndirect
MME071M	LED TYPE A/G-ARROW-R	Indirect		MMI005M	INSTALL FIBER OPTIC CABLE	Indirect		MMR062M	TRAFFIC CONTROL TRUCK WITH MOUNTED CRASH CUSHION	BRIndirect
MME072M	LED TYPE REPLACEMENT	Indirect		MMI006M	FIBER OPTIC SPLICE (1-8 FIBERS)	Indirect		MMR064M	LONG LIFE PATTERNED CONTRAST PAVEMENT MARKINGS TAPE, 7"	Indirect
MME078M	IMAGING DETECTION SYSTEM, 1 CAMERA	Indirect		MMI007M	FIBER OPTIC SPLICE (9-24 FIBERS)	Indirect		MMR065M	REMOVAL OF LONG LIFE PATTERNED CONTRAST PAVEMENT MARKIN	Indirect
MME079M	IMAGING DETECTION SYSTEM, 2 CAMERA	Indirect		MMI008M	FIBER OPTIC SPLICE (25-48 FIBERS)	Indirect		MMR066M	REMOVAL OF TRAFFIC STRIPES, LINES EPOXY RESIN	Indirect
MME080M	IMAGING DETECTION SYSTEM, 3 CAMERA	Indirect		MMI009M	CONDUIT REPAIR, GRASS AREA	Indirect		MMR067M	REMOVAL OF TRAFFIC MARKINGS, LINES	Indirect
MME081M	IMAGING DETECTION SYSTEM, 4 CAMERA	Indirect		MMI010M	CONDUIT REPAIR, ROADWAY	Indirect		MMR068M	REMOVAL OF TRAFFIC MARKINGS, SYMBOLS THE	Indirect
MME082M	IMAGING DETECTION SYSTEM, 5 CAMERA	Indirect		MMI011M	38" JUNCTION BOX LID REPLACEMENT	Indirect		MMR072M	INTERMEDIATE CABLE GUIDE WIRE ANCHOR ASSEMBLY	Indirect
MME084M	TRAFFIC DIRECTORS	Indirect		MMI013M	TRACE WIRE #14 AWG	Indirect		MMR073M	NON-VEGETATIVE SURFACE, POLYESTER MATTING	Indirect
MME085M	VISOR REPLACEMENT	Indirect		MMI014M	TRANSFORMER ASSEMBLY	Indirect		MMR076M	GUIDE RAIL EXTRUDERS	Indirect
MME089M	RWIS WIRELESS SENSOR INSTALLATION	Indirect		MMI015M	INSTALL CCTV CAMERA AND CONTROLLER ASSEMBLY	Indirect		MMR077M	MOBILIZATION OF MILLING AND HOT MIX ASPHALT PAVING EQUIPM	Indirect
MME090M	ROADWAY JUNCTION BOX	Indirect		MMI016M	CAMERA MAINTENANCE	Indirect		MMR078M	HMA MILLING, 2"	Indirect
MME091M	LED TYEP 12 RAG	Indirect		MMI019M	SAFETY FOR NJDOT MAINTENANCE STAFF	Indirect		MMR079M	HMA MILLING, 3"	Indirect
MME092M	LED TYPE 8 RAG	Indirect		MMI020M	SAFETY FOR NJDOT MAINTENANCE STAFF, NIGHTTIME	Indirect		MMR080M	CLEARING SITE	Direct
MME093M	LIGHTING ASSEMBLIES, TYPE L-R-X	Indirect		MMI021M	DMS MAINTENANCE	Indirect		MMR081M	TRAFFIC MARKINGS, LINES, THERMOPLASTIC	Indirect
MME094M	LIGHTING ASSEMBLIES, TYPE L-R-CF	Indirect		MMI022M	MISCELLANEOUS MATERIAL	Indirect		MMR083M	MOBILE RETROREFLECTOMETER SERVICE	Indirect
MME095M	LIGHTING ASSEMBLIES, TYPE L-R-V-X	Indirect		MMI023M	REPLACE HVAC SYSTEM	Indirect		MMR087M	MICROMILLING	Indirect
MME096M	LIGHTING ASSEMBLIES, TYPE L-R-MG	Indirect		MMI024M	RTMS MAINTENANCE	Indirect		MMR088M	SAWCUTTING	Indirect
MME097M	LIGHTING ASSEMBLIES, TYPE L-R-U-W	Indirect		MMI027M	WIM TEMPERATURE SENSOR	Indirect		MMR089M	PREPARATION OF ROADBED	Indirect
MME098M	LIGHTING ASSEMBLIES, TYPE L-R-U-P	Indirect		MMI031M	WIM AXLE SENSOR PIEZO	Indirect		MMR090M	LONG LIFE PATTERNED CONTRAST PAVEMENT MARKINGS TAPE, 9"	Indirect
MME099M	LIGHTING ASSEMBLIES, TYPE L-R-W	Indirect		MMI036M	FOUNDATION REMOVAL, ITS	Indirect		MMR091M	CONCRETE WASHOUT SYSTEM	Indirect
MME100M	TOWER RELAMPING	Indirect		MMI037M	REMOVAL OF JUNCTION BOXES	Indirect		MMR093M	FULL DEPTH CONCRETE PAVEMENT REPAIR, PRE-CAST	Indirect
MME101M	TOWER LUMINAIRE INSTALLATION	Indirect		MMI042M	REDEPLOYMENT OF PORTABLE TRAILER MOUNTED CCTV CAMERA	Indirect		MMR101M	STORM DRAIN MARKERS	Indirect
MME102M	LIGHTING ASSEMBLIES INSTALLATION	Indirect		MMI043M	REMOVAL ____	Indirect		MMR102M	THRIE BEAM POST	Indirect
MME103M	LIGHTING ASSEMBLIES, TYPE L-R-40-Y	Indirect		MMR001M	CONSTRUCTION LAYOUT	BRIndirect		MMR103M	STEEL POST FOR EXTRUDER TERMINAL	Indirect
MME108M	FUSE KITS	Indirect		MMR003M	MOBILIZATION, PREMIUM	Indirect		MMR104M	STEEL POST FOR FLARED END TERMINAL	Indirect

Line Item	Description	Type		Line Item	Description	Type		Line Item	Description	Type
MMR105M	ROADWAY EXCAVATION, UNCLASSIFIED,	Indirect								
MMR106M	CONCRETE MILLING, 2" OR LESS	Indirect								
MMR107M	Traffic Markings, Symbols, Long Life, Epoxy Resin	Indirect								
MMR108M	Hydro-Blasting	Indirect								
MMR109M	DIAMOND GRINDING, ASPHALT PAVEMENT	Indirect								

APPENDIX C – Unused Elements

Element #	Element Description
23	<Edit Long Element Description>
34	ConcDk prot w/membrane AC overlay coated bars-PREC
35	Conc Deck prot w/thin overlay & coated bars-PRECAS
53	Concrete Slab - Protected w/ Cathodic System
143	P/S Conc Arch
146	Cable - Uncoated (not embedded in concrete)
154	P/S Conc Floor Beam
156	Timber Floor Beam
213	Painted Steel Open Girder--Concrete Encased
214	Prestress Conc Abutment
242	Timber Culvert
260	Unpainted Steel Sheeting
261	Painted Steel Sheeting
262	Prestress Conc Sheeting
263	Reinf Conc Sheeting
264	Timber Sheeting
316	Steel Plate/Sliding Plate--Moveable Bearing
317	Rockers--Moveable Bearing
318	Other--Moveable Bearing
382	Reinf Conc Diaphragm
383	Timber Diaphragm
388	Wing Wall Footings
396	Reinforced Conc Pier Wall--Crash Wall

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	53 - Concrete Slab - Protected w/ Cathodic System
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	52 - Concrete Slab - Protected w/ Coated Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	44 - Concrete Slab - Protected w/ Thin Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	40 - Concrete Slab - Protected w/ AC Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	14 - Concrete Deck - Protected w/ AC Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	13 - Concrete Deck - Unprotected w/ AC Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	39 - Concrete Slab - Unprotected w/ AC Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	18 - Concrete Deck - Protected w/ Thin Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	22 - Concrete Deck - Protected w/ Rigid Overlay
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	26 - Concrete Deck - Protected w/ Coated Bars
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	27 - Concrete Deck - Protected w/ Cathodic System
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	38 - Concrete Slab - Bare
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	12 - Concrete Deck - Bare
020015R23C - REPAIR OF CONCRETE DECK, TYPE B Count	19
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	40 - Concrete Slab - Protected w/ AC Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	53 - Concrete Slab - Protected w/ Cathodic System
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	52 - Concrete Slab - Protected w/ Coated Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	44 - Concrete Slab - Protected w/ Thin Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	18 - Concrete Deck - Protected w/ Thin Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	39 - Concrete Slab - Unprotected w/ AC Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	14 - Concrete Deck - Protected w/ AC Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	22 - Concrete Deck - Protected w/ Rigid Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	26 - Concrete Deck - Protected w/ Coated Bars
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	12 - Concrete Deck - Bare
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	27 - Concrete Deck - Protected w/ Cathodic System
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	38 - Concrete Slab - Bare
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	13 - Concrete Deck - Unprotected w/ AC Overlay
020015R25C - REPAIR OF CONCRETE DECK, TYPE C Count	19
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	231 - Painted Steel Cap
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	131 - Painted Steel Deck Truss
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	202 - Painted Steel Column or Pile Extension
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	161 - Painted Steel Pin and/or Pin and Hanger Assembly
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	141 - Painted Steel Arch
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	121 - Painted Steel Bottom Chord Thru Truss
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	113 - Painted Steel Stringer
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	107 - Painted Steel Open Girder/Beam
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	102 - Painted Steel Closed Web/Box Girder
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	126 - Painted Steel Thru Truss (excl. bottom chord)
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	152 - Painted Steel Floor Beam
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING Count	11
02001MB086 - DECK CORROSION INHIBITOR	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
02001MB086 - DECK CORROSION INHIBITOR	39 - Concrete Slab - Unprotected w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	40 - Concrete Slab - Protected w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	44 - Concrete Slab - Protected w/ Thin Overlay
02001MB086 - DECK CORROSION INHIBITOR	48 - Concrete Slab - Protected w/ Rigid Overlay
02001MB086 - DECK CORROSION INHIBITOR	53 - Concrete Slab - Protected w/ Cathodic System
02001MB086 - DECK CORROSION INHIBITOR	55 - Timber Slab - w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
02001MB086 - DECK CORROSION INHIBITOR	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
02001MB086 - DECK CORROSION INHIBITOR	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
02001MB086 - DECK CORROSION INHIBITOR	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
02001MB086 - DECK CORROSION INHIBITOR	52 - Concrete Slab - Protected w/ Coated Bars
02001MB086 - DECK CORROSION INHIBITOR	38 - Concrete Slab - Bare
02001MB086 - DECK CORROSION INHIBITOR	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
02001MB086 - DECK CORROSION INHIBITOR	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
02001MB086 - DECK CORROSION INHIBITOR	54 - Timber Slab
02001MB086 - DECK CORROSION INHIBITOR	14 - Concrete Deck - Protected w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	18 - Concrete Deck - Protected w/ Thin Overlay
02001MB086 - DECK CORROSION INHIBITOR	22 - Concrete Deck - Protected w/ Rigid Overlay

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
02001MB086 - DECK CORROSION INHIBITOR	26 - Concrete Deck - Protected w/ Coated Bars
02001MB086 - DECK CORROSION INHIBITOR	28 - Steel Deck - Open Grid
02001MB086 - DECK CORROSION INHIBITOR	13 - Concrete Deck - Unprotected w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	29 - Steel Deck - Concrete Filled Grid
02001MB086 - DECK CORROSION INHIBITOR	12 - Concrete Deck - Bare
02001MB086 - DECK CORROSION INHIBITOR	30 - Steel Deck - Corrugated/Orthotropic/Etc.
02001MB086 - DECK CORROSION INHIBITOR	31 - Timber Deck - Bare
02001MB086 - DECK CORROSION INHIBITOR	32 - Timber Deck - w/ AC Overlay
02001MB086 - DECK CORROSION INHIBITOR	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
02001MB086 - DECK CORROSION INHIBITOR	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
02001MB086 - DECK CORROSION INHIBITOR	27 - Concrete Deck - Protected w/ Cathodic System
02001MB086 - DECK CORROSION INHIBITOR Count	30
02001MB087 - DECK JOINT RESEAL (SILICON)	305 - Finger Dams--Assembly Joint/Seal
02001MB087 - DECK JOINT RESEAL (SILICON)	307 - Other--Assembly Joint/Seal
02001MB087 - DECK JOINT RESEAL (SILICON)	306 - Sliding Plates--Assembly Joint/Seal
02001MB087 - DECK JOINT RESEAL (SILICON)	304 - Open Expansion Joint
02001MB087 - DECK JOINT RESEAL (SILICON)	302 - Compression Joint Seal
02001MB087 - DECK JOINT RESEAL (SILICON)	300 - Strip Seal Expansion Joint
02001MB087 - DECK JOINT RESEAL (SILICON)	301 - Pourable Joint Seal
02001MB087 - DECK JOINT RESEAL (SILICON) Count	7
02001MB088 - BRIDGE DECK CRACK SEALING	40 - Concrete Slab - Protected w/ AC Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
02001MB088 - BRIDGE DECK CRACK SEALING	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
02001MB088 - BRIDGE DECK CRACK SEALING	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
02001MB088 - BRIDGE DECK CRACK SEALING	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
02001MB088 - BRIDGE DECK CRACK SEALING	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
02001MB088 - BRIDGE DECK CRACK SEALING	53 - Concrete Slab - Protected w/ Cathodic System
02001MB088 - BRIDGE DECK CRACK SEALING	52 - Concrete Slab - Protected w/ Coated Bars
02001MB088 - BRIDGE DECK CRACK SEALING	44 - Concrete Slab - Protected w/ Thin Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	38 - Concrete Slab - Bare
02001MB088 - BRIDGE DECK CRACK SEALING	358 - Deck Cracking
02001MB088 - BRIDGE DECK CRACK SEALING	27 - Concrete Deck - Protected w/ Cathodic System
02001MB088 - BRIDGE DECK CRACK SEALING	26 - Concrete Deck - Protected w/ Coated Bars
02001MB088 - BRIDGE DECK CRACK SEALING	22 - Concrete Deck - Protected w/ Rigid Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	18 - Concrete Deck - Protected w/ Thin Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	14 - Concrete Deck - Protected w/ AC Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	13 - Concrete Deck - Unprotected w/ AC Overlay
02001MB088 - BRIDGE DECK CRACK SEALING	12 - Concrete Deck - Bare
02001MB088 - BRIDGE DECK CRACK SEALING	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
02001MB088 - BRIDGE DECK CRACK SEALING	39 - Concrete Slab - Unprotected w/ AC Overlay
02001MB088 - BRIDGE DECK CRACK SEALING Count	20
02001MB089 - DECK JOINT REPAIR	300 - Strip Seal Expansion Joint
02001MB089 - DECK JOINT REPAIR	307 - Other--Assembly Joint/Seal
02001MB089 - DECK JOINT REPAIR	306 - Sliding Plates--Assembly Joint/Seal
02001MB089 - DECK JOINT REPAIR	305 - Finger Dams--Assembly Joint/Seal
02001MB089 - DECK JOINT REPAIR	304 - Open Expansion Joint
02001MB089 - DECK JOINT REPAIR	303 - Assembly Joint/Seal (modular)
02001MB089 - DECK JOINT REPAIR	302 - Compression Joint Seal
02001MB089 - DECK JOINT REPAIR	301 - Pourable Joint Seal
02001MB089 - DECK JOINT REPAIR Count	8
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	304 - Open Expansion Joint
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	305 - Finger Dams--Assembly Joint/Seal
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	307 - Other--Assembly Joint/Seal
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	302 - Compression Joint Seal
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	301 - Pourable Joint Seal
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	300 - Strip Seal Expansion Joint
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	306 - Sliding Plates--Assembly Joint/Seal
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	303 - Assembly Joint/Seal (modular)
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT) Count	8
203111M - DEMONSTRATION STATIC LOAD TEST	225 - Unpainted Steel Submerged Pile
203111M - DEMONSTRATION STATIC LOAD TEST	226 - P/S Conc Submerged Pile
203111M - DEMONSTRATION STATIC LOAD TEST	227 - Reinforced Conc Submerged Pile
203111M - DEMONSTRATION STATIC LOAD TEST	228 - Timber Submerged Pile
203111M - DEMONSTRATION STATIC LOAD TEST	220 - Reinforced Conc Submerged Pile Cap/Footing
203111M - DEMONSTRATION STATIC LOAD TEST Count	5
401027M - POLYMERIZED JOINT ADHESIVE	39 - Concrete Slab - Unprotected w/ AC Overlay
401027M - POLYMERIZED JOINT ADHESIVE	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
401027M - POLYMERIZED JOINT ADHESIVE	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
401027M - POLYMERIZED JOINT ADHESIVE	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
401027M - POLYMERIZED JOINT ADHESIVE	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
401027M - POLYMERIZED JOINT ADHESIVE	55 - Timber Slab - w/ AC Overlay

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
401027M - POLYMERIZED JOINT ADHESIVE	44 - Concrete Slab - Protected w/ Thin Overlay
401027M - POLYMERIZED JOINT ADHESIVE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
401027M - POLYMERIZED JOINT ADHESIVE	13 - Concrete Deck - Unprotected w/ AC Overlay
401027M - POLYMERIZED JOINT ADHESIVE	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
401027M - POLYMERIZED JOINT ADHESIVE	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
401027M - POLYMERIZED JOINT ADHESIVE	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
401027M - POLYMERIZED JOINT ADHESIVE	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
401027M - POLYMERIZED JOINT ADHESIVE	32 - Timber Deck - w/ AC Overlay
401027M - POLYMERIZED JOINT ADHESIVE	18 - Concrete Deck - Protected w/ Thin Overlay
401027M - POLYMERIZED JOINT ADHESIVE	14 - Concrete Deck - Protected w/ AC Overlay
401027M - POLYMERIZED JOINT ADHESIVE	40 - Concrete Slab - Protected w/ AC Overlay
401027M - POLYMERIZED JOINT ADHESIVE Count	17
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	48 - Concrete Slab - Protected w/ Rigid Overlay
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	38 - Concrete Slab - Bare
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	12 - Concrete Deck - Bare
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	22 - Concrete Deck - Protected w/ Rigid Overlay
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE Count	7
405018M - CONTRACTION JOINT ASSEMBLY	305 - Finger Dams--Assembly Joint/Seal
405018M - CONTRACTION JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
405018M - CONTRACTION JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
405018M - CONTRACTION JOINT ASSEMBLY	302 - Compression Joint Seal
405018M - CONTRACTION JOINT ASSEMBLY	301 - Pourable Joint Seal
405018M - CONTRACTION JOINT ASSEMBLY	300 - Strip Seal Expansion Joint
405018M - CONTRACTION JOINT ASSEMBLY	306 - Sliding Plates--Assembly Joint/Seal
405018M - CONTRACTION JOINT ASSEMBLY Count	7
405021M - EXPANSION JOINT ASSEMBLY	306 - Sliding Plates--Assembly Joint/Seal
405021M - EXPANSION JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
405021M - EXPANSION JOINT ASSEMBLY	305 - Finger Dams--Assembly Joint/Seal
405021M - EXPANSION JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
405021M - EXPANSION JOINT ASSEMBLY	302 - Compression Joint Seal
405021M - EXPANSION JOINT ASSEMBLY	301 - Pourable Joint Seal
405021M - EXPANSION JOINT ASSEMBLY	300 - Strip Seal Expansion Joint
405021M - EXPANSION JOINT ASSEMBLY Count	7
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	38 - Concrete Slab - Bare
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	53 - Concrete Slab - Protected w/ Cathodic System
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	52 - Concrete Slab - Protected w/ Coated Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	44 - Concrete Slab - Protected w/ Thin Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	12 - Concrete Deck - Bare
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	40 - Concrete Slab - Protected w/ AC Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	27 - Concrete Deck - Protected w/ Cathodic System
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	26 - Concrete Deck - Protected w/ Coated Bars
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	22 - Concrete Deck - Protected w/ Rigid Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	18 - Concrete Deck - Protected w/ Thin Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	14 - Concrete Deck - Protected w/ AC Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	13 - Concrete Deck - Unprotected w/ AC Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	39 - Concrete Slab - Unprotected w/ AC Overlay
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V Count	19
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	52 - Concrete Slab - Protected w/ Coated Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	53 - Concrete Slab - Protected w/ Cathodic System
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	39 - Concrete Slab - Unprotected w/ AC Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	12 - Concrete Deck - Bare
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	38 - Concrete Slab - Bare
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	27 - Concrete Deck - Protected w/ Cathodic System
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	26 - Concrete Deck - Protected w/ Coated Bars
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	22 - Concrete Deck - Protected w/ Rigid Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	18 - Concrete Deck - Protected w/ Thin Overlay

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	14 - Concrete Deck - Protected w/ AC Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	13 - Concrete Deck - Unprotected w/ AC Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	40 - Concrete Slab - Protected w/ AC Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	44 - Concrete Slab - Protected w/ Thin Overlay
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA Count	19
501003P - TEMPORARY SHEETING	228 - Timber Submerged Pile
501003P - TEMPORARY SHEETING	227 - Reinforced Conc Submerged Pile
501003P - TEMPORARY SHEETING	226 - P/S Conc Submerged Pile
501003P - TEMPORARY SHEETING	225 - Unpainted Steel Submerged Pile
501003P - TEMPORARY SHEETING	220 - Reinforced Conc Submerged Pile Cap/Footing
501003P - TEMPORARY SHEETING Count	5
501006P - PERMANENT SHEETING	264 - Timber Sheeting
501006P - PERMANENT SHEETING	263 - Reinf Conc Sheeting
501006P - PERMANENT SHEETING	260 - Unpainted Steel Sheeting
501006P - PERMANENT SHEETING	262 - Prestress Conc Sheeting
501006P - PERMANENT SHEETING	261 - Painted Steel Sheeting
501006P - PERMANENT SHEETING Count	5
501008P - SHEET PILE WALL	260 - Unpainted Steel Sheeting
501008P - SHEET PILE WALL	261 - Painted Steel Sheeting
501008P - SHEET PILE WALL	262 - Prestress Conc Sheeting
501008P - SHEET PILE WALL	263 - Reinf Conc Sheeting
501008P - SHEET PILE WALL	264 - Timber Sheeting
501008P - SHEET PILE WALL Count	5
501012P - PERMANENT COFFERDAM	228 - Timber Submerged Pile
501012P - PERMANENT COFFERDAM	506 - Wingwalls - Abutment - Conc/Masonry/Timber
501012P - PERMANENT COFFERDAM	227 - Reinforced Conc Submerged Pile
501012P - PERMANENT COFFERDAM	226 - P/S Conc Submerged Pile
501012P - PERMANENT COFFERDAM	225 - Unpainted Steel Submerged Pile
501012P - PERMANENT COFFERDAM	217 - Other Material Abutment
501012P - PERMANENT COFFERDAM	216 - Timber Abutment
501012P - PERMANENT COFFERDAM	215 - Reinforced Conc Abutment
501012P - PERMANENT COFFERDAM	214 - Prestress Conc Abutment
501012P - PERMANENT COFFERDAM	220 - Reinforced Conc Submerged Pile Cap/Footing
501012P - PERMANENT COFFERDAM	508 - Wingwalls - Culvert - Conc/Masonry/Timber
501012P - PERMANENT COFFERDAM Count	11
502006M - PREBORED HOLE	225 - Unpainted Steel Submerged Pile
502006M - PREBORED HOLE	226 - P/S Conc Submerged Pile
502006M - PREBORED HOLE	227 - Reinforced Conc Submerged Pile
502006M - PREBORED HOLE	228 - Timber Submerged Pile
502006M - PREBORED HOLE Count	4
502009M - TEST PILE, FURNISHED	220 - Reinforced Conc Submerged Pile Cap/Footing
502009M - TEST PILE, FURNISHED	228 - Timber Submerged Pile
502009M - TEST PILE, FURNISHED	227 - Reinforced Conc Submerged Pile
502009M - TEST PILE, FURNISHED	225 - Unpainted Steel Submerged Pile
502009M - TEST PILE, FURNISHED	226 - P/S Conc Submerged Pile
502009M - TEST PILE, FURNISHED Count	5
502012M - TEST PILE, DRIVEN	220 - Reinforced Conc Submerged Pile Cap/Footing
502012M - TEST PILE, DRIVEN	225 - Unpainted Steel Submerged Pile
502012M - TEST PILE, DRIVEN	226 - P/S Conc Submerged Pile
502012M - TEST PILE, DRIVEN	227 - Reinforced Conc Submerged Pile
502012M - TEST PILE, DRIVEN	228 - Timber Submerged Pile
502012M - TEST PILE, DRIVEN Count	5
502015M - STATIC PILE LOAD TEST	226 - P/S Conc Submerged Pile
502015M - STATIC PILE LOAD TEST	227 - Reinforced Conc Submerged Pile
502015M - STATIC PILE LOAD TEST	220 - Reinforced Conc Submerged Pile Cap/Footing
502015M - STATIC PILE LOAD TEST	228 - Timber Submerged Pile
502015M - STATIC PILE LOAD TEST	225 - Unpainted Steel Submerged Pile
502015M - STATIC PILE LOAD TEST Count	5
502018M - DYNAMIC PILE LOAD TEST	220 - Reinforced Conc Submerged Pile Cap/Footing
502018M - DYNAMIC PILE LOAD TEST	225 - Unpainted Steel Submerged Pile
502018M - DYNAMIC PILE LOAD TEST	226 - P/S Conc Submerged Pile
502018M - DYNAMIC PILE LOAD TEST	227 - Reinforced Conc Submerged Pile
502018M - DYNAMIC PILE LOAD TEST	228 - Timber Submerged Pile
502018M - DYNAMIC PILE LOAD TEST Count	5
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	227 - Reinforced Conc Submerged Pile
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER Count	3
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	227 - Reinforced Conc Submerged Pile
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	270 - Conc Encased Steel Column or Pile Extension

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER Count	3
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	227 - Reinforced Conc Submerged Pile
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER Count	3
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	227 - Reinforced Conc Submerged Pile
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER Count	3
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	227 - Reinforced Conc Submerged Pile
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER Count	3
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	227 - Reinforced Conc Submerged Pile
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER Count	3
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	227 - Reinforced Conc Submerged Pile
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER Count	3
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	227 - Reinforced Conc Submerged Pile
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER Count	3
502152M - PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER	204 - P/S Conc Column or Pile Extension
502152M - PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER	226 - P/S Conc Submerged Pile
502152M - PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER Count	2
502155M - PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER	204 - P/S Conc Column or Pile Extension
502155M - PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER	226 - P/S Conc Submerged Pile
502155M - PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER Count	2
502157M - PRESTRESSED CONCRETE PILES, INSTALLED	204 - P/S Conc Column or Pile Extension
502157M - PRESTRESSED CONCRETE PILES, INSTALLED	226 - P/S Conc Submerged Pile
502157M - PRESTRESSED CONCRETE PILES, INSTALLED Count	2
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	202 - Painted Steel Column or Pile Extension
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	225 - Unpainted Steel Submerged Pile
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	201 - Unpainted Steel Column or Pile Extension
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53 Count	3
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	201 - Unpainted Steel Column or Pile Extension
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	202 - Painted Steel Column or Pile Extension
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	225 - Unpainted Steel Submerged Pile
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74 Count	3
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	201 - Unpainted Steel Column or Pile Extension
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	202 - Painted Steel Column or Pile Extension
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	225 - Unpainted Steel Submerged Pile
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73 Count	3
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	201 - Unpainted Steel Column or Pile Extension
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	202 - Painted Steel Column or Pile Extension
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	225 - Unpainted Steel Submerged Pile
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102 Count	3
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	201 - Unpainted Steel Column or Pile Extension
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	202 - Painted Steel Column or Pile Extension
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	225 - Unpainted Steel Submerged Pile
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117 Count	3
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	201 - Unpainted Steel Column or Pile Extension
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	225 - Unpainted Steel Submerged Pile
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	202 - Painted Steel Column or Pile Extension
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53 Count	3
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	201 - Unpainted Steel Column or Pile Extension
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	202 - Painted Steel Column or Pile Extension
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	225 - Unpainted Steel Submerged Pile
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74 Count	3
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	201 - Unpainted Steel Column or Pile Extension
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	202 - Painted Steel Column or Pile Extension
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	225 - Unpainted Steel Submerged Pile
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73 Count	3
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	201 - Unpainted Steel Column or Pile Extension
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	202 - Painted Steel Column or Pile Extension
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	225 - Unpainted Steel Submerged Pile
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102 Count	3
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	201 - Unpainted Steel Column or Pile Extension

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	202 - Painted Steel Column or Pile Extension
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	225 - Unpainted Steel Submerged Pile
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117 Count	3
502201M - SPLICE CAST-IN-PLACE PILE	205 - Reinforced Conc Column or Pile Extension
502201M - SPLICE CAST-IN-PLACE PILE	227 - Reinforced Conc Submerged Pile
502201M - SPLICE CAST-IN-PLACE PILE	270 - Conc Encased Steel Column or Pile Extension
502201M - SPLICE CAST-IN-PLACE PILE Count	3
502202M - SPLICE PRESTRESSED CONCRETE PILE	204 - P/S Conc Column or Pile Extension
502202M - SPLICE PRESTRESSED CONCRETE PILE	226 - P/S Conc Submerged Pile
502202M - SPLICE PRESTRESSED CONCRETE PILE Count	2
502204M - SPLICE STEEL H-PILE	201 - Unpainted Steel Column or Pile Extension
502204M - SPLICE STEEL H-PILE	202 - Painted Steel Column or Pile Extension
502204M - SPLICE STEEL H-PILE	225 - Unpainted Steel Submerged Pile
502204M - SPLICE STEEL H-PILE Count	3
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	201 - Unpainted Steel Column or Pile Extension
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	202 - Painted Steel Column or Pile Extension
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	225 - Unpainted Steel Submerged Pile
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE Count	3
502207M - PILE SHOE	226 - P/S Conc Submerged Pile
502207M - PILE SHOE	228 - Timber Submerged Pile
502207M - PILE SHOE	225 - Unpainted Steel Submerged Pile
502207M - PILE SHOE Count	3
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	202 - Painted Steel Column or Pile Extension
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	225 - Unpainted Steel Submerged Pile
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	201 - Unpainted Steel Column or Pile Extension
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH Count	3
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	201 - Unpainted Steel Column or Pile Extension
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	202 - Painted Steel Column or Pile Extension
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	225 - Unpainted Steel Submerged Pile
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN Count	3
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	227 - Reinforced Conc Submerged Pile
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER Count	3
502325M - STEEL KING PILES, W44X290, DRIVEN	260 - Unpainted Steel Sheeting
502325M - STEEL KING PILES, W44X290, DRIVEN	261 - Painted Steel Sheeting
502325M - STEEL KING PILES, W44X290, DRIVEN Count	2
502326M - STEEL KING PILES, W44X290, FURNISHED	260 - Unpainted Steel Sheeting
502326M - STEEL KING PILES, W44X290, FURNISHED	261 - Painted Steel Sheeting
502326M - STEEL KING PILES, W44X290, FURNISHED Count	2
502330M - STEEL KING PILES, W40X167 DRIVEN	260 - Unpainted Steel Sheeting
502330M - STEEL KING PILES, W40X167 DRIVEN	261 - Painted Steel Sheeting
502330M - STEEL KING PILES, W40X167 DRIVEN Count	2
502331M - STEEL KING PILES, W40X167 FURNISHED	261 - Painted Steel Sheeting
502331M - STEEL KING PILES, W40X167 FURNISHED	260 - Unpainted Steel Sheeting
502331M - STEEL KING PILES, W40X167 FURNISHED Count	2
502340M - STEEL KING PILES, HZ 1180M D-24, DRIVEN	260 - Unpainted Steel Sheeting
502340M - STEEL KING PILES, HZ 1180M D-24, DRIVEN	261 - Painted Steel Sheeting
502340M - STEEL KING PILES, HZ 1180M D-24, DRIVEN Count	2
502341M - STEEL KING PILES, HZ 1180M D-24, FURNISHED	260 - Unpainted Steel Sheeting
502341M - STEEL KING PILES, HZ 1180M D-24, FURNISHED	261 - Painted Steel Sheeting
502341M - STEEL KING PILES, HZ 1180M D-24, FURNISHED Count	2
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	205 - Reinforced Conc Column or Pile Extension
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	227 - Reinforced Conc Submerged Pile
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	270 - Conc Encased Steel Column or Pile Extension
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE Co	3
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	227 - Reinforced Conc Submerged Pile
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	205 - Reinforced Conc Column or Pile Extension
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER Cou	3
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	227 - Reinforced Conc Submerged Pile
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER Count	3
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	227 - Reinforced Conc Submerged Pile
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER Count	3
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	227 - Reinforced Conc Submerged Pile

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER Count	3
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	227 - Reinforced Conc Submerged Pile
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER Count	3
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	227 - Reinforced Conc Submerged Pile
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER Count	3
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	227 - Reinforced Conc Submerged Pile
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER Count	3
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	227 - Reinforced Conc Submerged Pile
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER Count	3
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	227 - Reinforced Conc Submerged Pile
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER Count	3
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	227 - Reinforced Conc Submerged Pile
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER Count	3
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	227 - Reinforced Conc Submerged Pile
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER Count	3
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	270 - Conc Encased Steel Column or Pile Extension
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	227 - Reinforced Conc Submerged Pile
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER Count	3
503055M - DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION	205 - Reinforced Conc Column or Pile Extension
503055M - DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION	227 - Reinforced Conc Submerged Pile
503055M - DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION Count	2
503060M - PERMANENT STEEL CASING, 96" DIAMETER	205 - Reinforced Conc Column or Pile Extension
503060M - PERMANENT STEEL CASING, 96" DIAMETER	227 - Reinforced Conc Submerged Pile
503060M - PERMANENT STEEL CASING, 96" DIAMETER Count	2
504003P - REINFORCEMENT STEEL	331 - Reinforced Conc Bridge Railing
504003P - REINFORCEMENT STEEL	39 - Concrete Slab - Unprotected w/ AC Overlay
504003P - REINFORCEMENT STEEL	388 - Wing Wall Footings
504003P - REINFORCEMENT STEEL	382 - Reinf Conc Diaphragm
504003P - REINFORCEMENT STEEL	38 - Concrete Slab - Bare
504003P - REINFORCEMENT STEEL	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
504003P - REINFORCEMENT STEEL	359 - Soffit of Concrete Deck or Slab
504003P - REINFORCEMENT STEEL	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
504003P - REINFORCEMENT STEEL	44 - Concrete Slab - Protected w/ Thin Overlay
504003P - REINFORCEMENT STEEL	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
504003P - REINFORCEMENT STEEL	262 - Prestress Conc Sheeting
504003P - REINFORCEMENT STEEL	271 - Concrete Encased Steel Pier Cap
504003P - REINFORCEMENT STEEL	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
504003P - REINFORCEMENT STEEL	27 - Concrete Deck - Protected w/ Cathodic System
504003P - REINFORCEMENT STEEL	263 - Reinf Conc Sheeting
504003P - REINFORCEMENT STEEL	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
504003P - REINFORCEMENT STEEL	52 - Concrete Slab - Protected w/ Coated Bars
504003P - REINFORCEMENT STEEL	270 - Conc Encased Steel Column or Pile Extension
504003P - REINFORCEMENT STEEL	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
504003P - REINFORCEMENT STEEL	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
504003P - REINFORCEMENT STEEL	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
504003P - REINFORCEMENT STEEL	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
504003P - REINFORCEMENT STEEL	396 - Reinforced Conc Pier Wall--Crash Wall
504003P - REINFORCEMENT STEEL	53 - Concrete Slab - Protected w/ Cathodic System
504003P - REINFORCEMENT STEEL	40 - Concrete Slab - Protected w/ AC Overlay
504003P - REINFORCEMENT STEEL	509 - Headwalls - Culvert - Concrete/Masonry
504003P - REINFORCEMENT STEEL	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504003P - REINFORCEMENT STEEL	507 - Headwalls - Other - Concrete/Masonry
504003P - REINFORCEMENT STEEL	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504003P - REINFORCEMENT STEEL	503 - Curbs/Sidewalks - Concrete
504003P - REINFORCEMENT STEEL	48 - Concrete Slab - Protected w/ Rigid Overlay
504003P - REINFORCEMENT STEEL	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
504003P - REINFORCEMENT STEEL	12 - Concrete Deck - Bare
504003P - REINFORCEMENT STEEL	171 - Concrete-Encased Steel Stringer
504003P - REINFORCEMENT STEEL	170 - Open Girder - Concrete Encased Steel
504003P - REINFORCEMENT STEEL	155 - Reinforced Conc Floor Beam
504003P - REINFORCEMENT STEEL	154 - P/S Conc Floor Beam
504003P - REINFORCEMENT STEEL	144 - Reinforced Conc Arch
504003P - REINFORCEMENT STEEL	143 - P/S Conc Arch
504003P - REINFORCEMENT STEEL	172 - Thru Truss - Bottom Chord - Conc Encased Steel
504003P - REINFORCEMENT STEEL	13 - Concrete Deck - Unprotected w/ AC Overlay
504003P - REINFORCEMENT STEEL	110 - Reinforced Conc Open Girder/Beam
504003P - REINFORCEMENT STEEL	116 - Reinforced Conc Stringer
504003P - REINFORCEMENT STEEL	115 - P/S Conc Stringer
504003P - REINFORCEMENT STEEL	109 - P/S Conc Open Girder/Beam
504003P - REINFORCEMENT STEEL	104 - P/S Conc Closed Web/Box Girder
504003P - REINFORCEMENT STEEL	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
504003P - REINFORCEMENT STEEL	26 - Concrete Deck - Protected w/ Coated Bars
504003P - REINFORCEMENT STEEL	14 - Concrete Deck - Protected w/ AC Overlay
504003P - REINFORCEMENT STEEL	22 - Concrete Deck - Protected w/ Rigid Overlay
504003P - REINFORCEMENT STEEL	105 - Reinforced Concrete Closed Webs/Box Girder
504003P - REINFORCEMENT STEEL	234 - Reinforced Conc Cap
504003P - REINFORCEMENT STEEL	241 - Reinforced Concrete Culvert
504003P - REINFORCEMENT STEEL	227 - Reinforced Conc Submerged Pile
504003P - REINFORCEMENT STEEL	173 - Arch - Concrete Encased Steel
504003P - REINFORCEMENT STEEL	220 - Reinforced Conc Submerged Pile Cap/Footing
504003P - REINFORCEMENT STEEL	233 - P/S Conc Cap
504003P - REINFORCEMENT STEEL	215 - Reinforced Conc Abutment
504003P - REINFORCEMENT STEEL	214 - Prestress Conc Abutment
504003P - REINFORCEMENT STEEL	213 - Painted Steel Open Girder--Concrete Encased
504003P - REINFORCEMENT STEEL	210 - Reinforced Conc Pier Wall
504003P - REINFORCEMENT STEEL	205 - Reinforced Conc Column or Pile Extension
504003P - REINFORCEMENT STEEL	204 - P/S Conc Column or Pile Extension
504003P - REINFORCEMENT STEEL	18 - Concrete Deck - Protected w/ Thin Overlay
504003P - REINFORCEMENT STEEL	174 - Floor Beam - Concrete Encased Steel
504003P - REINFORCEMENT STEEL	226 - P/S Conc Submerged Pile
504003P - REINFORCEMENT STEEL Count	66
504006P - REINFORCEMENT STEEL, EPOXY-COATED	359 - Soffit of Concrete Deck or Slab
504006P - REINFORCEMENT STEEL, EPOXY-COATED	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
504006P - REINFORCEMENT STEEL, EPOXY-COATED	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
504006P - REINFORCEMENT STEEL, EPOXY-COATED	331 - Reinforced Conc Bridge Railing
504006P - REINFORCEMENT STEEL, EPOXY-COATED	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
504006P - REINFORCEMENT STEEL, EPOXY-COATED	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
504006P - REINFORCEMENT STEEL, EPOXY-COATED	271 - Concrete Encased Steel Pier Cap
504006P - REINFORCEMENT STEEL, EPOXY-COATED	270 - Conc Encased Steel Column or Pile Extension
504006P - REINFORCEMENT STEEL, EPOXY-COATED	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
504006P - REINFORCEMENT STEEL, EPOXY-COATED	262 - Prestress Conc Sheeting
504006P - REINFORCEMENT STEEL, EPOXY-COATED	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	26 - Concrete Deck - Protected w/ Coated Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	263 - Reinf Conc Sheeting
504006P - REINFORCEMENT STEEL, EPOXY-COATED	382 - Reinf Conc Diaphragm
504006P - REINFORCEMENT STEEL, EPOXY-COATED	388 - Wing Wall Footings
504006P - REINFORCEMENT STEEL, EPOXY-COATED	396 - Reinforced Conc Pier Wall--Crash Wall
504006P - REINFORCEMENT STEEL, EPOXY-COATED	507 - Headwalls - Other - Concrete/Masonry
504006P - REINFORCEMENT STEEL, EPOXY-COATED	509 - Headwalls - Culvert - Concrete/Masonry
504006P - REINFORCEMENT STEEL, EPOXY-COATED	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
504006P - REINFORCEMENT STEEL, EPOXY-COATED	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
504006P - REINFORCEMENT STEEL, EPOXY-COATED	241 - Reinforced Concrete Culvert
504006P - REINFORCEMENT STEEL, EPOXY-COATED	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504006P - REINFORCEMENT STEEL, EPOXY-COATED	52 - Concrete Slab - Protected w/ Coated Bars
504006P - REINFORCEMENT STEEL, EPOXY-COATED	105 - Reinforced Concrete Closed Webs/Box Girder
504006P - REINFORCEMENT STEEL, EPOXY-COATED	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504006P - REINFORCEMENT STEEL, EPOXY-COATED	234 - Reinforced Conc Cap
504006P - REINFORCEMENT STEEL, EPOXY-COATED	104 - P/S Conc Closed Web/Box Girder
504006P - REINFORCEMENT STEEL, EPOXY-COATED	109 - P/S Conc Open Girder/Beam
504006P - REINFORCEMENT STEEL, EPOXY-COATED	110 - Reinforced Conc Open Girder/Beam
504006P - REINFORCEMENT STEEL, EPOXY-COATED	115 - P/S Conc Stringer
504006P - REINFORCEMENT STEEL, EPOXY-COATED	116 - Reinforced Conc Stringer
504006P - REINFORCEMENT STEEL, EPOXY-COATED	143 - P/S Conc Arch
504006P - REINFORCEMENT STEEL, EPOXY-COATED	144 - Reinforced Conc Arch

Line Item	Element
504006P - REINFORCEMENT STEEL, EPOXY-COATED	154 - P/S Conc Floor Beam
504006P - REINFORCEMENT STEEL, EPOXY-COATED	155 - Reinforced Conc Floor Beam
504006P - REINFORCEMENT STEEL, EPOXY-COATED	170 - Open Girder - Concrete Encased Steel
504006P - REINFORCEMENT STEEL, EPOXY-COATED	171 - Concrete-Encased Steel Stringer
504006P - REINFORCEMENT STEEL, EPOXY-COATED	233 - P/S Conc Cap
504006P - REINFORCEMENT STEEL, EPOXY-COATED	172 - Thru Truss - Bottom Chord - Conc Encased Steel
504006P - REINFORCEMENT STEEL, EPOXY-COATED	227 - Reinforced Conc Submerged Pile
504006P - REINFORCEMENT STEEL, EPOXY-COATED	226 - P/S Conc Submerged Pile
504006P - REINFORCEMENT STEEL, EPOXY-COATED	220 - Reinforced Conc Submerged Pile Cap/Footing
504006P - REINFORCEMENT STEEL, EPOXY-COATED	215 - Reinforced Conc Abutment
504006P - REINFORCEMENT STEEL, EPOXY-COATED	214 - Prestress Conc Abutment
504006P - REINFORCEMENT STEEL, EPOXY-COATED	210 - Reinforced Conc Pier Wall
504006P - REINFORCEMENT STEEL, EPOXY-COATED	205 - Reinforced Conc Column or Pile Extension
504006P - REINFORCEMENT STEEL, EPOXY-COATED	204 - P/S Conc Column or Pile Extension
504006P - REINFORCEMENT STEEL, EPOXY-COATED	174 - Floor Beam - Concrete Encased Steel
504006P - REINFORCEMENT STEEL, EPOXY-COATED	213 - Painted Steel Open Girder--Concrete Encased
504006P - REINFORCEMENT STEEL, EPOXY-COATED	173 - Arch - Concrete Encased Steel
504006P - REINFORCEMENT STEEL, EPOXY-COATED Count	53
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	234 - Reinforced Conc Cap
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	271 - Concrete Encased Steel Pier Cap
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	270 - Conc Encased Steel Column or Pile Extension
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	27 - Concrete Deck - Protected w/ Cathodic System
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	263 - Reinf Conc Sheeting
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	233 - P/S Conc Cap
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	241 - Reinforced Concrete Culvert
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	503 - Curbs/Sidewalks - Concrete
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	262 - Prestress Conc Sheeting
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	331 - Reinforced Conc Bridge Railing
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	359 - Soffit of Concrete Deck or Slab
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	382 - Reinf Conc Diaphragm
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	53 - Concrete Slab - Protected w/ Cathodic System
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	396 - Reinforced Conc Pier Wall--Crash Wall
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	507 - Headwalls - Other - Concrete/Masonry
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	509 - Headwalls - Culvert - Concrete/Masonry
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	227 - Reinforced Conc Submerged Pile
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	214 - Prestress Conc Abutment
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	388 - Wing Wall Footings
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	116 - Reinforced Conc Stringer
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	220 - Reinforced Conc Submerged Pile Cap/Footing
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	226 - P/S Conc Submerged Pile
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	104 - P/S Conc Closed Web/Box Girder
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	105 - Reinforced Concrete Closed Webs/Box Girder
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	109 - P/S Conc Open Girder/Beam
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	115 - P/S Conc Stringer
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	143 - P/S Conc Arch
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	144 - Reinforced Conc Arch
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	154 - P/S Conc Floor Beam
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	155 - Reinforced Conc Floor Beam
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	213 - Painted Steel Open Girder--Concrete Encased
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	110 - Reinforced Conc Open Girder/Beam
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	215 - Reinforced Conc Abutment
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	170 - Open Girder - Concrete Encased Steel
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	210 - Reinforced Conc Pier Wall
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	205 - Reinforced Conc Column or Pile Extension
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	204 - P/S Conc Column or Pile Extension
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	174 - Floor Beam - Concrete Encased Steel
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	173 - Arch - Concrete Encased Steel
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	172 - Thru Truss - Bottom Chord - Conc Encased Steel
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	171 - Concrete-Encased Steel Stringer
504008P - REINFORCEMENT STEEL, STAINLESS STEEL Count	45
504009P - REINFORCEMENT STEEL, GALVANIZED	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
504009P - REINFORCEMENT STEEL, GALVANIZED	226 - P/S Conc Submerged Pile
504009P - REINFORCEMENT STEEL, GALVANIZED	233 - P/S Conc Cap
504009P - REINFORCEMENT STEEL, GALVANIZED	234 - Reinforced Conc Cap
504009P - REINFORCEMENT STEEL, GALVANIZED	241 - Reinforced Concrete Culvert
504009P - REINFORCEMENT STEEL, GALVANIZED	262 - Prestress Conc Sheeting
504009P - REINFORCEMENT STEEL, GALVANIZED	263 - Reinf Conc Sheeting

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
504009P - REINFORCEMENT STEEL, GALVANIZED	27 - Concrete Deck - Protected w/ Cathodic System
504009P - REINFORCEMENT STEEL, GALVANIZED	270 - Conc Encased Steel Column or Pile Extension
504009P - REINFORCEMENT STEEL, GALVANIZED	271 - Concrete Encased Steel Pier Cap
504009P - REINFORCEMENT STEEL, GALVANIZED	227 - Reinforced Conc Submerged Pile
504009P - REINFORCEMENT STEEL, GALVANIZED	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
504009P - REINFORCEMENT STEEL, GALVANIZED	331 - Reinforced Conc Bridge Railing
504009P - REINFORCEMENT STEEL, GALVANIZED	359 - Soffit of Concrete Deck or Slab
504009P - REINFORCEMENT STEEL, GALVANIZED	388 - Wing Wall Footings
504009P - REINFORCEMENT STEEL, GALVANIZED	503 - Curbs/Sidewalks - Concrete
504009P - REINFORCEMENT STEEL, GALVANIZED	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504009P - REINFORCEMENT STEEL, GALVANIZED	507 - Headwalls - Other - Concrete/Masonry
504009P - REINFORCEMENT STEEL, GALVANIZED	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504009P - REINFORCEMENT STEEL, GALVANIZED	509 - Headwalls - Culvert - Concrete/Masonry
504009P - REINFORCEMENT STEEL, GALVANIZED	382 - Reinf Conc Diaphragm
504009P - REINFORCEMENT STEEL, GALVANIZED	53 - Concrete Slab - Protected w/ Cathodic System
504009P - REINFORCEMENT STEEL, GALVANIZED	109 - P/S Conc Open Girder/Beam
504009P - REINFORCEMENT STEEL, GALVANIZED	105 - Reinforced Concrete Closed Webs/Box Girder
504009P - REINFORCEMENT STEEL, GALVANIZED	110 - Reinforced Conc Open Girder/Beam
504009P - REINFORCEMENT STEEL, GALVANIZED	115 - P/S Conc Stringer
504009P - REINFORCEMENT STEEL, GALVANIZED	116 - Reinforced Conc Stringer
504009P - REINFORCEMENT STEEL, GALVANIZED	143 - P/S Conc Arch
504009P - REINFORCEMENT STEEL, GALVANIZED	396 - Reinforced Conc Pier Wall--Crash Wall
504009P - REINFORCEMENT STEEL, GALVANIZED	144 - Reinforced Conc Arch
504009P - REINFORCEMENT STEEL, GALVANIZED	104 - P/S Conc Closed Web/Box Girder
504009P - REINFORCEMENT STEEL, GALVANIZED	154 - P/S Conc Floor Beam
504009P - REINFORCEMENT STEEL, GALVANIZED	155 - Reinforced Conc Floor Beam
504009P - REINFORCEMENT STEEL, GALVANIZED	210 - Reinforced Conc Pier Wall
504009P - REINFORCEMENT STEEL, GALVANIZED	220 - Reinforced Conc Submerged Pile Cap/Footing
504009P - REINFORCEMENT STEEL, GALVANIZED	170 - Open Girder - Concrete Encased Steel
504009P - REINFORCEMENT STEEL, GALVANIZED	213 - Painted Steel Open Girder--Concrete Encased
504009P - REINFORCEMENT STEEL, GALVANIZED	215 - Reinforced Conc Abutment
504009P - REINFORCEMENT STEEL, GALVANIZED	205 - Reinforced Conc Column or Pile Extension
504009P - REINFORCEMENT STEEL, GALVANIZED	204 - P/S Conc Column or Pile Extension
504009P - REINFORCEMENT STEEL, GALVANIZED	174 - Floor Beam - Concrete Encased Steel
504009P - REINFORCEMENT STEEL, GALVANIZED	173 - Arch - Concrete Encased Steel
504009P - REINFORCEMENT STEEL, GALVANIZED	172 - Thru Truss - Bottom Chord - Conc Encased Steel
504009P - REINFORCEMENT STEEL, GALVANIZED	171 - Concrete-Encased Steel Stringer
504009P - REINFORCEMENT STEEL, GALVANIZED	214 - Prestress Conc Abutment
504009P - REINFORCEMENT STEEL, GALVANIZED Count	45
504010P - DRILL AND GROUT REINFORCEMENT STEEL	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504010P - DRILL AND GROUT REINFORCEMENT STEEL	48 - Concrete Slab - Protected w/ Rigid Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	44 - Concrete Slab - Protected w/ Thin Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	40 - Concrete Slab - Protected w/ AC Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	396 - Reinforced Conc Pier Wall--Crash Wall
504010P - DRILL AND GROUT REINFORCEMENT STEEL	39 - Concrete Slab - Unprotected w/ AC Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	388 - Wing Wall Footings
504010P - DRILL AND GROUT REINFORCEMENT STEEL	507 - Headwalls - Other - Concrete/Masonry
504010P - DRILL AND GROUT REINFORCEMENT STEEL	38 - Concrete Slab - Bare
504010P - DRILL AND GROUT REINFORCEMENT STEEL	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	382 - Reinf Conc Diaphragm
504010P - DRILL AND GROUT REINFORCEMENT STEEL	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504010P - DRILL AND GROUT REINFORCEMENT STEEL	509 - Headwalls - Culvert - Concrete/Masonry
504010P - DRILL AND GROUT REINFORCEMENT STEEL	52 - Concrete Slab - Protected w/ Coated Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	53 - Concrete Slab - Protected w/ Cathodic System
504010P - DRILL AND GROUT REINFORCEMENT STEEL	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
504010P - DRILL AND GROUT REINFORCEMENT STEEL	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
504010P - DRILL AND GROUT REINFORCEMENT STEEL	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
504010P - DRILL AND GROUT REINFORCEMENT STEEL	331 - Reinforced Conc Bridge Railing
504010P - DRILL AND GROUT REINFORCEMENT STEEL	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	18 - Concrete Deck - Protected w/ Thin Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	12 - Concrete Deck - Bare
504010P - DRILL AND GROUT REINFORCEMENT STEEL	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
504010P - DRILL AND GROUT REINFORCEMENT STEEL	14 - Concrete Deck - Protected w/ AC Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	359 - Soffit of Concrete Deck or Slab
504010P - DRILL AND GROUT REINFORCEMENT STEEL	205 - Reinforced Conc Column or Pile Extension
504010P - DRILL AND GROUT REINFORCEMENT STEEL	210 - Reinforced Conc Pier Wall
504010P - DRILL AND GROUT REINFORCEMENT STEEL	213 - Painted Steel Open Girder--Concrete Encased
504010P - DRILL AND GROUT REINFORCEMENT STEEL	215 - Reinforced Conc Abutment
504010P - DRILL AND GROUT REINFORCEMENT STEEL	22 - Concrete Deck - Protected w/ Rigid Overlay

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
504010P - DRILL AND GROUT REINFORCEMENT STEEL	220 - Reinforced Conc Submerged Pile Cap/Footing
504010P - DRILL AND GROUT REINFORCEMENT STEEL	227 - Reinforced Conc Submerged Pile
504010P - DRILL AND GROUT REINFORCEMENT STEEL	241 - Reinforced Concrete Culvert
504010P - DRILL AND GROUT REINFORCEMENT STEEL	26 - Concrete Deck - Protected w/ Coated Bars
504010P - DRILL AND GROUT REINFORCEMENT STEEL	263 - Reinf Conc Sheeting
504010P - DRILL AND GROUT REINFORCEMENT STEEL	27 - Concrete Deck - Protected w/ Cathodic System
504010P - DRILL AND GROUT REINFORCEMENT STEEL	270 - Conc Encased Steel Column or Pile Extension
504010P - DRILL AND GROUT REINFORCEMENT STEEL	271 - Concrete Encased Steel Pier Cap
504010P - DRILL AND GROUT REINFORCEMENT STEEL	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
504010P - DRILL AND GROUT REINFORCEMENT STEEL	13 - Concrete Deck - Unprotected w/ AC Overlay
504010P - DRILL AND GROUT REINFORCEMENT STEEL	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
504010P - DRILL AND GROUT REINFORCEMENT STEEL	234 - Reinforced Conc Cap
504010P - DRILL AND GROUT REINFORCEMENT STEEL Count	44
504012P - CONCRETE CULVERT, STRUCTURES	241 - Reinforced Concrete Culvert
504012P - CONCRETE CULVERT, STRUCTURES	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504012P - CONCRETE CULVERT, STRUCTURES	509 - Headwalls - Culvert - Concrete/Masonry
504012P - CONCRETE CULVERT, STRUCTURES Count	3
504015P - CONCRETE FOOTING	220 - Reinforced Conc Submerged Pile Cap/Footing
504015P - CONCRETE FOOTING	388 - Wing Wall Footings
504015P - CONCRETE FOOTING Count	2
504018P - CONCRETE WING WALL	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504018P - CONCRETE WING WALL	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504018P - CONCRETE WING WALL Count	2
504024P - CONCRETE ABUTMENT WALL	214 - Prestress Conc Abutment
504024P - CONCRETE ABUTMENT WALL	215 - Reinforced Conc Abutment
504024P - CONCRETE ABUTMENT WALL Count	2
504025P - MODIFICATION OF EXISTING ABUTMENTS	214 - Prestress Conc Abutment
504025P - MODIFICATION OF EXISTING ABUTMENTS	215 - Reinforced Conc Abutment
504025P - MODIFICATION OF EXISTING ABUTMENTS	216 - Timber Abutment
504025P - MODIFICATION OF EXISTING ABUTMENTS	217 - Other Material Abutment
504025P - MODIFICATION OF EXISTING ABUTMENTS Count	4
504026P - CONCRETE PIER COLUMN AND CAP, HPC	271 - Concrete Encased Steel Pier Cap
504026P - CONCRETE PIER COLUMN AND CAP, HPC	205 - Reinforced Conc Column or Pile Extension
504026P - CONCRETE PIER COLUMN AND CAP, HPC	220 - Reinforced Conc Submerged Pile Cap/Footing
504026P - CONCRETE PIER COLUMN AND CAP, HPC	234 - Reinforced Conc Cap
504026P - CONCRETE PIER COLUMN AND CAP, HPC Count	4
504027P - CONCRETE PIER COLUMN AND CAP	234 - Reinforced Conc Cap
504027P - CONCRETE PIER COLUMN AND CAP	271 - Concrete Encased Steel Pier Cap
504027P - CONCRETE PIER COLUMN AND CAP	205 - Reinforced Conc Column or Pile Extension
504027P - CONCRETE PIER COLUMN AND CAP	220 - Reinforced Conc Submerged Pile Cap/Footing
504027P - CONCRETE PIER COLUMN AND CAP Count	4
504028P - PIER CAP RECONSTRUCTION	220 - Reinforced Conc Submerged Pile Cap/Footing
504028P - PIER CAP RECONSTRUCTION	234 - Reinforced Conc Cap
504028P - PIER CAP RECONSTRUCTION	271 - Concrete Encased Steel Pier Cap
504028P - PIER CAP RECONSTRUCTION Count	3
504029P - CONCRETE SEAL	263 - Reinf Conc Sheeting
504029P - CONCRETE SEAL	270 - Conc Encased Steel Column or Pile Extension
504029P - CONCRETE SEAL	262 - Prestress Conc Sheeting
504029P - CONCRETE SEAL	241 - Reinforced Concrete Culvert
504029P - CONCRETE SEAL	234 - Reinforced Conc Cap
504029P - CONCRETE SEAL	233 - P/S Conc Cap
504029P - CONCRETE SEAL	227 - Reinforced Conc Submerged Pile
504029P - CONCRETE SEAL	220 - Reinforced Conc Submerged Pile Cap/Footing
504029P - CONCRETE SEAL	215 - Reinforced Conc Abutment
504029P - CONCRETE SEAL	214 - Prestress Conc Abutment
504029P - CONCRETE SEAL	210 - Reinforced Conc Pier Wall
504029P - CONCRETE SEAL	204 - P/S Conc Column or Pile Extension
504029P - CONCRETE SEAL	205 - Reinforced Conc Column or Pile Extension
504029P - CONCRETE SEAL	226 - P/S Conc Submerged Pile
504029P - CONCRETE SEAL	271 - Concrete Encased Steel Pier Cap
504029P - CONCRETE SEAL Count	15
504030P - CONCRETE PIER SHAFT	205 - Reinforced Conc Column or Pile Extension
504030P - CONCRETE PIER SHAFT	227 - Reinforced Conc Submerged Pile
504030P - CONCRETE PIER SHAFT	270 - Conc Encased Steel Column or Pile Extension
504030P - CONCRETE PIER SHAFT Count	3
504031P - MODIFICATION OF EXISTING PIERS	210 - Reinforced Conc Pier Wall
504031P - MODIFICATION OF EXISTING PIERS	396 - Reinforced Conc Pier Wall--Crash Wall
504031P - MODIFICATION OF EXISTING PIERS	211 - Other Material Pier Wall
504031P - MODIFICATION OF EXISTING PIERS	271 - Concrete Encased Steel Pier Cap
504031P - MODIFICATION OF EXISTING PIERS Count	4
504036P - EPOXY WATERPROOFING	227 - Reinforced Conc Submerged Pile

Line Item	Element
504036P - EPOXY WATERPROOFING	271 - Concrete Encased Steel Pier Cap
504036P - EPOXY WATERPROOFING	262 - Prestress Conc Sheeting
504036P - EPOXY WATERPROOFING	263 - Reinf Conc Sheeting
504036P - EPOXY WATERPROOFING	241 - Reinforced Concrete Culvert
504036P - EPOXY WATERPROOFING	234 - Reinforced Conc Cap
504036P - EPOXY WATERPROOFING	233 - P/S Conc Cap
504036P - EPOXY WATERPROOFING	220 - Reinforced Conc Submerged Pile Cap/Footing
504036P - EPOXY WATERPROOFING	215 - Reinforced Conc Abutment
504036P - EPOXY WATERPROOFING	214 - Prestress Conc Abutment
504036P - EPOXY WATERPROOFING	270 - Conc Encased Steel Column or Pile Extension
504036P - EPOXY WATERPROOFING	210 - Reinforced Conc Pier Wall
504036P - EPOXY WATERPROOFING	205 - Reinforced Conc Column or Pile Extension
504036P - EPOXY WATERPROOFING	204 - P/S Conc Column or Pile Extension
504036P - EPOXY WATERPROOFING	226 - P/S Conc Submerged Pile
504036P - EPOXY WATERPROOFING Count	15
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	226 - P/S Conc Submerged Pile
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	270 - Conc Encased Steel Column or Pile Extension
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	263 - Reinf Conc Sheeting
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	262 - Prestress Conc Sheeting
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	241 - Reinforced Concrete Culvert
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	234 - Reinforced Conc Cap
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	227 - Reinforced Conc Submerged Pile
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	271 - Concrete Encased Steel Pier Cap
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	220 - Reinforced Conc Submerged Pile Cap/Footing
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	215 - Reinforced Conc Abutment
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	214 - Prestress Conc Abutment
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	210 - Reinforced Conc Pier Wall
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	205 - Reinforced Conc Column or Pile Extension
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	204 - P/S Conc Column or Pile Extension
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	233 - P/S Conc Cap
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE Count	15
504038P - MEMBRANE WATERPROOFING	233 - P/S Conc Cap
504038P - MEMBRANE WATERPROOFING	270 - Conc Encased Steel Column or Pile Extension
504038P - MEMBRANE WATERPROOFING	271 - Concrete Encased Steel Pier Cap
504038P - MEMBRANE WATERPROOFING	263 - Reinf Conc Sheeting
504038P - MEMBRANE WATERPROOFING	262 - Prestress Conc Sheeting
504038P - MEMBRANE WATERPROOFING	241 - Reinforced Concrete Culvert
504038P - MEMBRANE WATERPROOFING	234 - Reinforced Conc Cap
504038P - MEMBRANE WATERPROOFING	226 - P/S Conc Submerged Pile
504038P - MEMBRANE WATERPROOFING	220 - Reinforced Conc Submerged Pile Cap/Footing
504038P - MEMBRANE WATERPROOFING	215 - Reinforced Conc Abutment
504038P - MEMBRANE WATERPROOFING	214 - Prestress Conc Abutment
504038P - MEMBRANE WATERPROOFING	210 - Reinforced Conc Pier Wall
504038P - MEMBRANE WATERPROOFING	204 - P/S Conc Column or Pile Extension
504038P - MEMBRANE WATERPROOFING	227 - Reinforced Conc Submerged Pile
504038P - MEMBRANE WATERPROOFING	205 - Reinforced Conc Column or Pile Extension
504038P - MEMBRANE WATERPROOFING Count	15
504040P - CONCRETE SURFACE TREATMENT	233 - P/S Conc Cap
504040P - CONCRETE SURFACE TREATMENT	234 - Reinforced Conc Cap
504040P - CONCRETE SURFACE TREATMENT	241 - Reinforced Concrete Culvert
504040P - CONCRETE SURFACE TREATMENT	262 - Prestress Conc Sheeting
504040P - CONCRETE SURFACE TREATMENT	263 - Reinf Conc Sheeting
504040P - CONCRETE SURFACE TREATMENT	270 - Conc Encased Steel Column or Pile Extension
504040P - CONCRETE SURFACE TREATMENT	205 - Reinforced Conc Column or Pile Extension
504040P - CONCRETE SURFACE TREATMENT	271 - Concrete Encased Steel Pier Cap
504040P - CONCRETE SURFACE TREATMENT	226 - P/S Conc Submerged Pile
504040P - CONCRETE SURFACE TREATMENT	220 - Reinforced Conc Submerged Pile Cap/Footing
504040P - CONCRETE SURFACE TREATMENT	215 - Reinforced Conc Abutment
504040P - CONCRETE SURFACE TREATMENT	204 - P/S Conc Column or Pile Extension
504040P - CONCRETE SURFACE TREATMENT	210 - Reinforced Conc Pier Wall
504040P - CONCRETE SURFACE TREATMENT	214 - Prestress Conc Abutment
504040P - CONCRETE SURFACE TREATMENT	227 - Reinforced Conc Submerged Pile
504040P - CONCRETE SURFACE TREATMENT Count	15
504046P - PAINTING OF CONCRETE SURFACE	263 - Reinf Conc Sheeting
504046P - PAINTING OF CONCRETE SURFACE	270 - Conc Encased Steel Column or Pile Extension
504046P - PAINTING OF CONCRETE SURFACE	262 - Prestress Conc Sheeting
504046P - PAINTING OF CONCRETE SURFACE	241 - Reinforced Concrete Culvert
504046P - PAINTING OF CONCRETE SURFACE	234 - Reinforced Conc Cap
504046P - PAINTING OF CONCRETE SURFACE	233 - P/S Conc Cap
504046P - PAINTING OF CONCRETE SURFACE	227 - Reinforced Conc Submerged Pile
504046P - PAINTING OF CONCRETE SURFACE	220 - Reinforced Conc Submerged Pile Cap/Footing

Line Item	Element
504046P - PAINTING OF CONCRETE SURFACE	215 - Reinforced Conc Abutment
504046P - PAINTING OF CONCRETE SURFACE	214 - Prestress Conc Abutment
504046P - PAINTING OF CONCRETE SURFACE	210 - Reinforced Conc Pier Wall
504046P - PAINTING OF CONCRETE SURFACE	204 - P/S Conc Column or Pile Extension
504046P - PAINTING OF CONCRETE SURFACE	205 - Reinforced Conc Column or Pile Extension
504046P - PAINTING OF CONCRETE SURFACE	271 - Concrete Encased Steel Pier Cap
504046P - PAINTING OF CONCRETE SURFACE	226 - P/S Conc Submerged Pile
504046P - PAINTING OF CONCRETE SURFACE Count	15
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	271 - Concrete Encased Steel Pier Cap
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	234 - Reinforced Conc Cap
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	241 - Reinforced Concrete Culvert
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	262 - Prestress Conc Sheeting
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	233 - P/S Conc Cap
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	270 - Conc Encased Steel Column or Pile Extension
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	215 - Reinforced Conc Abutment
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	263 - Reinf Conc Sheeting
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	227 - Reinforced Conc Submerged Pile
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	220 - Reinforced Conc Submerged Pile Cap/Footing
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	214 - Prestress Conc Abutment
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	210 - Reinforced Conc Pier Wall
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	205 - Reinforced Conc Column or Pile Extension
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	204 - P/S Conc Column or Pile Extension
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	226 - P/S Conc Submerged Pile
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME Count	15
504055P - CONCRETE BEAM	171 - Concrete-Encased Steel Stringer
504055P - CONCRETE BEAM	155 - Reinforced Conc Floor Beam
504055P - CONCRETE BEAM	105 - Reinforced Concrete Closed Webs/Box Girder
504055P - CONCRETE BEAM	110 - Reinforced Conc Open Girder/Beam
504055P - CONCRETE BEAM	174 - Floor Beam - Concrete Encased Steel
504055P - CONCRETE BEAM	116 - Reinforced Conc Stringer
504055P - CONCRETE BEAM Count	6
504064P - STONE VENEER	270 - Conc Encased Steel Column or Pile Extension
504064P - STONE VENEER	509 - Headwalls - Culvert - Concrete/Masonry
504064P - STONE VENEER	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504064P - STONE VENEER	507 - Headwalls - Other - Concrete/Masonry
504064P - STONE VENEER	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504064P - STONE VENEER	396 - Reinforced Conc Pier Wall--Crash Wall
504064P - STONE VENEER	331 - Reinforced Conc Bridge Railing
504064P - STONE VENEER	241 - Reinforced Concrete Culvert
504064P - STONE VENEER	204 - P/S Conc Column or Pile Extension
504064P - STONE VENEER	333 - Other Bridge Railing
504064P - STONE VENEER	202 - Painted Steel Column or Pile Extension
504064P - STONE VENEER	215 - Reinforced Conc Abutment
504064P - STONE VENEER	205 - Reinforced Conc Column or Pile Extension
504064P - STONE VENEER	210 - Reinforced Conc Pier Wall
504064P - STONE VENEER	211 - Other Material Pier Wall
504064P - STONE VENEER	214 - Prestress Conc Abutment
504064P - STONE VENEER	201 - Unpainted Steel Column or Pile Extension
504064P - STONE VENEER Count	17
504065P - BRICK VENEER	270 - Conc Encased Steel Column or Pile Extension
504065P - BRICK VENEER	201 - Unpainted Steel Column or Pile Extension
504065P - BRICK VENEER	507 - Headwalls - Other - Concrete/Masonry
504065P - BRICK VENEER	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504065P - BRICK VENEER	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504065P - BRICK VENEER	396 - Reinforced Conc Pier Wall--Crash Wall
504065P - BRICK VENEER	333 - Other Bridge Railing
504065P - BRICK VENEER	331 - Reinforced Conc Bridge Railing
504065P - BRICK VENEER	241 - Reinforced Concrete Culvert
504065P - BRICK VENEER	215 - Reinforced Conc Abutment
504065P - BRICK VENEER	214 - Prestress Conc Abutment
504065P - BRICK VENEER	211 - Other Material Pier Wall
504065P - BRICK VENEER	210 - Reinforced Conc Pier Wall
504065P - BRICK VENEER	205 - Reinforced Conc Column or Pile Extension
504065P - BRICK VENEER	202 - Painted Steel Column or Pile Extension
504065P - BRICK VENEER	509 - Headwalls - Culvert - Concrete/Masonry
504065P - BRICK VENEER	204 - P/S Conc Column or Pile Extension
504065P - BRICK VENEER Count	17
504067P - FORMLINER	333 - Other Bridge Railing
504067P - FORMLINER	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504067P - FORMLINER	509 - Headwalls - Culvert - Concrete/Masonry
504067P - FORMLINER	507 - Headwalls - Other - Concrete/Masonry

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
504067P - FORMLINER	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504067P - FORMLINER	396 - Reinforced Conc Pier Wall--Crash Wall
504067P - FORMLINER	270 - Conc Encased Steel Column or Pile Extension
504067P - FORMLINER	241 - Reinforced Concrete Culvert
504067P - FORMLINER	215 - Reinforced Conc Abutment
504067P - FORMLINER	205 - Reinforced Conc Column or Pile Extension
504067P - FORMLINER	210 - Reinforced Conc Pier Wall
504067P - FORMLINER	331 - Reinforced Conc Bridge Railing
504067P - FORMLINER Count	12
504075P - ARCHITECTURAL CAST STONE	331 - Reinforced Conc Bridge Railing
504075P - ARCHITECTURAL CAST STONE	509 - Headwalls - Culvert - Concrete/Masonry
504075P - ARCHITECTURAL CAST STONE	508 - Wingwalls - Culvert - Conc/Masonry/Timber
504075P - ARCHITECTURAL CAST STONE	507 - Headwalls - Other - Concrete/Masonry
504075P - ARCHITECTURAL CAST STONE	506 - Wingwalls - Abutment - Conc/Masonry/Timber
504075P - ARCHITECTURAL CAST STONE	333 - Other Bridge Railing
504075P - ARCHITECTURAL CAST STONE	270 - Conc Encased Steel Column or Pile Extension
504075P - ARCHITECTURAL CAST STONE	241 - Reinforced Concrete Culvert
504075P - ARCHITECTURAL CAST STONE	214 - Prestress Conc Abutment
504075P - ARCHITECTURAL CAST STONE	211 - Other Material Pier Wall
504075P - ARCHITECTURAL CAST STONE	210 - Reinforced Conc Pier Wall
504075P - ARCHITECTURAL CAST STONE	205 - Reinforced Conc Column or Pile Extension
504075P - ARCHITECTURAL CAST STONE	204 - P/S Conc Column or Pile Extension
504075P - ARCHITECTURAL CAST STONE	202 - Painted Steel Column or Pile Extension
504075P - ARCHITECTURAL CAST STONE	201 - Unpainted Steel Column or Pile Extension
504075P - ARCHITECTURAL CAST STONE	215 - Reinforced Conc Abutment
504075P - ARCHITECTURAL CAST STONE	396 - Reinforced Conc Pier Wall--Crash Wall
504075P - ARCHITECTURAL CAST STONE Count	17
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	154 - P/S Conc Floor Beam
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	109 - P/S Conc Open Girder/Beam
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	115 - P/S Conc Stringer
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36" Count	3
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	109 - P/S Conc Open Girder/Beam
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	115 - P/S Conc Stringer
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	154 - P/S Conc Floor Beam
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54" Count	3
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	109 - P/S Conc Open Girder/Beam
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	115 - P/S Conc Stringer
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	154 - P/S Conc Floor Beam
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63" Count	3
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	115 - P/S Conc Stringer
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	154 - P/S Conc Floor Beam
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	109 - P/S Conc Open Girder/Beam
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79" Count	3
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	154 - P/S Conc Floor Beam
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	109 - P/S Conc Open Girder/Beam
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	115 - P/S Conc Stringer
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27" Count	3
505057P - PRECAST CONCRETE CULVERT	241 - Reinforced Concrete Culvert
505057P - PRECAST CONCRETE CULVERT	243 - Other Culvert
505057P - PRECAST CONCRETE CULVERT Count	2
505058P - PRECAST CONCRETE CULVERT MODIFICATIONS	241 - Reinforced Concrete Culvert
505058P - PRECAST CONCRETE CULVERT MODIFICATIONS	243 - Other Culvert
505058P - PRECAST CONCRETE CULVERT MODIFICATIONS Count	2
505060P - PRECAST CONCRETE ARCH STRUCTURE	145 - Other Arch
505060P - PRECAST CONCRETE ARCH STRUCTURE	173 - Arch - Concrete Encased Steel
505060P - PRECAST CONCRETE ARCH STRUCTURE	143 - P/S Conc Arch
505060P - PRECAST CONCRETE ARCH STRUCTURE	144 - Reinforced Conc Arch
505060P - PRECAST CONCRETE ARCH STRUCTURE Count	4
505061P - PREFABRICATED SUBSTRUCTURE UNITS	270 - Conc Encased Steel Column or Pile Extension
505061P - PREFABRICATED SUBSTRUCTURE UNITS	235 - Timber Cap
505061P - PREFABRICATED SUBSTRUCTURE UNITS	240 - Unpainted Steel Culvert
505061P - PREFABRICATED SUBSTRUCTURE UNITS	241 - Reinforced Concrete Culvert
505061P - PREFABRICATED SUBSTRUCTURE UNITS	243 - Other Culvert
505061P - PREFABRICATED SUBSTRUCTURE UNITS	260 - Unpainted Steel Sheeting
505061P - PREFABRICATED SUBSTRUCTURE UNITS	261 - Painted Steel Sheeting
505061P - PREFABRICATED SUBSTRUCTURE UNITS	262 - Prestress Conc Sheeting
505061P - PREFABRICATED SUBSTRUCTURE UNITS	508 - Wingwalls - Culvert - Conc/Masonry/Timber
505061P - PREFABRICATED SUBSTRUCTURE UNITS	264 - Timber Sheeting
505061P - PREFABRICATED SUBSTRUCTURE UNITS	509 - Headwalls - Culvert - Concrete/Masonry
505061P - PREFABRICATED SUBSTRUCTURE UNITS	271 - Concrete Encased Steel Pier Cap
505061P - PREFABRICATED SUBSTRUCTURE UNITS	234 - Reinforced Conc Cap

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
505061P - PREFABRICATED SUBSTRUCTURE UNITS	506 - Wingwalls - Abutment - Conc/Masonry/Timber
505061P - PREFABRICATED SUBSTRUCTURE UNITS	507 - Headwalls - Other - Concrete/Masonry
505061P - PREFABRICATED SUBSTRUCTURE UNITS	206 - Timber Column or Pile Extension
505061P - PREFABRICATED SUBSTRUCTURE UNITS	263 - Reinf Conc Sheeting
505061P - PREFABRICATED SUBSTRUCTURE UNITS	202 - Painted Steel Column or Pile Extension
505061P - PREFABRICATED SUBSTRUCTURE UNITS	233 - P/S Conc Cap
505061P - PREFABRICATED SUBSTRUCTURE UNITS	201 - Unpainted Steel Column or Pile Extension
505061P - PREFABRICATED SUBSTRUCTURE UNITS	204 - P/S Conc Column or Pile Extension
505061P - PREFABRICATED SUBSTRUCTURE UNITS	211 - Other Material Pier Wall
505061P - PREFABRICATED SUBSTRUCTURE UNITS	214 - Prestress Conc Abutment
505061P - PREFABRICATED SUBSTRUCTURE UNITS	215 - Reinforced Conc Abutment
505061P - PREFABRICATED SUBSTRUCTURE UNITS	217 - Other Material Abutment
505061P - PREFABRICATED SUBSTRUCTURE UNITS	225 - Unpainted Steel Submerged Pile
505061P - PREFABRICATED SUBSTRUCTURE UNITS	226 - P/S Conc Submerged Pile
505061P - PREFABRICATED SUBSTRUCTURE UNITS	227 - Reinforced Conc Submerged Pile
505061P - PREFABRICATED SUBSTRUCTURE UNITS	228 - Timber Submerged Pile
505061P - PREFABRICATED SUBSTRUCTURE UNITS	230 - Unpainted Steel Cap
505061P - PREFABRICATED SUBSTRUCTURE UNITS	231 - Painted Steel Cap
505061P - PREFABRICATED SUBSTRUCTURE UNITS	220 - Reinforced Conc Submerged Pile Cap/Footing
505061P - PREFABRICATED SUBSTRUCTURE UNITS	213 - Painted Steel Open Girder--Concrete Encased
505061P - PREFABRICATED SUBSTRUCTURE UNITS Count	33
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	151 - Unpainted Steel Floor Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	173 - Arch - Concrete Encased Steel
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	172 - Thru Truss - Bottom Chord - Conc Encased Steel
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	171 - Concrete-Encased Steel Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	170 - Open Girder - Concrete Encased Steel
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	161 - Painted Steel Pin and/or Pin and Hanger Assembly
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	156 - Timber Floor Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	155 - Reinforced Conc Floor Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	152 - Painted Steel Floor Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	174 - Floor Beam - Concrete Encased Steel
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	154 - P/S Conc Floor Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	38 - Concrete Slab - Bare
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	39 - Concrete Slab - Unprotected w/ AC Overlay
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	40 - Concrete Slab - Protected w/ AC Overlay
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	44 - Concrete Slab - Protected w/ Thin Overlay
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	48 - Concrete Slab - Protected w/ Rigid Overlay
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	52 - Concrete Slab - Protected w/ Coated Bars
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	53 - Concrete Slab - Protected w/ Cathodic System
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	147 - Cable - Coated (not embedded in concrete)
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	141 - Painted Steel Arch
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	113 - Painted Steel Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	101 - Unpainted Steel Closed Web/Box Girder
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	102 - Painted Steel Closed Web/Box Girder
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	104 - P/S Conc Closed Web/Box Girder
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	105 - Reinforced Concrete Closed Webs/Box Girder
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	106 - Unpainted Steel Open Girder/Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	107 - Painted Steel Open Girder/Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	109 - P/S Conc Open Girder/Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	110 - Reinforced Conc Open Girder/Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	144 - Reinforced Conc Arch
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	112 - Unpainted Steel Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	146 - Cable - Uncoated (not embedded in concrete)
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	115 - P/S Conc Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	116 - Reinforced Conc Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	143 - P/S Conc Arch
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	120 - Unpainted Steel Bottom Chord Thru Truss
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	121 - Painted Steel Bottom Chord Thru Truss
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	125 - Unpainted Steel Thru Truss (excl. bottom chord)
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	126 - Painted Steel Thru Truss (excl. bottom chord)
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	130 - Unpainted Steel Deck Truss
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	131 - Painted Steel Deck Truss
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	135 - Timber Truss/Arch
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	140 - Unpainted Steel Arch

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	117 - Timber Stringer
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	111 - Timber Open Girder/Beam
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	145 - Other Arch
505063P - PREFABRICATED SUPERSTRUCTURE UNITS Count	53
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	53 - Concrete Slab - Protected w/ Cathodic System
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	154 - P/S Conc Floor Beam
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	52 - Concrete Slab - Protected w/ Coated Bars
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	109 - P/S Conc Open Girder/Beam
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	48 - Concrete Slab - Protected w/ Rigid Overlay
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	104 - P/S Conc Closed Web/Box Girder
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	39 - Concrete Slab - Unprotected w/ AC Overlay
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	115 - P/S Conc Stringer
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	143 - P/S Conc Arch
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	38 - Concrete Slab - Bare
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	40 - Concrete Slab - Protected w/ AC Overlay
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	44 - Concrete Slab - Protected w/ Thin Overlay
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS Cou	18
505072P - GIRDER JACKING	314 - Pot Bearing
505072P - GIRDER JACKING	363 - Section Loss
505072P - GIRDER JACKING	316 - Steel Plate/Sliding Plate--Moveable Bearing
505072P - GIRDER JACKING	317 - Rockers--Moveable Bearing
505072P - GIRDER JACKING	318 - Other--Moveable Bearing
505072P - GIRDER JACKING	360 - Settlement
505072P - GIRDER JACKING	315 - Disk Bearing
505072P - GIRDER JACKING	370 - Elastomeric Bearing with Teflon
505072P - GIRDER JACKING	313 - Fixed Bearing
505072P - GIRDER JACKING	375 - Pinned Bearing - Fixed
505072P - GIRDER JACKING	372 - Sliding Plate Bearing - Expansion/Moveable
505072P - GIRDER JACKING	376 - Spherical Bearing
505072P - GIRDER JACKING	520 - Isolation Bearing
505072P - GIRDER JACKING	521 - Bearing - Other
505072P - GIRDER JACKING	373 - Bond Breaker Bearing - Expansion/Moveable
505072P - GIRDER JACKING	217 - Other Material Abutment
505072P - GIRDER JACKING	374 - Rocker Bearing - Expansion/Moveable
505072P - GIRDER JACKING	210 - Reinforced Conc Pier Wall
505072P - GIRDER JACKING	211 - Other Material Pier Wall
505072P - GIRDER JACKING	214 - Prestress Conc Abutment
505072P - GIRDER JACKING	312 - Enclosed/Concealed Bearing
505072P - GIRDER JACKING	216 - Timber Abutment
505072P - GIRDER JACKING	220 - Reinforced Conc Submerged Pile Cap/Footing
505072P - GIRDER JACKING	230 - Unpainted Steel Cap
505072P - GIRDER JACKING	231 - Painted Steel Cap
505072P - GIRDER JACKING	233 - P/S Conc Cap
505072P - GIRDER JACKING	234 - Reinforced Conc Cap
505072P - GIRDER JACKING	235 - Timber Cap
505072P - GIRDER JACKING	271 - Concrete Encased Steel Pier Cap
505072P - GIRDER JACKING	310 - Elastomeric Bearing
505072P - GIRDER JACKING	311 - Moveable Bearing (roller, sliding, etc.)
505072P - GIRDER JACKING	215 - Reinforced Conc Abutment
505072P - GIRDER JACKING Count	32
505084P - PRECAST PIER	211 - Other Material Pier Wall
505084P - PRECAST PIER	233 - P/S Conc Cap
505084P - PRECAST PIER	234 - Reinforced Conc Cap
505084P - PRECAST PIER	271 - Concrete Encased Steel Pier Cap
505084P - PRECAST PIER Count	4
505088P - PRECAST PARAPET PANEL	333 - Other Bridge Railing
505088P - PRECAST PARAPET PANEL	507 - Headwalls - Other - Concrete/Masonry
505088P - PRECAST PARAPET PANEL	509 - Headwalls - Culvert - Concrete/Masonry
505088P - PRECAST PARAPET PANEL	331 - Reinforced Conc Bridge Railing
505088P - PRECAST PARAPET PANEL Count	4
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars

Line Item	Element
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC Count	6
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS Count	6
505094P - PRECAST CONCRETE STRUCTURE	155 - Reinforced Conc Floor Beam
505094P - PRECAST CONCRETE STRUCTURE	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
505094P - PRECAST CONCRETE STRUCTURE	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
505094P - PRECAST CONCRETE STRUCTURE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
505094P - PRECAST CONCRETE STRUCTURE	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
505094P - PRECAST CONCRETE STRUCTURE	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
505094P - PRECAST CONCRETE STRUCTURE	174 - Floor Beam - Concrete Encased Steel
505094P - PRECAST CONCRETE STRUCTURE	173 - Arch - Concrete Encased Steel
505094P - PRECAST CONCRETE STRUCTURE	172 - Thru Truss - Bottom Chord - Conc Encased Steel
505094P - PRECAST CONCRETE STRUCTURE	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
505094P - PRECAST CONCRETE STRUCTURE	105 - Reinforced Concrete Closed Webs/Box Girder
505094P - PRECAST CONCRETE STRUCTURE	170 - Open Girder - Concrete Encased Steel
505094P - PRECAST CONCRETE STRUCTURE	104 - P/S Conc Closed Web/Box Girder
505094P - PRECAST CONCRETE STRUCTURE	109 - P/S Conc Open Girder/Beam
505094P - PRECAST CONCRETE STRUCTURE	110 - Reinforced Conc Open Girder/Beam
505094P - PRECAST CONCRETE STRUCTURE	115 - P/S Conc Stringer
505094P - PRECAST CONCRETE STRUCTURE	116 - Reinforced Conc Stringer
505094P - PRECAST CONCRETE STRUCTURE	143 - P/S Conc Arch
505094P - PRECAST CONCRETE STRUCTURE	144 - Reinforced Conc Arch
505094P - PRECAST CONCRETE STRUCTURE	154 - P/S Conc Floor Beam
505094P - PRECAST CONCRETE STRUCTURE	171 - Concrete-Encased Steel Stringer
505094P - PRECAST CONCRETE STRUCTURE Count	21
506003P - STRUCTURAL STEEL	30 - Steel Deck - Corrugated/Orthotropic/Etc.
506003P - STRUCTURAL STEEL	213 - Painted Steel Open Girder--Concrete Encased
506003P - STRUCTURAL STEEL	161 - Painted Steel Pin and/or Pin and Hanger Assembly
506003P - STRUCTURAL STEEL	171 - Concrete-Encased Steel Stringer
506003P - STRUCTURAL STEEL	173 - Arch - Concrete Encased Steel
506003P - STRUCTURAL STEEL	174 - Floor Beam - Concrete Encased Steel
506003P - STRUCTURAL STEEL	201 - Unpainted Steel Column or Pile Extension
506003P - STRUCTURAL STEEL	202 - Painted Steel Column or Pile Extension
506003P - STRUCTURAL STEEL	217 - Other Material Abutment
506003P - STRUCTURAL STEEL	225 - Unpainted Steel Submerged Pile
506003P - STRUCTURAL STEEL	230 - Unpainted Steel Cap
506003P - STRUCTURAL STEEL	231 - Painted Steel Cap
506003P - STRUCTURAL STEEL	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
506003P - STRUCTURAL STEEL	29 - Steel Deck - Concrete Filled Grid
506003P - STRUCTURAL STEEL	170 - Open Girder - Concrete Encased Steel
506003P - STRUCTURAL STEEL	28 - Steel Deck - Open Grid
506003P - STRUCTURAL STEEL	106 - Unpainted Steel Open Girder/Beam
506003P - STRUCTURAL STEEL	101 - Unpainted Steel Closed Web/Box Girder
506003P - STRUCTURAL STEEL	172 - Thru Truss - Bottom Chord - Conc Encased Steel
506003P - STRUCTURAL STEEL	102 - Painted Steel Closed Web/Box Girder
506003P - STRUCTURAL STEEL	152 - Painted Steel Floor Beam
506003P - STRUCTURAL STEEL	107 - Painted Steel Open Girder/Beam
506003P - STRUCTURAL STEEL	112 - Unpainted Steel Stringer
506003P - STRUCTURAL STEEL	113 - Painted Steel Stringer
506003P - STRUCTURAL STEEL	120 - Unpainted Steel Bottom Chord Thru Truss
506003P - STRUCTURAL STEEL	121 - Painted Steel Bottom Chord Thru Truss
506003P - STRUCTURAL STEEL	126 - Painted Steel Thru Truss (excl. bottom chord)
506003P - STRUCTURAL STEEL	130 - Unpainted Steel Deck Truss
506003P - STRUCTURAL STEEL	131 - Painted Steel Deck Truss
506003P - STRUCTURAL STEEL	140 - Unpainted Steel Arch
506003P - STRUCTURAL STEEL	141 - Painted Steel Arch
506003P - STRUCTURAL STEEL	151 - Unpainted Steel Floor Beam
506003P - STRUCTURAL STEEL	125 - Unpainted Steel Thru Truss (excl. bottom chord)
506003P - STRUCTURAL STEEL Count	33
506004M - STRUCTURAL STEEL	230 - Unpainted Steel Cap
506004M - STRUCTURAL STEEL	161 - Painted Steel Pin and/or Pin and Hanger Assembly
506004M - STRUCTURAL STEEL	170 - Open Girder - Concrete Encased Steel
506004M - STRUCTURAL STEEL	171 - Concrete-Encased Steel Stringer
506004M - STRUCTURAL STEEL	172 - Thru Truss - Bottom Chord - Conc Encased Steel
506004M - STRUCTURAL STEEL	173 - Arch - Concrete Encased Steel

Line Item	Element
506004M - STRUCTURAL STEEL	174 - Floor Beam - Concrete Encased Steel
506004M - STRUCTURAL STEEL	201 - Unpainted Steel Column or Pile Extension
506004M - STRUCTURAL STEEL	202 - Painted Steel Column or Pile Extension
506004M - STRUCTURAL STEEL	213 - Painted Steel Open Girder--Concrete Encased
506004M - STRUCTURAL STEEL	29 - Steel Deck - Concrete Filled Grid
506004M - STRUCTURAL STEEL	225 - Unpainted Steel Submerged Pile
506004M - STRUCTURAL STEEL	231 - Painted Steel Cap
506004M - STRUCTURAL STEEL	28 - Steel Deck - Open Grid
506004M - STRUCTURAL STEEL	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
506004M - STRUCTURAL STEEL	217 - Other Material Abutment
506004M - STRUCTURAL STEEL	113 - Painted Steel Stringer
506004M - STRUCTURAL STEEL	152 - Painted Steel Floor Beam
506004M - STRUCTURAL STEEL	30 - Steel Deck - Corrugated/Orthotropic/Etc.
506004M - STRUCTURAL STEEL	101 - Unpainted Steel Closed Web/Box Girder
506004M - STRUCTURAL STEEL	102 - Painted Steel Closed Web/Box Girder
506004M - STRUCTURAL STEEL	106 - Unpainted Steel Open Girder/Beam
506004M - STRUCTURAL STEEL	112 - Unpainted Steel Stringer
506004M - STRUCTURAL STEEL	120 - Unpainted Steel Bottom Chord Thru Truss
506004M - STRUCTURAL STEEL	121 - Painted Steel Bottom Chord Thru Truss
506004M - STRUCTURAL STEEL	125 - Unpainted Steel Thru Truss (excl. bottom chord)
506004M - STRUCTURAL STEEL	126 - Painted Steel Thru Truss (excl. bottom chord)
506004M - STRUCTURAL STEEL	130 - Unpainted Steel Deck Truss
506004M - STRUCTURAL STEEL	131 - Painted Steel Deck Truss
506004M - STRUCTURAL STEEL	140 - Unpainted Steel Arch
506004M - STRUCTURAL STEEL	141 - Painted Steel Arch
506004M - STRUCTURAL STEEL	151 - Unpainted Steel Floor Beam
506004M - STRUCTURAL STEEL	107 - Painted Steel Open Girder/Beam
506004M - STRUCTURAL STEEL Count	33
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	372 - Sliding Plate Bearing - Expansion/Moveable
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	318 - Other--Moveable Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	521 - Bearing - Other
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	520 - Isolation Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	374 - Rocker Bearing - Expansion/Moveable
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	370 - Elastomeric Bearing with Teflon
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	316 - Steel Plate/Sliding Plate--Moveable Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	310 - Elastomeric Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	315 - Disk Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	314 - Pot Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	311 - Moveable Bearing (roller, sliding, etc.)
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	376 - Spherical Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	317 - Rockers--Moveable Bearing
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC Count	13
506006P - REINFORCED ELASTOMERIC BEARING ASSEMBLY	310 - Elastomeric Bearing
506006P - REINFORCED ELASTOMERIC BEARING ASSEMBLY	370 - Elastomeric Bearing with Teflon
506006P - REINFORCED ELASTOMERIC BEARING ASSEMBLY Count	2
506008P - RESET BEARING	317 - Rockers--Moveable Bearing
506008P - RESET BEARING	520 - Isolation Bearing
506008P - RESET BEARING	376 - Spherical Bearing
506008P - RESET BEARING	374 - Rocker Bearing - Expansion/Moveable
506008P - RESET BEARING	372 - Sliding Plate Bearing - Expansion/Moveable
506008P - RESET BEARING	318 - Other--Moveable Bearing
506008P - RESET BEARING	316 - Steel Plate/Sliding Plate--Moveable Bearing
506008P - RESET BEARING	315 - Disk Bearing
506008P - RESET BEARING	312 - Enclosed/Concealed Bearing
506008P - RESET BEARING	311 - Moveable Bearing (roller, sliding, etc.)
506008P - RESET BEARING	310 - Elastomeric Bearing
506008P - RESET BEARING	521 - Bearing - Other
506008P - RESET BEARING	370 - Elastomeric Bearing with Teflon
506008P - RESET BEARING Count	13
506009M - STRUCTURAL BEARING ASSEMBLY	370 - Elastomeric Bearing with Teflon
506009M - STRUCTURAL BEARING ASSEMBLY	520 - Isolation Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	521 - Bearing - Other
506009M - STRUCTURAL BEARING ASSEMBLY	376 - Spherical Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	375 - Pinned Bearing - Fixed
506009M - STRUCTURAL BEARING ASSEMBLY	374 - Rocker Bearing - Expansion/Moveable
506009M - STRUCTURAL BEARING ASSEMBLY	373 - Bond Breaker Bearing - Expansion/Moveable
506009M - STRUCTURAL BEARING ASSEMBLY	372 - Sliding Plate Bearing - Expansion/Moveable
506009M - STRUCTURAL BEARING ASSEMBLY	317 - Rockers--Moveable Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	316 - Steel Plate/Sliding Plate--Moveable Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	315 - Disk Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	314 - Pot Bearing

Line Item	Element
506009M - STRUCTURAL BEARING ASSEMBLY	313 - Fixed Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	312 - Enclosed/Concealed Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	310 - Elastomeric Bearing
506009M - STRUCTURAL BEARING ASSEMBLY	311 - Moveable Bearing (roller, sliding, etc.)
506009M - STRUCTURAL BEARING ASSEMBLY	318 - Other--Moveable Bearing
506009M - STRUCTURAL BEARING ASSEMBLY Count	17
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	316 - Steel Plate/Sliding Plate--Moveable Bearing
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	376 - Spherical Bearing
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	374 - Rocker Bearing - Expansion/Moveable
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	372 - Sliding Plate Bearing - Expansion/Moveable
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	317 - Rockers--Moveable Bearing
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	521 - Bearing - Other
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	314 - Pot Bearing
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	311 - Moveable Bearing (roller, sliding, etc.)
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	318 - Other--Moveable Bearing
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY Count	9
506011M - ANCHOR BOLT INSTALLATION	372 - Sliding Plate Bearing - Expansion/Moveable
506011M - ANCHOR BOLT INSTALLATION	520 - Isolation Bearing
506011M - ANCHOR BOLT INSTALLATION	521 - Bearing - Other
506011M - ANCHOR BOLT INSTALLATION	376 - Spherical Bearing
506011M - ANCHOR BOLT INSTALLATION	375 - Pinned Bearing - Fixed
506011M - ANCHOR BOLT INSTALLATION	374 - Rocker Bearing - Expansion/Moveable
506011M - ANCHOR BOLT INSTALLATION	373 - Bond Breaker Bearing - Expansion/Moveable
506011M - ANCHOR BOLT INSTALLATION	318 - Other--Moveable Bearing
506011M - ANCHOR BOLT INSTALLATION	317 - Rockers--Moveable Bearing
506011M - ANCHOR BOLT INSTALLATION	316 - Steel Plate/Sliding Plate--Moveable Bearing
506011M - ANCHOR BOLT INSTALLATION	315 - Disk Bearing
506011M - ANCHOR BOLT INSTALLATION	314 - Pot Bearing
506011M - ANCHOR BOLT INSTALLATION	313 - Fixed Bearing
506011M - ANCHOR BOLT INSTALLATION	311 - Moveable Bearing (roller, sliding, etc.)
506011M - ANCHOR BOLT INSTALLATION	312 - Enclosed/Concealed Bearing
506011M - ANCHOR BOLT INSTALLATION Count	15
506012P - SHEAR CONNECTOR	18 - Concrete Deck - Protected w/ Thin Overlay
506012P - SHEAR CONNECTOR	12 - Concrete Deck - Bare
506012P - SHEAR CONNECTOR	13 - Concrete Deck - Unprotected w/ AC Overlay
506012P - SHEAR CONNECTOR	22 - Concrete Deck - Protected w/ Rigid Overlay
506012P - SHEAR CONNECTOR	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
506012P - SHEAR CONNECTOR	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
506012P - SHEAR CONNECTOR	14 - Concrete Deck - Protected w/ AC Overlay
506012P - SHEAR CONNECTOR	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
506012P - SHEAR CONNECTOR	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
506012P - SHEAR CONNECTOR	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
506012P - SHEAR CONNECTOR	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
506012P - SHEAR CONNECTOR	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
506012P - SHEAR CONNECTOR	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
506012P - SHEAR CONNECTOR	26 - Concrete Deck - Protected w/ Coated Bars
506012P - SHEAR CONNECTOR	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
506012P - SHEAR CONNECTOR Count	15
506016P - GIRDER JACKING	372 - Sliding Plate Bearing - Expansion/Moveable
506016P - GIRDER JACKING	314 - Pot Bearing
506016P - GIRDER JACKING	315 - Disk Bearing
506016P - GIRDER JACKING	317 - Rockers--Moveable Bearing
506016P - GIRDER JACKING	360 - Settlement
506016P - GIRDER JACKING	363 - Section Loss
506016P - GIRDER JACKING	370 - Elastomeric Bearing with Teflon
506016P - GIRDER JACKING	373 - Bond Breaker Bearing - Expansion/Moveable
506016P - GIRDER JACKING	374 - Rocker Bearing - Expansion/Moveable
506016P - GIRDER JACKING	375 - Pinned Bearing - Fixed
506016P - GIRDER JACKING	376 - Spherical Bearing
506016P - GIRDER JACKING	313 - Fixed Bearing
506016P - GIRDER JACKING	521 - Bearing - Other
506016P - GIRDER JACKING	316 - Steel Plate/Sliding Plate--Moveable Bearing
506016P - GIRDER JACKING	520 - Isolation Bearing
506016P - GIRDER JACKING	233 - P/S Conc Cap
506016P - GIRDER JACKING	211 - Other Material Pier Wall
506016P - GIRDER JACKING	214 - Prestress Conc Abutment
506016P - GIRDER JACKING	215 - Reinforced Conc Abutment
506016P - GIRDER JACKING	216 - Timber Abutment
506016P - GIRDER JACKING	217 - Other Material Abutment
506016P - GIRDER JACKING	220 - Reinforced Conc Submerged Pile Cap/Footing
506016P - GIRDER JACKING	312 - Enclosed/Concealed Bearing

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
506016P - GIRDER JACKING	231 - Painted Steel Cap
506016P - GIRDER JACKING	234 - Reinforced Conc Cap
506016P - GIRDER JACKING	318 - Other--Moveable Bearing
506016P - GIRDER JACKING	235 - Timber Cap
506016P - GIRDER JACKING	210 - Reinforced Conc Pier Wall
506016P - GIRDER JACKING	271 - Concrete Encased Steel Pier Cap
506016P - GIRDER JACKING	310 - Elastomeric Bearing
506016P - GIRDER JACKING	311 - Moveable Bearing (roller, sliding, etc.)
506016P - GIRDER JACKING	230 - Unpainted Steel Cap
506016P - GIRDER JACKING Count	32
506021P - STEEL GRID FLOORING	28 - Steel Deck - Open Grid
506021P - STEEL GRID FLOORING	29 - Steel Deck - Concrete Filled Grid
506021P - STEEL GRID FLOORING Count	2
506040P - STEEL REPAIR, TYPE ___	225 - Unpainted Steel Submerged Pile
506040P - STEEL REPAIR, TYPE ___	161 - Painted Steel Pin and/or Pin and Hanger Assembly
506040P - STEEL REPAIR, TYPE ___	170 - Open Girder - Concrete Encased Steel
506040P - STEEL REPAIR, TYPE ___	171 - Concrete-Encased Steel Stringer
506040P - STEEL REPAIR, TYPE ___	172 - Thru Truss - Bottom Chord - Conc Encased Steel
506040P - STEEL REPAIR, TYPE ___	173 - Arch - Concrete Encased Steel
506040P - STEEL REPAIR, TYPE ___	174 - Floor Beam - Concrete Encased Steel
506040P - STEEL REPAIR, TYPE ___	201 - Unpainted Steel Column or Pile Extension
506040P - STEEL REPAIR, TYPE ___	213 - Painted Steel Open Girder--Concrete Encased
506040P - STEEL REPAIR, TYPE ___	231 - Painted Steel Cap
506040P - STEEL REPAIR, TYPE ___	28 - Steel Deck - Open Grid
506040P - STEEL REPAIR, TYPE ___	29 - Steel Deck - Concrete Filled Grid
506040P - STEEL REPAIR, TYPE ___	30 - Steel Deck - Corrugated/Orthotropic/Etc.
506040P - STEEL REPAIR, TYPE ___	217 - Other Material Abutment
506040P - STEEL REPAIR, TYPE ___	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
506040P - STEEL REPAIR, TYPE ___	202 - Painted Steel Column or Pile Extension
506040P - STEEL REPAIR, TYPE ___	106 - Unpainted Steel Open Girder/Beam
506040P - STEEL REPAIR, TYPE ___	152 - Painted Steel Floor Beam
506040P - STEEL REPAIR, TYPE ___	230 - Unpainted Steel Cap
506040P - STEEL REPAIR, TYPE ___	102 - Painted Steel Closed Web/Box Girder
506040P - STEEL REPAIR, TYPE ___	107 - Painted Steel Open Girder/Beam
506040P - STEEL REPAIR, TYPE ___	112 - Unpainted Steel Stringer
506040P - STEEL REPAIR, TYPE ___	113 - Painted Steel Stringer
506040P - STEEL REPAIR, TYPE ___	101 - Unpainted Steel Closed Web/Box Girder
506040P - STEEL REPAIR, TYPE ___	120 - Unpainted Steel Bottom Chord Thru Truss
506040P - STEEL REPAIR, TYPE ___	141 - Painted Steel Arch
506040P - STEEL REPAIR, TYPE ___	125 - Unpainted Steel Thru Truss (excl. bottom chord)
506040P - STEEL REPAIR, TYPE ___	126 - Painted Steel Thru Truss (excl. bottom chord)
506040P - STEEL REPAIR, TYPE ___	130 - Unpainted Steel Deck Truss
506040P - STEEL REPAIR, TYPE ___	131 - Painted Steel Deck Truss
506040P - STEEL REPAIR, TYPE ___	140 - Unpainted Steel Arch
506040P - STEEL REPAIR, TYPE ___	121 - Painted Steel Bottom Chord Thru Truss
506040P - STEEL REPAIR, TYPE ___	151 - Unpainted Steel Floor Beam
506040P - STEEL REPAIR, TYPE ___ Count	33
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	161 - Painted Steel Pin and/or Pin and Hanger Assembly
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	170 - Open Girder - Concrete Encased Steel
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	171 - Concrete-Encased Steel Stringer
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	172 - Thru Truss - Bottom Chord - Conc Encased Steel
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	173 - Arch - Concrete Encased Steel
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	174 - Floor Beam - Concrete Encased Steel
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	201 - Unpainted Steel Column or Pile Extension
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	202 - Painted Steel Column or Pile Extension
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	29 - Steel Deck - Concrete Filled Grid
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	230 - Unpainted Steel Cap
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	231 - Painted Steel Cap
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	28 - Steel Deck - Open Grid
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	213 - Painted Steel Open Girder--Concrete Encased
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	30 - Steel Deck - Corrugated/Orthotropic/Etc.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	217 - Other Material Abutment
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	112 - Unpainted Steel Stringer
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	225 - Unpainted Steel Submerged Pile
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	101 - Unpainted Steel Closed Web/Box Girder
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	102 - Painted Steel Closed Web/Box Girder
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	152 - Painted Steel Floor Beam
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	107 - Painted Steel Open Girder/Beam
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	113 - Painted Steel Stringer
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	120 - Unpainted Steel Bottom Chord Thru Truss

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	141 - Painted Steel Arch
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	125 - Unpainted Steel Thru Truss (excl. bottom chord)
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	126 - Painted Steel Thru Truss (excl. bottom chord)
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	130 - Unpainted Steel Deck Truss
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	131 - Painted Steel Deck Truss
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	140 - Unpainted Steel Arch
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	121 - Painted Steel Bottom Chord Thru Truss
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	151 - Unpainted Steel Floor Beam
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	106 - Unpainted Steel Open Girder/Beam
506041P - STRUCTURAL STEEL REPAIR, TYPE 1 Count	33
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	201 - Unpainted Steel Column or Pile Extension
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	174 - Floor Beam - Concrete Encased Steel
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	173 - Arch - Concrete Encased Steel
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	172 - Thru Truss - Bottom Chord - Conc Encased Steel
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	170 - Open Girder - Concrete Encased Steel
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	30 - Steel Deck - Corrugated/Orthotropic/Etc.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	171 - Concrete-Encased Steel Stringer
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	202 - Painted Steel Column or Pile Extension
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	213 - Painted Steel Open Girder--Concrete Encased
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	217 - Other Material Abutment
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	225 - Unpainted Steel Submerged Pile
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	230 - Unpainted Steel Cap
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	231 - Painted Steel Cap
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	161 - Painted Steel Pin and/or Pin and Hanger Assembly
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	29 - Steel Deck - Concrete Filled Grid
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	125 - Unpainted Steel Thru Truss (excl. bottom chord)
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	28 - Steel Deck - Open Grid
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	120 - Unpainted Steel Bottom Chord Thru Truss
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	101 - Unpainted Steel Closed Web/Box Girder
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	102 - Painted Steel Closed Web/Box Girder
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	106 - Unpainted Steel Open Girder/Beam
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	107 - Painted Steel Open Girder/Beam
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	130 - Unpainted Steel Deck Truss
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	113 - Painted Steel Stringer
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	121 - Painted Steel Bottom Chord Thru Truss
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	126 - Painted Steel Thru Truss (excl. bottom chord)
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	131 - Painted Steel Deck Truss
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	140 - Unpainted Steel Arch
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	141 - Painted Steel Arch
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	151 - Unpainted Steel Floor Beam
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	152 - Painted Steel Floor Beam
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	112 - Unpainted Steel Stringer
506042P - STRUCTURAL STEEL REPAIR, TYPE 2 Count	33
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	300 - Strip Seal Expansion Joint
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	302 - Compression Joint Seal
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY Count	4
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	300 - Strip Seal Expansion Joint
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	302 - Compression Joint Seal
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	303 - Assembly Joint/Seal (modular)
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	307 - Other--Assembly Joint/Seal
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER Count	4
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	300 - Strip Seal Expansion Joint
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	302 - Compression Joint Seal
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY Count	4
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	307 - Other--Assembly Joint/Seal
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	303 - Assembly Joint/Seal (modular)
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	300 - Strip Seal Expansion Joint
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	302 - Compression Joint Seal
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER Count	4
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	300 - Strip Seal Expansion Joint
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	302 - Compression Joint Seal
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	303 - Assembly Joint/Seal (modular)
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	307 - Other--Assembly Joint/Seal
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER Count	4
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	300 - Strip Seal Expansion Joint

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	302 - Compression Joint Seal
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY Count	4
507014P - NEOPRENE STRIP SEAL GLAND	300 - Strip Seal Expansion Joint
507014P - NEOPRENE STRIP SEAL GLAND	307 - Other--Assembly Joint/Seal
507014P - NEOPRENE STRIP SEAL GLAND Count	2
507015P - STRIP SEAL EXPANSION JOINT ASSEMBLY	300 - Strip Seal Expansion Joint
507015P - STRIP SEAL EXPANSION JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
507015P - STRIP SEAL EXPANSION JOINT ASSEMBLY Count	2
507016P - FINGER JOINT EXPANSION DAM	307 - Other--Assembly Joint/Seal
507016P - FINGER JOINT EXPANSION DAM	303 - Assembly Joint/Seal (modular)
507016P - FINGER JOINT EXPANSION DAM	305 - Finger Dams--Assembly Joint/Seal
507016P - FINGER JOINT EXPANSION DAM Count	3
507018P - MODULAR EXPANSION JOINT ASSEMBLY	303 - Assembly Joint/Seal (modular)
507018P - MODULAR EXPANSION JOINT ASSEMBLY	307 - Other--Assembly Joint/Seal
507018P - MODULAR EXPANSION JOINT ASSEMBLY Count	2
507021P - CONCRETE BRIDGE DECK	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507021P - CONCRETE BRIDGE DECK	18 - Concrete Deck - Protected w/ Thin Overlay
507021P - CONCRETE BRIDGE DECK	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507021P - CONCRETE BRIDGE DECK	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507021P - CONCRETE BRIDGE DECK	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
507021P - CONCRETE BRIDGE DECK	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507021P - CONCRETE BRIDGE DECK	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507021P - CONCRETE BRIDGE DECK	12 - Concrete Deck - Bare
507021P - CONCRETE BRIDGE DECK	22 - Concrete Deck - Protected w/ Rigid Overlay
507021P - CONCRETE BRIDGE DECK	27 - Concrete Deck - Protected w/ Cathodic System
507021P - CONCRETE BRIDGE DECK	14 - Concrete Deck - Protected w/ AC Overlay
507021P - CONCRETE BRIDGE DECK	13 - Concrete Deck - Unprotected w/ AC Overlay
507021P - CONCRETE BRIDGE DECK	26 - Concrete Deck - Protected w/ Coated Bars
507021P - CONCRETE BRIDGE DECK Count	13
507022P - CONCRETE BRIDGE SEATS, HES	215 - Reinforced Conc Abutment
507022P - CONCRETE BRIDGE SEATS, HES	234 - Reinforced Conc Cap
507022P - CONCRETE BRIDGE SEATS, HES Count	2
507023P - CONCRETE BRIDGE APPROACH, HES	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507023P - CONCRETE BRIDGE APPROACH, HES	48 - Concrete Slab - Protected w/ Rigid Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	52 - Concrete Slab - Protected w/ Coated Bars
507023P - CONCRETE BRIDGE APPROACH, HES	53 - Concrete Slab - Protected w/ Cathodic System
507023P - CONCRETE BRIDGE APPROACH, HES	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507023P - CONCRETE BRIDGE APPROACH, HES	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507023P - CONCRETE BRIDGE APPROACH, HES	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
507023P - CONCRETE BRIDGE APPROACH, HES	39 - Concrete Slab - Unprotected w/ AC Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507023P - CONCRETE BRIDGE APPROACH, HES	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507023P - CONCRETE BRIDGE APPROACH, HES	13 - Concrete Deck - Unprotected w/ AC Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	44 - Concrete Slab - Protected w/ Thin Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	12 - Concrete Deck - Bare
507023P - CONCRETE BRIDGE APPROACH, HES	40 - Concrete Slab - Protected w/ AC Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	14 - Concrete Deck - Protected w/ AC Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	18 - Concrete Deck - Protected w/ Thin Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	22 - Concrete Deck - Protected w/ Rigid Overlay
507023P - CONCRETE BRIDGE APPROACH, HES	26 - Concrete Deck - Protected w/ Coated Bars
507023P - CONCRETE BRIDGE APPROACH, HES	27 - Concrete Deck - Protected w/ Cathodic System
507023P - CONCRETE BRIDGE APPROACH, HES	38 - Concrete Slab - Bare
507023P - CONCRETE BRIDGE APPROACH, HES Count	20
507024P - CONCRETE BRIDGE DECK, HPC	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507024P - CONCRETE BRIDGE DECK, HPC	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507024P - CONCRETE BRIDGE DECK, HPC	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
507024P - CONCRETE BRIDGE DECK, HPC	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507024P - CONCRETE BRIDGE DECK, HPC	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507024P - CONCRETE BRIDGE DECK, HPC	27 - Concrete Deck - Protected w/ Cathodic System
507024P - CONCRETE BRIDGE DECK, HPC	26 - Concrete Deck - Protected w/ Coated Bars
507024P - CONCRETE BRIDGE DECK, HPC	22 - Concrete Deck - Protected w/ Rigid Overlay
507024P - CONCRETE BRIDGE DECK, HPC	18 - Concrete Deck - Protected w/ Thin Overlay
507024P - CONCRETE BRIDGE DECK, HPC	14 - Concrete Deck - Protected w/ AC Overlay
507024P - CONCRETE BRIDGE DECK, HPC	13 - Concrete Deck - Unprotected w/ AC Overlay
507024P - CONCRETE BRIDGE DECK, HPC	12 - Concrete Deck - Bare
507024P - CONCRETE BRIDGE DECK, HPC	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507024P - CONCRETE BRIDGE DECK, HPC Count	13
507025P - CONCRETE BRIDGE DECK, HES	14 - Concrete Deck - Protected w/ AC Overlay
507025P - CONCRETE BRIDGE DECK, HES	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507025P - CONCRETE BRIDGE DECK, HES	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
507025P - CONCRETE BRIDGE DECK, HES	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507025P - CONCRETE BRIDGE DECK, HES	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507025P - CONCRETE BRIDGE DECK, HES	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507025P - CONCRETE BRIDGE DECK, HES	27 - Concrete Deck - Protected w/ Cathodic System
507025P - CONCRETE BRIDGE DECK, HES	26 - Concrete Deck - Protected w/ Coated Bars
507025P - CONCRETE BRIDGE DECK, HES	18 - Concrete Deck - Protected w/ Thin Overlay
507025P - CONCRETE BRIDGE DECK, HES	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507025P - CONCRETE BRIDGE DECK, HES	13 - Concrete Deck - Unprotected w/ AC Overlay
507025P - CONCRETE BRIDGE DECK, HES	12 - Concrete Deck - Bare
507025P - CONCRETE BRIDGE DECK, HES	22 - Concrete Deck - Protected w/ Rigid Overlay
507025P - CONCRETE BRIDGE DECK, HES Count	13
507028M - ENCASEMENT CONCRETE	270 - Conc Encased Steel Column or Pile Extension
507028M - ENCASEMENT CONCRETE	271 - Concrete Encased Steel Pier Cap
507028M - ENCASEMENT CONCRETE Count	2
507042P - 4-BAR OPEN STEEL PARAPET	330 - Metal Bridge Railing - Uncoated
507042P - 4-BAR OPEN STEEL PARAPET	333 - Other Bridge Railing
507042P - 4-BAR OPEN STEEL PARAPET	334 - Metal Bridge Railing - Coated
507042P - 4-BAR OPEN STEEL PARAPET Count	3
507050M - CONCRETE SLEEPER SLAB	48 - Concrete Slab - Protected w/ Rigid Overlay
507050M - CONCRETE SLEEPER SLAB	53 - Concrete Slab - Protected w/ Cathodic System
507050M - CONCRETE SLEEPER SLAB	52 - Concrete Slab - Protected w/ Coated Bars
507050M - CONCRETE SLEEPER SLAB	40 - Concrete Slab - Protected w/ AC Overlay
507050M - CONCRETE SLEEPER SLAB	38 - Concrete Slab - Bare
507050M - CONCRETE SLEEPER SLAB	39 - Concrete Slab - Unprotected w/ AC Overlay
507050M - CONCRETE SLEEPER SLAB	44 - Concrete Slab - Protected w/ Thin Overlay
507050M - CONCRETE SLEEPER SLAB Count	7
507051P - CONCRETE BRIDGE APPROACH	38 - Concrete Slab - Bare
507051P - CONCRETE BRIDGE APPROACH	39 - Concrete Slab - Unprotected w/ AC Overlay
507051P - CONCRETE BRIDGE APPROACH	40 - Concrete Slab - Protected w/ AC Overlay
507051P - CONCRETE BRIDGE APPROACH	44 - Concrete Slab - Protected w/ Thin Overlay
507051P - CONCRETE BRIDGE APPROACH	48 - Concrete Slab - Protected w/ Rigid Overlay
507051P - CONCRETE BRIDGE APPROACH	52 - Concrete Slab - Protected w/ Coated Bars
507051P - CONCRETE BRIDGE APPROACH	53 - Concrete Slab - Protected w/ Cathodic System
507051P - CONCRETE BRIDGE APPROACH Count	7
507052M - CONCRETE MOMENT SLAB	52 - Concrete Slab - Protected w/ Coated Bars
507052M - CONCRETE MOMENT SLAB	53 - Concrete Slab - Protected w/ Cathodic System
507052M - CONCRETE MOMENT SLAB	48 - Concrete Slab - Protected w/ Rigid Overlay
507052M - CONCRETE MOMENT SLAB	40 - Concrete Slab - Protected w/ AC Overlay
507052M - CONCRETE MOMENT SLAB	39 - Concrete Slab - Unprotected w/ AC Overlay
507052M - CONCRETE MOMENT SLAB	38 - Concrete Slab - Bare
507052M - CONCRETE MOMENT SLAB	44 - Concrete Slab - Protected w/ Thin Overlay
507052M - CONCRETE MOMENT SLAB Count	7
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	52 - Concrete Slab - Protected w/ Coated Bars
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	38 - Concrete Slab - Bare
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	53 - Concrete Slab - Protected w/ Cathodic System
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	48 - Concrete Slab - Protected w/ Rigid Overlay
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	44 - Concrete Slab - Protected w/ Thin Overlay
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	40 - Concrete Slab - Protected w/ AC Overlay
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	39 - Concrete Slab - Unprotected w/ AC Overlay
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC Count	7
507055P - FIVE BAR OPEN STEEL PARAPET	333 - Other Bridge Railing
507055P - FIVE BAR OPEN STEEL PARAPET	334 - Metal Bridge Railing - Coated
507055P - FIVE BAR OPEN STEEL PARAPET	330 - Metal Bridge Railing - Uncoated
507055P - FIVE BAR OPEN STEEL PARAPET Count	3
507065P - CONCRETE CAST-IN-PLACE SLABS	48 - Concrete Slab - Protected w/ Rigid Overlay
507065P - CONCRETE CAST-IN-PLACE SLABS	44 - Concrete Slab - Protected w/ Thin Overlay
507065P - CONCRETE CAST-IN-PLACE SLABS	53 - Concrete Slab - Protected w/ Cathodic System
507065P - CONCRETE CAST-IN-PLACE SLABS	40 - Concrete Slab - Protected w/ AC Overlay
507065P - CONCRETE CAST-IN-PLACE SLABS	52 - Concrete Slab - Protected w/ Coated Bars
507065P - CONCRETE CAST-IN-PLACE SLABS	39 - Concrete Slab - Unprotected w/ AC Overlay
507065P - CONCRETE CAST-IN-PLACE SLABS	38 - Concrete Slab - Bare
507065P - CONCRETE CAST-IN-PLACE SLABS Count	7
507066P - PRECAST CONCRETE BRIDGE APPROACH	38 - Concrete Slab - Bare
507066P - PRECAST CONCRETE BRIDGE APPROACH	39 - Concrete Slab - Unprotected w/ AC Overlay
507066P - PRECAST CONCRETE BRIDGE APPROACH	40 - Concrete Slab - Protected w/ AC Overlay
507066P - PRECAST CONCRETE BRIDGE APPROACH	44 - Concrete Slab - Protected w/ Thin Overlay
507066P - PRECAST CONCRETE BRIDGE APPROACH	48 - Concrete Slab - Protected w/ Rigid Overlay
507066P - PRECAST CONCRETE BRIDGE APPROACH	52 - Concrete Slab - Protected w/ Coated Bars
507066P - PRECAST CONCRETE BRIDGE APPROACH	53 - Concrete Slab - Protected w/ Cathodic System
507066P - PRECAST CONCRETE BRIDGE APPROACH Count	7
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	22 - Concrete Deck - Protected w/ Rigid Overlay
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	14 - Concrete Deck - Protected w/ AC Overlay
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	18 - Concrete Deck - Protected w/ Thin Overlay
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE Count	6
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	40 - Concrete Slab - Protected w/ AC Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	44 - Concrete Slab - Protected w/ Thin Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	13 - Concrete Deck - Unprotected w/ AC Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	22 - Concrete Deck - Protected w/ Rigid Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	18 - Concrete Deck - Protected w/ Thin Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	14 - Concrete Deck - Protected w/ AC Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	48 - Concrete Slab - Protected w/ Rigid Overlay
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE Count	14
507123P - CONCRETE BRIDGE DECK, UHPC	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
507123P - CONCRETE BRIDGE DECK, UHPC	48 - Concrete Slab - Protected w/ Rigid Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	52 - Concrete Slab - Protected w/ Coated Bars
507123P - CONCRETE BRIDGE DECK, UHPC	53 - Concrete Slab - Protected w/ Cathodic System
507123P - CONCRETE BRIDGE DECK, UHPC	44 - Concrete Slab - Protected w/ Thin Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
507123P - CONCRETE BRIDGE DECK, UHPC	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
507123P - CONCRETE BRIDGE DECK, UHPC	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
507123P - CONCRETE BRIDGE DECK, UHPC	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
507123P - CONCRETE BRIDGE DECK, UHPC	40 - Concrete Slab - Protected w/ AC Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	13 - Concrete Deck - Unprotected w/ AC Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
507123P - CONCRETE BRIDGE DECK, UHPC	38 - Concrete Slab - Bare
507123P - CONCRETE BRIDGE DECK, UHPC	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
507123P - CONCRETE BRIDGE DECK, UHPC	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
507123P - CONCRETE BRIDGE DECK, UHPC	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
507123P - CONCRETE BRIDGE DECK, UHPC	27 - Concrete Deck - Protected w/ Cathodic System
507123P - CONCRETE BRIDGE DECK, UHPC	26 - Concrete Deck - Protected w/ Coated Bars
507123P - CONCRETE BRIDGE DECK, UHPC	22 - Concrete Deck - Protected w/ Rigid Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	14 - Concrete Deck - Protected w/ AC Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	18 - Concrete Deck - Protected w/ Thin Overlay
507123P - CONCRETE BRIDGE DECK, UHPC	12 - Concrete Deck - Bare
507123P - CONCRETE BRIDGE DECK, UHPC	39 - Concrete Slab - Unprotected w/ AC Overlay
507123P - CONCRETE BRIDGE DECK, UHPC Count	23
509131P - METAL MEDIAN BARRIER	330 - Metal Bridge Railing - Uncoated
509131P - METAL MEDIAN BARRIER	334 - Metal Bridge Railing - Coated
509131P - METAL MEDIAN BARRIER Count	2
509132P - METAL HALF MEDIAN BARRIER	330 - Metal Bridge Railing - Uncoated
509132P - METAL HALF MEDIAN BARRIER	334 - Metal Bridge Railing - Coated
509132P - METAL HALF MEDIAN BARRIER Count	2
511006P - STEEL SHEET PILING	261 - Painted Steel Sheeting
511006P - STEEL SHEET PILING	260 - Unpainted Steel Sheeting
511006P - STEEL SHEET PILING Count	2
513015P - LANDSCAPE RETAINING WALL	506 - Wingwalls - Abutment - Conc/Masonry/Timber
513015P - LANDSCAPE RETAINING WALL	509 - Headwalls - Culvert - Concrete/Masonry
513015P - LANDSCAPE RETAINING WALL	507 - Headwalls - Other - Concrete/Masonry
513015P - LANDSCAPE RETAINING WALL	508 - Wingwalls - Culvert - Conc/Masonry/Timber
513015P - LANDSCAPE RETAINING WALL Count	4
513022P - CONCRETE COPING	506 - Wingwalls - Abutment - Conc/Masonry/Timber
513022P - CONCRETE COPING	507 - Headwalls - Other - Concrete/Masonry
513022P - CONCRETE COPING	508 - Wingwalls - Culvert - Conc/Masonry/Timber
513022P - CONCRETE COPING	509 - Headwalls - Culvert - Concrete/Masonry
513022P - CONCRETE COPING Count	4
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM Count	5
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	34 - ConcDk prot w/membrane AC overlay coated bars-PREC

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT Count	5
551003M - REPAIR OF CONCRETE DECK, TYPE A	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
551003M - REPAIR OF CONCRETE DECK, TYPE A	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
551003M - REPAIR OF CONCRETE DECK, TYPE A	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
551003M - REPAIR OF CONCRETE DECK, TYPE A	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
551003M - REPAIR OF CONCRETE DECK, TYPE A	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
551003M - REPAIR OF CONCRETE DECK, TYPE A	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
551003M - REPAIR OF CONCRETE DECK, TYPE A	12 - Concrete Deck - Bare
551003M - REPAIR OF CONCRETE DECK, TYPE A	14 - Concrete Deck - Protected w/ AC Overlay
551003M - REPAIR OF CONCRETE DECK, TYPE A	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
551003M - REPAIR OF CONCRETE DECK, TYPE A	18 - Concrete Deck - Protected w/ Thin Overlay
551003M - REPAIR OF CONCRETE DECK, TYPE A	22 - Concrete Deck - Protected w/ Rigid Overlay
551003M - REPAIR OF CONCRETE DECK, TYPE A	26 - Concrete Deck - Protected w/ Coated Bars
551003M - REPAIR OF CONCRETE DECK, TYPE A	27 - Concrete Deck - Protected w/ Cathodic System
551003M - REPAIR OF CONCRETE DECK, TYPE A	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
551003M - REPAIR OF CONCRETE DECK, TYPE A	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
551003M - REPAIR OF CONCRETE DECK, TYPE A	13 - Concrete Deck - Unprotected w/ AC Overlay
551003M - REPAIR OF CONCRETE DECK, TYPE A Count	16
551006M - REPAIR OF CONCRETE DECK, TYPE B	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
551006M - REPAIR OF CONCRETE DECK, TYPE B	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
551006M - REPAIR OF CONCRETE DECK, TYPE B	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
551006M - REPAIR OF CONCRETE DECK, TYPE B	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
551006M - REPAIR OF CONCRETE DECK, TYPE B	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
551006M - REPAIR OF CONCRETE DECK, TYPE B	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
551006M - REPAIR OF CONCRETE DECK, TYPE B	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
551006M - REPAIR OF CONCRETE DECK, TYPE B	14 - Concrete Deck - Protected w/ AC Overlay
551006M - REPAIR OF CONCRETE DECK, TYPE B	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
551006M - REPAIR OF CONCRETE DECK, TYPE B	13 - Concrete Deck - Unprotected w/ AC Overlay
551006M - REPAIR OF CONCRETE DECK, TYPE B	18 - Concrete Deck - Protected w/ Thin Overlay
551006M - REPAIR OF CONCRETE DECK, TYPE B	22 - Concrete Deck - Protected w/ Rigid Overlay
551006M - REPAIR OF CONCRETE DECK, TYPE B	26 - Concrete Deck - Protected w/ Coated Bars
551006M - REPAIR OF CONCRETE DECK, TYPE B	27 - Concrete Deck - Protected w/ Cathodic System
551006M - REPAIR OF CONCRETE DECK, TYPE B	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
551006M - REPAIR OF CONCRETE DECK, TYPE B	12 - Concrete Deck - Bare
551006M - REPAIR OF CONCRETE DECK, TYPE B Count	16
551007M - REPAIR OF CONCRETE DECK, TYPE B1	12 - Concrete Deck - Bare
551007M - REPAIR OF CONCRETE DECK, TYPE B1	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
551007M - REPAIR OF CONCRETE DECK, TYPE B1	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
551007M - REPAIR OF CONCRETE DECK, TYPE B1	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
551007M - REPAIR OF CONCRETE DECK, TYPE B1	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
551007M - REPAIR OF CONCRETE DECK, TYPE B1	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
551007M - REPAIR OF CONCRETE DECK, TYPE B1	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
551007M - REPAIR OF CONCRETE DECK, TYPE B1	13 - Concrete Deck - Unprotected w/ AC Overlay
551007M - REPAIR OF CONCRETE DECK, TYPE B1	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
551007M - REPAIR OF CONCRETE DECK, TYPE B1	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
551007M - REPAIR OF CONCRETE DECK, TYPE B1	27 - Concrete Deck - Protected w/ Cathodic System
551007M - REPAIR OF CONCRETE DECK, TYPE B1	26 - Concrete Deck - Protected w/ Coated Bars
551007M - REPAIR OF CONCRETE DECK, TYPE B1	22 - Concrete Deck - Protected w/ Rigid Overlay
551007M - REPAIR OF CONCRETE DECK, TYPE B1	18 - Concrete Deck - Protected w/ Thin Overlay
551007M - REPAIR OF CONCRETE DECK, TYPE B1	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
551007M - REPAIR OF CONCRETE DECK, TYPE B1	14 - Concrete Deck - Protected w/ AC Overlay
551007M - REPAIR OF CONCRETE DECK, TYPE B1 Count	16
551009M - REPAIR OF CONCRETE DECK, TYPE C	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
551009M - REPAIR OF CONCRETE DECK, TYPE C	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
551009M - REPAIR OF CONCRETE DECK, TYPE C	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
551009M - REPAIR OF CONCRETE DECK, TYPE C	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
551009M - REPAIR OF CONCRETE DECK, TYPE C	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
551009M - REPAIR OF CONCRETE DECK, TYPE C	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
551009M - REPAIR OF CONCRETE DECK, TYPE C	12 - Concrete Deck - Bare
551009M - REPAIR OF CONCRETE DECK, TYPE C	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
551009M - REPAIR OF CONCRETE DECK, TYPE C	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
551009M - REPAIR OF CONCRETE DECK, TYPE C	27 - Concrete Deck - Protected w/ Cathodic System
551009M - REPAIR OF CONCRETE DECK, TYPE C	26 - Concrete Deck - Protected w/ Coated Bars
551009M - REPAIR OF CONCRETE DECK, TYPE C	22 - Concrete Deck - Protected w/ Rigid Overlay
551009M - REPAIR OF CONCRETE DECK, TYPE C	18 - Concrete Deck - Protected w/ Thin Overlay
551009M - REPAIR OF CONCRETE DECK, TYPE C	13 - Concrete Deck - Unprotected w/ AC Overlay
551009M - REPAIR OF CONCRETE DECK, TYPE C	14 - Concrete Deck - Protected w/ AC Overlay

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
551009M - REPAIR OF CONCRETE DECK, TYPE C	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
551009M - REPAIR OF CONCRETE DECK, TYPE C Count	16
551014P - DECK JOINT RESEAL	301 - Pourable Joint Seal
551014P - DECK JOINT RESEAL	305 - Finger Dams--Assembly Joint/Seal
551014P - DECK JOINT RESEAL	306 - Sliding Plates--Assembly Joint/Seal
551014P - DECK JOINT RESEAL	307 - Other--Assembly Joint/Seal
551014P - DECK JOINT RESEAL Count	4
551015M - DECK JOINT RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
551015M - DECK JOINT RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
551015M - DECK JOINT RECONSTRUCTION	300 - Strip Seal Expansion Joint
551015M - DECK JOINT RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
551015M - DECK JOINT RECONSTRUCTION	302 - Compression Joint Seal
551015M - DECK JOINT RECONSTRUCTION	304 - Open Expansion Joint
551015M - DECK JOINT RECONSTRUCTION	307 - Other--Assembly Joint/Seal
551015M - DECK JOINT RECONSTRUCTION Count	7
551017M - CONCRETE OVERLAY, HPC	12 - Concrete Deck - Bare
551017M - CONCRETE OVERLAY, HPC	38 - Concrete Slab - Bare
551017M - CONCRETE OVERLAY, HPC Count	2
551021M - HEADER RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	307 - Other--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	307 - Other--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	507 - Headwalls - Other - Concrete/Masonry
551021M - HEADER RECONSTRUCTION	509 - Headwalls - Culvert - Concrete/Masonry
551021M - HEADER RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
551021M - HEADER RECONSTRUCTION	215 - Reinforced Conc Abutment
551021M - HEADER RECONSTRUCTION	304 - Open Expansion Joint
551021M - HEADER RECONSTRUCTION	300 - Strip Seal Expansion Joint
551021M - HEADER RECONSTRUCTION	214 - Prestress Conc Abutment
551021M - HEADER RECONSTRUCTION	217 - Other Material Abutment
551021M - HEADER RECONSTRUCTION	300 - Strip Seal Expansion Joint
551021M - HEADER RECONSTRUCTION	302 - Compression Joint Seal
551021M - HEADER RECONSTRUCTION	302 - Compression Joint Seal
551021M - HEADER RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
551021M - HEADER RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
551021M - HEADER RECONSTRUCTION	304 - Open Expansion Joint
551021M - HEADER RECONSTRUCTION Count	19
551022M - HEADER RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
551022M - HEADER RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	507 - Headwalls - Other - Concrete/Masonry
551022M - HEADER RECONSTRUCTION	307 - Other--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	307 - Other--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
551022M - HEADER RECONSTRUCTION	509 - Headwalls - Culvert - Concrete/Masonry
551022M - HEADER RECONSTRUCTION	304 - Open Expansion Joint
551022M - HEADER RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
551022M - HEADER RECONSTRUCTION	302 - Compression Joint Seal
551022M - HEADER RECONSTRUCTION	302 - Compression Joint Seal
551022M - HEADER RECONSTRUCTION	300 - Strip Seal Expansion Joint
551022M - HEADER RECONSTRUCTION	300 - Strip Seal Expansion Joint
551022M - HEADER RECONSTRUCTION	217 - Other Material Abutment
551022M - HEADER RECONSTRUCTION	215 - Reinforced Conc Abutment
551022M - HEADER RECONSTRUCTION	214 - Prestress Conc Abutment
551022M - HEADER RECONSTRUCTION	304 - Open Expansion Joint
551022M - HEADER RECONSTRUCTION Count	19
551045M - PARAPET MODIFICATIONS	331 - Reinforced Conc Bridge Railing
551045M - PARAPET MODIFICATIONS	334 - Metal Bridge Railing - Coated
551045M - PARAPET MODIFICATIONS	330 - Metal Bridge Railing - Uncoated
551045M - PARAPET MODIFICATIONS	332 - Timber Bridge Railing
551045M - PARAPET MODIFICATIONS	333 - Other Bridge Railing
551045M - PARAPET MODIFICATIONS Count	5
551070M - MISCELLANEOUS CONCRETE	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
551070M - MISCELLANEOUS CONCRETE	40 - Concrete Slab - Protected w/ AC Overlay
551070M - MISCELLANEOUS CONCRETE	39 - Concrete Slab - Unprotected w/ AC Overlay
551070M - MISCELLANEOUS CONCRETE	382 - Reinf Conc Diaphragm
551070M - MISCELLANEOUS CONCRETE	359 - Soffit of Concrete Deck or Slab
551070M - MISCELLANEOUS CONCRETE	27 - Concrete Deck - Protected w/ Cathodic System
551070M - MISCELLANEOUS CONCRETE	503 - Curbs/Sidewalks - Concrete

Line Item	Element
551070M - MISCELLANEOUS CONCRETE	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
551070M - MISCELLANEOUS CONCRETE	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly
551070M - MISCELLANEOUS CONCRETE	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly
551070M - MISCELLANEOUS CONCRETE	271 - Concrete Encased Steel Pier Cap
551070M - MISCELLANEOUS CONCRETE	270 - Conc Encased Steel Column or Pile Extension
551070M - MISCELLANEOUS CONCRETE	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
551070M - MISCELLANEOUS CONCRETE	48 - Concrete Slab - Protected w/ Rigid Overlay
551070M - MISCELLANEOUS CONCRETE	506 - Wingwalls - Abutment - Conc/Masonry/Timber
551070M - MISCELLANEOUS CONCRETE	507 - Headwalls - Other - Concrete/Masonry
551070M - MISCELLANEOUS CONCRETE	508 - Wingwalls - Culvert - Conc/Masonry/Timber
551070M - MISCELLANEOUS CONCRETE	509 - Headwalls - Culvert - Concrete/Masonry
551070M - MISCELLANEOUS CONCRETE	52 - Concrete Slab - Protected w/ Coated Bars
551070M - MISCELLANEOUS CONCRETE	53 - Concrete Slab - Protected w/ Cathodic System
551070M - MISCELLANEOUS CONCRETE	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
551070M - MISCELLANEOUS CONCRETE	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
551070M - MISCELLANEOUS CONCRETE	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
551070M - MISCELLANEOUS CONCRETE	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
551070M - MISCELLANEOUS CONCRETE	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
551070M - MISCELLANEOUS CONCRETE	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
551070M - MISCELLANEOUS CONCRETE	44 - Concrete Slab - Protected w/ Thin Overlay
551070M - MISCELLANEOUS CONCRETE	12 - Concrete Deck - Bare
551070M - MISCELLANEOUS CONCRETE	104 - P/S Conc Closed Web/Box Girder
551070M - MISCELLANEOUS CONCRETE	26 - Concrete Deck - Protected w/ Coated Bars
551070M - MISCELLANEOUS CONCRETE	38 - Concrete Slab - Bare
551070M - MISCELLANEOUS CONCRETE	105 - Reinforced Concrete Closed Webs/Box Girder
551070M - MISCELLANEOUS CONCRETE	109 - P/S Conc Open Girder/Beam
551070M - MISCELLANEOUS CONCRETE	110 - Reinforced Conc Open Girder/Beam
551070M - MISCELLANEOUS CONCRETE	116 - Reinforced Conc Stringer
551070M - MISCELLANEOUS CONCRETE	13 - Concrete Deck - Unprotected w/ AC Overlay
551070M - MISCELLANEOUS CONCRETE	14 - Concrete Deck - Protected w/ AC Overlay
551070M - MISCELLANEOUS CONCRETE	143 - P/S Conc Arch
551070M - MISCELLANEOUS CONCRETE	144 - Reinforced Conc Arch
551070M - MISCELLANEOUS CONCRETE	145 - Other Arch
551070M - MISCELLANEOUS CONCRETE	154 - P/S Conc Floor Beam
551070M - MISCELLANEOUS CONCRETE	155 - Reinforced Conc Floor Beam
551070M - MISCELLANEOUS CONCRETE	234 - Reinforced Conc Cap
551070M - MISCELLANEOUS CONCRETE	115 - P/S Conc Stringer
551070M - MISCELLANEOUS CONCRETE	170 - Open Girder - Concrete Encased Steel
551070M - MISCELLANEOUS CONCRETE	233 - P/S Conc Cap
551070M - MISCELLANEOUS CONCRETE	22 - Concrete Deck - Protected w/ Rigid Overlay
551070M - MISCELLANEOUS CONCRETE	214 - Prestress Conc Abutment
551070M - MISCELLANEOUS CONCRETE	210 - Reinforced Conc Pier Wall
551070M - MISCELLANEOUS CONCRETE	241 - Reinforced Concrete Culvert
551070M - MISCELLANEOUS CONCRETE	205 - Reinforced Conc Column or Pile Extension
551070M - MISCELLANEOUS CONCRETE	204 - P/S Conc Column or Pile Extension
551070M - MISCELLANEOUS CONCRETE	18 - Concrete Deck - Protected w/ Thin Overlay
551070M - MISCELLANEOUS CONCRETE	174 - Floor Beam - Concrete Encased Steel
551070M - MISCELLANEOUS CONCRETE	173 - Arch - Concrete Encased Steel
551070M - MISCELLANEOUS CONCRETE	172 - Thru Truss - Bottom Chord - Conc Encased Steel
551070M - MISCELLANEOUS CONCRETE	171 - Concrete-Encased Steel Stringer
551070M - MISCELLANEOUS CONCRETE	215 - Reinforced Conc Abutment
551070M - MISCELLANEOUS CONCRETE Count	58
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	107 - Painted Steel Open Girder/Beam
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	202 - Painted Steel Column or Pile Extension
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	261 - Painted Steel Sheeting
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	231 - Painted Steel Cap
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	213 - Painted Steel Open Girder--Concrete Encased
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	161 - Painted Steel Pin and/or Pin and Hanger Assembly
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	152 - Painted Steel Floor Beam
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	141 - Painted Steel Arch
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	131 - Painted Steel Deck Truss
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	113 - Painted Steel Stringer
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	102 - Painted Steel Closed Web/Box Girder
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	121 - Painted Steel Bottom Chord Thru Truss
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING Count	12
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	202 - Painted Steel Column or Pile Extension
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	121 - Painted Steel Bottom Chord Thru Truss
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	261 - Painted Steel Sheeting
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	231 - Painted Steel Cap
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	213 - Painted Steel Open Girder--Concrete Encased
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	152 - Painted Steel Floor Beam

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	102 - Painted Steel Closed Web/Box Girder
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	131 - Painted Steel Deck Truss
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	107 - Painted Steel Open Girder/Beam
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	141 - Painted Steel Arch
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	113 - Painted Steel Stringer
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	161 - Painted Steel Pin and/or Pin and Hanger Assembly
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING, Count	12
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	170 - Open Girder - Concrete Encased Steel
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	171 - Concrete-Encased Steel Stringer
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	172 - Thru Truss - Bottom Chord - Conc Encased Steel
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	173 - Arch - Concrete Encased Steel
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	174 - Floor Beam - Concrete Encased Steel
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	270 - Conc Encased Steel Column or Pile Extension
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	271 - Concrete Encased Steel Pier Cap
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING Count	7
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	171 - Concrete-Encased Steel Stringer
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	172 - Thru Truss - Bottom Chord - Conc Encased Steel
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	173 - Arch - Concrete Encased Steel
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	174 - Floor Beam - Concrete Encased Steel
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	270 - Conc Encased Steel Column or Pile Extension
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	271 - Concrete Encased Steel Pier Cap
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	170 - Open Girder - Concrete Encased Steel
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING Count	7
554027P - CLEANING AND PAINTING OF BEARING	317 - Rockers--Moveable Bearing
554027P - CLEANING AND PAINTING OF BEARING	311 - Moveable Bearing (roller, sliding, etc.)
554027P - CLEANING AND PAINTING OF BEARING	318 - Other--Moveable Bearing
554027P - CLEANING AND PAINTING OF BEARING	315 - Disk Bearing
554027P - CLEANING AND PAINTING OF BEARING	314 - Pot Bearing
554027P - CLEANING AND PAINTING OF BEARING	312 - Enclosed/Concealed Bearing
554027P - CLEANING AND PAINTING OF BEARING	316 - Steel Plate/Sliding Plate--Moveable Bearing
554027P - CLEANING AND PAINTING OF BEARING	313 - Fixed Bearing
554027P - CLEANING AND PAINTING OF BEARING Count	8
555003M - SUBSTRUCTURE CONCRETE REPAIR	234 - Reinforced Conc Cap
555003M - SUBSTRUCTURE CONCRETE REPAIR	508 - Wingwalls - Culvert - Conc/Masonry/Timber
555003M - SUBSTRUCTURE CONCRETE REPAIR	506 - Wingwalls - Abutment - Conc/Masonry/Timber
555003M - SUBSTRUCTURE CONCRETE REPAIR	396 - Reinforced Conc Pier Wall--Crash Wall
555003M - SUBSTRUCTURE CONCRETE REPAIR	388 - Wing Wall Footings
555003M - SUBSTRUCTURE CONCRETE REPAIR	241 - Reinforced Concrete Culvert
555003M - SUBSTRUCTURE CONCRETE REPAIR	215 - Reinforced Conc Abutment
555003M - SUBSTRUCTURE CONCRETE REPAIR	214 - Prestress Conc Abutment
555003M - SUBSTRUCTURE CONCRETE REPAIR	210 - Reinforced Conc Pier Wall
555003M - SUBSTRUCTURE CONCRETE REPAIR	205 - Reinforced Conc Column or Pile Extension
555003M - SUBSTRUCTURE CONCRETE REPAIR	204 - P/S Conc Column or Pile Extension
555003M - SUBSTRUCTURE CONCRETE REPAIR	270 - Conc Encased Steel Column or Pile Extension
555003M - SUBSTRUCTURE CONCRETE REPAIR Count	12
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	22 - Concrete Deck - Protected w/ Rigid Overlay
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	14 - Concrete Deck - Protected w/ AC Overlay
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	18 - Concrete Deck - Protected w/ Thin Overlay
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE Count	5
555008P - CULVERT REPAIR, TYPE ___	240 - Unpainted Steel Culvert
555008P - CULVERT REPAIR, TYPE ___	241 - Reinforced Concrete Culvert
555008P - CULVERT REPAIR, TYPE ___	242 - Timber Culvert
555008P - CULVERT REPAIR, TYPE ___	243 - Other Culvert
555008P - CULVERT REPAIR, TYPE ___ Count	4
555009M - CONCRETE SPALL REPAIR, TYPE 1	271 - Concrete Encased Steel Pier Cap
555009M - CONCRETE SPALL REPAIR, TYPE 1	44 - Concrete Slab - Protected w/ Thin Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	39 - Concrete Slab - Unprotected w/ AC Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	38 - Concrete Slab - Bare
555009M - CONCRETE SPALL REPAIR, TYPE 1	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
555009M - CONCRETE SPALL REPAIR, TYPE 1	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
555009M - CONCRETE SPALL REPAIR, TYPE 1	48 - Concrete Slab - Protected w/ Rigid Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	331 - Reinforced Conc Bridge Railing
555009M - CONCRETE SPALL REPAIR, TYPE 1	508 - Wingwalls - Culvert - Conc/Masonry/Timber
555009M - CONCRETE SPALL REPAIR, TYPE 1	270 - Conc Encased Steel Column or Pile Extension
555009M - CONCRETE SPALL REPAIR, TYPE 1	27 - Concrete Deck - Protected w/ Cathodic System
555009M - CONCRETE SPALL REPAIR, TYPE 1	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
555009M - CONCRETE SPALL REPAIR, TYPE 1	503 - Curbs/Sidewalks - Concrete
555009M - CONCRETE SPALL REPAIR, TYPE 1	263 - Reinf Conc Sheeting
555009M - CONCRETE SPALL REPAIR, TYPE 1	507 - Headwalls - Other - Concrete/Masonry

Line Item	Element
555009M - CONCRETE SPALL REPAIR, TYPE 1	382 - Reinf Conc Diaphragm
555009M - CONCRETE SPALL REPAIR, TYPE 1	509 - Headwalls - Culvert - Concrete/Masonry
555009M - CONCRETE SPALL REPAIR, TYPE 1	52 - Concrete Slab - Protected w/ Coated Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	53 - Concrete Slab - Protected w/ Cathodic System
555009M - CONCRETE SPALL REPAIR, TYPE 1	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
555009M - CONCRETE SPALL REPAIR, TYPE 1	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
555009M - CONCRETE SPALL REPAIR, TYPE 1	506 - Wingwalls - Abutment - Conc/Masonry/Timber
555009M - CONCRETE SPALL REPAIR, TYPE 1	204 - P/S Conc Column or Pile Extension
555009M - CONCRETE SPALL REPAIR, TYPE 1	115 - P/S Conc Stringer
555009M - CONCRETE SPALL REPAIR, TYPE 1	262 - Prestress Conc Sheeting
555009M - CONCRETE SPALL REPAIR, TYPE 1	12 - Concrete Deck - Bare
555009M - CONCRETE SPALL REPAIR, TYPE 1	13 - Concrete Deck - Unprotected w/ AC Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	14 - Concrete Deck - Protected w/ AC Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	143 - P/S Conc Arch
555009M - CONCRETE SPALL REPAIR, TYPE 1	144 - Reinforced Conc Arch
555009M - CONCRETE SPALL REPAIR, TYPE 1	154 - P/S Conc Floor Beam
555009M - CONCRETE SPALL REPAIR, TYPE 1	155 - Reinforced Conc Floor Beam
555009M - CONCRETE SPALL REPAIR, TYPE 1	171 - Concrete-Encased Steel Stringer
555009M - CONCRETE SPALL REPAIR, TYPE 1	172 - Thru Truss - Bottom Chord - Conc Encased Steel
555009M - CONCRETE SPALL REPAIR, TYPE 1	173 - Arch - Concrete Encased Steel
555009M - CONCRETE SPALL REPAIR, TYPE 1	40 - Concrete Slab - Protected w/ AC Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	18 - Concrete Deck - Protected w/ Thin Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	116 - Reinforced Conc Stringer
555009M - CONCRETE SPALL REPAIR, TYPE 1	205 - Reinforced Conc Column or Pile Extension
555009M - CONCRETE SPALL REPAIR, TYPE 1	210 - Reinforced Conc Pier Wall
555009M - CONCRETE SPALL REPAIR, TYPE 1	214 - Prestress Conc Abutment
555009M - CONCRETE SPALL REPAIR, TYPE 1	215 - Reinforced Conc Abutment
555009M - CONCRETE SPALL REPAIR, TYPE 1	22 - Concrete Deck - Protected w/ Rigid Overlay
555009M - CONCRETE SPALL REPAIR, TYPE 1	220 - Reinforced Conc Submerged Pile Cap/Footing
555009M - CONCRETE SPALL REPAIR, TYPE 1	226 - P/S Conc Submerged Pile
555009M - CONCRETE SPALL REPAIR, TYPE 1	227 - Reinforced Conc Submerged Pile
555009M - CONCRETE SPALL REPAIR, TYPE 1	233 - P/S Conc Cap
555009M - CONCRETE SPALL REPAIR, TYPE 1	234 - Reinforced Conc Cap
555009M - CONCRETE SPALL REPAIR, TYPE 1	241 - Reinforced Concrete Culvert
555009M - CONCRETE SPALL REPAIR, TYPE 1	26 - Concrete Deck - Protected w/ Coated Bars
555009M - CONCRETE SPALL REPAIR, TYPE 1	174 - Floor Beam - Concrete Encased Steel
555009M - CONCRETE SPALL REPAIR, TYPE 1 Count	55
555012M - CONCRETE SPALL REPAIR, TYPE 2	506 - Wingwalls - Abutment - Conc/Masonry/Timber
555012M - CONCRETE SPALL REPAIR, TYPE 2	26 - Concrete Deck - Protected w/ Coated Bars
555012M - CONCRETE SPALL REPAIR, TYPE 2	262 - Prestress Conc Sheeting
555012M - CONCRETE SPALL REPAIR, TYPE 2	263 - Reinf Conc Sheeting
555012M - CONCRETE SPALL REPAIR, TYPE 2	27 - Concrete Deck - Protected w/ Cathodic System
555012M - CONCRETE SPALL REPAIR, TYPE 2	270 - Conc Encased Steel Column or Pile Extension
555012M - CONCRETE SPALL REPAIR, TYPE 2	271 - Concrete Encased Steel Pier Cap
555012M - CONCRETE SPALL REPAIR, TYPE 2	331 - Reinforced Conc Bridge Railing
555012M - CONCRETE SPALL REPAIR, TYPE 2	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
555012M - CONCRETE SPALL REPAIR, TYPE 2	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
555012M - CONCRETE SPALL REPAIR, TYPE 2	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
555012M - CONCRETE SPALL REPAIR, TYPE 2	241 - Reinforced Concrete Culvert
555012M - CONCRETE SPALL REPAIR, TYPE 2	503 - Curbs/Sidewalks - Concrete
555012M - CONCRETE SPALL REPAIR, TYPE 2	507 - Headwalls - Other - Concrete/Masonry
555012M - CONCRETE SPALL REPAIR, TYPE 2	508 - Wingwalls - Culvert - Conc/Masonry/Timber
555012M - CONCRETE SPALL REPAIR, TYPE 2	509 - Headwalls - Culvert - Concrete/Masonry
555012M - CONCRETE SPALL REPAIR, TYPE 2	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
555012M - CONCRETE SPALL REPAIR, TYPE 2	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
555012M - CONCRETE SPALL REPAIR, TYPE 2	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
555012M - CONCRETE SPALL REPAIR, TYPE 2	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
555012M - CONCRETE SPALL REPAIR, TYPE 2	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
555012M - CONCRETE SPALL REPAIR, TYPE 2	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
555012M - CONCRETE SPALL REPAIR, TYPE 2	382 - Reinf Conc Diaphragm
555012M - CONCRETE SPALL REPAIR, TYPE 2	144 - Reinforced Conc Arch
555012M - CONCRETE SPALL REPAIR, TYPE 2	116 - Reinforced Conc Stringer
555012M - CONCRETE SPALL REPAIR, TYPE 2	115 - P/S Conc Stringer
555012M - CONCRETE SPALL REPAIR, TYPE 2	12 - Concrete Deck - Bare
555012M - CONCRETE SPALL REPAIR, TYPE 2	13 - Concrete Deck - Unprotected w/ AC Overlay
555012M - CONCRETE SPALL REPAIR, TYPE 2	234 - Reinforced Conc Cap
555012M - CONCRETE SPALL REPAIR, TYPE 2	143 - P/S Conc Arch

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
555012M - CONCRETE SPALL REPAIR, TYPE 2	154 - P/S Conc Floor Beam
555012M - CONCRETE SPALL REPAIR, TYPE 2	155 - Reinforced Conc Floor Beam
555012M - CONCRETE SPALL REPAIR, TYPE 2	171 - Concrete-Encased Steel Stringer
555012M - CONCRETE SPALL REPAIR, TYPE 2	172 - Thru Truss - Bottom Chord - Conc Encased Steel
555012M - CONCRETE SPALL REPAIR, TYPE 2	173 - Arch - Concrete Encased Steel
555012M - CONCRETE SPALL REPAIR, TYPE 2	22 - Concrete Deck - Protected w/ Rigid Overlay
555012M - CONCRETE SPALL REPAIR, TYPE 2	14 - Concrete Deck - Protected w/ AC Overlay
555012M - CONCRETE SPALL REPAIR, TYPE 2	233 - P/S Conc Cap
555012M - CONCRETE SPALL REPAIR, TYPE 2	174 - Floor Beam - Concrete Encased Steel
555012M - CONCRETE SPALL REPAIR, TYPE 2	227 - Reinforced Conc Submerged Pile
555012M - CONCRETE SPALL REPAIR, TYPE 2	220 - Reinforced Conc Submerged Pile Cap/Footing
555012M - CONCRETE SPALL REPAIR, TYPE 2	215 - Reinforced Conc Abutment
555012M - CONCRETE SPALL REPAIR, TYPE 2	214 - Prestress Conc Abutment
555012M - CONCRETE SPALL REPAIR, TYPE 2	210 - Reinforced Conc Pier Wall
555012M - CONCRETE SPALL REPAIR, TYPE 2	205 - Reinforced Conc Column or Pile Extension
555012M - CONCRETE SPALL REPAIR, TYPE 2	204 - P/S Conc Column or Pile Extension
555012M - CONCRETE SPALL REPAIR, TYPE 2	18 - Concrete Deck - Protected w/ Thin Overlay
555012M - CONCRETE SPALL REPAIR, TYPE 2	226 - P/S Conc Submerged Pile
555012M - CONCRETE SPALL REPAIR, TYPE 2 Count	48
555013M - CONCRETE SPALL REPAIR	44 - Concrete Slab - Protected w/ Thin Overlay
555013M - CONCRETE SPALL REPAIR	27 - Concrete Deck - Protected w/ Cathodic System
555013M - CONCRETE SPALL REPAIR	38 - Concrete Slab - Bare
555013M - CONCRETE SPALL REPAIR	40 - Concrete Slab - Protected w/ AC Overlay
555013M - CONCRETE SPALL REPAIR	39 - Concrete Slab - Unprotected w/ AC Overlay
555013M - CONCRETE SPALL REPAIR	382 - Reinf Conc Diaphragm
555013M - CONCRETE SPALL REPAIR	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
555013M - CONCRETE SPALL REPAIR	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
555013M - CONCRETE SPALL REPAIR	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
555013M - CONCRETE SPALL REPAIR	331 - Reinforced Conc Bridge Railing
555013M - CONCRETE SPALL REPAIR	270 - Conc Encased Steel Column or Pile Extension
555013M - CONCRETE SPALL REPAIR	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
555013M - CONCRETE SPALL REPAIR	271 - Concrete Encased Steel Pier Cap
555013M - CONCRETE SPALL REPAIR	503 - Curbs/Sidewalks - Concrete
555013M - CONCRETE SPALL REPAIR	507 - Headwalls - Other - Concrete/Masonry
555013M - CONCRETE SPALL REPAIR	508 - Wingwalls - Culvert - Conc/Masonry/Timber
555013M - CONCRETE SPALL REPAIR	509 - Headwalls - Culvert - Concrete/Masonry
555013M - CONCRETE SPALL REPAIR	52 - Concrete Slab - Protected w/ Coated Bars
555013M - CONCRETE SPALL REPAIR	53 - Concrete Slab - Protected w/ Cathodic System
555013M - CONCRETE SPALL REPAIR	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
555013M - CONCRETE SPALL REPAIR	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
555013M - CONCRETE SPALL REPAIR	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
555013M - CONCRETE SPALL REPAIR	263 - Reinf Conc Sheeting
555013M - CONCRETE SPALL REPAIR	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
555013M - CONCRETE SPALL REPAIR	48 - Concrete Slab - Protected w/ Rigid Overlay
555013M - CONCRETE SPALL REPAIR	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
555013M - CONCRETE SPALL REPAIR	155 - Reinforced Conc Floor Beam
555013M - CONCRETE SPALL REPAIR	506 - Wingwalls - Abutment - Conc/Masonry/Timber
555013M - CONCRETE SPALL REPAIR	115 - P/S Conc Stringer
555013M - CONCRETE SPALL REPAIR	116 - Reinforced Conc Stringer
555013M - CONCRETE SPALL REPAIR	12 - Concrete Deck - Bare
555013M - CONCRETE SPALL REPAIR	13 - Concrete Deck - Unprotected w/ AC Overlay
555013M - CONCRETE SPALL REPAIR	14 - Concrete Deck - Protected w/ AC Overlay
555013M - CONCRETE SPALL REPAIR	143 - P/S Conc Arch
555013M - CONCRETE SPALL REPAIR	262 - Prestress Conc Sheeting
555013M - CONCRETE SPALL REPAIR	154 - P/S Conc Floor Beam
555013M - CONCRETE SPALL REPAIR	171 - Concrete-Encased Steel Stringer
555013M - CONCRETE SPALL REPAIR	172 - Thru Truss - Bottom Chord - Conc Encased Steel
555013M - CONCRETE SPALL REPAIR	173 - Arch - Concrete Encased Steel
555013M - CONCRETE SPALL REPAIR	174 - Floor Beam - Concrete Encased Steel
555013M - CONCRETE SPALL REPAIR	234 - Reinforced Conc Cap
555013M - CONCRETE SPALL REPAIR	26 - Concrete Deck - Protected w/ Coated Bars
555013M - CONCRETE SPALL REPAIR	144 - Reinforced Conc Arch
555013M - CONCRETE SPALL REPAIR	241 - Reinforced Concrete Culvert
555013M - CONCRETE SPALL REPAIR	18 - Concrete Deck - Protected w/ Thin Overlay
555013M - CONCRETE SPALL REPAIR	233 - P/S Conc Cap
555013M - CONCRETE SPALL REPAIR	227 - Reinforced Conc Submerged Pile
555013M - CONCRETE SPALL REPAIR	226 - P/S Conc Submerged Pile
555013M - CONCRETE SPALL REPAIR	22 - Concrete Deck - Protected w/ Rigid Overlay
555013M - CONCRETE SPALL REPAIR	215 - Reinforced Conc Abutment
555013M - CONCRETE SPALL REPAIR	214 - Prestress Conc Abutment
555013M - CONCRETE SPALL REPAIR	210 - Reinforced Conc Pier Wall

Line Item	Element
555013M - CONCRETE SPALL REPAIR	205 - Reinforced Conc Column or Pile Extension
555013M - CONCRETE SPALL REPAIR	204 - P/S Conc Column or Pile Extension
555013M - CONCRETE SPALL REPAIR	220 - Reinforced Conc Submerged Pile Cap/Footing
555013M - CONCRETE SPALL REPAIR Count	55
555015M - SUPERSTRUCTURE CONCRETE REPAIR	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
555015M - SUPERSTRUCTURE CONCRETE REPAIR	27 - Concrete Deck - Protected w/ Cathodic System
555015M - SUPERSTRUCTURE CONCRETE REPAIR	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
555015M - SUPERSTRUCTURE CONCRETE REPAIR	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
555015M - SUPERSTRUCTURE CONCRETE REPAIR	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
555015M - SUPERSTRUCTURE CONCRETE REPAIR	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
555015M - SUPERSTRUCTURE CONCRETE REPAIR	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
555015M - SUPERSTRUCTURE CONCRETE REPAIR	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
555015M - SUPERSTRUCTURE CONCRETE REPAIR	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
555015M - SUPERSTRUCTURE CONCRETE REPAIR	26 - Concrete Deck - Protected w/ Coated Bars
555015M - SUPERSTRUCTURE CONCRETE REPAIR	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
555015M - SUPERSTRUCTURE CONCRETE REPAIR	12 - Concrete Deck - Bare
555015M - SUPERSTRUCTURE CONCRETE REPAIR	22 - Concrete Deck - Protected w/ Rigid Overlay
555015M - SUPERSTRUCTURE CONCRETE REPAIR	29 - Steel Deck - Concrete Filled Grid
555015M - SUPERSTRUCTURE CONCRETE REPAIR	116 - Reinforced Conc Stringer
555015M - SUPERSTRUCTURE CONCRETE REPAIR	13 - Concrete Deck - Unprotected w/ AC Overlay
555015M - SUPERSTRUCTURE CONCRETE REPAIR	14 - Concrete Deck - Protected w/ AC Overlay
555015M - SUPERSTRUCTURE CONCRETE REPAIR	115 - P/S Conc Stringer
555015M - SUPERSTRUCTURE CONCRETE REPAIR	154 - P/S Conc Floor Beam
555015M - SUPERSTRUCTURE CONCRETE REPAIR	155 - Reinforced Conc Floor Beam
555015M - SUPERSTRUCTURE CONCRETE REPAIR	171 - Concrete-Encased Steel Stringer
555015M - SUPERSTRUCTURE CONCRETE REPAIR	172 - Thru Truss - Bottom Chord - Conc Encased Steel
555015M - SUPERSTRUCTURE CONCRETE REPAIR	18 - Concrete Deck - Protected w/ Thin Overlay
555015M - SUPERSTRUCTURE CONCRETE REPAIR Count	23
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	214 - Prestress Conc Abutment
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	214 - Prestress Conc Abutment
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	215 - Reinforced Conc Abutment
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	215 - Reinforced Conc Abutment
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	270 - Conc Encased Steel Column or Pile Extension
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	271 - Concrete Encased Steel Pier Cap
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	396 - Reinforced Conc Pier Wall--Crash Wall
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	210 - Reinforced Conc Pier Wall
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL Count	8
555023P - PIER RECONSTRUCTION	210 - Reinforced Conc Pier Wall
555023P - PIER RECONSTRUCTION	211 - Other Material Pier Wall
555023P - PIER RECONSTRUCTION Count	2
555035M - MASONRY REPOINTING	145 - Other Arch
555035M - MASONRY REPOINTING	217 - Other Material Abutment
555035M - MASONRY REPOINTING	243 - Other Culvert
555035M - MASONRY REPOINTING	506 - Wingwalls - Abutment - Conc/Masonry/Timber
555035M - MASONRY REPOINTING	507 - Headwalls - Other - Concrete/Masonry
555035M - MASONRY REPOINTING	508 - Wingwalls - Culvert - Conc/Masonry/Timber
555035M - MASONRY REPOINTING	509 - Headwalls - Culvert - Concrete/Masonry
555035M - MASONRY REPOINTING Count	7
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	330 - Metal Bridge Railing - Uncoated
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	334 - Metal Bridge Railing - Coated
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	333 - Other Bridge Railing
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	331 - Reinforced Conc Bridge Railing
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	332 - Timber Bridge Railing
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT Count	5
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	30 - Steel Deck - Corrugated/Orthotropic/Etc.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	29 - Steel Deck - Concrete Filled Grid
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	240 - Unpainted Steel Culvert
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	28 - Steel Deck - Open Grid
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	230 - Unpainted Steel Cap
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	161 - Painted Steel Pin and/or Pin and Hanger Assembly
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	152 - Painted Steel Floor Beam
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	151 - Unpainted Steel Floor Beam
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	231 - Painted Steel Cap
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	202 - Painted Steel Column or Pile Extension
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED Count	11
559003P - SUBSTRUCTURE CONCRETE REPAIR	205 - Reinforced Conc Column or Pile Extension
559003P - SUBSTRUCTURE CONCRETE REPAIR	210 - Reinforced Conc Pier Wall
559003P - SUBSTRUCTURE CONCRETE REPAIR	214 - Prestress Conc Abutment
559003P - SUBSTRUCTURE CONCRETE REPAIR	215 - Reinforced Conc Abutment
559003P - SUBSTRUCTURE CONCRETE REPAIR	234 - Reinforced Conc Cap

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
559003P - SUBSTRUCTURE CONCRETE REPAIR	241 - Reinforced Concrete Culvert
559003P - SUBSTRUCTURE CONCRETE REPAIR	270 - Conc Encased Steel Column or Pile Extension
559003P - SUBSTRUCTURE CONCRETE REPAIR	388 - Wing Wall Footings
559003P - SUBSTRUCTURE CONCRETE REPAIR	396 - Reinforced Conc Pier Wall--Crash Wall
559003P - SUBSTRUCTURE CONCRETE REPAIR	506 - Wingwalls - Abutment - Conc/Masonry/Timber
559003P - SUBSTRUCTURE CONCRETE REPAIR	508 - Wingwalls - Culvert - Conc/Masonry/Timber
559003P - SUBSTRUCTURE CONCRETE REPAIR	204 - P/S Conc Column or Pile Extension
559003P - SUBSTRUCTURE CONCRETE REPAIR Count	12
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	52 - Concrete Slab - Protected w/ Coated Bars
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	48 - Concrete Slab - Protected w/ Rigid Overlay
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	38 - Concrete Slab - Bare
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	40 - Concrete Slab - Protected w/ AC Overlay
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	39 - Concrete Slab - Unprotected w/ AC Overlay
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	44 - Concrete Slab - Protected w/ Thin Overlay
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	53 - Concrete Slab - Protected w/ Cathodic System
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE Count	7
602215M - CAPPING EXISTING DRAINAGE STRUCTURES	506 - Wingwalls - Abutment - Conc/Masonry/Timber
602215M - CAPPING EXISTING DRAINAGE STRUCTURES	508 - Wingwalls - Culvert - Conc/Masonry/Timber
602215M - CAPPING EXISTING DRAINAGE STRUCTURES Count	2
609004M - BEAM GUIDE RAIL, BRIDGE	330 - Metal Bridge Railing - Uncoated
609004M - BEAM GUIDE RAIL, BRIDGE	334 - Metal Bridge Railing - Coated
609004M - BEAM GUIDE RAIL, BRIDGE	333 - Other Bridge Railing
609004M - BEAM GUIDE RAIL, BRIDGE	331 - Reinforced Conc Bridge Railing
609004M - BEAM GUIDE RAIL, BRIDGE	332 - Timber Bridge Railing
609004M - BEAM GUIDE RAIL, BRIDGE Count	5
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	330 - Metal Bridge Railing - Uncoated
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	331 - Reinforced Conc Bridge Railing
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	332 - Timber Bridge Railing
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	333 - Other Bridge Railing
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	334 - Metal Bridge Railing - Coated
609015M - THRIE BEAM GUIDE RAIL, BRIDGE Count	5
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	331 - Reinforced Conc Bridge Railing
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	332 - Timber Bridge Railing
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	333 - Other Bridge Railing
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	334 - Metal Bridge Railing - Coated
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	330 - Metal Bridge Railing - Uncoated
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE Count	5
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	334 - Metal Bridge Railing - Coated
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	333 - Other Bridge Railing
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	332 - Timber Bridge Railing
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	330 - Metal Bridge Railing - Uncoated
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	331 - Reinforced Conc Bridge Railing
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE Count	5
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	331 - Reinforced Conc Bridge Railing
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	332 - Timber Bridge Railing
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	333 - Other Bridge Railing
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	334 - Metal Bridge Railing - Coated
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	330 - Metal Bridge Railing - Uncoated
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE Count	5
MMB002M - REPAIR CATEGORY "A", CONCRETE	220 - Reinforced Conc Submerged Pile Cap/Footing
MMB002M - REPAIR CATEGORY "A", CONCRETE	226 - P/S Conc Submerged Pile
MMB002M - REPAIR CATEGORY "A", CONCRETE	227 - Reinforced Conc Submerged Pile
MMB002M - REPAIR CATEGORY "A", CONCRETE Count	3
MMB067M - STRUCTURAL STEEL REPAIR	306 - Sliding Plates--Assembly Joint/Seal
MMB067M - STRUCTURAL STEEL REPAIR	315 - Disk Bearing
MMB067M - STRUCTURAL STEEL REPAIR	316 - Steel Plate/Sliding Plate--Moveable Bearing
MMB067M - STRUCTURAL STEEL REPAIR	317 - Rockers--Moveable Bearing
MMB067M - STRUCTURAL STEEL REPAIR	372 - Sliding Plate Bearing - Expansion/Moveable
MMB067M - STRUCTURAL STEEL REPAIR	374 - Rocker Bearing - Expansion/Moveable
MMB067M - STRUCTURAL STEEL REPAIR	376 - Spherical Bearing
MMB067M - STRUCTURAL STEEL REPAIR	314 - Pot Bearing
MMB067M - STRUCTURAL STEEL REPAIR	375 - Pinned Bearing - Fixed
MMB067M - STRUCTURAL STEEL REPAIR	311 - Moveable Bearing (roller, sliding, etc.)
MMB067M - STRUCTURAL STEEL REPAIR	305 - Finger Dams--Assembly Joint/Seal
MMB067M - STRUCTURAL STEEL REPAIR	303 - Assembly Joint/Seal (modular)
MMB067M - STRUCTURAL STEEL REPAIR	30 - Steel Deck - Corrugated/Orthotropic/Etc.
MMB067M - STRUCTURAL STEEL REPAIR	29 - Steel Deck - Concrete Filled Grid
MMB067M - STRUCTURAL STEEL REPAIR	28 - Steel Deck - Open Grid
MMB067M - STRUCTURAL STEEL REPAIR	174 - Floor Beam - Concrete Encased Steel
MMB067M - STRUCTURAL STEEL REPAIR	312 - Enclosed/Concealed Bearing
MMB067M - STRUCTURAL STEEL REPAIR	313 - Fixed Bearing

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
MMB067M - STRUCTURAL STEEL REPAIR Count	18
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	26 - Concrete Deck - Protected w/ Coated Bars
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	331 - Reinforced Conc Bridge Railing
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	271 - Concrete Encased Steel Pier Cap
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	270 - Conc Encased Steel Column or Pile Extension
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	27 - Concrete Deck - Protected w/ Cathodic System
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	262 - Prestress Conc Sheeting
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	241 - Reinforced Concrete Culvert
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	263 - Reinf Conc Sheeting
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	382 - Reinf Conc Diaphragm
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	234 - Reinforced Conc Cap
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	506 - Wingwalls - Abutment - Conc/Masonry/Timber
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	509 - Headwalls - Culvert - Concrete/Masonry
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	503 - Curbs/Sidewalks - Concrete
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	508 - Wingwalls - Culvert - Conc/Masonry/Timber
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	115 - P/S Conc Stringer
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	233 - P/S Conc Cap
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	507 - Headwalls - Other - Concrete/Masonry
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	116 - Reinforced Conc Stringer
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	12 - Concrete Deck - Bare
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	13 - Concrete Deck - Unprotected w/ AC Overlay
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	14 - Concrete Deck - Protected w/ AC Overlay
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	143 - P/S Conc Arch
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	144 - Reinforced Conc Arch
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	154 - P/S Conc Floor Beam
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	155 - Reinforced Conc Floor Beam
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	171 - Concrete-Encased Steel Stringer
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	172 - Thru Truss - Bottom Chord - Conc Encased Steel
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	215 - Reinforced Conc Abutment
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	173 - Arch - Concrete Encased Steel
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	227 - Reinforced Conc Submerged Pile
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	22 - Concrete Deck - Protected w/ Rigid Overlay
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	226 - P/S Conc Submerged Pile
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	214 - Prestress Conc Abutment
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	210 - Reinforced Conc Pier Wall
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	205 - Reinforced Conc Column or Pile Extension
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	204 - P/S Conc Column or Pile Extension
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	18 - Concrete Deck - Protected w/ Thin Overlay
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	174 - Floor Beam - Concrete Encased Steel
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	220 - Reinforced Conc Submerged Pile Cap/Footing
MMB071M - CONCRETE SPALL REPAIR, TYPE 1 Count	48
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	331 - Reinforced Conc Bridge Railing
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	271 - Concrete Encased Steel Pier Cap
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	270 - Conc Encased Steel Column or Pile Extension
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	27 - Concrete Deck - Protected w/ Cathodic System
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	263 - Reinf Conc Sheeting
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	26 - Concrete Deck - Protected w/ Coated Bars
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	262 - Prestress Conc Sheeting
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	382 - Reinf Conc Diaphragm
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	503 - Curbs/Sidewalks - Concrete
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	506 - Wingwalls - Abutment - Conc/Masonry/Timber
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	507 - Headwalls - Other - Concrete/Masonry
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	509 - Headwalls - Culvert - Concrete/Masonry
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	241 - Reinforced Concrete Culvert
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	214 - Prestress Conc Abutment
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	508 - Wingwalls - Culvert - Conc/Masonry/Timber
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	154 - P/S Conc Floor Beam
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	115 - P/S Conc Stringer
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	116 - Reinforced Conc Stringer
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	12 - Concrete Deck - Bare
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	13 - Concrete Deck - Unprotected w/ AC Overlay
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	14 - Concrete Deck - Protected w/ AC Overlay
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	22 - Concrete Deck - Protected w/ Rigid Overlay
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	144 - Reinforced Conc Arch
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	234 - Reinforced Conc Cap
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	155 - Reinforced Conc Floor Beam
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	171 - Concrete-Encased Steel Stringer
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	172 - Thru Truss - Bottom Chord - Conc Encased Steel
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	227 - Reinforced Conc Submerged Pile
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	174 - Floor Beam - Concrete Encased Steel
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	18 - Concrete Deck - Protected w/ Thin Overlay
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	204 - P/S Conc Column or Pile Extension
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	205 - Reinforced Conc Column or Pile Extension
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	210 - Reinforced Conc Pier Wall
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	233 - P/S Conc Cap
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	215 - Reinforced Conc Abutment
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	220 - Reinforced Conc Submerged Pile Cap/Footing
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	226 - P/S Conc Submerged Pile
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	173 - Arch - Concrete Encased Steel
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	143 - P/S Conc Arch
MMB072M - CONCRETE SPALL REPAIR, TYPE 2 Count	48
MMB075M - DECK JOINT RECONSTRUCTION	307 - Other--Assembly Joint/Seal
MMB075M - DECK JOINT RECONSTRUCTION	306 - Sliding Plates--Assembly Joint/Seal
MMB075M - DECK JOINT RECONSTRUCTION	305 - Finger Dams--Assembly Joint/Seal
MMB075M - DECK JOINT RECONSTRUCTION	304 - Open Expansion Joint
MMB075M - DECK JOINT RECONSTRUCTION	303 - Assembly Joint/Seal (modular)
MMB075M - DECK JOINT RECONSTRUCTION	302 - Compression Joint Seal
MMB075M - DECK JOINT RECONSTRUCTION	300 - Strip Seal Expansion Joint
MMB075M - DECK JOINT RECONSTRUCTION	301 - Pourable Joint Seal
MMB075M - DECK JOINT RECONSTRUCTION Count	8
MMB076M - CONCRETE BALUSTRADE REPAIR	331 - Reinforced Conc Bridge Railing
MMB076M - CONCRETE BALUSTRADE REPAIR	333 - Other Bridge Railing
MMB076M - CONCRETE BALUSTRADE REPAIR Count	2
MMB077M - REPAIR CONCRETE CURB	502 - Curbs/Sidewalks - Painted Steel
MMB077M - REPAIR CONCRETE CURB	501 - Curbs/Sidewalks - Unpainted Steel
MMB077M - REPAIR CONCRETE CURB	503 - Curbs/Sidewalks - Concrete
MMB077M - REPAIR CONCRETE CURB	504 - Curbs/Sidewalks - Timber
MMB077M - REPAIR CONCRETE CURB Count	4
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	375 - Pinned Bearing - Fixed
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	311 - Moveable Bearing (roller, sliding, etc.)
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	315 - Disk Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	317 - Rockers--Moveable Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	318 - Other--Moveable Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	334 - Metal Bridge Railing - Coated
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	372 - Sliding Plate Bearing - Expansion/Moveable
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	307 - Other--Assembly Joint/Seal
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	374 - Rocker Bearing - Expansion/Moveable
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	314 - Pot Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	376 - Spherical Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	502 - Curbs/Sidewalks - Painted Steel
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	520 - Isolation Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	373 - Bond Breaker Bearing - Expansion/Moveable
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	152 - Painted Steel Floor Beam
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	107 - Painted Steel Open Girder/Beam
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	113 - Painted Steel Stringer
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	121 - Painted Steel Bottom Chord Thru Truss
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	126 - Painted Steel Thru Truss (excl. bottom chord)
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	316 - Steel Plate/Sliding Plate--Moveable Bearing
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	141 - Painted Steel Arch
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	306 - Sliding Plates--Assembly Joint/Seal
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	161 - Painted Steel Pin and/or Pin and Hanger Assembly
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	231 - Painted Steel Cap
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	261 - Painted Steel Sheeting
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	300 - Strip Seal Expansion Joint

APPENDIX D - Line Items Matched to >1 Element

Line Item	Element
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	302 - Compression Joint Seal
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	303 - Assembly Joint/Seal (modular)
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	305 - Finger Dams--Assembly Joint/Seal
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	126 - Painted Steel Thru Truss (excl. bottom chord)
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING Count	30
MMB086M - DECK CORROSION INHIBITOR	12 - Concrete Deck - Bare
MMB086M - DECK CORROSION INHIBITOR	13 - Concrete Deck - Unprotected w/ AC Overlay
MMB086M - DECK CORROSION INHIBITOR	38 - Concrete Slab - Bare
MMB086M - DECK CORROSION INHIBITOR	39 - Concrete Slab - Unprotected w/ AC Overlay
MMB086M - DECK CORROSION INHIBITOR Count	4
MMB087M - DECK JOINT RESEAL (SILICON)	304 - Open Expansion Joint
MMB087M - DECK JOINT RESEAL (SILICON)	305 - Finger Dams--Assembly Joint/Seal
MMB087M - DECK JOINT RESEAL (SILICON)	306 - Sliding Plates--Assembly Joint/Seal
MMB087M - DECK JOINT RESEAL (SILICON)	302 - Compression Joint Seal
MMB087M - DECK JOINT RESEAL (SILICON)	301 - Pourable Joint Seal
MMB087M - DECK JOINT RESEAL (SILICON)	300 - Strip Seal Expansion Joint
MMB087M - DECK JOINT RESEAL (SILICON)	307 - Other--Assembly Joint/Seal
MMB087M - DECK JOINT RESEAL (SILICON)	303 - Assembly Joint/Seal (modular)
MMB087M - DECK JOINT RESEAL (SILICON) Count	8
MMB088M - BRIDGE DECK CRACK SEALING	35 - Conc Deck prot w/thin overlay & coated bars-PRECAS
MMB088M - BRIDGE DECK CRACK SEALING	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA
MMB088M - BRIDGE DECK CRACK SEALING	34 - ConcDk prot w/membrane AC overlay coated bars-PREC
MMB088M - BRIDGE DECK CRACK SEALING	27 - Concrete Deck - Protected w/ Cathodic System
MMB088M - BRIDGE DECK CRACK SEALING	22 - Concrete Deck - Protected w/ Rigid Overlay
MMB088M - BRIDGE DECK CRACK SEALING	18 - Concrete Deck - Protected w/ Thin Overlay
MMB088M - BRIDGE DECK CRACK SEALING	12 - Concrete Deck - Bare
MMB088M - BRIDGE DECK CRACK SEALING	14 - Concrete Deck - Protected w/ AC Overlay
MMB088M - BRIDGE DECK CRACK SEALING	13 - Concrete Deck - Unprotected w/ AC Overlay
MMB088M - BRIDGE DECK CRACK SEALING	26 - Concrete Deck - Protected w/ Coated Bars
MMB088M - BRIDGE DECK CRACK SEALING Count	10
MMB089M - DECK JOINT REPAIR	303 - Assembly Joint/Seal (modular)
MMB089M - DECK JOINT REPAIR	306 - Sliding Plates--Assembly Joint/Seal
MMB089M - DECK JOINT REPAIR	300 - Strip Seal Expansion Joint
MMB089M - DECK JOINT REPAIR	304 - Open Expansion Joint
MMB089M - DECK JOINT REPAIR	302 - Compression Joint Seal
MMB089M - DECK JOINT REPAIR	301 - Pourable Joint Seal
MMB089M - DECK JOINT REPAIR	307 - Other--Assembly Joint/Seal
MMB089M - DECK JOINT REPAIR	305 - Finger Dams--Assembly Joint/Seal
MMB089M - DECK JOINT REPAIR Count	8
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	304 - Open Expansion Joint
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	307 - Other--Assembly Joint/Seal
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	306 - Sliding Plates--Assembly Joint/Seal
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	305 - Finger Dams--Assembly Joint/Seal
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	301 - Pourable Joint Seal
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	302 - Compression Joint Seal
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	300 - Strip Seal Expansion Joint
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	303 - Assembly Joint/Seal (modular)
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT) Count	8
Grand Count	2247

APPENDIX E – Unmatched Line Items

Line Item	Unmatched reason
020012A21C - CLEARING SITE	No matching element
551001M - DECK EDGE STABILIZATION	Insufficient description
02001MB070 - BRIDGE HEADER REPAIR	No matching element
159020P - CONSTRUCTION BARRIER CURB, BRIDGE	No matching element
162003M - CONDITION SURVEY	No matching element
162005P - VIBRATION MONITORING	No matching element
201003P - CLEARING SITE	No matching element
201006P - CLEARING SITE, BRIDGE (___)	No matching element
201009P - CLEARING SITE, STRUCTURE (___)	No matching element
201037P - ASBESTOS REMOVAL, BRIDGE NO. ___	No matching element
201039P - TEMPORARY SHIELDING	No matching element
513009M - COARSE AGGREGATE LAYER	Insufficient description
502352M - FIBERGLASS PIPE PILE, FURNISHED, 12" DIAMETER	No matching element
502353M - FIBERGLASS PIPE PILE, DRIVEN, 12" DIAMETER	No matching element
504033P - CONCRETE PEDESTRIAN BRIDGE	No matching element
504053P - CONCRETE PYLON	No matching element
504073P - CAST STONE CAP	No matching element
504080P - CONCRETE SPILLWAY	No matching element
506018P - STEEL PEDESTRIAN BRIDGE	No matching element
506024P - MECHANICAL CONNECTOR	No matching element
507002P - ELASTOMERIC CONCRETE BRIDGE JOINT SYSTEM	No matching element
507027M - DATE PANEL	No matching element
507046M - 15" BY 32" CONCRETE BARRIER CURB, BRIDGE, HPC	No matching element
507048M - 24" BY 32" CONCRETE BARRIER CURB, BRIDGE	No matching element
507056P - CONCRETE MEDIAN BARRIER, HES	No matching element
507058P - CONCRETE MEDIAN SLAB, HPC	No matching element
507059P - CONCRETE MEDIAN BARRIER, HPC	No matching element
507062M - CAST-IN-PLACE EXODERMIC BRIDGE DECK SYSTEM, HPC	No matching element
507101P - CONCRETE CLOSURE POUR	No matching element
508003M - INLET FRAME AND GRATE	No matching element
508004M - NEW SCUPPER IN EXISTING DECK	No matching element
508005M - CLEAN EXISTING SCUPPERS AND PIPES	No matching element
508006M - SCUPPER	No matching element
508007M - SCUPPER RESET	No matching element
508008P - 6" STEEL ALLOY PIPE	No matching element
508009P - 8" STEEL ALLOY PIPE	No matching element
508012P - 10" STEEL ALLOY PIPE	No matching element
508017P - STANDPIPE	No matching element
508018M - STANDPIPE	No matching element
508020P - MANHOLE ON STRUCTURE	No matching element
508900P - FIBERGLASS DRAIN PIPE	No matching element

APPENDIX E – Unmatched Line Items

Line Item	Unmatched reason
508902P - _____" FIBERGLASS PIPE	No matching element
509024P - CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 4' 0" HIGH	No matching element
509030P - CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 0" HIGH	No matching element
509033P - CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 3" HIGH	No matching element
509039P - CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 8' 6" HIGH	No matching element
509042P - CHAIN-LINK FENCE, PVC-COATED STEEL, BRIDGE, 6' 3" HIGH	No matching element
509051P - CHAIN-LINK FENCE, GALVANIZED STEEL, BRID	No matching element
509057P - CHAIN-LINK FENCE, GALVANIZED STEEL, BRIDGE, 6' 3" HIGH	No matching element
509058P - CHAIN-LINK FENCE, TYPE I, ZINC-COATED STEEL, BRIDGE, 6' 3" H	No matching element
509065P - CHAIN-LINK FENCE, TYPE I, ZINC-COATED, STEEL, BRIDGE, 13'-0"	No matching element
509078P - CHAIN-LINK FENCE, ALUMINUM-COATED STEEL, BRIDGE, 6' 3"	No matching element
509079P - CHAIN-LINK FENCE, TYPE II, ALUMINUM-COATED STEEL, BRIDGE, 6'	No matching element
509083P - CHAIN-LINK FENCE, TYPE IV	No matching element
509084P - CHAIN-LINK FENCE, PVC-COATED STEEL, BRIDGE, 6' 3" HIGH, CURV	No matching element
509085P - PICKET FENCE, STEEL, BRIDGE, 6' 3" HIGH, CURVED TOP	No matching element
509086P - PICKET FENCE, STEEL, BRIDGE, 4' 0" HIGH	No matching element
509096P - CHAIN-LINK FENCE, GALVANIZED STEEL, BRIDGE, 6' 3" HIGH,	No matching element
509097P - CHAIN-LINK FENCE,TYPE I,ZINC-COATED STEEL,BRIDGE,6' 3" HIGH,	No matching element
509102P - PICKET FENCE, STEEL, BRIDGE, 6' 3" HIGH	No matching element
509111P - RELOCATE CHAIN-LINK FENCE, TYPE I, ZINC-COATED STEEL, BRIDGE	No matching element
509120P - FISH LADDER	No matching element
509123P - FIBERGLASS REINFORCED PLASTIC GRATING	No matching element
509127P - CHAIN-LINK FENCE, TYPE IV, PVC-COATED STEEL, BRIDGE, 6' 3" H	No matching element
511012M - COMPOSITE PILE, ___ INCH DIAMETER	No matching element
511015P - FIBERGLASS REINFORCED PLASTIC LUMBER	No matching element
511019M - TIDE CLEARANCE GAUGE	No matching element
511020P - FENDER SYSTEM	No matching element
511023P - FALL PROTECTION SYSTEM	No matching element
511025M - TIE-ROD SYSTEM	No matching element
512003M - CANTILEVER SIGN SUPPORT, STRUCTURE NO. ___	No matching element
512004P - RELOCATE CANTILEVER SIGN SUPPORT, STRUCTURE NO. ___	No matching element
512006M - BRIDGE MOUNTED SIGN SUPPORT, STRUCTURE NO. ___	No matching element
512007P - REMOVE/REINSTALL EXISTING BRIDGE MOUNTED SIGN	No matching element
512009M - BUTTERFLY SIGN SUPPORT, STRUCTURE	No matching element
512012M - OVERHEAD SIGN SUPPORT, STRUCTURE NO. ___	No matching element
513003P - RETAINING WALL, LOCATION NO. ___	No matching element
513006P - RETAINING WALL, CAST-IN-PLACE, LOCATION NO. ___	No matching element
513007P - STAGE LINE MSE RETAINING WALL	No matching element
517003P - HYBRID-COMPOSITE BEAMS, FURNISHING AND TESTING	No matching element
517006P - HYBRID-COMPOSITE BEAMS, ERECTING	No matching element

APPENDIX E – Unmatched Line Items

Line Item	Unmatched reason
520003P - PERMANENT GROUND ANCHOR	No matching element
554017P - CONCRETE ENCASEMENT REMOVAL AND PAINTING (BEAMS)	No matching element
554018P - CONCRETE ENCASEMENT REMOVAL AND PAINTING (EXPANSION	No matching element
555010M - REPAIR OF CONCRETE, TYPE E	Insufficient description
555011M - REPAIR OF CONCRETE, TYPE D	Insufficient description
557007P - FLOORBEAM REPAIR, __ VIADUCT, TYPE FB1	No matching element
557008M - FLOORBEAM REPAIR, __ VIADUCT, TYPE FB2, IF AND WHERE DIRECTE	No matching element
557012P - BRACING REPLACEMENT, TYPE BR1	No matching element
557013P - BRACING REPLACEMENT, TYPE BR2	No matching element
557014P - BRACING REPLACEMENT, TYPE BR3	No matching element
557018P - TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR1	No matching element
557019P - TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR2	No matching element
557020P - TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR3	No matching element
557021M - TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR4	No matching element
557022M - TRUSS REPAIR, CONRAIL VIADUCT, TYPE TR5, IF AND WHERE DIRECT	No matching element
558003P - DECK SLURRY OVERLAY SYSTEM	No matching element
558005P - RIVET REPLACEMENT	No matching element
612041P - BRIDGE VERTICAL UNDERCLEARANCE SIGN	No matching element
613005P - NOISE BARRIER, BRIDGE	No matching element
613010P - NOISE BARRIER FOUNDATION	No matching element
651523P - 24" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	No matching element
651524P - 30" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	No matching element
651525P - 36" DUCTILE IRON WATER PIPE BRIDGE, CLASS 350	No matching element
652293P - 12" STEEL SEWER PIPE, BRIDGE	No matching element
653048P - 6" GAS MAIN, BRIDGE	No matching element
653051P - 8" GAS MAIN, BRIDGE	No matching element
653057P - 12" GAS MAIN, BRIDGE	No matching element
703021M - SIGN LIGHTING, STRUCTURE NO. ___	No matching element
703024M - UNDERDECK LIGHTING TYPE W	No matching element
703027M - UNDERDECK LIGHTING TYPE P	No matching element
706019M - BARRIER GATE	No matching element
706022M - BRAKE REPLACEMENT	No matching element
750038P - DRAW BRIDGE BARRIER GATE	No matching element
MMB022M - REPAIR CATEGORY "A", MOVABLE	Insufficient description
MMB024M - REPAIR CATEGORY "B", MOVABLE	Insufficient description
MMB070M - BRIDGE HEADER REPAIR	No matching element
MMB181M - REPAIR CATEGORY "A"	Insufficient description
MMB182M - REPAIR CATEGORY "B"	Insufficient description
MMB183M - REPAIR CATEGORY "C"	Insufficient description
MMB184M - REPAIR CATEGORY "D"	Insufficient description

APPENDIX E – Unmatched Line Items

Line Item	Unmatched reason
MMD035M - CLEARING SITE	No matching element
MMD049M - REPAIR OF STRUCTURE	No matching element
MME127M - UNDERDECK LIGHTING ASSEMBLIES INSTALLATI	No matching element
MMR044M - ATTACHMENT TO STRUCTURE, BALUSTRADE, DETAIL NO.CD-609-10.1	Insufficient description
MMR045M - ATTACHMENT TO STRUCTURE, DETAIL NO. CD-609-10.2	Insufficient description
MMR046M - ATTACHMENT TO STRUCTURE, DETAIL NO. CD-609-10.2, W/	Insufficient description
MMR047M - ATTACHMENT TO STRUCTURE, TYPE A	Insufficient description
MMR048M - ATTACHMENT TO STRUCTURE, TYPE B	Insufficient description
MMR080M - CLEARING SITE	No matching element

Line Item	Line Item Unit	Element	Element Unit	Factor
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
020015R23C - REPAIR OF CONCRETE DECK, TYPE B	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
020015R25C - REPAIR OF CONCRETE DECK, TYPE C	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
02001MB077 - REPAIR CONCRETE CURB	LF	503 - Curbs/Sidewalks - Concrete	m.	0.3048
02001MB086 - DECK CORROSION INHIBITOR	SY	12 - Concrete Deck - Bare	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	28 - Steel Deck - Open Grid	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	29 - Steel Deck - Concrete Filled Grid	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	31 - Timber Deck - Bare	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	32 - Timber Deck - w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	54 - Timber Slab	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	55 - Timber Slab - w/ AC Overlay	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.8361
02001MB086 - DECK CORROSION INHIBITOR	SY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.8361
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	300 - Strip Seal Expansion Joint	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	301 - Pourable Joint Seal	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	302 - Compression Joint Seal	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	304 - Open Expansion Joint	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
02001MB087 - DECK JOINT RESEAL (SILICON)	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	300 - Strip Seal Expansion Joint	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	301 - Pourable Joint Seal	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	302 - Compression Joint Seal	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	304 - Open Expansion Joint	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
02001MB089 - DECK JOINT REPAIR	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	300 - Strip Seal Expansion Joint	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	301 - Pourable Joint Seal	m.	0.3048

Line Item	Line Item Unit	Element	Element Unit	Factor
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	302 - Compression Joint Seal	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	304 - Open Expansion Joint	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
02001MB090 - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
203111M - DEMONSTRATION STATIC LOAD TEST	U	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.	1
203111M - DEMONSTRATION STATIC LOAD TEST	U	225 - Unpainted Steel Submerged Pile	ea.	1
203111M - DEMONSTRATION STATIC LOAD TEST	U	226 - P/S Conc Submerged Pile	ea.	1
203111M - DEMONSTRATION STATIC LOAD TEST	U	227 - Reinforced Conc Submerged Pile	ea.	1
203111M - DEMONSTRATION STATIC LOAD TEST	U	228 - Timber Submerged Pile	ea.	1
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	12 - Concrete Deck - Bare	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.8361
453005M - FULL DEPTH CONCRETE PAVEMENT REPAIR, CONCRETE CLASS V	SY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	12 - Concrete Deck - Bare	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.8361
453006M - FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	SY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.8361
502015M - STATIC PILE LOAD TEST	U	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.	1
502015M - STATIC PILE LOAD TEST	U	225 - Unpainted Steel Submerged Pile	ea.	1
502015M - STATIC PILE LOAD TEST	U	226 - P/S Conc Submerged Pile	ea.	1
502015M - STATIC PILE LOAD TEST	U	227 - Reinforced Conc Submerged Pile	ea.	1
502015M - STATIC PILE LOAD TEST	U	228 - Timber Submerged Pile	ea.	1
502018M - DYNAMIC PILE LOAD TEST	U	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.	1
502018M - DYNAMIC PILE LOAD TEST	U	225 - Unpainted Steel Submerged Pile	ea.	1
502018M - DYNAMIC PILE LOAD TEST	U	226 - P/S Conc Submerged Pile	ea.	1
502018M - DYNAMIC PILE LOAD TEST	U	227 - Reinforced Conc Submerged Pile	ea.	1
502018M - DYNAMIC PILE LOAD TEST	U	228 - Timber Submerged Pile	ea.	1
502067P - RESTRIKE CAST-IN-PLACE CONCRETE PILE, 12" DIAMETER	U	227 - Reinforced Conc Submerged Pile	ea.	1
502201M - SPLICE CAST-IN-PLACE PILE	U	205 - Reinforced Conc Column or Pile Extension	ea.	1
502201M - SPLICE CAST-IN-PLACE PILE	U	227 - Reinforced Conc Submerged Pile	ea.	1
502201M - SPLICE CAST-IN-PLACE PILE	U	270 - Conc Encased Steel Column or Pile Extension	ea.	1
502202M - SPLICE PRESTRESSED CONCRETE PILE	U	204 - P/S Conc Column or Pile Extension	ea.	1
502202M - SPLICE PRESTRESSED CONCRETE PILE	U	226 - P/S Conc Submerged Pile	ea.	1
502204M - SPLICE STEEL H-PILE	U	201 - Unpainted Steel Column or Pile Extension	ea.	1
502204M - SPLICE STEEL H-PILE	U	202 - Painted Steel Column or Pile Extension	ea.	1
502204M - SPLICE STEEL H-PILE	U	225 - Unpainted Steel Submerged Pile	ea.	1
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	U	201 - Unpainted Steel Column or Pile Extension	ea.	1
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	U	202 - Painted Steel Column or Pile Extension	ea.	1
502205M - SPLICE CONCRETE FILLED STEEL PIPE PILE	U	225 - Unpainted Steel Submerged Pile	ea.	1
502207M - PILE SHOE	U	225 - Unpainted Steel Submerged Pile	ea.	1
502207M - PILE SHOE	U	226 - P/S Conc Submerged Pile	ea.	1
502207M - PILE SHOE	U	228 - Timber Submerged Pile	ea.	1
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	210 - Reinforced Conc Pier Wall	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	215 - Reinforced Conc Abutment	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	234 - Reinforced Conc Cap	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	241 - Reinforced Concrete Culvert	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	271 - Concrete Encased Steel Pier Cap	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	507 - Headwalls - Other - Concrete/Masonry	m.	0.3048
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.	0.3048

APPENDIX F - Matched Unit Detail

Line Item	Line Item Unit	Element	Element Unit	Factor
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	509 - Headwalls - Culvert - Concrete/Masonry	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	210 - Reinforced Conc Pier Wall	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	211 - Other Material Pier Wall	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	215 - Reinforced Conc Abutment	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	241 - Reinforced Concrete Culvert	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	333 - Other Bridge Railing	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	507 - Headwalls - Other - Concrete/Masonry	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.	0.3048
504075P - ARCHITECTURAL CAST STONE	LF	509 - Headwalls - Culvert - Concrete/Masonry	m.	0.3048
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	LF	109 - P/S Conc Open Girder/Beam	m.	0.3048
505004P - PRETENSIONED PRESTRESSED CONCRETE BEAM 36"	LF	115 - P/S Conc Stringer	m.	0.3048
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	LF	109 - P/S Conc Open Girder/Beam	m.	0.3048
505006P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 54"	LF	115 - P/S Conc Stringer	m.	0.3048
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	LF	109 - P/S Conc Open Girder/Beam	m.	0.3048
505009P - PRETENSIONED PRESTRESSED CONCRETE BEAM, 63"	LF	115 - P/S Conc Stringer	m.	0.3048
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	LF	109 - P/S Conc Open Girder/Beam	m.	0.3048
505011P - PRETENSIONED-PRESTRESSED CONCRETE BEAM, 79"	LF	115 - P/S Conc Stringer	m.	0.3048
505015P - PRESTRESSED CONCRETE BOX BEAM, (TYPE BI-36), 36" X 27"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505027P - PRESTRESSED CONCRETE BOX BEAM, (TYPE BI-48), 48" X 27"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505030P - PRESTRESSED CONCRETE BOX BEAM, (TYPE BII-48), 48" X 33"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505039P - PRESTRESSED CONCRETE SLAB BEAM, (TYPE SII-36), 36" X 15"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505045P - PRESTRESSED CONCRETE SLAB BEAM, (TYPE SIV-36), 36" X 21"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505048P - PRESTRESSED CONCRETE SLAB BEAM, (TYPE SII-48), 48" X 15"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505054P - PRESTRESSED CONCRETE SLAB BEAM, (TYPE SIV-48), 48" X 21"	LF	104 - P/S Conc Closed Web/Box Girder	m.	0.3048
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	LF	109 - P/S Conc Open Girder/Beam	m.	0.3048
505055P - PRESTRESSED CONCRETE CHANNEL BEAM, 54"X27"	LF	115 - P/S Conc Stringer	m.	0.3048
505057P - PRECAST CONCRETE CULVERT	LF	241 - Reinforced Concrete Culvert	m.	0.3048
505057P - PRECAST CONCRETE CULVERT	LF	243 - Other Culvert	m.	0.3048
505060P - PRECAST CONCRETE ARCH STRUCTURE	LF	144 - Reinforced Conc Arch	m.	0.3048
505060P - PRECAST CONCRETE ARCH STRUCTURE	LF	145 - Other Arch	m.	0.3048
505060P - PRECAST CONCRETE ARCH STRUCTURE	LF	173 - Arch - Concrete Encased Steel	m.	0.3048
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
505088P - PRECAST PARAPET PANEL	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
505088P - PRECAST PARAPET PANEL	LF	333 - Other Bridge Railing	m.	0.3048
505088P - PRECAST PARAPET PANEL	LF	507 - Headwalls - Other - Concrete/Masonry	m.	0.3048
505088P - PRECAST PARAPET PANEL	LF	509 - Headwalls - Culvert - Concrete/Masonry	m.	0.3048
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
505090P - PRESTRESSED CONCRETE DECK PANELS, HPC	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
505091P - PRECAST LIGHTWEIGHT CONCRETE DECK PANELS	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	310 - Elastomeric Bearing	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	314 - Pot Bearing	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	315 - Disk Bearing	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	370 - Elastomeric Bearing with Teflon	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	372 - Sliding Plate Bearing - Expansion/Moveable	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	374 - Rocker Bearing - Expansion/Moveable	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	376 - Spherical Bearing	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	520 - Isolation Bearing	ea.	1
506005P - STRUCTURAL BEARING ASSEMBLY, SEISMIC	U	521 - Bearing - Other	ea.	1
506006P - REINFORCED ELASTOMERIC BEARING ASSEMBLY	U	310 - Elastomeric Bearing	ea.	1
506006P - REINFORCED ELASTOMERIC BEARING ASSEMBLY	U	370 - Elastomeric Bearing with Teflon	ea.	1

Line Item	Line Item Unit	Element	Element Unit	Factor
506008P - RESET BEARING	U	310 - Elastomeric Bearing	ea.	1
506008P - RESET BEARING	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
506008P - RESET BEARING	U	312 - Enclosed/Concealed Bearing	ea.	1
506008P - RESET BEARING	U	315 - Disk Bearing	ea.	1
506008P - RESET BEARING	U	370 - Elastomeric Bearing with Teflon	ea.	1
506008P - RESET BEARING	U	372 - Sliding Plate Bearing - Expansion/Moveable	ea.	1
506008P - RESET BEARING	U	374 - Rocker Bearing - Expansion/Moveable	ea.	1
506008P - RESET BEARING	U	376 - Spherical Bearing	ea.	1
506008P - RESET BEARING	U	520 - Isolation Bearing	ea.	1
506008P - RESET BEARING	U	521 - Bearing - Other	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	310 - Elastomeric Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	312 - Enclosed/Concealed Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	313 - Fixed Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	314 - Pot Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	315 - Disk Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	370 - Elastomeric Bearing with Teflon	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	372 - Sliding Plate Bearing - Expansion/Moveable	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	373 - Bond Breaker Bearing - Expansion/Moveable	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	374 - Rocker Bearing - Expansion/Moveable	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	375 - Pinned Bearing - Fixed	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	376 - Spherical Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	520 - Isolation Bearing	ea.	1
506009M - STRUCTURAL BEARING ASSEMBLY	U	521 - Bearing - Other	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	314 - Pot Bearing	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	372 - Sliding Plate Bearing - Expansion/Moveable	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	374 - Rocker Bearing - Expansion/Moveable	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	376 - Spherical Bearing	ea.	1
506010M - HIGH LOAD MULTIROTATIONAL BEARING ASSEMBLY	U	521 - Bearing - Other	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	312 - Enclosed/Concealed Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	313 - Fixed Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	314 - Pot Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	315 - Disk Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	372 - Sliding Plate Bearing - Expansion/Moveable	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	373 - Bond Breaker Bearing - Expansion/Moveable	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	374 - Rocker Bearing - Expansion/Moveable	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	375 - Pinned Bearing - Fixed	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	376 - Spherical Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	520 - Isolation Bearing	ea.	1
506011M - ANCHOR BOLT INSTALLATION	U	521 - Bearing - Other	ea.	1
506021P - STEEL GRID FLOORING	SF	28 - Steel Deck - Open Grid	sq.m.	0.0929
506021P - STEEL GRID FLOORING	SF	29 - Steel Deck - Concrete Filled Grid	sq.m.	0.0929
506040P - STEEL REPAIR, TYPE ___	U	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.	1
506040P - STEEL REPAIR, TYPE ___	U	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.	1
506040P - STEEL REPAIR, TYPE ___	U	201 - Unpainted Steel Column or Pile Extension	ea.	1
506040P - STEEL REPAIR, TYPE ___	U	202 - Painted Steel Column or Pile Extension	ea.	1
506040P - STEEL REPAIR, TYPE ___	U	225 - Unpainted Steel Submerged Pile	ea.	1
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	302 - Compression Joint Seal	m.	0.3048
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507003P - 1 3/4" BY 1 3/4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	LF	302 - Compression Joint Seal	m.	0.3048
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507004P - 1 3/4" X 1 3/4" PREFOREMD ELASTOMERIC JOINT SEALER	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	302 - Compression Joint Seal	m.	0.3048
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507006P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	LF	302 - Compression Joint Seal	m.	0.3048
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507007P - 2 1/2" BY 2 1/2" PREFORMED ELASTOMERIC JOINT SEALER	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	LF	302 - Compression Joint Seal	m.	0.3048
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507008P - 4" X 4" PREFORMED ELASTOMERIC JOINT SEALER	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	302 - Compression Joint Seal	m.	0.3048
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507009P - 4" BY 4" PREFORMED ELASTOMERIC JOINT ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507014P - NEOPRENE STRIP SEAL GLAND	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507014P - NEOPRENE STRIP SEAL GLAND	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507015P - STRIP SEAL EXPANSION JOINT ASSEMBLY	LF	300 - Strip Seal Expansion Joint	m.	0.3048
507015P - STRIP SEAL EXPANSION JOINT ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507016P - FINGER JOINT EXPANSION DAM	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
507016P - FINGER JOINT EXPANSION DAM	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
507016P - FINGER JOINT EXPANSION DAM	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507018P - MODULAR EXPANSION JOINT ASSEMBLY	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048

Line Item	Line Item Unit	Element	Element Unit	Factor
507018P - MODULAR EXPANSION JOINT ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507020P - ASPHALTIC BRIDGE JOINT SYSTEM	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507034P - CONCRETE BARRIER CURB	LF	333 - Other Bridge Railing	m.	0.3048
507036P - CONCRETE BRIDGE PARAPET	LF	333 - Other Bridge Railing	m.	0.3048
507037P - BARRIER PARAPET MODIFICATIONS	LF	333 - Other Bridge Railing	m.	0.3048
507038P - CONCRETE BRIDGE PARAPET WITH MOMENT SLAB, HPC	LF	333 - Other Bridge Railing	m.	0.3048
507039P - CONCRETE BRIDGE PARAPET, HPC	LF	333 - Other Bridge Railing	m.	0.3048
507040P - CONCRETE BRIDGE PARAPET, HES	LF	333 - Other Bridge Railing	m.	0.3048
507042P - 4-BAR OPEN STEEL PARAPET	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
507042P - 4-BAR OPEN STEEL PARAPET	LF	333 - Other Bridge Railing	m.	0.3048
507042P - 4-BAR OPEN STEEL PARAPET	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.8361
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.8361
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.8361
507054P - CONCRETE BRIDGE RELIEF SLAB, HPC	SY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.8361
507055P - FIVE BAR OPEN STEEL PARAPET	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
507055P - FIVE BAR OPEN STEEL PARAPET	LF	333 - Other Bridge Railing	m.	0.3048
507055P - FIVE BAR OPEN STEEL PARAPET	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.8361
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.8361
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.8361
507066P - PRECAST CONCRETE BRIDGE APPROACH	SY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.8361
507067P - CONCRETE BALUSTRADE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
507073M - DIAMOND GRINDING, CONCRETE DECK SURFACE	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
507095P - PREFORMED JOINT FILLER ASSEMBLY	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
507096P - SLIDING PLATE EXPANSION JOINT ASSEMBLY	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
509003P - BRIDGE RAILING (1 RAIL, ALUMINUM)	LF	333 - Other Bridge Railing	m.	0.3048
509006P - BRIDGE RAILING (2 RAIL, ALUMINUM)	LF	333 - Other Bridge Railing	m.	0.3048
509007P - ALUMINIUM RAILING, BRIDGE, 5'-6" HIGH	LF	333 - Other Bridge Railing	m.	0.3048
509008P - ALUMINIUM RAILING, BRIDGE, 7'-0" HIGH	LF	333 - Other Bridge Railing	m.	0.3048
509009P - BRIDGE RAILING (1 RAIL, STEEL)	LF	333 - Other Bridge Railing	m.	0.3048
509010P - STEEL BRIDGE RAILIN, TWO-RAIL	LF	333 - Other Bridge Railing	m.	0.3048
509011P - STEEL BRIDGE RAILING, THREE-RAIL	LF	333 - Other Bridge Railing	m.	0.3048
509012P - BRIDGE RAILING (2 RAIL, STEEL)	LF	333 - Other Bridge Railing	m.	0.3048
509013P - 2-BAR STEEL BRIDGE RAILING	LF	333 - Other Bridge Railing	m.	0.3048
509100P - ORNAMENTAL RAILING	LF	333 - Other Bridge Railing	m.	0.3048
509101P - PIPE RAIL	LF	333 - Other Bridge Railing	m.	0.3048
509131P - METAL MEDIAN BARRIER	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
509131P - METAL MEDIAN BARRIER	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
509132P - METAL HALF MEDIAN BARRIER	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
509132P - METAL HALF MEDIAN BARRIER	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
513022P - CONCRETE COPING	LF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.	0.3048
513022P - CONCRETE COPING	LF	507 - Headwalls - Other - Concrete/Masonry	m.	0.3048
513022P - CONCRETE COPING	LF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.	0.3048
513022P - CONCRETE COPING	LF	509 - Headwalls - Culvert - Concrete/Masonry	m.	0.3048
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
516003P - PRECAST EXODERMIC BRIDGE DECK SYSTEM	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
516004P - PRECAST EXODERMIC BRIDGE DECK SYSTEM, LIGHTWEIGHT	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929

APPENDIX F - Matched Unit Detail

Line Item	Line Item Unit	Element	Element Unit	Factor
551003M - REPAIR OF CONCRETE DECK, TYPE A	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
551006M - REPAIR OF CONCRETE DECK, TYPE B	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
551007M - REPAIR OF CONCRETE DECK, TYPE B1	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
551009M - REPAIR OF CONCRETE DECK, TYPE C	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
551014P - DECK JOINT RESEAL	LF	301 - Pourable Joint Seal	m.	0.3048
551014P - DECK JOINT RESEAL	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
551014P - DECK JOINT RESEAL	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
551014P - DECK JOINT RESEAL	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	300 - Strip Seal Expansion Joint	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	302 - Compression Joint Seal	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	304 - Open Expansion Joint	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
551015M - DECK JOINT RECONSTRUCTION	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
551017M - CONCRETE OVERLAY, HPC	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
551017M - CONCRETE OVERLAY, HPC	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
551018M - CRACK SPANNING MEMBRANE	LF	301 - Pourable Joint Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	215 - Reinforced Conc Abutment	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	217 - Other Material Abutment	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	300 - Strip Seal Expansion Joint	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	300 - Strip Seal Expansion Joint	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	302 - Compression Joint Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	302 - Compression Joint Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	304 - Open Expansion Joint	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	304 - Open Expansion Joint	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	507 - Headwalls - Other - Concrete/Masonry	m.	0.3048
551021M - HEADER RECONSTRUCTION	LF	509 - Headwalls - Culvert - Concrete/Masonry	m.	0.3048
551030M - CURB RECONSTRUCTION, BRIDGE	LF	503 - Curbs/Sidewalks - Concrete	m.	0.3048
551045M - PARAPET MODIFICATIONS	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
551045M - PARAPET MODIFICATIONS	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
551045M - PARAPET MODIFICATIONS	LF	332 - Timber Bridge Railing	m.	0.3048
551045M - PARAPET MODIFICATIONS	LF	333 - Other Bridge Railing	m.	0.3048
551045M - PARAPET MODIFICATIONS	LF	334 - Metal Bridge Railing - Coated	m.	0.3048

Line Item	Line Item Unit	Element	Element Unit	Factor
554027P - CLEANING AND PAINTING OF BEARING	U	311 - Moveable Bearing (roller, sliding, etc.)	ea.	1
554027P - CLEANING AND PAINTING OF BEARING	U	312 - Enclosed/Concealed Bearing	ea.	1
554027P - CLEANING AND PAINTING OF BEARING	U	313 - Fixed Bearing	ea.	1
554027P - CLEANING AND PAINTING OF BEARING	U	314 - Pot Bearing	ea.	1
554027P - CLEANING AND PAINTING OF BEARING	U	315 - Disk Bearing	ea.	1
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	38 - Concrete Slab - Bare	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
555013M - CONCRETE SPALL REPAIR	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	29 - Steel Deck - Concrete Filled Grid	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	U	270 - Conc Encased Steel Column or Pile Extension	ea.	1
555025P - RETROFIT STRIP SEAL JOINT SYSTEM	LF	300 - Strip Seal Expansion Joint	m.	0.3048
609004M - BEAM GUIDE RAIL, BRIDGE	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
609004M - BEAM GUIDE RAIL, BRIDGE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
609004M - BEAM GUIDE RAIL, BRIDGE	LF	332 - Timber Bridge Railing	m.	0.3048
609004M - BEAM GUIDE RAIL, BRIDGE	LF	333 - Other Bridge Railing	m.	0.3048
609004M - BEAM GUIDE RAIL, BRIDGE	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	LF	332 - Timber Bridge Railing	m.	0.3048
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	LF	333 - Other Bridge Railing	m.	0.3048
609015M - THRIE BEAM GUIDE RAIL, BRIDGE	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	332 - Timber Bridge Railing	m.	0.3048
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	333 - Other Bridge Railing	m.	0.3048
701013P - 1 1/2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	332 - Timber Bridge Railing	m.	0.3048
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	333 - Other Bridge Railing	m.	0.3048

Line Item	Line Item Unit	Element	Element Unit	Factor
701019P - 2" RIGID METALLIC CONDUIT ON STRUCTURE	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	LF	330 - Metal Bridge Railing - Uncoated	m.	0.3048
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	LF	332 - Timber Bridge Railing	m.	0.3048
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	LF	333 - Other Bridge Railing	m.	0.3048
701022P - 3" RIGID METALLIC CONDUIT ON STRUCTURE	LF	334 - Metal Bridge Railing - Coated	m.	0.3048
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	12 - Concrete Deck - Bare	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.	0.0929
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.	0.0929
MMB075M - DECK JOINT RECONSTRUCTION	LF	300 - Strip Seal Expansion Joint	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	301 - Pourable Joint Seal	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	302 - Compression Joint Seal	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	304 - Open Expansion Joint	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
MMB075M - DECK JOINT RECONSTRUCTION	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
MMB076M - CONCRETE BALUSTRADE REPAIR	LF	331 - Reinforced Conc Bridge Railing	m.	0.3048
MMB076M - CONCRETE BALUSTRADE REPAIR	LF	333 - Other Bridge Railing	m.	0.3048
MMB077M - REPAIR CONCRETE CURB	LF	501 - Curbs/Sidewalks - Unpainted Steel	m.	0.3048
MMB077M - REPAIR CONCRETE CURB	LF	502 - Curbs/Sidewalks - Painted Steel	m.	0.3048
MMB077M - REPAIR CONCRETE CURB	LF	503 - Curbs/Sidewalks - Concrete	m.	0.3048
MMB077M - REPAIR CONCRETE CURB	LF	504 - Curbs/Sidewalks - Timber	m.	0.3048
MMB086M - DECK CORROSION INHIBITOR	SY	12 - Concrete Deck - Bare	sq.m.	0.8361
MMB086M - DECK CORROSION INHIBITOR	SY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.	0.8361
MMB086M - DECK CORROSION INHIBITOR	SY	38 - Concrete Slab - Bare	sq.m.	0.8361
MMB086M - DECK CORROSION INHIBITOR	SY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.	0.8361
MMB087M - DECK JOINT RESEAL (SILICON)	LF	300 - Strip Seal Expansion Joint	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	301 - Pourable Joint Seal	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	302 - Compression Joint Seal	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	304 - Open Expansion Joint	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
MMB087M - DECK JOINT RESEAL (SILICON)	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	300 - Strip Seal Expansion Joint	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	301 - Pourable Joint Seal	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	302 - Compression Joint Seal	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	304 - Open Expansion Joint	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
MMB089M - DECK JOINT REPAIR	LF	307 - Other--Assembly Joint/Seal	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	300 - Strip Seal Expansion Joint	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	301 - Pourable Joint Seal	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	302 - Compression Joint Seal	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	303 - Assembly Joint/Seal (modular)	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	304 - Open Expansion Joint	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	305 - Finger Dams--Assembly Joint/Seal	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	306 - Sliding Plates--Assembly Joint/Seal	m.	0.3048
MMB090M - DECK JOINT RESEAL (RUBBER ASPHALT)	LF	307 - Other--Assembly Joint/Seal	m.	0.3048

Line Item	Line Item Unit	Element	Element Unit
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	102 - Painted Steel Closed Web/Box Girder	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	107 - Painted Steel Open Girder/Beam	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	113 - Painted Steel Stringer	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	121 - Painted Steel Bottom Chord Thru Truss	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	131 - Painted Steel Deck Truss	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	141 - Painted Steel Arch	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	152 - Painted Steel Floor Beam	m.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	202 - Painted Steel Column or Pile Extension	ea.
02001MB084 - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	231 - Painted Steel Cap	m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	12 - Concrete Deck - Bare	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	358 - Deck Cracking	ea.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	38 - Concrete Slab - Bare	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
02001MB088 - BRIDGE DECK CRACK SEALING	LF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
158087M - TEMPORARY RIPRAP	CY	361 - Scour	ea.
401027M - POLYMERIZED JOINT ADHESIVE	LF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	32 - Timber Deck - w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
401027M - POLYMERIZED JOINT ADHESIVE	LF	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
401027M - POLYMERIZED JOINT ADHESIVE	LF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	55 - Timber Slab - w/ AC Overlay	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
401027M - POLYMERIZED JOINT ADHESIVE	LF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	12 - Concrete Deck - Bare	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	38 - Concrete Slab - Bare	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
401112M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
405018M - CONTRACTION JOINT ASSEMBLY	U	300 - Strip Seal Expansion Joint	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	301 - Pourable Joint Seal	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	302 - Compression Joint Seal	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	303 - Assembly Joint/Seal (modular)	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	305 - Finger Dams--Assembly Joint/Seal	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	306 - Sliding Plates--Assembly Joint/Seal	m.
405018M - CONTRACTION JOINT ASSEMBLY	U	307 - Other--Assembly Joint/Seal	m.
405021M - EXPANSION JOINT ASSEMBLY	U	300 - Strip Seal Expansion Joint	m.
405021M - EXPANSION JOINT ASSEMBLY	U	301 - Pourable Joint Seal	m.
405021M - EXPANSION JOINT ASSEMBLY	U	302 - Compression Joint Seal	m.
405021M - EXPANSION JOINT ASSEMBLY	U	303 - Assembly Joint/Seal (modular)	m.
405021M - EXPANSION JOINT ASSEMBLY	U	305 - Finger Dams--Assembly Joint/Seal	m.
405021M - EXPANSION JOINT ASSEMBLY	U	306 - Sliding Plates--Assembly Joint/Seal	m.
405021M - EXPANSION JOINT ASSEMBLY	U	307 - Other--Assembly Joint/Seal	m.
501003P - TEMPORARY SHEETING	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
501003P - TEMPORARY SHEETING	SF	225 - Unpainted Steel Submerged Pile	ea.
501003P - TEMPORARY SHEETING	SF	226 - P/S Conc Submerged Pile	ea.
501003P - TEMPORARY SHEETING	SF	227 - Reinforced Conc Submerged Pile	ea.
501003P - TEMPORARY SHEETING	SF	228 - Timber Submerged Pile	ea.
501007P - PERMANENT SHEETING, FIBERGLASS	SF	217 - Other Material Abutment	m.

Line Item	Line Item Unit	Element	Element Unit
501012P - PERMANENT COFFERDAM	LS	215 - Reinforced Conc Abutment	m.
501012P - PERMANENT COFFERDAM	LS	216 - Timber Abutment	m.
501012P - PERMANENT COFFERDAM	LS	217 - Other Material Abutment	m.
501012P - PERMANENT COFFERDAM	LS	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
501012P - PERMANENT COFFERDAM	LS	225 - Unpainted Steel Submerged Pile	ea.
501012P - PERMANENT COFFERDAM	LS	226 - P/S Conc Submerged Pile	ea.
501012P - PERMANENT COFFERDAM	LS	227 - Reinforced Conc Submerged Pile	ea.
501012P - PERMANENT COFFERDAM	LS	228 - Timber Submerged Pile	ea.
501012P - PERMANENT COFFERDAM	LS	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
501012P - PERMANENT COFFERDAM	LS	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
502006M - PREBORED HOLE	LF	225 - Unpainted Steel Submerged Pile	ea.
502006M - PREBORED HOLE	LF	226 - P/S Conc Submerged Pile	ea.
502006M - PREBORED HOLE	LF	227 - Reinforced Conc Submerged Pile	ea.
502006M - PREBORED HOLE	LF	228 - Timber Submerged Pile	ea.
502009M - TEST PILE, FURNISHED	LF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
502009M - TEST PILE, FURNISHED	LF	225 - Unpainted Steel Submerged Pile	ea.
502009M - TEST PILE, FURNISHED	LF	226 - P/S Conc Submerged Pile	ea.
502009M - TEST PILE, FURNISHED	LF	227 - Reinforced Conc Submerged Pile	ea.
502009M - TEST PILE, FURNISHED	LF	228 - Timber Submerged Pile	ea.
502012M - TEST PILE, DRIVEN	LF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
502012M - TEST PILE, DRIVEN	LF	225 - Unpainted Steel Submerged Pile	ea.
502012M - TEST PILE, DRIVEN	LF	226 - P/S Conc Submerged Pile	ea.
502012M - TEST PILE, DRIVEN	LF	227 - Reinforced Conc Submerged Pile	ea.
502012M - TEST PILE, DRIVEN	LF	228 - Timber Submerged Pile	ea.
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502021M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 12" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502024M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 14" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502027M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 16" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502036M - CAST-IN-PLACE CONCRETE PILE, FURNISHED, 24" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502045M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 12" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502048M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 14" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502051M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 16" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502060M - CAST-IN-PLACE CONCRETE PILE, DRIVEN, 24" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502152M - PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER	LF	204 - P/S Conc Column or Pile Extension	ea.
502152M - PRESTRESSED CONCRETE PILE, FURNISHED, 36" DIAMETER	LF	226 - P/S Conc Submerged Pile	ea.
502155M - PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER	LF	204 - P/S Conc Column or Pile Extension	ea.
502155M - PRESTRESSED CONCRETE PILE, DRIVEN, 36" DIAMETER	LF	226 - P/S Conc Submerged Pile	ea.
502157M - PRESTRESSED CONCRETE PILES, INSTALLED	LF	204 - P/S Conc Column or Pile Extension	ea.
502157M - PRESTRESSED CONCRETE PILES, INSTALLED	LF	226 - P/S Conc Submerged Pile	ea.
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	LF	202 - Painted Steel Column or Pile Extension	ea.
502165M - STEEL H-PILE, FURNISHED, HP 12 X 53	LF	225 - Unpainted Steel Submerged Pile	ea.
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	LF	202 - Painted Steel Column or Pile Extension	ea.
502168M - STEEL H-PILE, FURNISHED, HP 12 X 74	LF	225 - Unpainted Steel Submerged Pile	ea.
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	LF	202 - Painted Steel Column or Pile Extension	ea.
502171M - STEEL H-PILE, FURNISHED, HP 14 X 73	LF	225 - Unpainted Steel Submerged Pile	ea.
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	LF	202 - Painted Steel Column or Pile Extension	ea.
502173M - STEEL H-PILE, FURNISHED, HP 14 X 102	LF	225 - Unpainted Steel Submerged Pile	ea.
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	LF	202 - Painted Steel Column or Pile Extension	ea.
502174M - STEEL H-PILE, FURNISHED, HP 14 X 117	LF	225 - Unpainted Steel Submerged Pile	ea.
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	LF	202 - Painted Steel Column or Pile Extension	ea.
502183M - STEEL H-PILE, DRIVEN, HP 12 X 53	LF	225 - Unpainted Steel Submerged Pile	ea.
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	LF	201 - Unpainted Steel Column or Pile Extension	ea.

Line Item	Line Item Unit	Element	Element Unit
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	LF	202 - Painted Steel Column or Pile Extension	ea.
502186M - STEEL H-PILE, DRIVEN, HP 12 X 74	LF	225 - Unpainted Steel Submerged Pile	ea.
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	LF	202 - Painted Steel Column or Pile Extension	ea.
502189M - STEEL H-PILE, DRIVEN, HP 14 X 73	LF	225 - Unpainted Steel Submerged Pile	ea.
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	LF	202 - Painted Steel Column or Pile Extension	ea.
502191M - STEEL H-PILE, DRIVEN, HP 14 X 102	LF	225 - Unpainted Steel Submerged Pile	ea.
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	LF	202 - Painted Steel Column or Pile Extension	ea.
502192M - STEEL H-PILE, DRIVEN, HP 14 X 117	LF	225 - Unpainted Steel Submerged Pile	ea.
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	LF	202 - Painted Steel Column or Pile Extension	ea.
502208M - CONCRETE-FILLED STEEL PIPE PILE, FURNISH	LF	225 - Unpainted Steel Submerged Pile	ea.
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	LF	201 - Unpainted Steel Column or Pile Extension	ea.
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	LF	202 - Painted Steel Column or Pile Extension	ea.
502209M - CONCRETE-FILLED STEEL PIPE PILE, DRIVEN	LF	225 - Unpainted Steel Submerged Pile	ea.
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502310M - CAST-IN-PLACE CONCRETE PILE, DRILLED, 24" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	LF	227 - Reinforced Conc Submerged Pile	ea.
502350M - CONCRETE-FILLED FIBERGLASS PIPE PILE, FURNISHED, 16" DIAMETE	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
502351M - CONCRETE-FILLED FIBERGLASS PIPE PILE, DRIVEN, 16" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503017M - DRILLED SHAFT IN SOIL, 30" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503018M - DRILLED SHAFT IN SOIL 36" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503019M - DRILLED SHAFT IN SOIL, 42" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503021M - DRILLED SHAFT IN SOIL 48" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503024M - DRILLED SHAFT IN SOIL 54" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503030M - DRILLED SHAFT IN SOIL 72" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503031M - DRILLED SHAFT IN SOIL, 96" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503033M - DRILLED SHAFT IN ROCK 36" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503035M - DRILLED SHAFT IN ROCK, 42" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503036M - DRILLED SHAFT IN ROCK 48" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
503046M - DRILLED SHAFT IN ROCK, 90" DIAMETER	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
503055M - DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503055M - DRILLED SHAFT FOR SIGN STRUCTURE FOUNDATION	LF	227 - Reinforced Conc Submerged Pile	ea.
503060M - PERMANENT STEEL CASING, 96" DIAMETER	LF	205 - Reinforced Conc Column or Pile Extension	ea.
503060M - PERMANENT STEEL CASING, 96" DIAMETER	LF	227 - Reinforced Conc Submerged Pile	ea.
504003P - REINFORCEMENT STEEL	LB	104 - P/S Conc Closed Web/Box Girder	m.
504003P - REINFORCEMENT STEEL	LB	105 - Reinforced Concrete Closed Webs/Box Girder	m.
504003P - REINFORCEMENT STEEL	LB	109 - P/S Conc Open Girder/Beam	m.
504003P - REINFORCEMENT STEEL	LB	110 - Reinforced Conc Open Girder/Beam	m.
504003P - REINFORCEMENT STEEL	LB	115 - P/S Conc Stringer	m.
504003P - REINFORCEMENT STEEL	LB	116 - Reinforced Conc Stringer	m.
504003P - REINFORCEMENT STEEL	LB	12 - Concrete Deck - Bare	sq.m.
504003P - REINFORCEMENT STEEL	LB	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	144 - Reinforced Conc Arch	m.

Line Item	Line Item Unit	Element	Element Unit
504003P - REINFORCEMENT STEEL	LB	155 - Reinforced Conc Floor Beam	m.
504003P - REINFORCEMENT STEEL	LB	170 - Open Girder - Concrete Encased Steel	m.
504003P - REINFORCEMENT STEEL	LB	171 - Concrete-Encased Steel Stringer	m.
504003P - REINFORCEMENT STEEL	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
504003P - REINFORCEMENT STEEL	LB	173 - Arch - Concrete Encased Steel	m.
504003P - REINFORCEMENT STEEL	LB	174 - Floor Beam - Concrete Encased Steel	m.
504003P - REINFORCEMENT STEEL	LB	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	204 - P/S Conc Column or Pile Extension	ea.
504003P - REINFORCEMENT STEEL	LB	205 - Reinforced Conc Column or Pile Extension	ea.
504003P - REINFORCEMENT STEEL	LB	210 - Reinforced Conc Pier Wall	m.
504003P - REINFORCEMENT STEEL	LB	215 - Reinforced Conc Abutment	m.
504003P - REINFORCEMENT STEEL	LB	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504003P - REINFORCEMENT STEEL	LB	226 - P/S Conc Submerged Pile	ea.
504003P - REINFORCEMENT STEEL	LB	227 - Reinforced Conc Submerged Pile	ea.
504003P - REINFORCEMENT STEEL	LB	233 - P/S Conc Cap	m.
504003P - REINFORCEMENT STEEL	LB	234 - Reinforced Conc Cap	m.
504003P - REINFORCEMENT STEEL	LB	241 - Reinforced Concrete Culvert	m.
504003P - REINFORCEMENT STEEL	LB	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
504003P - REINFORCEMENT STEEL	LB	270 - Conc Encased Steel Column or Pile Extension	ea.
504003P - REINFORCEMENT STEEL	LB	271 - Concrete Encased Steel Pier Cap	m.
504003P - REINFORCEMENT STEEL	LB	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
504003P - REINFORCEMENT STEEL	LB	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
504003P - REINFORCEMENT STEEL	LB	331 - Reinforced Conc Bridge Railing	m.
504003P - REINFORCEMENT STEEL	LB	359 - Soffit of Concrete Deck or Slab	ea.
504003P - REINFORCEMENT STEEL	LB	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
504003P - REINFORCEMENT STEEL	LB	38 - Concrete Slab - Bare	sq.m.
504003P - REINFORCEMENT STEEL	LB	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
504003P - REINFORCEMENT STEEL	LB	503 - Curbs/Sidewalks - Concrete	m.
504003P - REINFORCEMENT STEEL	LB	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504003P - REINFORCEMENT STEEL	LB	507 - Headwalls - Other - Concrete/Masonry	m.
504003P - REINFORCEMENT STEEL	LB	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504003P - REINFORCEMENT STEEL	LB	509 - Headwalls - Culvert - Concrete/Masonry	m.
504003P - REINFORCEMENT STEEL	LB	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
504003P - REINFORCEMENT STEEL	LB	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
504003P - REINFORCEMENT STEEL	LB	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	104 - P/S Conc Closed Web/Box Girder	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	105 - Reinforced Concrete Closed Webs/Box Girder	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	109 - P/S Conc Open Girder/Beam	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	110 - Reinforced Conc Open Girder/Beam	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	115 - P/S Conc Stringer	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	116 - Reinforced Conc Stringer	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	144 - Reinforced Conc Arch	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	155 - Reinforced Conc Floor Beam	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	170 - Open Girder - Concrete Encased Steel	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	171 - Concrete-Encased Steel Stringer	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	173 - Arch - Concrete Encased Steel	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	174 - Floor Beam - Concrete Encased Steel	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	204 - P/S Conc Column or Pile Extension	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	205 - Reinforced Conc Column or Pile Extension	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	210 - Reinforced Conc Pier Wall	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	215 - Reinforced Conc Abutment	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	226 - P/S Conc Submerged Pile	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	227 - Reinforced Conc Submerged Pile	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	233 - P/S Conc Cap	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	234 - Reinforced Conc Cap	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	241 - Reinforced Concrete Culvert	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	270 - Conc Encased Steel Column or Pile Extension	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	271 - Concrete Encased Steel Pier Cap	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	331 - Reinforced Conc Bridge Railing	m.

Line Item	Line Item Unit	Element	Element Unit
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	359 - Soffit of Concrete Deck or Slab	ea.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	507 - Headwalls - Other - Concrete/Masonry	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	509 - Headwalls - Culvert - Concrete/Masonry	m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
504006P - REINFORCEMENT STEEL, EPOXY-COATED	LB	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	104 - P/S Conc Closed Web/Box Girder	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	105 - Reinforced Concrete Closed Webs/Box Girder	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	109 - P/S Conc Open Girder/Beam	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	110 - Reinforced Conc Open Girder/Beam	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	115 - P/S Conc Stringer	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	116 - Reinforced Conc Stringer	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	144 - Reinforced Conc Arch	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	155 - Reinforced Conc Floor Beam	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	170 - Open Girder - Concrete Encased Steel	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	171 - Concrete-Encased Steel Stringer	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	173 - Arch - Concrete Encased Steel	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	174 - Floor Beam - Concrete Encased Steel	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	204 - P/S Conc Column or Pile Extension	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	205 - Reinforced Conc Column or Pile Extension	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	210 - Reinforced Conc Pier Wall	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	215 - Reinforced Conc Abutment	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	226 - P/S Conc Submerged Pile	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	227 - Reinforced Conc Submerged Pile	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	233 - P/S Conc Cap	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	234 - Reinforced Conc Cap	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	241 - Reinforced Concrete Culvert	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	270 - Conc Encased Steel Column or Pile Extension	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	271 - Concrete Encased Steel Pier Cap	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	331 - Reinforced Conc Bridge Railing	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	359 - Soffit of Concrete Deck or Slab	ea.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	503 - Curbs/Sidewalks - Concrete	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	507 - Headwalls - Other - Concrete/Masonry	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504008P - REINFORCEMENT STEEL, STAINLESS STEEL	LB	509 - Headwalls - Culvert - Concrete/Masonry	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	104 - P/S Conc Closed Web/Box Girder	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	105 - Reinforced Concrete Closed Webs/Box Girder	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	109 - P/S Conc Open Girder/Beam	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	110 - Reinforced Conc Open Girder/Beam	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	115 - P/S Conc Stringer	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	116 - Reinforced Conc Stringer	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	144 - Reinforced Conc Arch	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	155 - Reinforced Conc Floor Beam	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	170 - Open Girder - Concrete Encased Steel	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	171 - Concrete-Encased Steel Stringer	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	173 - Arch - Concrete Encased Steel	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	174 - Floor Beam - Concrete Encased Steel	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	204 - P/S Conc Column or Pile Extension	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	205 - Reinforced Conc Column or Pile Extension	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	210 - Reinforced Conc Pier Wall	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	215 - Reinforced Conc Abutment	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	226 - P/S Conc Submerged Pile	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	227 - Reinforced Conc Submerged Pile	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	233 - P/S Conc Cap	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	234 - Reinforced Conc Cap	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	241 - Reinforced Concrete Culvert	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	270 - Conc Encased Steel Column or Pile Extension	ea.

Line Item	Line Item Unit	Element	Element Unit
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	271 - Concrete Encased Steel Pier Cap	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	331 - Reinforced Conc Bridge Railing	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	359 - Soffit of Concrete Deck or Slab	ea.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	503 - Curbs/Sidewalks - Concrete	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	507 - Headwalls - Other - Concrete/Masonry	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504009P - REINFORCEMENT STEEL, GALVANIZED	LB	509 - Headwalls - Culvert - Concrete/Masonry	m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	12 - Concrete Deck - Bare	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	205 - Reinforced Conc Column or Pile Extension	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	227 - Reinforced Conc Submerged Pile	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	359 - Soffit of Concrete Deck or Slab	ea.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	38 - Concrete Slab - Bare	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
504010P - DRILL AND GROUT REINFORCEMENT STEEL	LF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
504012P - CONCRETE CULVERT, STRUCTURES	CY	241 - Reinforced Concrete Culvert	m.
504012P - CONCRETE CULVERT, STRUCTURES	CY	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504012P - CONCRETE CULVERT, STRUCTURES	CY	509 - Headwalls - Culvert - Concrete/Masonry	m.
504015P - CONCRETE FOOTING	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504018P - CONCRETE WING WALL	CY	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504018P - CONCRETE WING WALL	CY	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504024P - CONCRETE ABUTMENT WALL	CY	215 - Reinforced Conc Abutment	m.
504025P - MODIFICATION OF EXISTING ABUTMENTS	CY	215 - Reinforced Conc Abutment	m.
504025P - MODIFICATION OF EXISTING ABUTMENTS	CY	216 - Timber Abutment	m.
504025P - MODIFICATION OF EXISTING ABUTMENTS	CY	217 - Other Material Abutment	m.
504026P - CONCRETE PIER COLUMN AND CAP, HPC	CY	205 - Reinforced Conc Column or Pile Extension	ea.
504026P - CONCRETE PIER COLUMN AND CAP, HPC	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504026P - CONCRETE PIER COLUMN AND CAP, HPC	CY	234 - Reinforced Conc Cap	m.
504026P - CONCRETE PIER COLUMN AND CAP, HPC	CY	271 - Concrete Encased Steel Pier Cap	m.
504027P - CONCRETE PIER COLUMN AND CAP	CY	205 - Reinforced Conc Column or Pile Extension	ea.
504027P - CONCRETE PIER COLUMN AND CAP	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504027P - CONCRETE PIER COLUMN AND CAP	CY	234 - Reinforced Conc Cap	m.
504027P - CONCRETE PIER COLUMN AND CAP	CY	271 - Concrete Encased Steel Pier Cap	m.
504028P - PIER CAP RECONSTRUCTION	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504028P - PIER CAP RECONSTRUCTION	CY	234 - Reinforced Conc Cap	m.
504028P - PIER CAP RECONSTRUCTION	CY	271 - Concrete Encased Steel Pier Cap	m.
504029P - CONCRETE SEAL	CY	204 - P/S Conc Column or Pile Extension	ea.
504029P - CONCRETE SEAL	CY	205 - Reinforced Conc Column or Pile Extension	ea.
504029P - CONCRETE SEAL	CY	210 - Reinforced Conc Pier Wall	m.
504029P - CONCRETE SEAL	CY	215 - Reinforced Conc Abutment	m.
504029P - CONCRETE SEAL	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504029P - CONCRETE SEAL	CY	226 - P/S Conc Submerged Pile	ea.
504029P - CONCRETE SEAL	CY	227 - Reinforced Conc Submerged Pile	ea.
504029P - CONCRETE SEAL	CY	233 - P/S Conc Cap	m.
504029P - CONCRETE SEAL	CY	234 - Reinforced Conc Cap	m.
504029P - CONCRETE SEAL	CY	241 - Reinforced Concrete Culvert	m.
504029P - CONCRETE SEAL	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
504029P - CONCRETE SEAL	CY	271 - Concrete Encased Steel Pier Cap	m.
504030P - CONCRETE PIER SHAFT	CY	205 - Reinforced Conc Column or Pile Extension	ea.
504030P - CONCRETE PIER SHAFT	CY	227 - Reinforced Conc Submerged Pile	ea.
504030P - CONCRETE PIER SHAFT	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
504031P - MODIFICATION OF EXISTING PIERS	CY	210 - Reinforced Conc Pier Wall	m.

Line Item	Line Item Unit	Element	Element Unit
504031P - MODIFICATION OF EXISTING PIERS	CY	211 - Other Material Pier Wall	m.
504031P - MODIFICATION OF EXISTING PIERS	CY	271 - Concrete Encased Steel Pier Cap	m.
504036P - EPOXY WATERPROOFING	SY	204 - P/S Conc Column or Pile Extension	ea.
504036P - EPOXY WATERPROOFING	SY	205 - Reinforced Conc Column or Pile Extension	ea.
504036P - EPOXY WATERPROOFING	SY	210 - Reinforced Conc Pier Wall	m.
504036P - EPOXY WATERPROOFING	SY	215 - Reinforced Conc Abutment	m.
504036P - EPOXY WATERPROOFING	SY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504036P - EPOXY WATERPROOFING	SY	226 - P/S Conc Submerged Pile	ea.
504036P - EPOXY WATERPROOFING	SY	227 - Reinforced Conc Submerged Pile	ea.
504036P - EPOXY WATERPROOFING	SY	233 - P/S Conc Cap	m.
504036P - EPOXY WATERPROOFING	SY	234 - Reinforced Conc Cap	m.
504036P - EPOXY WATERPROOFING	SY	241 - Reinforced Concrete Culvert	m.
504036P - EPOXY WATERPROOFING	SY	270 - Conc Encased Steel Column or Pile Extension	ea.
504036P - EPOXY WATERPROOFING	SY	271 - Concrete Encased Steel Pier Cap	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	204 - P/S Conc Column or Pile Extension	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	205 - Reinforced Conc Column or Pile Extension	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	210 - Reinforced Conc Pier Wall	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	215 - Reinforced Conc Abutment	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	226 - P/S Conc Submerged Pile	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	227 - Reinforced Conc Submerged Pile	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	233 - P/S Conc Cap	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	234 - Reinforced Conc Cap	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	241 - Reinforced Concrete Culvert	m.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
504037P - SPRAY APPLIED WATERPROOFING MEMBRANE	SF	271 - Concrete Encased Steel Pier Cap	m.
504038P - MEMBRANE WATERPROOFING	SY	204 - P/S Conc Column or Pile Extension	ea.
504038P - MEMBRANE WATERPROOFING	SY	205 - Reinforced Conc Column or Pile Extension	ea.
504038P - MEMBRANE WATERPROOFING	SY	210 - Reinforced Conc Pier Wall	m.
504038P - MEMBRANE WATERPROOFING	SY	215 - Reinforced Conc Abutment	m.
504038P - MEMBRANE WATERPROOFING	SY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504038P - MEMBRANE WATERPROOFING	SY	226 - P/S Conc Submerged Pile	ea.
504038P - MEMBRANE WATERPROOFING	SY	227 - Reinforced Conc Submerged Pile	ea.
504038P - MEMBRANE WATERPROOFING	SY	233 - P/S Conc Cap	m.
504038P - MEMBRANE WATERPROOFING	SY	234 - Reinforced Conc Cap	m.
504038P - MEMBRANE WATERPROOFING	SY	241 - Reinforced Concrete Culvert	m.
504038P - MEMBRANE WATERPROOFING	SY	270 - Conc Encased Steel Column or Pile Extension	ea.
504038P - MEMBRANE WATERPROOFING	SY	271 - Concrete Encased Steel Pier Cap	m.
504040P - CONCRETE SURFACE TREATMENT	SY	204 - P/S Conc Column or Pile Extension	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	205 - Reinforced Conc Column or Pile Extension	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	210 - Reinforced Conc Pier Wall	m.
504040P - CONCRETE SURFACE TREATMENT	SY	215 - Reinforced Conc Abutment	m.
504040P - CONCRETE SURFACE TREATMENT	SY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	226 - P/S Conc Submerged Pile	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	227 - Reinforced Conc Submerged Pile	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	233 - P/S Conc Cap	m.
504040P - CONCRETE SURFACE TREATMENT	SY	234 - Reinforced Conc Cap	m.
504040P - CONCRETE SURFACE TREATMENT	SY	241 - Reinforced Concrete Culvert	m.
504040P - CONCRETE SURFACE TREATMENT	SY	270 - Conc Encased Steel Column or Pile Extension	ea.
504040P - CONCRETE SURFACE TREATMENT	SY	271 - Concrete Encased Steel Pier Cap	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	204 - P/S Conc Column or Pile Extension	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	205 - Reinforced Conc Column or Pile Extension	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	210 - Reinforced Conc Pier Wall	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	215 - Reinforced Conc Abutment	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	226 - P/S Conc Submerged Pile	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	227 - Reinforced Conc Submerged Pile	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	233 - P/S Conc Cap	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	234 - Reinforced Conc Cap	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	241 - Reinforced Concrete Culvert	m.
504046P - PAINTING OF CONCRETE SURFACE	SY	270 - Conc Encased Steel Column or Pile Extension	ea.
504046P - PAINTING OF CONCRETE SURFACE	SY	271 - Concrete Encased Steel Pier Cap	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	204 - P/S Conc Column or Pile Extension	ea.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	205 - Reinforced Conc Column or Pile Extension	ea.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	210 - Reinforced Conc Pier Wall	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	215 - Reinforced Conc Abutment	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	226 - P/S Conc Submerged Pile	ea.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	227 - Reinforced Conc Submerged Pile	ea.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	233 - P/S Conc Cap	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	234 - Reinforced Conc Cap	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	241 - Reinforced Concrete Culvert	m.
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	270 - Conc Encased Steel Column or Pile Extension	ea.

Line Item	Line Item Unit	Element	Element Unit
504047P - CONCRETE STAIN AND ANTI-GRAFFITI TREATME	SF	271 - Concrete Encased Steel Pier Cap	m.
504055P - CONCRETE BEAM	CY	105 - Reinforced Concrete Closed Webs/Box Girder	m.
504055P - CONCRETE BEAM	CY	110 - Reinforced Conc Open Girder/Beam	m.
504055P - CONCRETE BEAM	CY	116 - Reinforced Conc Stringer	m.
504055P - CONCRETE BEAM	CY	155 - Reinforced Conc Floor Beam	m.
504055P - CONCRETE BEAM	CY	171 - Concrete-Encased Steel Stringer	m.
504055P - CONCRETE BEAM	CY	174 - Floor Beam - Concrete Encased Steel	m.
504064P - STONE VENEER	SY	201 - Unpainted Steel Column or Pile Extension	ea.
504064P - STONE VENEER	SY	202 - Painted Steel Column or Pile Extension	ea.
504064P - STONE VENEER	SY	204 - P/S Conc Column or Pile Extension	ea.
504064P - STONE VENEER	SY	205 - Reinforced Conc Column or Pile Extension	ea.
504064P - STONE VENEER	SY	210 - Reinforced Conc Pier Wall	m.
504064P - STONE VENEER	SY	211 - Other Material Pier Wall	m.
504064P - STONE VENEER	SY	215 - Reinforced Conc Abutment	m.
504064P - STONE VENEER	SY	241 - Reinforced Concrete Culvert	m.
504064P - STONE VENEER	SY	270 - Conc Encased Steel Column or Pile Extension	ea.
504064P - STONE VENEER	SY	331 - Reinforced Conc Bridge Railing	m.
504064P - STONE VENEER	SY	333 - Other Bridge Railing	m.
504064P - STONE VENEER	SY	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504064P - STONE VENEER	SY	507 - Headwalls - Other - Concrete/Masonry	m.
504064P - STONE VENEER	SY	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504064P - STONE VENEER	SY	509 - Headwalls - Culvert - Concrete/Masonry	m.
504065P - BRICK VENEER	SF	201 - Unpainted Steel Column or Pile Extension	ea.
504065P - BRICK VENEER	SF	202 - Painted Steel Column or Pile Extension	ea.
504065P - BRICK VENEER	SF	204 - P/S Conc Column or Pile Extension	ea.
504065P - BRICK VENEER	SF	205 - Reinforced Conc Column or Pile Extension	ea.
504065P - BRICK VENEER	SF	210 - Reinforced Conc Pier Wall	m.
504065P - BRICK VENEER	SF	211 - Other Material Pier Wall	m.
504065P - BRICK VENEER	SF	215 - Reinforced Conc Abutment	m.
504065P - BRICK VENEER	SF	241 - Reinforced Concrete Culvert	m.
504065P - BRICK VENEER	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
504065P - BRICK VENEER	SF	331 - Reinforced Conc Bridge Railing	m.
504065P - BRICK VENEER	SF	333 - Other Bridge Railing	m.
504065P - BRICK VENEER	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504065P - BRICK VENEER	SF	507 - Headwalls - Other - Concrete/Masonry	m.
504065P - BRICK VENEER	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504065P - BRICK VENEER	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
504067P - FORMLINER	SF	205 - Reinforced Conc Column or Pile Extension	ea.
504067P - FORMLINER	SF	210 - Reinforced Conc Pier Wall	m.
504067P - FORMLINER	SF	215 - Reinforced Conc Abutment	m.
504067P - FORMLINER	SF	241 - Reinforced Concrete Culvert	m.
504067P - FORMLINER	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
504067P - FORMLINER	SF	331 - Reinforced Conc Bridge Railing	m.
504067P - FORMLINER	SF	333 - Other Bridge Railing	m.
504067P - FORMLINER	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
504067P - FORMLINER	SF	507 - Headwalls - Other - Concrete/Masonry	m.
504067P - FORMLINER	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
504067P - FORMLINER	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
504075P - ARCHITECTURAL CAST STONE	LF	201 - Unpainted Steel Column or Pile Extension	ea.
504075P - ARCHITECTURAL CAST STONE	LF	202 - Painted Steel Column or Pile Extension	ea.
504075P - ARCHITECTURAL CAST STONE	LF	204 - P/S Conc Column or Pile Extension	ea.
504075P - ARCHITECTURAL CAST STONE	LF	205 - Reinforced Conc Column or Pile Extension	ea.
504075P - ARCHITECTURAL CAST STONE	LF	270 - Conc Encased Steel Column or Pile Extension	ea.
505058P - PRECAST CONCRETE CULVERT MODIFICATIONS	LS	241 - Reinforced Concrete Culvert	m.
505058P - PRECAST CONCRETE CULVERT MODIFICATIONS	LS	243 - Other Culvert	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	201 - Unpainted Steel Column or Pile Extension	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	202 - Painted Steel Column or Pile Extension	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	204 - P/S Conc Column or Pile Extension	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	206 - Timber Column or Pile Extension	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	211 - Other Material Pier Wall	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	215 - Reinforced Conc Abutment	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	217 - Other Material Abutment	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	225 - Unpainted Steel Submerged Pile	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	226 - P/S Conc Submerged Pile	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	227 - Reinforced Conc Submerged Pile	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	228 - Timber Submerged Pile	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	230 - Unpainted Steel Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	231 - Painted Steel Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	233 - P/S Conc Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	234 - Reinforced Conc Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	235 - Timber Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	240 - Unpainted Steel Culvert	m.

Line Item	Line Item Unit	Element	Element Unit
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	241 - Reinforced Concrete Culvert	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	243 - Other Culvert	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	271 - Concrete Encased Steel Pier Cap	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	507 - Headwalls - Other - Concrete/Masonry	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
505061P - PREFABRICATED SUBSTRUCTURE UNITS	CY	509 - Headwalls - Culvert - Concrete/Masonry	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	101 - Unpainted Steel Closed Web/Box Girder	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	102 - Painted Steel Closed Web/Box Girder	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	104 - P/S Conc Closed Web/Box Girder	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	105 - Reinforced Concrete Closed Webs/Box Girder	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	106 - Unpainted Steel Open Girder/Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	107 - Painted Steel Open Girder/Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	109 - P/S Conc Open Girder/Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	110 - Reinforced Conc Open Girder/Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	111 - Timber Open Girder/Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	112 - Unpainted Steel Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	113 - Painted Steel Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	115 - P/S Conc Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	116 - Reinforced Conc Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	117 - Timber Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	120 - Unpainted Steel Bottom Chord Thru Truss	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	121 - Painted Steel Bottom Chord Thru Truss	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	130 - Unpainted Steel Deck Truss	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	131 - Painted Steel Deck Truss	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	135 - Timber Truss/Arch	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	140 - Unpainted Steel Arch	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	141 - Painted Steel Arch	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	144 - Reinforced Conc Arch	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	145 - Other Arch	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	147 - Cable - Coated (not embedded in concrete)	ea.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	151 - Unpainted Steel Floor Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	152 - Painted Steel Floor Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	155 - Reinforced Conc Floor Beam	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	170 - Open Girder - Concrete Encased Steel	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	171 - Concrete-Encased Steel Stringer	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	173 - Arch - Concrete Encased Steel	m.
505063P - PREFABRICATED SUPERSTRUCTURE UNITS	SF	174 - Floor Beam - Concrete Encased Steel	m.
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	104 - P/S Conc Closed Web/Box Girder	m.
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	109 - P/S Conc Open Girder/Beam	m.
505064P - PREFABRICATED PRESTRESSED CONCRET SUPERSTRUCTURE UNITS	SF	115 - P/S Conc Stringer	m.
505072P - GIRDER JACKING	LS	210 - Reinforced Conc Pier Wall	m.
505072P - GIRDER JACKING	LS	211 - Other Material Pier Wall	m.
505072P - GIRDER JACKING	LS	215 - Reinforced Conc Abutment	m.
505072P - GIRDER JACKING	LS	216 - Timber Abutment	m.
505072P - GIRDER JACKING	LS	217 - Other Material Abutment	m.
505072P - GIRDER JACKING	LS	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
505072P - GIRDER JACKING	LS	230 - Unpainted Steel Cap	m.
505072P - GIRDER JACKING	LS	231 - Painted Steel Cap	m.
505072P - GIRDER JACKING	LS	233 - P/S Conc Cap	m.
505072P - GIRDER JACKING	LS	234 - Reinforced Conc Cap	m.
505072P - GIRDER JACKING	LS	235 - Timber Cap	m.
505072P - GIRDER JACKING	LS	271 - Concrete Encased Steel Pier Cap	m.
505072P - GIRDER JACKING	LS	310 - Elastomeric Bearing	ea.
505072P - GIRDER JACKING	LS	311 - Moveable Bearing (roller, sliding, etc.)	ea.
505072P - GIRDER JACKING	LS	312 - Enclosed/Concealed Bearing	ea.
505072P - GIRDER JACKING	LS	313 - Fixed Bearing	ea.
505072P - GIRDER JACKING	LS	314 - Pot Bearing	ea.
505072P - GIRDER JACKING	LS	315 - Disk Bearing	ea.
505072P - GIRDER JACKING	LS	360 - Settlement	ea.
505072P - GIRDER JACKING	LS	363 - Section Loss	ea.
505072P - GIRDER JACKING	LS	370 - Elastomeric Bearing with Teflon	ea.
505072P - GIRDER JACKING	LS	372 - Sliding Plate Bearing - Expansion/Moveable	ea.
505072P - GIRDER JACKING	LS	373 - Bond Breaker Bearing - Expansion/Moveable	ea.
505072P - GIRDER JACKING	LS	374 - Rocker Bearing - Expansion/Moveable	ea.
505072P - GIRDER JACKING	LS	375 - Pinned Bearing - Fixed	ea.
505072P - GIRDER JACKING	LS	376 - Spherical Bearing	ea.

Line Item	Line Item Unit	Element	Element Unit
505072P - GIRDER JACKING	LS	520 - Isolation Bearing	ea.
505072P - GIRDER JACKING	LS	521 - Bearing - Other	ea.
505084P - PRECAST PIER	LS	211 - Other Material Pier Wall	m.
505084P - PRECAST PIER	LS	233 - P/S Conc Cap	m.
505084P - PRECAST PIER	LS	234 - Reinforced Conc Cap	m.
505084P - PRECAST PIER	LS	271 - Concrete Encased Steel Pier Cap	m.
505094P - PRECAST CONCRETE STRUCTURE	U	104 - P/S Conc Closed Web/Box Girder	m.
505094P - PRECAST CONCRETE STRUCTURE	U	105 - Reinforced Concrete Closed Webs/Box Girder	m.
505094P - PRECAST CONCRETE STRUCTURE	U	109 - P/S Conc Open Girder/Beam	m.
505094P - PRECAST CONCRETE STRUCTURE	U	110 - Reinforced Conc Open Girder/Beam	m.
505094P - PRECAST CONCRETE STRUCTURE	U	115 - P/S Conc Stringer	m.
505094P - PRECAST CONCRETE STRUCTURE	U	116 - Reinforced Conc Stringer	m.
505094P - PRECAST CONCRETE STRUCTURE	U	144 - Reinforced Conc Arch	m.
505094P - PRECAST CONCRETE STRUCTURE	U	155 - Reinforced Conc Floor Beam	m.
505094P - PRECAST CONCRETE STRUCTURE	U	170 - Open Girder - Concrete Encased Steel	m.
505094P - PRECAST CONCRETE STRUCTURE	U	171 - Concrete-Encased Steel Stringer	m.
505094P - PRECAST CONCRETE STRUCTURE	U	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
505094P - PRECAST CONCRETE STRUCTURE	U	173 - Arch - Concrete Encased Steel	m.
505094P - PRECAST CONCRETE STRUCTURE	U	174 - Floor Beam - Concrete Encased Steel	m.
505094P - PRECAST CONCRETE STRUCTURE	U	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
505094P - PRECAST CONCRETE STRUCTURE	U	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
505094P - PRECAST CONCRETE STRUCTURE	U	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
505094P - PRECAST CONCRETE STRUCTURE	U	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
506003P - STRUCTURAL STEEL	LS	101 - Unpainted Steel Closed Web/Box Girder	m.
506003P - STRUCTURAL STEEL	LS	102 - Painted Steel Closed Web/Box Girder	m.
506003P - STRUCTURAL STEEL	LS	106 - Unpainted Steel Open Girder/Beam	m.
506003P - STRUCTURAL STEEL	LS	107 - Painted Steel Open Girder/Beam	m.
506003P - STRUCTURAL STEEL	LS	112 - Unpainted Steel Stringer	m.
506003P - STRUCTURAL STEEL	LS	113 - Painted Steel Stringer	m.
506003P - STRUCTURAL STEEL	LS	120 - Unpainted Steel Bottom Chord Thru Truss	m.
506003P - STRUCTURAL STEEL	LS	121 - Painted Steel Bottom Chord Thru Truss	m.
506003P - STRUCTURAL STEEL	LS	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
506003P - STRUCTURAL STEEL	LS	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
506003P - STRUCTURAL STEEL	LS	130 - Unpainted Steel Deck Truss	m.
506003P - STRUCTURAL STEEL	LS	131 - Painted Steel Deck Truss	m.
506003P - STRUCTURAL STEEL	LS	140 - Unpainted Steel Arch	m.
506003P - STRUCTURAL STEEL	LS	141 - Painted Steel Arch	m.
506003P - STRUCTURAL STEEL	LS	151 - Unpainted Steel Floor Beam	m.
506003P - STRUCTURAL STEEL	LS	152 - Painted Steel Floor Beam	m.
506003P - STRUCTURAL STEEL	LS	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
506003P - STRUCTURAL STEEL	LS	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
506003P - STRUCTURAL STEEL	LS	170 - Open Girder - Concrete Encased Steel	m.
506003P - STRUCTURAL STEEL	LS	171 - Concrete-Encased Steel Stringer	m.
506003P - STRUCTURAL STEEL	LS	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
506003P - STRUCTURAL STEEL	LS	173 - Arch - Concrete Encased Steel	m.
506003P - STRUCTURAL STEEL	LS	174 - Floor Beam - Concrete Encased Steel	m.
506003P - STRUCTURAL STEEL	LS	201 - Unpainted Steel Column or Pile Extension	ea.
506003P - STRUCTURAL STEEL	LS	202 - Painted Steel Column or Pile Extension	ea.
506003P - STRUCTURAL STEEL	LS	217 - Other Material Abutment	m.
506003P - STRUCTURAL STEEL	LS	225 - Unpainted Steel Submerged Pile	ea.
506003P - STRUCTURAL STEEL	LS	230 - Unpainted Steel Cap	m.
506003P - STRUCTURAL STEEL	LS	231 - Painted Steel Cap	m.
506003P - STRUCTURAL STEEL	LS	28 - Steel Deck - Open Grid	sq.m.
506003P - STRUCTURAL STEEL	LS	29 - Steel Deck - Concrete Filled Grid	sq.m.
506003P - STRUCTURAL STEEL	LS	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
506004M - STRUCTURAL STEEL	LB	101 - Unpainted Steel Closed Web/Box Girder	m.
506004M - STRUCTURAL STEEL	LB	102 - Painted Steel Closed Web/Box Girder	m.
506004M - STRUCTURAL STEEL	LB	106 - Unpainted Steel Open Girder/Beam	m.
506004M - STRUCTURAL STEEL	LB	107 - Painted Steel Open Girder/Beam	m.
506004M - STRUCTURAL STEEL	LB	112 - Unpainted Steel Stringer	m.
506004M - STRUCTURAL STEEL	LB	113 - Painted Steel Stringer	m.
506004M - STRUCTURAL STEEL	LB	120 - Unpainted Steel Bottom Chord Thru Truss	m.
506004M - STRUCTURAL STEEL	LB	121 - Painted Steel Bottom Chord Thru Truss	m.
506004M - STRUCTURAL STEEL	LB	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
506004M - STRUCTURAL STEEL	LB	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
506004M - STRUCTURAL STEEL	LB	130 - Unpainted Steel Deck Truss	m.
506004M - STRUCTURAL STEEL	LB	131 - Painted Steel Deck Truss	m.
506004M - STRUCTURAL STEEL	LB	140 - Unpainted Steel Arch	m.
506004M - STRUCTURAL STEEL	LB	141 - Painted Steel Arch	m.
506004M - STRUCTURAL STEEL	LB	151 - Unpainted Steel Floor Beam	m.
506004M - STRUCTURAL STEEL	LB	152 - Painted Steel Floor Beam	m.
506004M - STRUCTURAL STEEL	LB	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
506004M - STRUCTURAL STEEL	LB	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.

Line Item	Line Item Unit	Element	Element Unit
506004M - STRUCTURAL STEEL	LB	170 - Open Girder - Concrete Encased Steel	m.
506004M - STRUCTURAL STEEL	LB	171 - Concrete-Encased Steel Stringer	m.
506004M - STRUCTURAL STEEL	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
506004M - STRUCTURAL STEEL	LB	173 - Arch - Concrete Encased Steel	m.
506004M - STRUCTURAL STEEL	LB	174 - Floor Beam - Concrete Encased Steel	m.
506004M - STRUCTURAL STEEL	LB	201 - Unpainted Steel Column or Pile Extension	ea.
506004M - STRUCTURAL STEEL	LB	202 - Painted Steel Column or Pile Extension	ea.
506004M - STRUCTURAL STEEL	LB	217 - Other Material Abutment	m.
506004M - STRUCTURAL STEEL	LB	225 - Unpainted Steel Submerged Pile	ea.
506004M - STRUCTURAL STEEL	LB	230 - Unpainted Steel Cap	m.
506004M - STRUCTURAL STEEL	LB	231 - Painted Steel Cap	m.
506004M - STRUCTURAL STEEL	LB	28 - Steel Deck - Open Grid	sq.m.
506004M - STRUCTURAL STEEL	LB	29 - Steel Deck - Concrete Filled Grid	sq.m.
506004M - STRUCTURAL STEEL	LB	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
506012P - SHEAR CONNECTOR	U	12 - Concrete Deck - Bare	sq.m.
506012P - SHEAR CONNECTOR	U	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
506012P - SHEAR CONNECTOR	U	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
506012P - SHEAR CONNECTOR	U	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
506012P - SHEAR CONNECTOR	U	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
506012P - SHEAR CONNECTOR	U	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
506012P - SHEAR CONNECTOR	U	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
506012P - SHEAR CONNECTOR	U	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
506012P - SHEAR CONNECTOR	U	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
506012P - SHEAR CONNECTOR	U	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
506012P - SHEAR CONNECTOR	U	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
506012P - SHEAR CONNECTOR	U	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
506012P - SHEAR CONNECTOR	U	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
506015P - SHEAR CONNECTOR, GALVANIZED	U	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
506016P - GIRDER JACKING	LS	210 - Reinforced Conc Pier Wall	m.
506016P - GIRDER JACKING	LS	211 - Other Material Pier Wall	m.
506016P - GIRDER JACKING	LS	215 - Reinforced Conc Abutment	m.
506016P - GIRDER JACKING	LS	216 - Timber Abutment	m.
506016P - GIRDER JACKING	LS	217 - Other Material Abutment	m.
506016P - GIRDER JACKING	LS	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
506016P - GIRDER JACKING	LS	230 - Unpainted Steel Cap	m.
506016P - GIRDER JACKING	LS	231 - Painted Steel Cap	m.
506016P - GIRDER JACKING	LS	233 - P/S Conc Cap	m.
506016P - GIRDER JACKING	LS	234 - Reinforced Conc Cap	m.
506016P - GIRDER JACKING	LS	235 - Timber Cap	m.
506016P - GIRDER JACKING	LS	271 - Concrete Encased Steel Pier Cap	m.
506016P - GIRDER JACKING	LS	310 - Elastomeric Bearing	ea.
506016P - GIRDER JACKING	LS	311 - Moveable Bearing (roller, sliding, etc.)	ea.
506016P - GIRDER JACKING	LS	312 - Enclosed/Concealed Bearing	ea.
506016P - GIRDER JACKING	LS	313 - Fixed Bearing	ea.
506016P - GIRDER JACKING	LS	314 - Pot Bearing	ea.
506016P - GIRDER JACKING	LS	315 - Disk Bearing	ea.
506016P - GIRDER JACKING	LS	360 - Settlement	ea.
506016P - GIRDER JACKING	LS	363 - Section Loss	ea.
506016P - GIRDER JACKING	LS	370 - Elastomeric Bearing with Teflon	ea.
506016P - GIRDER JACKING	LS	372 - Sliding Plate Bearing - Expansion/Moveable	ea.
506016P - GIRDER JACKING	LS	373 - Bond Breaker Bearing - Expansion/Moveable	ea.
506016P - GIRDER JACKING	LS	374 - Rocker Bearing - Expansion/Moveable	ea.
506016P - GIRDER JACKING	LS	375 - Pinned Bearing - Fixed	ea.
506016P - GIRDER JACKING	LS	376 - Spherical Bearing	ea.
506016P - GIRDER JACKING	LS	520 - Isolation Bearing	ea.
506016P - GIRDER JACKING	LS	521 - Bearing - Other	ea.
506040P - STEEL REPAIR, TYPE ___	U	101 - Unpainted Steel Closed Web/Box Girder	m.
506040P - STEEL REPAIR, TYPE ___	U	102 - Painted Steel Closed Web/Box Girder	m.
506040P - STEEL REPAIR, TYPE ___	U	106 - Unpainted Steel Open Girder/Beam	m.
506040P - STEEL REPAIR, TYPE ___	U	107 - Painted Steel Open Girder/Beam	m.
506040P - STEEL REPAIR, TYPE ___	U	112 - Unpainted Steel Stringer	m.
506040P - STEEL REPAIR, TYPE ___	U	113 - Painted Steel Stringer	m.
506040P - STEEL REPAIR, TYPE ___	U	120 - Unpainted Steel Bottom Chord Thru Truss	m.
506040P - STEEL REPAIR, TYPE ___	U	121 - Painted Steel Bottom Chord Thru Truss	m.
506040P - STEEL REPAIR, TYPE ___	U	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
506040P - STEEL REPAIR, TYPE ___	U	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
506040P - STEEL REPAIR, TYPE ___	U	130 - Unpainted Steel Deck Truss	m.
506040P - STEEL REPAIR, TYPE ___	U	131 - Painted Steel Deck Truss	m.
506040P - STEEL REPAIR, TYPE ___	U	140 - Unpainted Steel Arch	m.
506040P - STEEL REPAIR, TYPE ___	U	141 - Painted Steel Arch	m.
506040P - STEEL REPAIR, TYPE ___	U	151 - Unpainted Steel Floor Beam	m.
506040P - STEEL REPAIR, TYPE ___	U	152 - Painted Steel Floor Beam	m.
506040P - STEEL REPAIR, TYPE ___	U	170 - Open Girder - Concrete Encased Steel	m.

Line Item	Line Item Unit	Element	Element Unit
506040P - STEEL REPAIR, TYPE ___	U	171 - Concrete-Encased Steel Stringer	m.
506040P - STEEL REPAIR, TYPE ___	U	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
506040P - STEEL REPAIR, TYPE ___	U	173 - Arch - Concrete Encased Steel	m.
506040P - STEEL REPAIR, TYPE ___	U	174 - Floor Beam - Concrete Encased Steel	m.
506040P - STEEL REPAIR, TYPE ___	U	217 - Other Material Abutment	m.
506040P - STEEL REPAIR, TYPE ___	U	230 - Unpainted Steel Cap	m.
506040P - STEEL REPAIR, TYPE ___	U	231 - Painted Steel Cap	m.
506040P - STEEL REPAIR, TYPE ___	U	28 - Steel Deck - Open Grid	sq.m.
506040P - STEEL REPAIR, TYPE ___	U	29 - Steel Deck - Concrete Filled Grid	sq.m.
506040P - STEEL REPAIR, TYPE ___	U	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	101 - Unpainted Steel Closed Web/Box Girder	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	102 - Painted Steel Closed Web/Box Girder	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	106 - Unpainted Steel Open Girder/Beam	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	107 - Painted Steel Open Girder/Beam	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	112 - Unpainted Steel Stringer	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	113 - Painted Steel Stringer	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	120 - Unpainted Steel Bottom Chord Thru Truss	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	121 - Painted Steel Bottom Chord Thru Truss	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	130 - Unpainted Steel Deck Truss	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	131 - Painted Steel Deck Truss	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	140 - Unpainted Steel Arch	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	141 - Painted Steel Arch	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	151 - Unpainted Steel Floor Beam	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	152 - Painted Steel Floor Beam	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	170 - Open Girder - Concrete Encased Steel	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	171 - Concrete-Encased Steel Stringer	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	173 - Arch - Concrete Encased Steel	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	174 - Floor Beam - Concrete Encased Steel	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	201 - Unpainted Steel Column or Pile Extension	ea.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	202 - Painted Steel Column or Pile Extension	ea.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	217 - Other Material Abutment	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	225 - Unpainted Steel Submerged Pile	ea.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	230 - Unpainted Steel Cap	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	231 - Painted Steel Cap	m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	28 - Steel Deck - Open Grid	sq.m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	29 - Steel Deck - Concrete Filled Grid	sq.m.
506041P - STRUCTURAL STEEL REPAIR, TYPE 1	LB	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	101 - Unpainted Steel Closed Web/Box Girder	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	102 - Painted Steel Closed Web/Box Girder	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	106 - Unpainted Steel Open Girder/Beam	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	107 - Painted Steel Open Girder/Beam	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	112 - Unpainted Steel Stringer	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	113 - Painted Steel Stringer	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	120 - Unpainted Steel Bottom Chord Thru Truss	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	121 - Painted Steel Bottom Chord Thru Truss	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	125 - Unpainted Steel Thru Truss (excl. bottom chord)	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	130 - Unpainted Steel Deck Truss	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	131 - Painted Steel Deck Truss	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	140 - Unpainted Steel Arch	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	141 - Painted Steel Arch	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	151 - Unpainted Steel Floor Beam	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	152 - Painted Steel Floor Beam	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	170 - Open Girder - Concrete Encased Steel	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	171 - Concrete-Encased Steel Stringer	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	173 - Arch - Concrete Encased Steel	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	174 - Floor Beam - Concrete Encased Steel	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	201 - Unpainted Steel Column or Pile Extension	ea.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	202 - Painted Steel Column or Pile Extension	ea.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	217 - Other Material Abutment	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	225 - Unpainted Steel Submerged Pile	ea.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	230 - Unpainted Steel Cap	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	231 - Painted Steel Cap	m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	28 - Steel Deck - Open Grid	sq.m.
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	29 - Steel Deck - Concrete Filled Grid	sq.m.

Line Item	Line Item Unit	Element	Element Unit
506042P - STRUCTURAL STEEL REPAIR, TYPE 2	LB	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	12 - Concrete Deck - Bare	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
507021P - CONCRETE BRIDGE DECK	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
507022P - CONCRETE BRIDGE SEATS, HES	CY	215 - Reinforced Conc Abutment	m.
507022P - CONCRETE BRIDGE SEATS, HES	CY	234 - Reinforced Conc Cap	m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	12 - Concrete Deck - Bare	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	38 - Concrete Slab - Bare	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
507023P - CONCRETE BRIDGE APPROACH, HES	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	12 - Concrete Deck - Bare	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
507024P - CONCRETE BRIDGE DECK, HPC	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	12 - Concrete Deck - Bare	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
507025P - CONCRETE BRIDGE DECK, HES	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
507028M - ENCASEMENT CONCRETE	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
507028M - ENCASEMENT CONCRETE	CY	271 - Concrete Encased Steel Pier Cap	m.
507030P - CONCRETE BRIDGE SIDEWALK	CY	503 - Curbs/Sidewalks - Concrete	m.
507031P - CONCRETE CRACK SEAL, SAFETY WALK	SF	503 - Curbs/Sidewalks - Concrete	m.
507032P - CONCRETE BRIDGE SIDEWALK, HES	CY	503 - Curbs/Sidewalks - Concrete	m.
507033P - CONCRETE BRIDGE SIDEWALK, HPC	CY	503 - Curbs/Sidewalks - Concrete	m.
507050M - CONCRETE SLEEPER SLAB	CY	38 - Concrete Slab - Bare	sq.m.
507050M - CONCRETE SLEEPER SLAB	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507050M - CONCRETE SLEEPER SLAB	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507050M - CONCRETE SLEEPER SLAB	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507050M - CONCRETE SLEEPER SLAB	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507050M - CONCRETE SLEEPER SLAB	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.

Line Item	Line Item Unit	Element	Element Unit
507051P - CONCRETE BRIDGE APPROACH	CY	38 - Concrete Slab - Bare	sq.m.
507051P - CONCRETE BRIDGE APPROACH	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507051P - CONCRETE BRIDGE APPROACH	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507051P - CONCRETE BRIDGE APPROACH	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507051P - CONCRETE BRIDGE APPROACH	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507051P - CONCRETE BRIDGE APPROACH	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	38 - Concrete Slab - Bare	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507052M - CONCRETE MOMENT SLAB	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	38 - Concrete Slab - Bare	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507065P - CONCRETE CAST-IN-PLACE SLABS	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507070M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	12 - Concrete Deck - Bare	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	38 - Concrete Slab - Bare	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
507123P - CONCRETE BRIDGE DECK, UHPC	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
513008P - RETAINING SYSTEM	SF	360 - Settlement	ea.
513015P - LANDSCAPE RETAINING WALL	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
513015P - LANDSCAPE RETAINING WALL	SF	507 - Headwalls - Other - Concrete/Masonry	m.
513015P - LANDSCAPE RETAINING WALL	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
513015P - LANDSCAPE RETAINING WALL	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
551002M - CONCRETE DECK CRACK REPAIR	LF	358 - Deck Cracking	ea.
551022M - HEADER RECONSTRUCTION	CY	215 - Reinforced Conc Abutment	m.
551022M - HEADER RECONSTRUCTION	CY	217 - Other Material Abutment	m.
551022M - HEADER RECONSTRUCTION	CY	300 - Strip Seal Expansion Joint	m.
551022M - HEADER RECONSTRUCTION	CY	300 - Strip Seal Expansion Joint	m.
551022M - HEADER RECONSTRUCTION	CY	302 - Compression Joint Seal	m.
551022M - HEADER RECONSTRUCTION	CY	302 - Compression Joint Seal	m.
551022M - HEADER RECONSTRUCTION	CY	303 - Assembly Joint/Seal (modular)	m.
551022M - HEADER RECONSTRUCTION	CY	303 - Assembly Joint/Seal (modular)	m.
551022M - HEADER RECONSTRUCTION	CY	304 - Open Expansion Joint	m.
551022M - HEADER RECONSTRUCTION	CY	304 - Open Expansion Joint	m.
551022M - HEADER RECONSTRUCTION	CY	305 - Finger Dams--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	305 - Finger Dams--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	306 - Sliding Plates--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	306 - Sliding Plates--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	307 - Other--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	307 - Other--Assembly Joint/Seal	m.
551022M - HEADER RECONSTRUCTION	CY	507 - Headwalls - Other - Concrete/Masonry	m.
551022M - HEADER RECONSTRUCTION	CY	509 - Headwalls - Culvert - Concrete/Masonry	m.
551033M - CONCRETE BRIDGE SIDEWALK REPAIR	SF	503 - Curbs/Sidewalks - Concrete	m.
551070M - MISCELLANEOUS CONCRETE	CY	104 - P/S Conc Closed Web/Box Girder	m.
551070M - MISCELLANEOUS CONCRETE	CY	105 - Reinforced Concrete Closed Webs/Box Girder	m.
551070M - MISCELLANEOUS CONCRETE	CY	109 - P/S Conc Open Girder/Beam	m.
551070M - MISCELLANEOUS CONCRETE	CY	110 - Reinforced Conc Open Girder/Beam	m.
551070M - MISCELLANEOUS CONCRETE	CY	115 - P/S Conc Stringer	m.
551070M - MISCELLANEOUS CONCRETE	CY	116 - Reinforced Conc Stringer	m.

APPENDIX G - Unmatched Unit Detail

Line Item	Line Item Unit	Element	Element Unit
551070M - MISCELLANEOUS CONCRETE	CY	12 - Concrete Deck - Bare	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	144 - Reinforced Conc Arch	m.
551070M - MISCELLANEOUS CONCRETE	CY	145 - Other Arch	m.
551070M - MISCELLANEOUS CONCRETE	CY	155 - Reinforced Conc Floor Beam	m.
551070M - MISCELLANEOUS CONCRETE	CY	170 - Open Girder - Concrete Encased Steel	m.
551070M - MISCELLANEOUS CONCRETE	CY	171 - Concrete-Encased Steel Stringer	m.
551070M - MISCELLANEOUS CONCRETE	CY	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
551070M - MISCELLANEOUS CONCRETE	CY	173 - Arch - Concrete Encased Steel	m.
551070M - MISCELLANEOUS CONCRETE	CY	174 - Floor Beam - Concrete Encased Steel	m.
551070M - MISCELLANEOUS CONCRETE	CY	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	204 - P/S Conc Column or Pile Extension	ea.
551070M - MISCELLANEOUS CONCRETE	CY	205 - Reinforced Conc Column or Pile Extension	ea.
551070M - MISCELLANEOUS CONCRETE	CY	210 - Reinforced Conc Pier Wall	m.
551070M - MISCELLANEOUS CONCRETE	CY	215 - Reinforced Conc Abutment	m.
551070M - MISCELLANEOUS CONCRETE	CY	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	233 - P/S Conc Cap	m.
551070M - MISCELLANEOUS CONCRETE	CY	234 - Reinforced Conc Cap	m.
551070M - MISCELLANEOUS CONCRETE	CY	241 - Reinforced Concrete Culvert	m.
551070M - MISCELLANEOUS CONCRETE	CY	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
551070M - MISCELLANEOUS CONCRETE	CY	271 - Concrete Encased Steel Pier Cap	m.
551070M - MISCELLANEOUS CONCRETE	CY	320 - P/S Concrete Approach Slab w/ or w-o/AC Ovly	ea.
551070M - MISCELLANEOUS CONCRETE	CY	321 - Reinforced Conc Approach Slab w/ or w/o AC Ovly	ea.
551070M - MISCELLANEOUS CONCRETE	CY	359 - Soffit of Concrete Deck or Slab	ea.
551070M - MISCELLANEOUS CONCRETE	CY	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	38 - Concrete Slab - Bare	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	503 - Curbs/Sidewalks - Concrete	m.
551070M - MISCELLANEOUS CONCRETE	CY	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
551070M - MISCELLANEOUS CONCRETE	CY	507 - Headwalls - Other - Concrete/Masonry	m.
551070M - MISCELLANEOUS CONCRETE	CY	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
551070M - MISCELLANEOUS CONCRETE	CY	509 - Headwalls - Culvert - Concrete/Masonry	m.
551070M - MISCELLANEOUS CONCRETE	CY	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
551070M - MISCELLANEOUS CONCRETE	CY	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
552003M - PRESSURE INJECTION, CONCRETE CRACKS	LF	358 - Deck Cracking	ea.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	102 - Painted Steel Closed Web/Box Girder	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	107 - Painted Steel Open Girder/Beam	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	113 - Painted Steel Stringer	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	121 - Painted Steel Bottom Chord Thru Truss	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	131 - Painted Steel Deck Truss	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	141 - Painted Steel Arch	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	152 - Painted Steel Floor Beam	m.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	202 - Painted Steel Column or Pile Extension	ea.
554009P - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	231 - Painted Steel Cap	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	102 - Painted Steel Closed Web/Box Girder	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	107 - Painted Steel Open Girder/Beam	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	113 - Painted Steel Stringer	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	121 - Painted Steel Bottom Chord Thru Truss	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	131 - Painted Steel Deck Truss	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	141 - Painted Steel Arch	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	152 - Painted Steel Floor Beam	m.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	202 - Painted Steel Column or Pile Extension	ea.
554010P - NEAR-WHITE BLAST CLEANING AND PAINTING,	LS	231 - Painted Steel Cap	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	170 - Open Girder - Concrete Encased Steel	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	171 - Concrete-Encased Steel Stringer	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	173 - Arch - Concrete Encased Steel	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	174 - Floor Beam - Concrete Encased Steel	m.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	270 - Conc Encased Steel Column or Pile Extension	ea.
554016P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	CY	271 - Concrete Encased Steel Pier Cap	m.

Line Item	Line Item Unit	Element	Element Unit
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	170 - Open Girder - Concrete Encased Steel	m.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	171 - Concrete-Encased Steel Stringer	m.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	173 - Arch - Concrete Encased Steel	m.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	174 - Floor Beam - Concrete Encased Steel	m.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
554019P - CONCRETE ENCASEMENT REMOVAL AND PAINTING	SF	271 - Concrete Encased Steel Pier Cap	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	204 - P/S Conc Column or Pile Extension	ea.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	205 - Reinforced Conc Column or Pile Extension	ea.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	210 - Reinforced Conc Pier Wall	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	215 - Reinforced Conc Abutment	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	234 - Reinforced Conc Cap	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	241 - Reinforced Concrete Culvert	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
555003M - SUBSTRUCTURE CONCRETE REPAIR	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
555006M - BRIDGE DECK WATERPROOF SURFACE COURSE	T	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
555008P - CULVERT REPAIR, TYPE ___	LS	240 - Unpainted Steel Culvert	m.
555008P - CULVERT REPAIR, TYPE ___	LS	241 - Reinforced Concrete Culvert	m.
555008P - CULVERT REPAIR, TYPE ___	LS	243 - Other Culvert	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	115 - P/S Conc Stringer	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	116 - Reinforced Conc Stringer	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	144 - Reinforced Conc Arch	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	155 - Reinforced Conc Floor Beam	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	171 - Concrete-Encased Steel Stringer	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	173 - Arch - Concrete Encased Steel	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	174 - Floor Beam - Concrete Encased Steel	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	204 - P/S Conc Column or Pile Extension	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	205 - Reinforced Conc Column or Pile Extension	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	210 - Reinforced Conc Pier Wall	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	215 - Reinforced Conc Abutment	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	226 - P/S Conc Submerged Pile	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	227 - Reinforced Conc Submerged Pile	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	233 - P/S Conc Cap	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	234 - Reinforced Conc Cap	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	241 - Reinforced Concrete Culvert	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	271 - Concrete Encased Steel Pier Cap	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	331 - Reinforced Conc Bridge Railing	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	503 - Curbs/Sidewalks - Concrete	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	507 - Headwalls - Other - Concrete/Masonry	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
555009M - CONCRETE SPALL REPAIR, TYPE 1	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	115 - P/S Conc Stringer	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	116 - Reinforced Conc Stringer	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	12 - Concrete Deck - Bare	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	144 - Reinforced Conc Arch	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	155 - Reinforced Conc Floor Beam	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	171 - Concrete-Encased Steel Stringer	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	173 - Arch - Concrete Encased Steel	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	174 - Floor Beam - Concrete Encased Steel	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	204 - P/S Conc Column or Pile Extension	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	205 - Reinforced Conc Column or Pile Extension	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	210 - Reinforced Conc Pier Wall	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	215 - Reinforced Conc Abutment	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	226 - P/S Conc Submerged Pile	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	227 - Reinforced Conc Submerged Pile	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	233 - P/S Conc Cap	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	234 - Reinforced Conc Cap	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	241 - Reinforced Concrete Culvert	m.

APPENDIX G - Unmatched Unit Detail

Line Item	Line Item Unit	Element	Element Unit
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	270 - Conc Encased Steel Column or Pile Extension	ea.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	271 - Concrete Encased Steel Pier Cap	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	331 - Reinforced Conc Bridge Railing	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	503 - Curbs/Sidewalks - Concrete	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	507 - Headwalls - Other - Concrete/Masonry	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	509 - Headwalls - Culvert - Concrete/Masonry	m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
555012M - CONCRETE SPALL REPAIR, TYPE 2	CF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
555013M - CONCRETE SPALL REPAIR	SF	115 - P/S Conc Stringer	m.
555013M - CONCRETE SPALL REPAIR	SF	116 - Reinforced Conc Stringer	m.
555013M - CONCRETE SPALL REPAIR	SF	144 - Reinforced Conc Arch	m.
555013M - CONCRETE SPALL REPAIR	SF	155 - Reinforced Conc Floor Beam	m.
555013M - CONCRETE SPALL REPAIR	SF	171 - Concrete-Encased Steel Stringer	m.
555013M - CONCRETE SPALL REPAIR	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
555013M - CONCRETE SPALL REPAIR	SF	173 - Arch - Concrete Encased Steel	m.
555013M - CONCRETE SPALL REPAIR	SF	174 - Floor Beam - Concrete Encased Steel	m.
555013M - CONCRETE SPALL REPAIR	SF	204 - P/S Conc Column or Pile Extension	ea.
555013M - CONCRETE SPALL REPAIR	SF	205 - Reinforced Conc Column or Pile Extension	ea.
555013M - CONCRETE SPALL REPAIR	SF	210 - Reinforced Conc Pier Wall	m.
555013M - CONCRETE SPALL REPAIR	SF	215 - Reinforced Conc Abutment	m.
555013M - CONCRETE SPALL REPAIR	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
555013M - CONCRETE SPALL REPAIR	SF	226 - P/S Conc Submerged Pile	ea.
555013M - CONCRETE SPALL REPAIR	SF	227 - Reinforced Conc Submerged Pile	ea.
555013M - CONCRETE SPALL REPAIR	SF	233 - P/S Conc Cap	m.
555013M - CONCRETE SPALL REPAIR	SF	234 - Reinforced Conc Cap	m.
555013M - CONCRETE SPALL REPAIR	SF	241 - Reinforced Concrete Culvert	m.
555013M - CONCRETE SPALL REPAIR	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
555013M - CONCRETE SPALL REPAIR	SF	271 - Concrete Encased Steel Pier Cap	m.
555013M - CONCRETE SPALL REPAIR	SF	331 - Reinforced Conc Bridge Railing	m.
555013M - CONCRETE SPALL REPAIR	SF	503 - Curbs/Sidewalks - Concrete	m.
555013M - CONCRETE SPALL REPAIR	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
555013M - CONCRETE SPALL REPAIR	SF	507 - Headwalls - Other - Concrete/Masonry	m.
555013M - CONCRETE SPALL REPAIR	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
555013M - CONCRETE SPALL REPAIR	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	115 - P/S Conc Stringer	m.
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	116 - Reinforced Conc Stringer	m.
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	155 - Reinforced Conc Floor Beam	m.
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	171 - Concrete-Encased Steel Stringer	m.
555015M - SUPERSTRUCTURE CONCRETE REPAIR	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	U	210 - Reinforced Conc Pier Wall	m.
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	U	215 - Reinforced Conc Abutment	m.
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	U	215 - Reinforced Conc Abutment	m.
555020M - SUBSTRUCTURE CONCRETE REPAIR, BEARING PEDESTAL	U	271 - Concrete Encased Steel Pier Cap	m.
555023P - PIER RECONSTRUCTION	L S	210 - Reinforced Conc Pier Wall	m.
555023P - PIER RECONSTRUCTION	L S	211 - Other Material Pier Wall	m.
555035M - MASONRY REPOINTING	SF	145 - Other Arch	m.
555035M - MASONRY REPOINTING	SF	217 - Other Material Abutment	m.
555035M - MASONRY REPOINTING	SF	243 - Other Culvert	m.
555035M - MASONRY REPOINTING	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
555035M - MASONRY REPOINTING	SF	507 - Headwalls - Other - Concrete/Masonry	m.
555035M - MASONRY REPOINTING	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
555035M - MASONRY REPOINTING	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	LS	330 - Metal Bridge Railing - Uncoated	m.
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	LS	331 - Reinforced Conc Bridge Railing	m.
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	LS	332 - Timber Bridge Railing	m.
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	LS	333 - Other Bridge Railing	m.
555040P - SUPPORT AND PROTECTION OF RIGID CONDUIT	LS	334 - Metal Bridge Railing - Coated	m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	151 - Unpainted Steel Floor Beam	m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	152 - Painted Steel Floor Beam	m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	160 - Unpainted Steel Pin and/or Pin and Hanger Assembly	ea.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	202 - Painted Steel Column or Pile Extension	ea.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	230 - Unpainted Steel Cap	m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	231 - Painted Steel Cap	m.

Line Item	Line Item Unit	Element	Element Unit
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	240 - Unpainted Steel Culvert	m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	28 - Steel Deck - Open Grid	sq.m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	29 - Steel Deck - Concrete Filled Grid	sq.m.
557004M - STRUCTURAL STEEL REPAIRS, WHERE DIRECTED	LB	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	204 - P/S Conc Column or Pile Extension	ea.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	205 - Reinforced Conc Column or Pile Extension	ea.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	210 - Reinforced Conc Pier Wall	m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	215 - Reinforced Conc Abutment	m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	234 - Reinforced Conc Cap	m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	241 - Reinforced Concrete Culvert	m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
559003P - SUBSTRUCTURE CONCRETE REPAIR	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	38 - Concrete Slab - Bare	sq.m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	39 - Concrete Slab - Unprotected w/ AC Overlay	sq.m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	40 - Concrete Slab - Protected w/ AC Overlay	sq.m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	44 - Concrete Slab - Protected w/ Thin Overlay	sq.m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	48 - Concrete Slab - Protected w/ Rigid Overlay	sq.m.
601410P - 8" CORRUGATED STEEL UNDERDRAIN PIPE	LF	52 - Concrete Slab - Protected w/ Coated Bars	sq.m.
602215M - CAPPING EXISTING DRAINAGE STRUCTURES	U	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
602215M - CAPPING EXISTING DRAINAGE STRUCTURES	U	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
603016P - STONE SLOPE PROTECTION	SY	505 - Slope Protection	ea.
607030P - 12" X 13" CONCRETE SLOPING CURB	LF	505 - Slope Protection	ea.
MMB002M - REPAIR CATEGORY "A", CONCRETE	HOUR	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
MMB002M - REPAIR CATEGORY "A", CONCRETE	HOUR	226 - P/S Conc Submerged Pile	ea.
MMB002M - REPAIR CATEGORY "A", CONCRETE	HOUR	227 - Reinforced Conc Submerged Pile	ea.
MMB004M - REPAIR CATEGORY "A", TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB005M - REPAIR CATEGORY "A" (WD), TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB006M - REPAIR CATEGORY "B", TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB007M - REPAIR CATEGORY "B" (WD), TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB008M - REPAIR CATEGORY "C", TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB009M - REPAIR CATEGORY "C" (WD), TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB010M - REPAIR CATEGORY "D", TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB011M - REPAIR CATEGORY "D" (WD), TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB020M - DIVING CREW, TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB021M - DIVING CREW (WD), TIMBER	HOUR	228 - Timber Submerged Pile	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	174 - Floor Beam - Concrete Encased Steel	m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	28 - Steel Deck - Open Grid	sq.m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	29 - Steel Deck - Concrete Filled Grid	sq.m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	30 - Steel Deck - Corrugated/Orthotropic/Etc.	sq.m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	303 - Assembly Joint/Seal (modular)	m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	305 - Finger Dams--Assembly Joint/Seal	m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	306 - Sliding Plates--Assembly Joint/Seal	m.
MMB067M - STRUCTURAL STEEL REPAIR	LB	311 - Moveable Bearing (roller, sliding, etc.)	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	312 - Enclosed/Concealed Bearing	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	313 - Fixed Bearing	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	314 - Pot Bearing	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	315 - Disk Bearing	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	372 - Sliding Plate Bearing - Expansion/Moveable	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	374 - Rocker Bearing - Expansion/Moveable	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	375 - Pinned Bearing - Fixed	ea.
MMB067M - STRUCTURAL STEEL REPAIR	LB	376 - Spherical Bearing	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	115 - P/S Conc Stringer	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	116 - Reinforced Conc Stringer	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	144 - Reinforced Conc Arch	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	155 - Reinforced Conc Floor Beam	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	171 - Concrete-Encased Steel Stringer	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	173 - Arch - Concrete Encased Steel	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	174 - Floor Beam - Concrete Encased Steel	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	204 - P/S Conc Column or Pile Extension	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	205 - Reinforced Conc Column or Pile Extension	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	210 - Reinforced Conc Pier Wall	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	215 - Reinforced Conc Abutment	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	226 - P/S Conc Submerged Pile	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	227 - Reinforced Conc Submerged Pile	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	233 - P/S Conc Cap	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	234 - Reinforced Conc Cap	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	241 - Reinforced Concrete Culvert	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	270 - Conc Encased Steel Column or Pile Extension	ea.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	271 - Concrete Encased Steel Pier Cap	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	331 - Reinforced Conc Bridge Railing	m.

Line Item	Line Item Unit	Element	Element Unit
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	503 - Curbs/Sidewalks - Concrete	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	507 - Headwalls - Other - Concrete/Masonry	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
MMB071M - CONCRETE SPALL REPAIR, TYPE 1	SF	509 - Headwalls - Culvert - Concrete/Masonry	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	115 - P/S Conc Stringer	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	116 - Reinforced Conc Stringer	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	12 - Concrete Deck - Bare	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	144 - Reinforced Conc Arch	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	155 - Reinforced Conc Floor Beam	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	171 - Concrete-Encased Steel Stringer	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	172 - Thru Truss - Bottom Chord - Conc Encased Steel	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	173 - Arch - Concrete Encased Steel	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	174 - Floor Beam - Concrete Encased Steel	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	204 - P/S Conc Column or Pile Extension	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	205 - Reinforced Conc Column or Pile Extension	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	210 - Reinforced Conc Pier Wall	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	215 - Reinforced Conc Abutment	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	220 - Reinforced Conc Submerged Pile Cap/Footing	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	226 - P/S Conc Submerged Pile	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	227 - Reinforced Conc Submerged Pile	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	233 - P/S Conc Cap	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	234 - Reinforced Conc Cap	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	241 - Reinforced Concrete Culvert	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	270 - Conc Encased Steel Column or Pile Extension	ea.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	271 - Concrete Encased Steel Pier Cap	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	331 - Reinforced Conc Bridge Railing	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	503 - Curbs/Sidewalks - Concrete	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	506 - Wingwalls - Abutment - Conc/Masonry/Timber	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	507 - Headwalls - Other - Concrete/Masonry	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	508 - Wingwalls - Culvert - Conc/Masonry/Timber	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	509 - Headwalls - Culvert - Concrete/Masonry	m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	70 - Conc Deck Prot w/ Mem, AC Ovly, C-I-P Coated Bars	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	71 - Conc Deck Prot w/ Mem, AC Ovly, Precast Coat Bars	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	73 - Conc Deck Prot w/ Thin Ovly, C-I-P Coated Bars	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	74 - Conc Deck Prot w/ Thin Ovly, Precast Coated Bars	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	76 - Conc Deck Prot w/ Coated Bars & C-I-P Rigid Ovly	sq.m.
MMB072M - CONCRETE SPALL REPAIR, TYPE 2	CF	77 - Conc Deck Prot w/ Coated Bars & Precast Rigid Ovly	sq.m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	107 - Painted Steel Open Girder/Beam	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	113 - Painted Steel Stringer	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	121 - Painted Steel Bottom Chord Thru Truss	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	126 - Painted Steel Thru Truss (excl. bottom chord)	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	141 - Painted Steel Arch	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	152 - Painted Steel Floor Beam	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	161 - Painted Steel Pin and/or Pin and Hanger Assembly	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	231 - Painted Steel Cap	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	300 - Strip Seal Expansion Joint	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	302 - Compression Joint Seal	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	303 - Assembly Joint/Seal (modular)	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	305 - Finger Dams--Assembly Joint/Seal	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	306 - Sliding Plates--Assembly Joint/Seal	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	307 - Other--Assembly Joint/Seal	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	311 - Moveable Bearing (roller, sliding, etc.)	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	314 - Pot Bearing	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	315 - Disk Bearing	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	334 - Metal Bridge Railing - Coated	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	372 - Sliding Plate Bearing - Expansion/Moveable	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	373 - Bond Breaker Bearing - Expansion/Moveable	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	374 - Rocker Bearing - Expansion/Moveable	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	375 - Pinned Bearing - Fixed	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	376 - Spherical Bearing	ea.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	502 - Curbs/Sidewalks - Painted Steel	m.
MMB084M - NEAR-WHITE BLAST CLEANING AND PAINTING	LS	520 - Isolation Bearing	ea.
MMB088M - BRIDGE DECK CRACK SEALING	LF	12 - Concrete Deck - Bare	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	13 - Concrete Deck - Unprotected w/ AC Overlay	sq.m.

Line Item	Line Item Unit	Element	Element Unit
MMB088M - BRIDGE DECK CRACK SEALING	LF	14 - Concrete Deck - Protected w/ AC Overlay	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	18 - Concrete Deck - Protected w/ Thin Overlay	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	22 - Concrete Deck - Protected w/ Rigid Overlay	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	26 - Concrete Deck - Protected w/ Coated Bars	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	27 - Concrete Deck - Protected w/ Cathodic System	sq.m.
MMB088M - BRIDGE DECK CRACK SEALING	LF	36 - Conc Deck prot w/coated bars & rigid overlay-PRECA	sq.m.

APPENDIX 1C
RISK BASED PRIORITIZATION

NJDOT Risk-Based Prioritization of Structurally Deficient Bridges

Intelligent Infrastructure Systems

Prepared For:
Rutgers University
New Jersey Department of Transportation

Guidance Report Submitted:
December 20, 2013
Resubmitted: February 19, 2014

1. Introduction and Background

A common challenge shared among bridge owners and managers is the widening of the gap between the amount of funds required for replacement and reparative maintenance of their bridge populations and the amount of funds available. This has led to an increased demand in selectively targeting bridges that justify an elevated level of resources. The conditions upon which these decisions are based can be founded upon a multitude of bridge metrics, including deterioration, traffic features, average daily traffic ADT, etc. Most DOTs use some combination of these metrics in the form of one or more analytical tools used to prioritize bridges based upon their need for attention. The underlying principles of these analytical tools range from straightforward and transparent (i.e. Sufficiency Ratings) to complex and somewhat opaque (i.e. AASHTO's Pontis). Although these tools are generally successful at what they seek to accomplish, the majority of them fail to explicitly account for the various risk factors that may ultimately cause a bridge to fail to perform adequately. To bridge this gap, the research reported herein aims to develop a prioritization tool that explicitly addresses the hazards, vulnerabilities and exposures associated with a wide range of limit states using only available data.

2. Scope of Project

The scope of this project was to develop a framework for risk-based prioritization of bridges for New Jersey Department of Transportation (NJDOT) and apply it to 100 structurally deficient bridges within New Jersey's bridge population. The 100 bridges represent 16 percent of total structurally deficient population within the NJ bridge inventory. This was accomplished through four project tasks:

2.1 Sample Selection

The bridges included in this study were selected from publicly available National Bridge Inventory (NBI) data. NBI data for New Jersey was first filtered to include only structurally deficient bridges. One hundred bridges were then randomly selected from this population for inclusion in this project.

2.2 Data Collection and Integration

This task involved first identifying desirable criteria to be used in the Risk-Based Prioritization (RBP) framework, and subsequently locating sources by which to obtain the data necessary to adequately and reliably characterize these criteria. The initial source of this list of criteria was "A Risk Based Process to Assign Priority for Bridge Replacement" (Moon et al., 2009). This paper proposes and describes a methodology for classifying large populations of bridges based upon the relative risk they pose across a number of different limit states.

The majority of the criteria presented in this paper can be characterized using data readily available in the NBI Database and NJDOT-supplied Inspection Reports. For several of the remaining criteria it was necessary to identify external data sources and references. Furthermore, some desirable criteria required data that was simply unavailable or required a level of research that was inappropriate for the scope of this project. After developing these criteria and locating sources, data was compiled in a Microsoft Excel Spreadsheet for each of the 100 selected bridges for use in Task 3.3.

2.3 Data Assessment and Prioritization

After collecting and integrating the data, a series of prioritization schemes were employed and each was assessed for its sensitivity and variability of results. The core of this scheme is described in Appendix 9.7. The prioritization schemes were evaluated by selecting 20 of the 100 total bridges, applying a series of schemes to this data, and identifying the most effective prioritization scheme of those applied. The success of each scheme was judged by how well the quantitative rankings agreed with engineering heuristics. Only then was the selected scheme applied to the remaining 80 bridges.

2.4 Reporting and Recommendations

Currently, all 100 bridges have been evaluated using the final framework that was developed and identified in Task 2.3, the details and results of which are presented, discussed, and summarized within this report.

3. Risk-Based Prioritization Concept

The core methodology for the RBP framework is described in (Moon et al., 2009). This paper describes a prioritization approach that is founded upon identifying relevant bridge performance limit states, and determining the risk associated with that limit state, as outlined in this section.

3.1 Bridge Performance Limit States

The approach taken classifies risk according to four performance limit states, listed below:

1. Safety: Structural
2. Safety: Geotechnical/Hydraulic
3. Serviceability and Durability
4. Operations

Structural safety is subdivided into Structural and Geotechnical/Hydraulic criteria, with Structural Safety encompassing conditions related to the superstructure and Geotechnical/Hydraulic covering conditions concerning substructure. The performance limit states extend beyond basic safety and serviceability to also include criteria regarding the operational functionality/safety of the bridge.

3.2 Risk-Based Approach

Establishing a global risk value for each bridge requires the combination of risk values for each identified limit state. Within each limit state there exist several failure modes which also have an associated risk value and each failure mode has associated hazards, vulnerabilities and exposures that go into determining the risk value for each failure mode. Hazards, vulnerabilities and exposures are the three main components of risk, herein referred to as risk components. A hierarchy of risk is shown in Figure 1 and the risk components are defined below.

Hazard – Generally defined as a threat to the structure. Probabilistically, it is defined as the likelihood of a hazard to occur.

Vulnerability – A preexisting condition of the structure by which a hazard is more or less likely to enable or mobilize failure. Probabilistically, it is defined as the likelihood of a failure to occur given the existence of a hazard.

Exposure – The consequences associated with a failure.

Uncertainty Premium – An adjustment factor to account for the inherent uncertainty of the evaluation methods used to generate the data that is used for prioritization. In the case of this project, an uncertainty premium of two is assigned for every structure evaluated. This value is consistent for this project because of the uniformity of the data sources available for each bridge. The RBP Microsoft Excel document includes the capability of changing the uncertainty premium, which would be essential if applying the methodology to a sample population with varying available data sources.

It is important to note that the definition of failure will differ with each limit state. For Structural and Geotechnical/Hydraulic Safety limit states, failure is defined as the partial or total collapse of one or more of the critical elements of the bridge's structural system. For the Serviceability and Durability limit state, failure is defined as the inability of a structure to perform adequately given its requisite functional requirements. For the Operational Limit State, failure is defined as the inability of the structure to adequately serve the demands of its users.

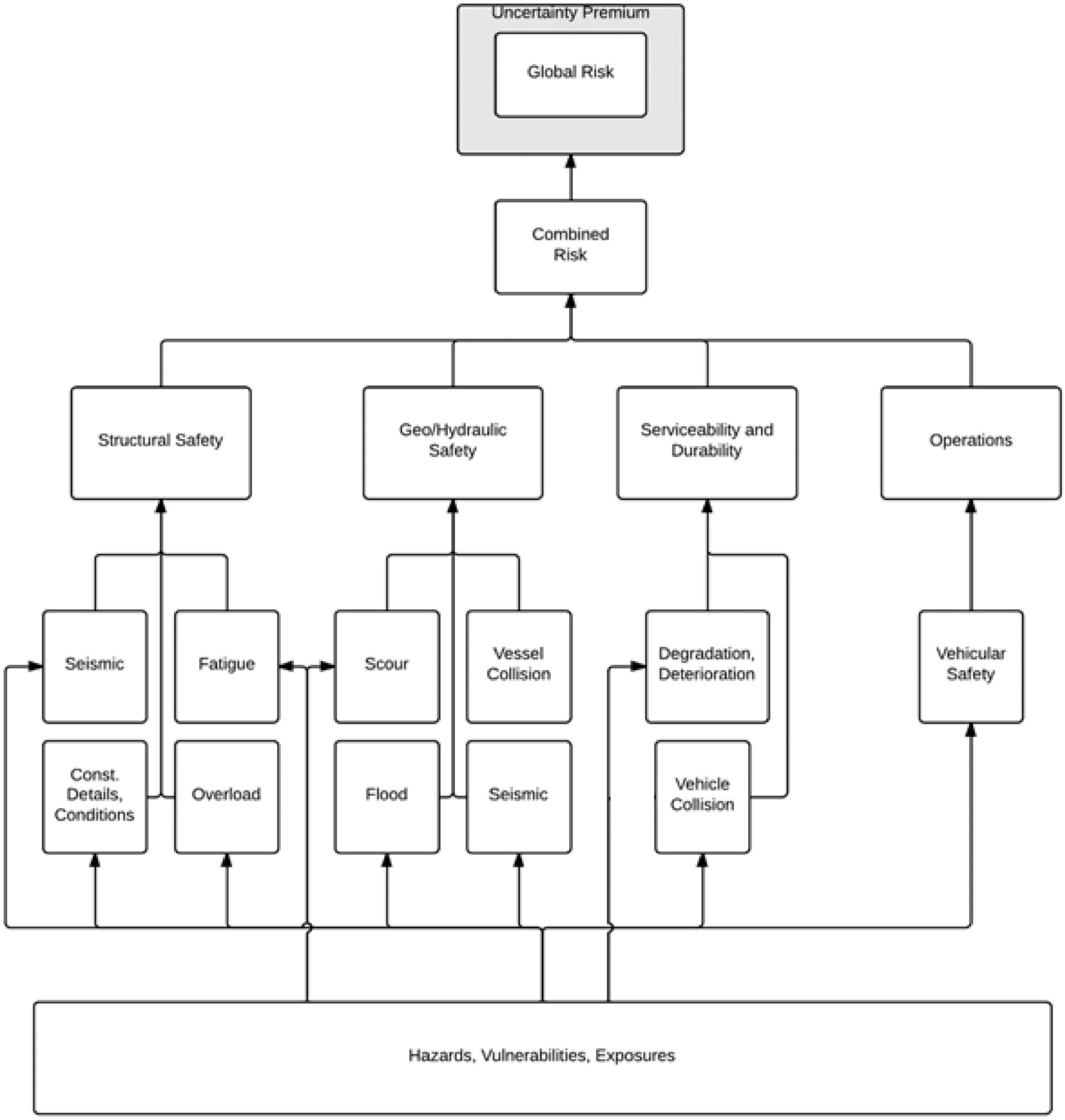


Figure 1: Risk Hierarchy

4. Risk-Based Prioritization Methodology

This final methodology developed through this project is represented by the flowchart in Figure 2. This flowchart graphically represents each step of the RBP methodology, starting with raw data sources and ending with a final normalized risk value and risk level. The relative line thickness represents the volume of data being processed. Solid lines represent data with multiple rows, columns or attributes. Dashed lines represent a single value. This flowchart is divided into the five sections or steps, each of which corresponds to the subsection in which that step is explained. Figure 2 should serve as a guide to the remainder of this section, which describes in detail the methodology and reasoning for each of these steps.

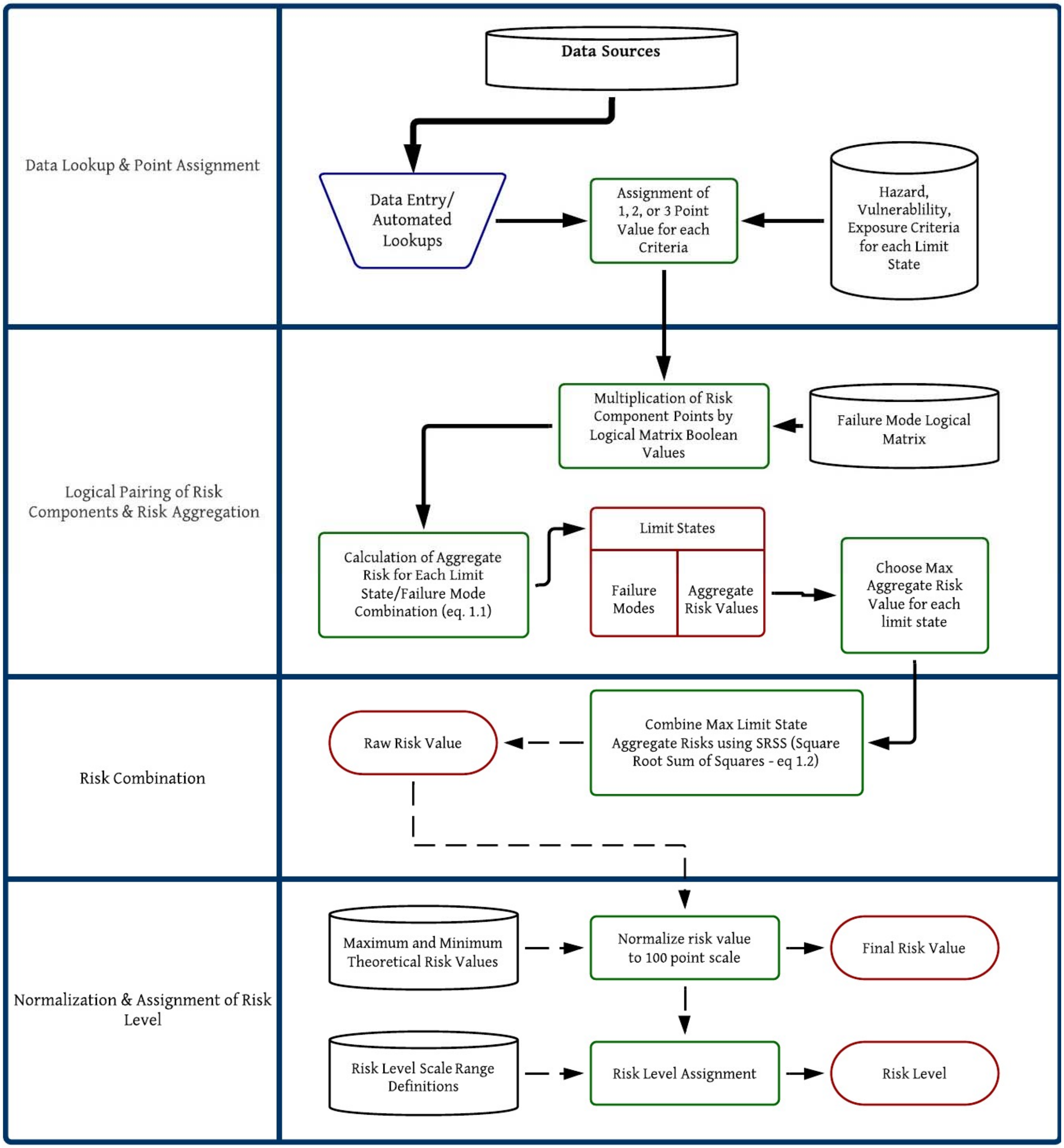


Figure 2: RBP Methodology Process Flowchart

4.1 Data Sources and Point Assignment

The initial step of the process is data lookup and point assignment. In this step, data is compiled from multiple sources in one central location. This data is then entered either manually or automatically and sorted into its corresponding limit state and risk component. The method for this data entry varies depending upon the data source. NBI data entry is easily automated using vertical lookups functions in Microsoft Excel. Other sources, such as commentary and photographs from Inspection Reports, require human interaction and therefore are manually entered. Geographic data taken from maps (i.e. functional classification of spanned roadway from maps) also require manual entry. However, this process was expedited whenever possible with image overlays and GPS coordinate data in Google Earth. After gathering this data for each bridge, a point value is assigned for the severity of risk associated with that condition based upon a set or range of typical conditions identified for each data source. At the conclusion of this step, each piece of data has been assigned four attributes:

1. Limit State (Geotechnical/Hydraulic Safety, Structural Safety, Serviceability and Durability, or Operations)
2. Risk Component (Hazard, Vulnerability, or Exposure)
3. Typical Range or Condition Classification
4. Point Value

For example, consider a bridge that is located within a 100 year flood plain. In this step of the process, the following attributes would be assigned:

- Limit State – Geotechnical/Hydraulic Safety
- Risk Component – Hazard
- Condition Classification – Within a 100 year Flood Plain
- Point value: 3

The starting point for development of the data, point assignment scale, and typical ranges and conditions was the criteria and point assignments listed in the reference included in Appendix 9.7. This list was then customized and improved over the course of the project. Desirable data sources were removed due to lack of available data, data sources were added due to modifications to the RBP methodology and point values were changed for various conditions. Furthermore, this list was developed with a broad geographical scope in mind and therefore contains some items that are either irrelevant or have insignificant variation when constrained to a geographically small region such as New Jersey. The final list of criteria used is shown in Table 1.

Table 1: Risk Component/Limit State Criteria

Performance Limit States	Hazards	Vulnerabilities	Exposures
Safety: Geotechnical/ Hydraulic	• Flood Plain • Seismic Design Category • Marine Traffic • Storm Surge Category • Underwater Substructure Flowrate	• Foundation Bearing Conditions • Pier Protection Standards • Scour Critical • Evidence of Substructure Settlement • Superstructure Above/Below Flood Level	• Replacement Cost • Coastal Evacuation Route • Distance of Detour Route • STRAHNET Route • Utility Disruption
Safety: Structural	• Average Daily Truck Traffic ADTT • Seismic Design Category	• Structural Assembly Classification • Fatigue Details • History of Displacements and Vibrations • Evidence of Structural Damage • Spanned Roadway Functional Classification • Fracture Critical Details • Exposed Prestressing Strands • Rocker Bearings • Percentage of Legal Truck Weight Posted • Does/Does not span roadway • Primary Construction Material	• Loss of Life • Loss of use or partial loss of use
Serviceability, Durability	• ADTT of Spanned Roadways • Average Annual Snowfall • Use of Deicing Salts • Freeze-Thaw Cycle • Proximity to Coast • History of Vehicular Collisions	• Water Penetration/Corrosion • Bearing Conditions • Expansion Joint Condition • Condition Rating of Approach • Condition Rating of Superstructure • Condition Rating of Substructure • Condition Rating of Deck • Underclearance of Spanned Roadways	• Maintenance Costs
Operations	• History of Fatal Accidents • Utilities on Structure	• Lane Width • Line Striping Condition • Traffic Safety Feature Adequacy • Breakdown Lanes/Shoulders	• History of Congestion • Current or Future Congestion

		• Percentage of Legal Truck Weight Posted

4.2 Logical Pairing of Risk components and Risk aggregation

After compiling and classifying the data based upon the criteria described in Section 4.1, the result is a collection of data categorized according to the hierarchy shown in Figure 3 . Data is first categorized into its limit state, then further categorized by its risk component (hazard, vulnerability, and exposure, represented by H, V, and E in Figure 3). The result is a series of tables each with two columns – the condition and its point value. In reality, these tables are too voluminous to show in this report, so they are represented by the table icons on the bottom tier of the hierarchy shown in Figure 3.

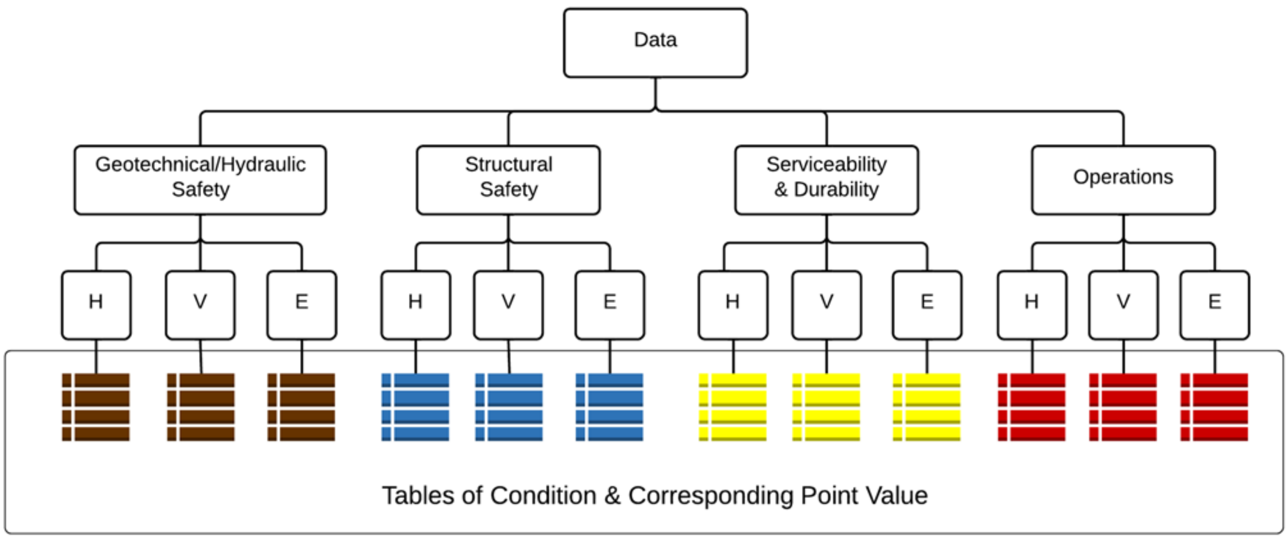
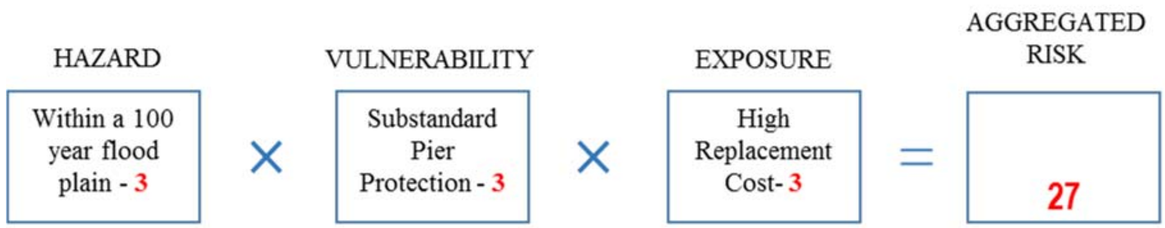


Figure 3: Data Relational Hierarchy

Having compiled and categorized this data, the next step is to calculate an aggregate risk for each limit state using the following equation:

$$Aggregate\ Risk(H) = (Hazard_{critical})(Vulnerability_{critical})(Exposure_{critical})(Uncertainty\ Premium) \qquad (1.1)$$

For successful implementation of equation 1.1, a critical hazard, vulnerability and exposure need to be identified from each limit state to calculate an aggregate risk value. As illustrated in Figure 3, each limit state has a range of conditions and points. A simple scheme such as choosing the maximum or averaging these values into a critical value was originally envisioned, but it soon became clear that this approach led to overly inflated risk values due to the inclusion of unrelated risk components as the critical values in equation 1.1. It became clear that the underlying modes of failure, herein referred to as **failure modes**, needed to be considered in the RBP scheme. To illustrate the importance of utilizing failure modes, consider a case where the critical value is chosen simply by picking the maximum value. The following scenario is a possible outcome for the geotechnical/hydraulic limit state:



In this case, a flood plain hazard is being influenced by pier protection vulnerability. Although the two share the same limit state, it is nearly impossible to imagine a scenario where pier protection affects flood risk. The identification of this problem made clear the need to create a RBP process that linked together related risk components and isolated those that were irrelevant. The list of failure modes for each limit state is shown below:

GeoTechnical/Hydraulic Safety:

- Seismic Liquefaction
- Flood
- Scour
- Vessel Collision

Structural Safety:

- Seismic
- Fatigue
- Construction Details or Conditions
- Overload

Serviceability and Durability

- Degradation
- Vehicle Collision

Operations

- Vehicular Safety

These failure modes impose a constraint on the sorted data that defines its interrelation by linking together related risk components and ignoring those that are unrelated. The next step was to define the interrelation of these failure modes. This interrelation is illustrated for the Geotechnical/Hydraulic Safety and Structural Safety Limit States in Figure 4. The tables contain risk components and their associated criteria. Each figure represents a limit state (Structural Safety and Geotechnical/Hydraulic Safety). The lines represent failure mode and their interrelation to each risk components' criteria and are differentiated by pattern and color.

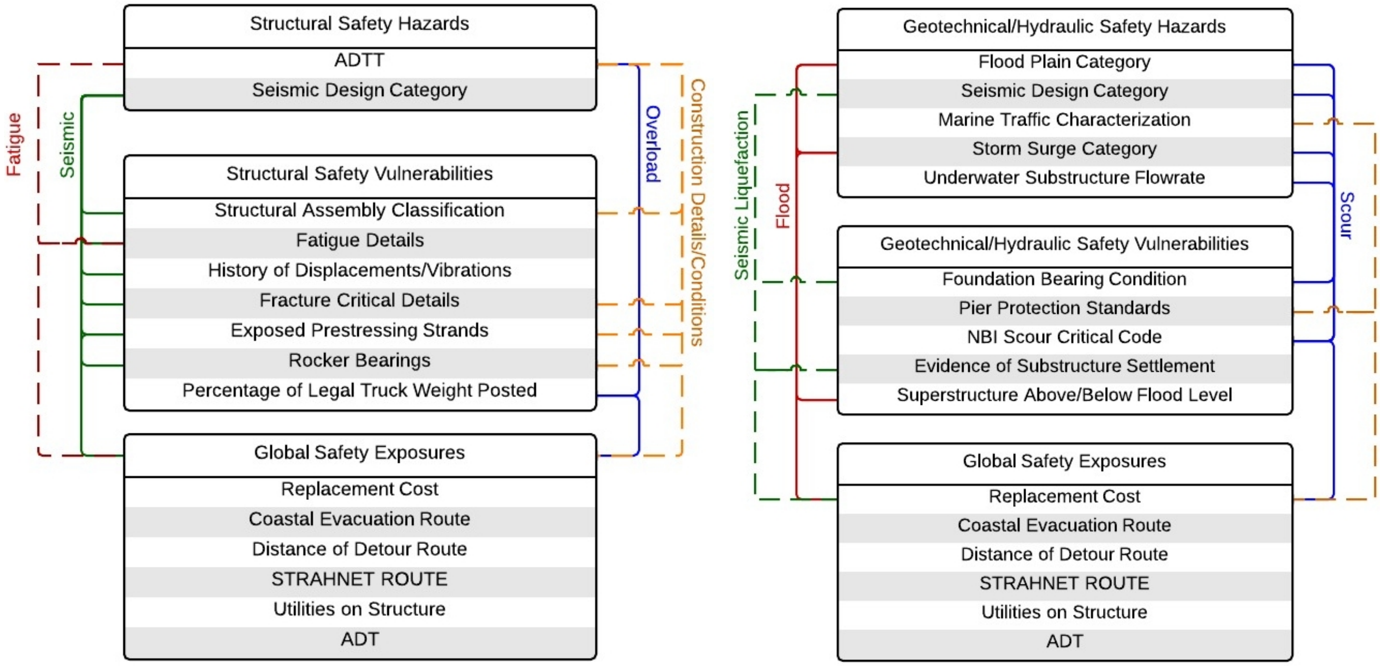


Figure 4: Failure Mode Interrelationship Diagrams

Numerically, this is accomplished through a logical matrix, with the rows relating to failure modes and the columns relating to risk components. Each entry of this matrix thus corresponds to a failure mode/risk component combination. If the risk component relates to the failure mode, a value of 1 is assigned. If the two are unrelated, a value of 0 is assigned. Multiplying this matrix by each structure's risk component point values allows for the calculation of an aggregate risk for each failure mode. The final result is a table containing an aggregate risk value for each failure mode. For examples of this table, see the results presented in Section 5. The controlling aggregate risk value is then identified by choosing the failure mode with the max aggregate risk value.

4.3 Risk Combination

After identifying a controlling aggregate risk for each limit state by the process described in Section the controlling risk values are then combined into one total risk value using square-root-sum-of-squares, as shown in equation 1.2.

$$Combined\ Risk = \sqrt{Geo/Hydra\ Safety^2 + \ Structural\ Safety^2 + Service\ and\ Durability^2 + Operations^2}$$

The result is a total risk value that can be used as a single metric to prioritize a bridge based on its overall risk.

4.4 Normalization and Assignment of Risk Level

Each risk value is normalized to a 100 point scale system using the maximum and minimum theoretical risk values for a given point assignment scheme and chosen uncertainty premium. A discrete ranking system is then used to classify these risk values into one of five categories, as found below in Table 2.

Table 2: Discrete Risk Level Raking Scale

Risk Level	Risk Value Range
Severe	80-100
High	60-80
Elevated	40-60
Guarded	20-40
Low	0-20

5. Discussion of Selected Results

For the purpose of illustrating the RBP methodology described in the preceding sections, one bridge has been selected from each risk level for in depth discussion. For a table summarizing the results of the complete set of 100 bridges, see Appendix A.2.

5.1 Low Risk Level– Structure No. 1913154 NJ 15 Northbound over Main Street

Structure No. 1913154 is a single span, composite, multiple rolled steel stringer bridge. It is located in Sparta Township, Sussex County, NJ. It carries NJ 15 Northbound, an urban principal arterial roadway with an ADT of 11,000 and an ADTT of 440. It spans Main Street, an urban collector with an ADT of 3,200. It has been identified as structurally deficient due to its deck condition rating of 4. Its sufficiency rating is 93.9.

Figure 5: Structure No. 1913154 (Cherry Weber & Associates, PC, 2010)

This structure has a total risk rating of 13, which puts it close to the middle of the "low" risk level range. It has low hazard and vulnerability risk values for its geotechnical/hydraulic safety limit state simply because it is not in a flood plain or spanning any bodies of water. It has low hazard values for its structural safety limit state because of its low ADTT. Its structural safety vulnerability values are also relatively low, the exception being a value of 3 for its E and E' fatigue details. Its maximum exposure value is 2 for its replacement cost and ADT.

Table 3 shows the results of the risk analysis for Structure No. 1913154. It is common for multiple failure modes to have identical values of aggregate risk. This is particularly common with lower risk bridges. As can be seen below, seismic liquefaction, flood, scour, and vessel collision – every geotechnical/hydraulic failure mode- all have aggregate risk values of 2. This is because the hazard and vulnerability values are all equal to 1, with a maximum exposure value of 2. The uncertainty premium for every structure in this project, as previously explained is 2. Therefore, when calculating equation 1.1 for these failure modes, the product is 4.

Table 3: Structure No. 1913154 Results

Results				
Failure Mode	**Aggregate Risk**			
	Safety: Geo/Hydra	**Safety: Structural**	**Serviceability and Durability**	**Operations**
Seismic Liquefaction	2	-	-	-
Flood	2	-	-	-
Scour	2	-	-	-
Vessel Collision	2	-	-	-
Seismic	-	4	-	-
Fatigue Failure	-	6	-	-
Construction Details or Conditions	-	4	-	-
Overload	-	2	-	-
Degradation	-	-	2	-
Vehicle Collision	-	-	2	-
Vehicular Safety	-	-	2	2
Max H*V*E	**2**	**6**	**2**	**2**
Uncertainty Premium	**2**	**2**	**2**	**2**
Max H*V*E*UP	**4**	**12**	**4**	**4**
			Total Risk	13
			Risk Level	Low

Figure 6: Structure No. 1913154 Aerial View

5.2 Guarded Risk Level – Structure No. 1817156 I-78 Westbound over US 202-206

Structure No. 1817156 is a three span, simply supported, composite, prestressed concrete bridge located in Bedminster, Somerset County, NJ. The bridge carries the westbound lanes of I-78, an Interstate Highway with an ADT of 19,700 and an ADTT of 800. It spans US 202-206, an urban principal arterial road with an ADT of 39,000 and an ADTT of 3,510. It is structurally deficient due to its deck condition rating of 3. Its sufficiency rating is 79.

Structure No. 1817156 has a low aggregate risk value for its aggregate Geotechnical/Hydraulic risk value, simply because it is outside of any flood plain and does not span a body of water. Its only geo/hydra hazard or vulnerability point value greater than 1 is a value of 2 for "Evidence of Substructure Settlement". This is because the Inspection Report notes 4" of settlement in some of its embankment panels. Its critical structural safety hazard component is a 3 due to its high ADT, and its critical structural safety vulnerability component is a 2 due to the presence of mildly exposed prestressing strands as noted in the Inspection Report. Its maximum exposure value is a 2 because of its ADT and STRAHNET Route designation.

Figure 7: Structure No. 1817156 (Management, 2011)

Table 5 shows the results of the risk analysis for structure No. 1817156. The controlling limit state for the Geo/Hydra limit state is Seismic Liquefaction due the structure's moderate history of substructure settlement. The controlling structural safety limit state is "Construction Details or Conditions", due to its exposed prestressing strands and high ADTT.

Table 4: Structure No. 1817156 Results

Results				
Failure Mode	**Aggregate Risk**			
	Safety: Geo/Hydra	**Safety: Structural**	**Serviceability and Durability**	**Operations**
Seismic Liquefaction	4	-	-	-
Flood	2	-	-	-
Scour	2	-	-	-
Vessel Collision	2	-	-	-
Seismic	-	2	-	-
Fatigue Failure	-	6	-	-
Construction Details or Conditions	-	12	-	-
Overload	-	6	-	-
Degradation	-	-	2	-
Vehicle Collision	-	-	2	-
Vehicular Safety	-	-	1	1
Max H*V*E	**4**	**12**	**2**	**1**
Uncertainty Premium	**2**	**2**	**2**	**2**
Max H*V*E*UP	**8**	**24**	**4**	**2**
			Total Risk	29
			Risk Level	Guarded

Figure 8: Structure No. 1817156 Aerial View

5.3 Elevated Risk – Structure No. 1516152 Route NJ 166 over North Channel of Toms River

Structure number 1516152 Route NJ 166 over the North Channel of Toms River is a single span, simply supported multiple concrete encased steel stringer bridge located in Dover, Ocean County, NJ. It carries Route NJ 166, an urban minor arterial road with an ADT of about 25,000 and an ADTT of less than 1000. It spans the North Channel of Toms River, a non-permit required waterway. It has been classified as structurally deficient due to its deck condition rating of 4 (poor). Its sufficiency rating is 64.3.

Figure 9: Structure No. 1516152 (LS Engineering Associates Corporation, 2010)

Table 5 shows the results of the risk analysis for Structure No. 1516152. The combined risk value for the structure is 50, which puts it in the middle of the "Elevated" risk range. The controlling failure mode for the geo/hydra limit state is scour. This bridge is located within a 100 year flood plain, falls within a storm surge region for category 1 and 2 hurricanes, and has been identified as a scour critical bridge. The controlling failure mode for the structural safety limit state is fatigue. This is due to a mid-range ADT paired with C and D fatigue details. The critical global safety exposure value is 2, due to the structure's replacement cost and ADT.

Table 5: Structure No. 1516152

Results				
Failure Mode	**Aggregate Risk**			
	Safety: Geo/Hydra	**Safety: Structural**	**Serviceability and Durability**	**Operations**
Seismic Liquefaction	4	-	-	-
Flood	12	-	-	-
Scour	18	-	-	-
Vessel Collision	2	-	-	-
Seismic	-	2	-	-
Fatigue Failure	-	8	-	-
Construction Details or Conditions	-	4	-	-
Overload	-	4	-	-
Degradation	-	-	2	-
Vehicle Collision	-	-	1	-
Vehicular Safety	-	-	4	4
Max H*V*E	**18**	**8**	**4**	**4**
Uncertainty Premium	**2**	**2**	**2**	**2**
Max H*V*E*UP	**36**	**16**	**8**	**8**
			Total Risk	50
			Risk Level	Elevated

Figure 10: Structure No. 1516152 Aerial View

5.4 High Risk– Structure No. 2003150 Route US 22 over Park Avenue (CR655)

Structure No. 2003150 is a two span, simply supported rolled steel beam bridge located in Scotch Plains Township, Union County, NJ. It carries US 22, an urban principal arterial highway with ADT of 25,300 and an ADTT just over 1,000. It spans Park Avenue, an urban principal arterial highway with an ADT of 57,700 and an ADTT of about 1,000. The bridge is considered structurally deficient due to its substructure condition rating of 4. Its sufficiency rating is 48.1.

Figure 11: Structure No. 2003150 (Stantec Consulting Services, 2009)

Structure No. 2003150 received relatively high risk ratings for both the structural safety and geo/hydra limit states. Although it does not span a body of water, it is located within a 100 year flood plain. Its superstructure is located below the Base Flood Elevation (BSEL) for a 100 year flood, making it vulnerable to overtopping during a flooding event. The structure is subjected to significant truck traffic, and has E and E' fatigue details. Its maximum exposure value is 2 due to its mid-range ADT and replacement cost. Table 6 shows the results of the risk analysis for structure No. 2003150. The total risk value is 63, putting it at the lower end of the range for a "high" risk level.

Table 6: Structure No. 2003150 Results

Results				
Failure Mode	**Aggregate Risk**			
	Safety: Geo/Hydra	**Safety: Structural**	**Serviceability and Durability**	**Operations**
Seismic Liquefaction	2	-	-	-
Flood	18	-	-	-
Scour	6	-	-	-
Vessel Collision	2	-	-	-
Seismic	-	6	-	-
Fatigue Failure	-	18	-	-
Construction Details or Conditions	-	12	-	-
Overload	-	6	-	-
Degradation	-	-	2	-
Vehicle Collision	-	-	2	-
Vehicular Safety	-	-	2	2
Max H*V*E	**18**	**18**	**2**	**2**
Uncertainty Premium	**2**	**2**	**2**	**2**
Max H*V*E*UP	**36**	**36**	**4**	**4**
			Total Risk	63
			Risk Level	High

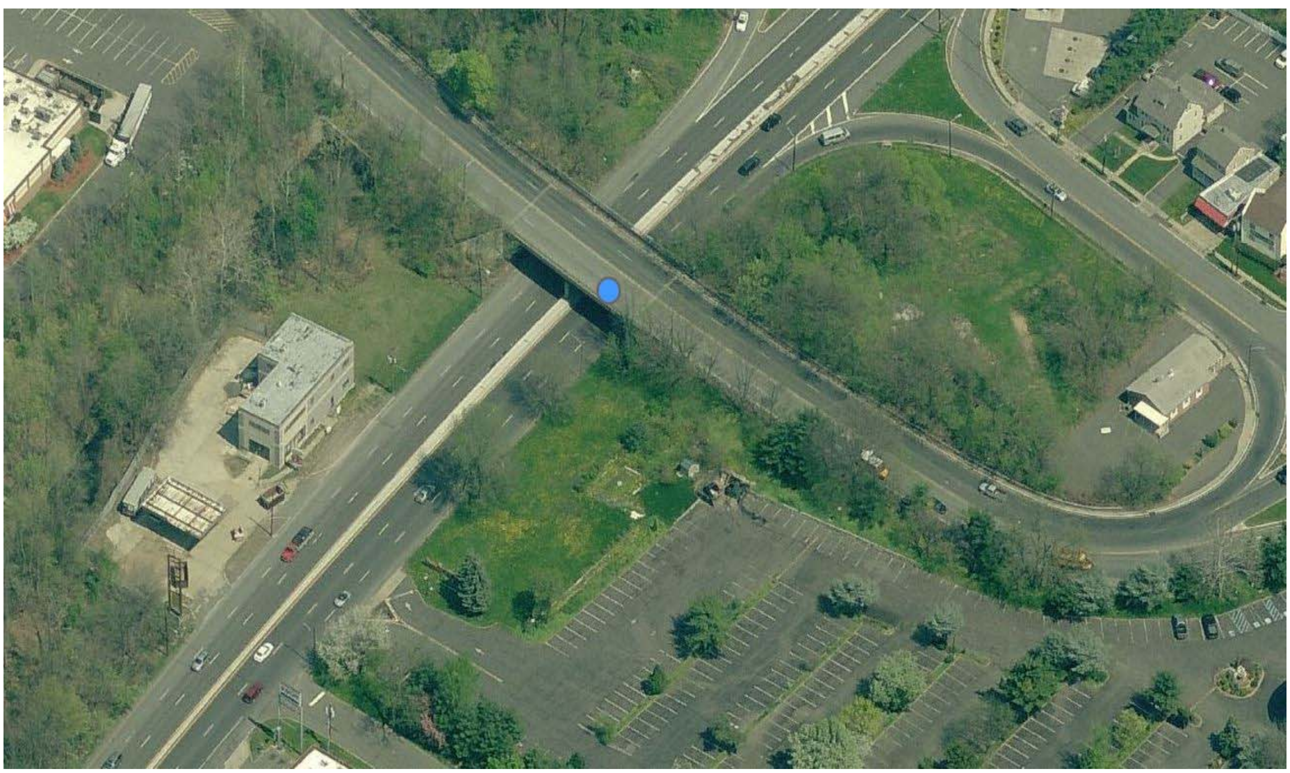


Figure 12: Structure No. 2003150 Aerial View

5.5 Severe Risk– Structure No. 0103153 US Route 30 over Duck Thorofare

Structure No. 0103153 is a 25-span (5 span continuous) reinforced concrete slab bridge with a substructure consisting of timber piles and pile caps located in Atlantic City, Atlantic County, NJ. It carries Route 30, an urban principal arterial route with ADT of 58,410 and ADTT of about 2,300. It spans Duck Thorofare, a low velocity tidal channel between Absecon and Lakes Bay. The bridge is considered structurally deficient due to the poor condition of its substructure. Its sufficiency rating is 35.4.

Figure 13: Structure No. 0103153 (Structural Evaluation, 2011)

Structure No. 0103153 received the highest risk rating of the sample population with value of 97. Its aggregate risk for the geo/hydra safety and structural limit states are both 27, the maximum possible value before multiplying by the uncertainty premium. It is located within a 100 year flood plain and is within a storm surge region for category 1 and 2 hurricanes. Its superstructure is below the flood level for a 100 year storm, making it vulnerable to overtopping in a flooding event. ADTT across the structure is about 2,300 trucks per day. At the time of the last available Inspection Report, several of the timber piles exhibit 100% section loss. Its maximum exposure value is 3, due to its high replacement cost and ADT.

Table 7: Structure No. 0103153 Results

Results				
Failure Mode	**Aggregate Risk**			
	Safety: Geo/Hydra	**Safety: Structural**	**Serviceability and Durability**	**Operations**
Seismic Liquefaction	3	-	-	-
Flood	27	-	-	-
Scour	9	-	-	-
Vessel Collision	3	-	-	-
Seismic	-	9	-	-
Fatigue Failure	-	9	-	-
Construction Details or Conditions	-	27	-	-
Overload	-	9	-	-
Degradation	-	-	2	-
Vehicle Collision	-	-	1	-
Vehicular Safety	-	-	1	1
Max H*V*E	**27**	**27**	**2**	**1**
Uncertainty Premium	**2**	**2**	**2**	**2**
Max H*V*E*UP	**54**	**54**	**4**	**2**
			Total Risk	97
			Risk Level	Severe

Figure 14: Structure No. 0103150 Aerial View

6. Discussion of Overall Results

The final risk values as calculated by the methodology described in this report can be found in Appendix A. The average combined risk value is 37, with a standard deviation of 21. The maximum value is 99 and the minimum value is 5. Figure 15 shows a histogram of the assigned risk levels for the total 100 bridges. The mid-range risk Level, "Elevated", is the most common, accounting for 31 of the 100 bridges. "Low" and "Guarded" are nearly equal, with 29 and 26 occurrences, respectively. "High" and "Severe" are the least frequent risk levels, with 10 and 4 occurrences, respectively.

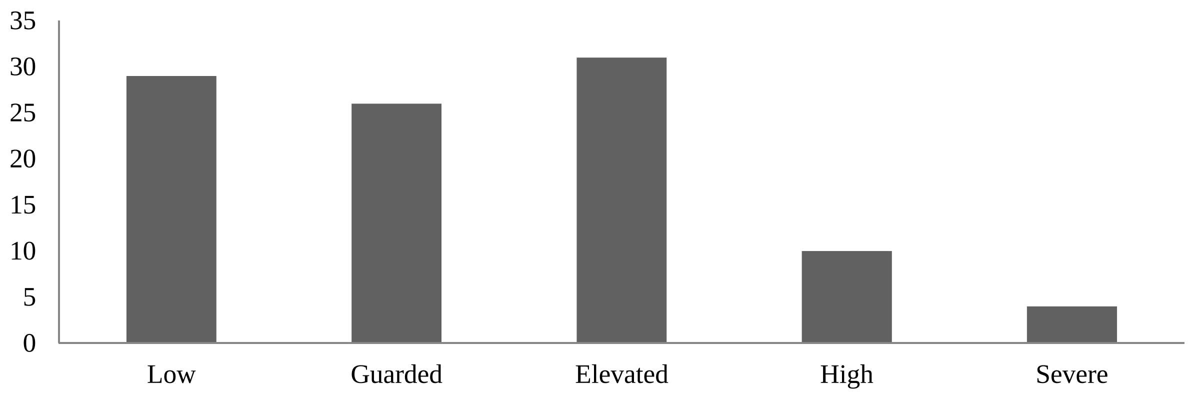


Figure 15: Risk Level Histogram

Figure 16 and Figure 17 are histograms of the controlling failure modes for the safety based limit states. It is important to note that the total number of occurrences in each of these histograms exceeds 100, the total number of bridges. Due to the discrete ranking methodology that was applied in this project, in many cases there are identical aggregate risk values for each limit state, and therefore picking the maximum value resulted in a tie. It is also important to note that these histograms neglect the severity of each controlling aggregate risk. For example, consider the frequency of seismic as the controlling failure mode in Figure 17. New Jersey as a whole is at a relatively low hazard for seismic activity. However, a seismic hazard can nonetheless be elevated if a seismic vulnerability is present, such as rocker bearings or other seismically vulnerable construction details. In this case, it is possible for the seismic failure mode to be tied as the max value with other aggregate risks, even though its aggregate risk value is likely very low. Hydraulic Failure modes predominantly control the Geotechnical/Hydraulic Limit State (Flood, Scour, and Seismic Liquefaction), with Vessel Collision as the least frequent controlling limit state.

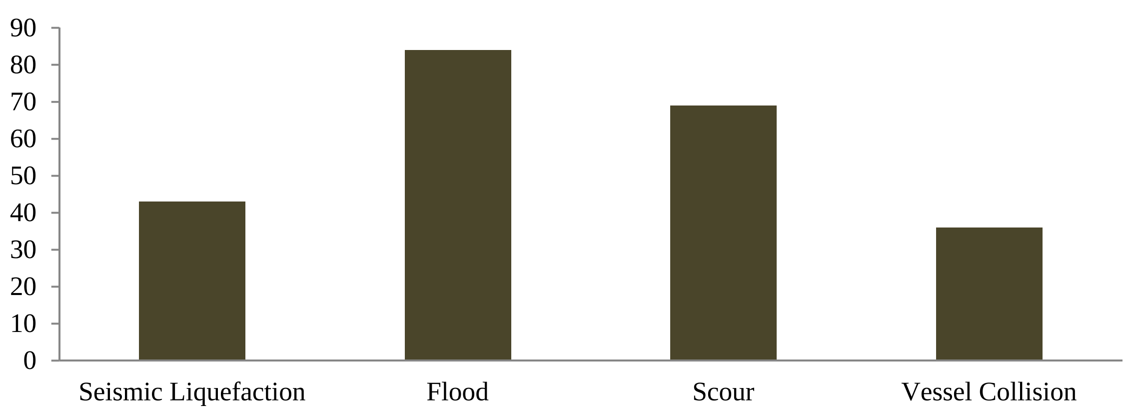


Figure 16: Geotechnical/Hydraulic Safety Controlling Failure Mode Diagram

Construction Details or Conditions is the predominant controlling limit state for the Structural Safety Limit State, followed by fatigue, overload, and seismic, respectively.

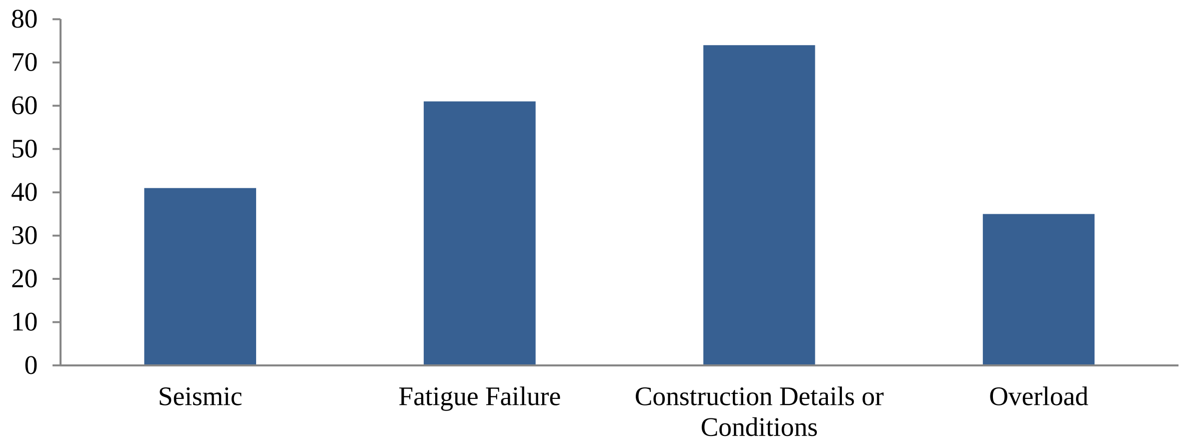


Figure 17: Structural Safety Controlling Failure Mode Diagram

An alternative way to conceptualize the RBP risk definitions is to consider the combined hazard and vulnerability values as a "reliability index", and to consider the combined exposure values as an "importance factor". A scatterplot with reliability index on the y axis and combined exposure, or importance factor, on the x axis is shown below in Figure 18. The data used to generate this scatterplot can be found in Appendix 9.6.

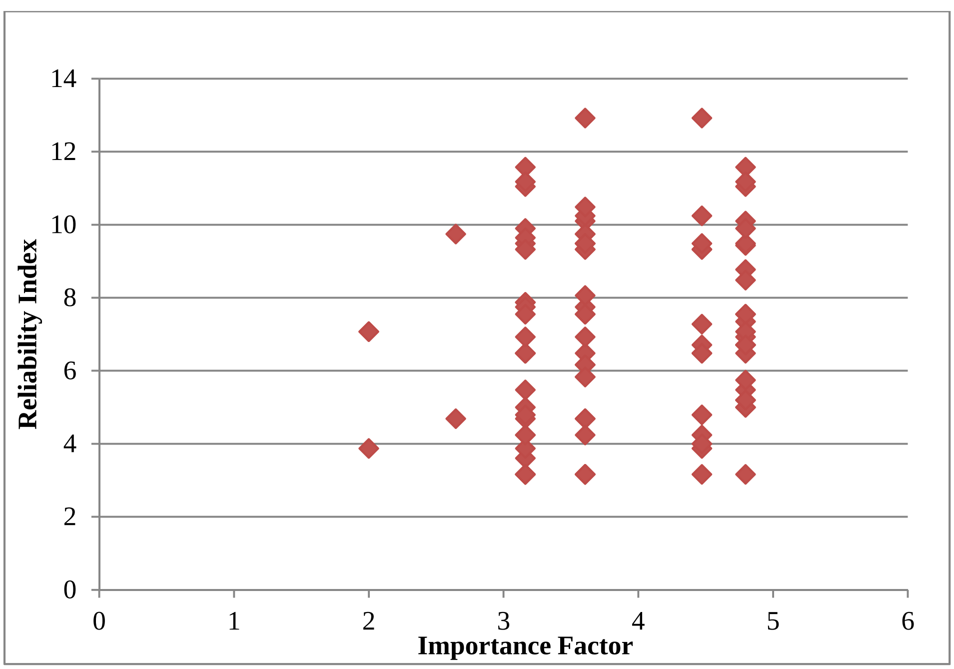


Figure 18: Reliability Index vs. Importance Factor

7. Potential Future Work

If applied on a larger scale, the framework discussed in this report would greatly benefit from further effort toward automating the data entry and lookup process. Any information found in the NBI Database is easily automated using lookup functions, but any data gathered from cross referencing geospatial sources such as FEMA maps, NOAA maps, etc. currently requires manual entry. Qualitative information found in Inspection Reports also requires manual effort for interpretation and entry. Automating both of these processes would allow for expedited implementation on a statewide or even nationwide scale.

8. Works Cited

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9. Appendices

9.1 Data Sources and Point Assignment

This section describes each data source, categorized into their corresponding limit state and risk component. The number at the end of each data source title refers its detailed source information, found in Appendix 9.3. A table is included in each subsection that lists typical conditions that were encountered for each criteria and a corresponding point value for each. Several of the criteria listed in this section have point values that are consistent across a range of conditions. This is simply because the framework of the Prioritization Excel Document is structured so that changes to these values can be made quickly and easily by simply changing the value of a single cell. Incorporating a comprehensive range of possible conditions, even if one or more has identical risk values, allows flexibility in optimizing the RBP scheme.

9.1.2 Geotechnical/Hydraulic Safety Limit State Conditions and Points

Geotechnical/Hydraulic Safety Hazards

Flood Plain [1] [2]

Flood plain classification was determined using FEMA Flood Insurance Rate (FIRM) Maps. For the sake of convenience, the FEMA FIRM Google Earth plugin was used to expedite this process whenever possible. If insufficient data was available through this source, then the FEMA FIRM map web-based tool was used.

Seismic Design Category [4]

Seismic design values were calculated per AASHTO specifications, and were relatively consistent across NJ using available data.

Marine Traffic [1]

Marine traffic was characterized using the NJ Coast Guard Designation field in the NJDOT SI&A sheets. If the bridge is a "permit-required" structure, it means that it spans a body of water that has been deemed navigable by the Coast Guard. The size of the ships that can navigate the spanned channel is determined from observing aerial maps and the structure's horizontal and vertical clearance.

Storm Surge Category [1] [1]

The probability of a structure being exposed to a hurricane storm surge was estimated using the New Jersey Office of Emergency Management (NJ OEM) Hurricane Storm Surge Maps. This map indicates which areas within NJ are susceptible to Hurricane Storm Surge, and the category of Hurricane at which they become vulnerable.

Underwater Substructure Flow rate [9]

For structures that span a body of water, approximate flow rate on the day of inspection is noted in the Inspection Report.

Table 8: Geotechnical/Hydraulic Safety Hazards

Hazard	Conditions	Risk Value
Flood Plain	Outside of a 500 year flood plain	1
	Outside of a 100 year flood plain	2
	Within a 100 year flood plain	3
Seismic Design Category	A	1
	B, C	2
	D, E, F	3
Marine Traffic Characterization	Over a Non-navigable/Non-Permit Channel	1
	Permit-Required Channel	2
	Permit-Required channel for large vessels	3
Storm Surge Category	Not in Storm Surge Region	1
	Storm Surge Category 3 or 4	2
	Storm Surge Category 1 or 2	3
Underwater Substructure Flowrate	No underwater substructure	1
	Underwater substructure subjected to no flow; stagnant water	1
	Substructure subjected to low to moderate flow rates	2
	Substructure subjected to significant flow rates	3

Geotechnical/Hydraulic Safety Vulnerabilities

Foundation Bearing Conditions

A reliable data source was not found for the bearing conditions of NJDOT Bridges. This criterion was left in the Prioritization framework and simply given a value of 1 for every structure.

Pier Protection Standards [1]

Pier protection standards were taken from NBI Code Item 111 “Pier or Abutment Protection”. The condition of the pier protection system represents a critical vulnerability to ship impact of substructure elements.

Scour Critical [1]

Scour vulnerability was represented using NBI Code 113, “Scour Critical Bridges”. Table 9 lists the risk point values for each code found in the NBI data for this item

Evidence of Substructure Settlement [9]

Substructure Settlement was qualitatively estimated from commentary in the Inspection Reports regarding settlement, misalignment and displacement of structural elements.

Superstructure Above/Below Flood Level [9] [2]

Whether the superstructure is above or below the flood level poses a critical vulnerability to overtopping during a flooding event. This criterion was represented by FEMA Flood Rate Insurance Maps (FIRM) Base Flood Elevation (BFE). The BFE was then compared to the elevation of the bottom of the superstructure, which could typically be obtained or estimated from altitude maps and Inspection Reports.

Table 9: Geotechnical/Hydraulic Safety Vulnerabilities

Vulnerabilities	Conditions	Risk Value
Foundation Bearing Conditions	Founded on deep foundations or bedrock	1
	Founded on shallow foundations on cohesive soil	2
	Founded on shallow foundations on non-cohesive soil	3
Pier Protection Standards	Navigation protection not required	1
	In place and functioning	1
	In place but in a deteriorated condition	2
	In place but re-evaluation of design suggested	2
	None present but re-evaluation suggested	2
Scour Critical	N, U, T, 9 , 8, 7	1
	6, 5, 4	2
	3, 2, 1, 0	3
Evidence of Substructure Settlement	No History/Evidence	1
	Moderate History/Evidence	2
	Moderate - Significant History/Evidence	3
Superstructure Above/Below Flood Level	Not in Flood Plain	1
	Underwater substructure subjected to no flow; stagnant water	1
	Above	2
	Below	3

9.1.3 Structural Safety Limit State Conditions and Points

Structural Safety Limit State Hazards

Average Daily Truck Traffic (ADTT) [1]
Values for average daily truck traffic were taken from NBI data. Repeated heavy loading from trucks is one of the most significant and quantifiable structural hazards experienced by bridge superstructures, which led to its inclusion in this report.

Seismic Design Category [4]
Seismic design values were calculated per AASHTO specifications, and were consistent across NJ using available data.

Table 10: Structural Safety Limit State Hazards

Hazard	Conditions	Risk Value
ADTT	Founded on deep foundations or bedrock	1
	Founded on shallow foundations on cohesive soil	2
	Founded on shallow foundations on non-cohesive soil	3
Seismic Design Category	A	1
	B, C	2
	D, E, F	3

Structural Safety Limit State Vulnerabilities

Structural Assembly Classification [9]

This criterion was determined by observing photographs contained in the Inspection Reports and classifying the structural system based upon its overall redundancy.

Fatigue Details [9]

The classification of fatigue details for steel structures within the selected population was taken directly from the Inspection Reports.

History of Displacements and Vibrations [9]

A field in the Inspection Reports notes the structure's vibration and deflection as experienced by the inspector on the day of the inspection.

Evidence of Structural Damage [9]

Inspection Reports were reviewed for any source of structural damage, the severity of which was then classified to assign a risk value**.**

Fracture Critical Details [1]

This data was taken from NBI Code Item 92A "Fracture Critical Details". This field provides a simple "yes" or "no" regarding the presence of fracture critical details in the structure.

Exposed Prestressing Strands [9]

This information was gathered from element condition commentary and photographs in Inspection Reports.

Rocker Bearings [9]

The presence of rocker bearings was determined from NJDOT SI&A sheets.

Percentage of Legal Truck Weight Posted [1]

The percentage of legal truck weight posted was determined from NBI Code Item 70 "Bridge Posting".

Table 11: Structural Safety Limit State Vulnerabilities

Vulnerabilities	Conditions	Risk Value
Structural Assembly Classification	Structure displays bidirectional redundancy	1
	Simply-supported constructed with transverse distribution capabilities;	2
	Non-composite construction	3
	Simply-supported construction with minimal transverse distribution capabilities	3
Fatigue Details	No fatigue details	1
	A and B fatigue details	1
	C and D fatigue details	2
	E and E' fatigue details	3
History of Displacements and Vibrations	No history of excessive displacements or vibrations	1
	History of significant displacements or vibrations	2
	History of excessive displacements or vibrations	3
Evidence of Structural Damage	No evidence of structural damage	1
	Minor evidence of structural damage within the critical load path	2
	Evidence of structural damage within the critical load path	3
Fracture Critical Details	FALSE	1
	TRUE	2
Exposed Prestressing Strands	FALSE	1
	TRUE	2
Rocker Bearings	FALSE	1
	TRUE	2
Percentage of Legal Truck Weight Posted	Equal to or above legal loads	1
	00.1 - 09.9 % below	2
	10.0 - 19.9 % below	3
	20.0 - 29.9 % below	3
	30.0 - 39.9 % below	3
	> 39.9% below	3

9.1.4 Geotechnical/Hydraulic and Structural Safety Limit State Shared Exposure Conditions and Points

While it is useful to differentiate between geotechnical/hydraulic and structural limit states for hazard and vulnerability risk components, the impact due to a failure is consistent for both. For this reason, exposure criteria for both geotechnical/hydraulic safety and structural safety are shared between these limit states.

Replacement Cost [1]

Replacement cost was assumed to be the total replacement cost of the bridge, not including approach roadways. In the case of bridges where replacement is the recommended course of action, this can be gathered from NBI Data using NBI Item 75A "Type of Work" and Item 94, "Bridge Improvement Cost". In case other forms of rehabilitation or widening are recommended, this data is unavailable. In these cases, replacement cost was estimated using multivariate linear regression. First, several NBI Items were identified that were correlated to bridge replacement cost per square foot of deck, including materials type, skew, number of spans, length of maximum span, setting (urban or rural), vertical underclearance, ADT and feature spanned. This data was gathered for structurally deficient bridges with replacement as the recommended course of action for the Mid-Atlantic Region Bridge Population (New Jersey, Delaware, and Maryland). Pennsylvania was omitted due to lack of necessary data. After performing multivariate linear regression on these bridges with known replacement costs, an equation was developed to predict the cost per square foot of deck, which could then be used to calculate the replacement cost of the bridge. Eighty percent of the sample population set was used for generating the linear regression formula. The remaining twenty percent was used as validation by calculating the replacement cost for these bridges using the linear regression formula and then comparing the results to the known replacement cost and calculating a margin of error. This margin of error was calculated to be 21% on average.

Coastal Evacuation Route [1] [11]

Whether a bridge was located on a coastal evacuation route was determined from NJ Office of Emergency Management's (OEM) Costal Evacuation Route Maps. Higher exposure is assigned if a structure is located on a coastal evacuation route that has been identified as non-redundant.

Distance of Detour Route [1]

Distance of detour route is taken from NBI Item 19, and is defined as "the total additional travel for a vehicle which would result from closing of the bridge" (FHWA, 1995).

Strategic Highway Corridor Network (STRAHNET) Route [1]

STRAHNET routes are roadways that have been identified as strategically significant to the Department of Defense, and include the Federal Interstate Highway network and other non-interstate highways that serve as connectors from interstates to important military installations. This data was gathered from NBI Item 100 "STRAHNET Highway Designation".

Utilities on Structure [9]

Whether a bridge supports utility lines was considered due to the impact to surrounding communities in the event of a failure that resulted in an interruption of service. This data was gathered from SI&A sheets.

Average Daily Traffic (ADT) [1]

ADT was used as a metric to account for potential for loss of life in the event of a failure.

Table 12: Global Safety Exposures

Exposure	Conditions	Risk Value
Replacement Cost	Less than $10,000,000	1
	Between $10,000,000 and $50,000,000	2
	Greater than $50,000,000	3
Coastal Evacuation Route	Not on a critical route (lifeline, evacuation route)	1
	Not on a critical, nonredundant route (life line evacuation route)	1
	On a critical, non-redundant route	2
Distance of Detour Route	Less than 1 (miles)	1
	Less than 2 (miles)	1
	Greater than 2 (miles)	1
STRAHNET Route	The inventory route is not a STRAHNET route.	1
	The inventory route is on an Interstate STRAHNET route.	2
	The inventory route is on a Non-Interstate STRAHNET route.	2
	The inventory route is on a STRAHNET connector route.	2
Utility Disruption	None, telephone conduit, sanitary sewer	1
	Electrical Conduit, Gas Main, Fiber Optic Cable	2
	Water Main	3
Average Daily Traffic (ADT)	Less than 10,000	1
	Between 10,000 and 50,000	2
	Greater than 50,000	3

9.1.5 Serviceability and Durability Limit State Conditions and Points

Serviceability and Durability Hazards

ADTT of Spanned Roadways [1]
Average Daily Truck Traffic of spanned roadway was obtained from SI&A sheets. This information is important in estimating risk of vehicle collision to superstructure elements.

Use of Deicing Salts [12]
Use of deicing salts was assumed to have a direct correlation to average annual snowfall. Average annual snowfall for each bridge location was obtained from maps that delineate regions of annual snowfall ranges. This then allowed for determination of severity of deicing salt use.

Freeze-Thaw Cycle [3]
Freeze thaw cycles were determined from the thaw cycle severity map found shown in Figure 1 of ASTM D5312-04. This map indicated relatively consistent freeze-thaw cycle frequency across New Jersey.

Proximity to Coast
The proximity of a structure to a saline body of water has a significant effect on the rate and severity of corrosion that can be expected. In the case of New Jersey, these include the Atlantic Ocean, Barrier Island bays and Inlets, the Delaware Bay and the Lower New York Bay.

History of Vehicular Collisions [9]
This information is gathered from Inspection Reports, which will typically document any structural damage related to vehicular collision.

Table 13: Serviceability and Durability Hazards

Hazard	Conditions	Risk Value
ADTT of Spanned Roadways	Less than 500	1
	Between 500 and 10,000	2
	Greater than 10,000	2
Use of Deicing Salts	No Use	1
	Moderate Use	1
	Severe Use	2
Freeze-Thaw Cycle	Low Number	1
	Moderate Number	1
	High Number	1
Proximity to Coast	Less than 1	1
	Between 1 and 5	2
	Greater than 5	3
History of Vehicular Collisions	None	1
	Isolated Collisions	2
	Repeated Collisions	2

Serviceability and Durability Vulnerabilities

Water Penetration/Corrosion, Bearing Conditions, and Expansion Joint Condition [9]

These three criteria were assessed by qualitatively reviewing Inspection Reports items such as condition ratings, condition rating commentary and photographs.

Condition of Approach [9]

This data was represented by NJDOT's SI&A Item BA "Condition of Approach", which is simply a numeric condition rating.

Condition Rating of Superstructure, Condition Rating of Substructure, Condition Rating of Deck [1]

These three criteria were gathered from NBI data items 58, 59, and 60, respectively.

Underclearance of Spanned Roadways [1]

To determine the adequacy of the structure's clearance, the difference between the minimum vertical underclearance of the bridge and the NJDOT recommended underclearance for the structure was calculated. A structure's minimum underclearance was taken from NBI Code Item 54A. NJDOT Recommended Underclearance and is dependent upon the functional classification of the roadway spanned. For this piece of data, NJDOT functional classification maps were used in conjunction with Table 3.3.2 on page 3-3 of the NJDOT Design Manual for Bridges and Structures, 5th Edition.

Table 14: Serviceability and Durability Vulnerabilities

Vulnerabilities	Conditions	Risk Value
Water Penetration/Corrosion	Little or no evidence of reinforcement/structural steel corrosion	1
	Some evidence of reinforcement/structural steel corrosion	1
	Evidence of wide-spread reinforcement/structural steel corrosion	1
Bearing Conditions	Bearings in good operating condition	1
	Bearings with minor damage or Corrosion	1
	Bearings with Extensive Damage of Corrosion	1
Expansion Joint Condition	Joints in good operating condition	1
	Joints with minor evidence of damage/leaking	1
	Failed expansion joints	1
Condition of Approach	9, 8, 7	1
	6, 5, 4	1
	3, 2, 1, 0	2
Condition Ratings of Superstructure, Substructure and Deck	9, 8, 7	1
	6, 5, 4	1
	3, 2, 1, 0	2
Difference in Underclearance and NJDOT Recommended Underclearance	Meets Clearance Requirements	1
	Within 2' of required clearance	1
	2' or more below recommended clearance	2

Serviceability and Durability Exposures

Maintenance Cost

A reliable source for maintenance cost data could not be determined. Therefore, all bridges were given a risk rating of one for this criterion, pending the identification of a reliable source of data.

Table 15: Serviceability and Durability Exposures

Exposure	Conditions	Risk Value
Maintenance Costs	Data Not Available	1
	Less than $1,000,000	1
	Less than 3,000,000	1
	Greater than $3,000,000	1

9.1.6 Operations Limit State Conditions and Points

Operations Hazards

History of Fatal Accidents [6]

This data was gathered from The Fatality Analysis Reporting System (FARS). As described on their website, FARS is a "nationwide census providing NHTSA, Congress and the American public yearly data regarding fatal injuries suffered in motor vehicle traffic crashes. (NHTSA, n.d.)" FARS data is available from 1977-2011, but only the most recent ten years of data was used in the prioritization scheme. Fatal Accidents both on the structure and the area of roadway spanned by the structure were considered.

Table 16: Operations Hazards

Hazard	Conditions	Risk Value
History of Fatal Accidents	No History of Fatal Accidents	1
	Isolated Fatal Accidents	1
	Repeated Fatal Accidents	2
History of Vehicular Collisions	None	1
	Isolated Collisions	1
	Repeated Collisions	2

Operations Vulnerabilities

Lane Width [9] [1]

Whenever available, dimensioned roadway cross section drawings were used to determine the lane width. If unavailable, NBI item 51 "Bridge roadway width, Curb to Curb" with NBI Item 28A, "Lanes on Structure" was used.

Line Striping Condition [9]

Line Striping condition was determined qualitatively from observing deck photographs.

Traffic Safety Feature Adequacy [1]

Traffic Safety Feature Adequacy was determined from NBI Items 36A, 36B, 36C and 36D, "Bridge Railings", "Transitions", "Approach Guardrail" and "Approach Guardrail Ends", respectively. These items are given a value of 1 if deemed adequate and 0 if deemed inadequate. These four values were averaged to obtain a value ranging from 0-1.

Breakdown Lanes/Shoulders [9]
The presence of breakdown lanes was determined from Inspection Report photographs.

Percentage of Legal Truck Weight Posted [1]
The percentage of legal truck weight posted was determined from NBI Code Item 70 “Bridge Posting”.

Table 17: Operations Vulnerabilities

Vulnerability	Conditions	Risk Value
Lane Width	Roadway geometry up to current standards	1
	Less than standard lane width by no more than 2’.	1
	More than 2’ less than standard lane width	1
Line Striping Condition	Road paint in good condition	1
	Road paint in fair condition	1
	Road paint in poor condition	1
Traffic Safety Feature Adequacy	Greater than .75	1
	Between 0.5 and 0.75	1
	Less than 0.5	1
Breakdown Lanes/Shoulders	Breakdown lane/shoulders	1
	Breakdown lane/ shoulders not present	1
Percentage of Legal Truck Weight Posted	00.1 - 09.9 % below	1
	10.0 - 19.9 % below	1
	20.0 - 29.9 % below	1
	30.0 - 39.9 % below	1
	> 39.9% below	1

Operations Exposures

History of Congestion [5]
This data was gathered from Google Maps. A feature within Google maps allows the user to view traffic history at a certain location at a given time and day of the week. By scanning through each day of the week, the severity of congestion at each location was able to be qualitatively determined.

Table 18: Operations Exposures

Exposure	Conditions	Risk Value
History of Congestion	No Congestion	1
	Moderate Congestion	1
	Severe Congestion	2

9.2 Overall Results Summary

Structure No.	Structure Description		Risk Value	Risk Level
	Spanning	Carrying		
2153161	'STRANTON BRANCH (ABAN) '	'Dirt Farm Road '	5	Low
0350162	'ROBINSVILLE SECONDARY '	'FARNSWORTH AVE. '	8	Low
3001151	'D&R FEEDER CANAL '	'US 206 (N. BROAD) '	4	Low
2108155	'ABAND. HUDSON BR.CONRAIL'	'ROUTE US 46 '	8	Low
1850166	'NEW YORK BRNCH (CONRAIL)'	'HAMILTON ROAD '	8	Low
1337150	'SOUTHERN DIVISION '	'NJ RTS 33-34 EB. '	11	Low
1426150	'NJ TRANSIT(MORR LINE) '	'NJ 183 '	9	Low
1650161	'PASSAIC-NY BR (ABAN) '	'PIAGET AVE(CR 628)'	9	Low
2117161	'ABANDONED NYS&WRR '	'ROUTE NJ 94 '	9	Low
1014162	'I-78 '	'PATTENBURG RD-614 '	9	Low
1101163	'NEW YORK AVENUE '	'ROUTE US 1B '	9	Low
2105153	'BRANCH LOPATCONG CREEK '	'ROUTE NJ 57 '	9	Low
1249165	'AMTRAK NE CORRIDOR '	'ADAMS LANE (CR608)'	13	Low
1850160	'TRENTON LINE (CSX) '	'CAMP MEETING AVE. '	13	Low
1850164	'TRENTON LINE(CSXT) '	'HOMESTEAD ROAD '	13	Low
1913154	'MAIN STREET '	'NJ 15 NB '	13	Low
2101150	'RAMP TO US 22 EB '	'US 22 WB '	8	Low
0114155	'ROUTE US 322 '	'ROUTE NJ 54 '	12	Low
1419169	'I-287 '	'JAMES ST.(CO.663) '	14	Low
3001176	'D&R CANAL FEEDER '	'BARBERS FARM BR. '	14	Low
3001175	'D&R CANAL FEEDER '	'CORYELL STREET '	14	Low
0222153	'U.S. ROUTE 46 '	'BROAD AVENUE '	14	Low
0823150	'NJ 45 SB '	'I-295 RAMP S '	18	Low
1607161	'ROUTE US 46 '	'MAIN AVENUE(CR601)'	18	Low
0815150	'I-295 '	'ROUTE NJ 47 '	19	Low
1711156	'I-295 SB '	'RAMP K '	19	Low
0317155	'CRAFTS CREEK '	'US 130 '	15	Low
1223153	'PERTH AMBOY CONN (RT440)'	'N.J RT 35 '	18	Low
2154160	'Washington Sec (Conrail)'	'S Main St '	18	Low
0731160	'RAMP B (I-280) '	'NJ 21 RAMP A '	22	Guarded
0220154	'ERIE-LACKAWANNA RAILROAD'	'U.S. ROUTE 46 '	21	Guarded
0731156	'RAMPS C & D '	'RT. I-280 RAMP E '	22	Guarded
0206173	'TEANECK ROAD '	'ROUTE NJ 4 '	21	Guarded
1005151	'CENTRAL RAIL ROAD OF NJ '	'ROUTE US 22 '	21	Guarded

3000163	'D&R CANAL '	'GRIGGSTOWN CAUSWY '	21	Guarded
3001159	'DE.& RARITAN CANAL FDR '	'HERMITAGE AVENUE '	24	Guarded
0214159	'CENTRAL AVENUE '	'ROUTE NJ 17 '	27	Guarded
1502152	'WARETOWN CREEK '	'ROUTE US 9 '	25	Guarded
1817155	'US 202&206 '	'I-78 EB '	28	Guarded
1007159	'WICKECHEOKE CREEK '	'ROUTE NJ 29 '	28	Guarded
0730152	'PROSPECT AV NB (CR 577) '	'I-280 RAMP 2P '	28	Guarded
1009150	'COPPER CREEK '	'ROUTE NJ 29 '	28	Guarded
1817156	'US 202-206 '	'I-78 WESTBOUND '	29	Guarded
1149160	'AMTRAK NE CORRIDOR '	'CENTER STREET '	29	Guarded
0202159	'JONES ROAD '	'US 1 9 AND 46 '	28	Guarded
3001162	'DEL. & RAR. CANAL FEEDER'	'LOWER FERRY ROAD '	29	Guarded
0821160	'ROUTE I-295 '	'DEMOCRAT ROAD '	31	Guarded
1211152	'NEW JERSEY ROUTE 18 '	'RT 516(MATAWAN RD)'	32	Guarded
0203153	'NJ ROUTE 3 '	'ORIENT WAY '	31	Guarded
1310155	'N BRANCH WRECK POND '	'RT 35 '	29	Guarded
1105151	'MILLSTONE RIVER '	'OLD RT 27 '	31	Guarded
0221152	'ROUTE NJ 17 NORTHBOUND '	'ROUTE US 46 '	32	Guarded
0314151	'ROUTE NJ 73 '	'CO RT 537 '	31	Guarded
0907152	'NJ 3 '	'PATERSON PLANK RD '	34	Guarded
3000169	'DELAWARE & RARITAN CANAL'	'LANDING LANE (609)'	36	Guarded
2003166	'CHESTNUT STREET(CR626) '	'US 22 '	44	Elevated
2106151	'SHABBACONG CREEK '	'NJ ROUTE 57 '	44	Elevated
1308154	'BIG BROOK '	'NJ ROUTE 34 '	45	Elevated
0302151	'JOBS CREEK '	'U.S.ROUTE 9 '	44	Elevated
2117160	'PAULINS KILL '	'ROUTE NJ 94 '	44	Elevated
1809150	'N BRANCH RARITAN RIVER '	'US202 '	44	Elevated
2105152	'LOPATCONG CREEK '	'NJ 57 '	45	Elevated
0731154	'MARTIN LUTHER KING BLVD.'	'I-280 '	44	Elevated
1601162	'NJ TRANSIT & SERVICE RD '	'NJ RT 3 '	44	Elevated
0906156	'RR ST PAUL AVE '	'US 1+9T '	46	Elevated
1231168	'I-287 '	'RIVER ROAD CR 622 '	46	Elevated
1253164	'LEHIGH VALLEY LINE '	'OAK TREE RD(CR604)'	46	Elevated
1304151	'MILLSTONE RIVER '	'OLD ROAD (NJ 33) '	45	Elevated
1515150	'BEAVER DAM CREEK '	'NJ RT 88 '	45	Elevated
1809158	'PASSAIC RIVER '	'ROUTE US 202 '	45	Elevated
0807152	'RACCOON CREEK '	'ROUTE NJ 45 '	45	Elevated
0601152	'MENANTICO CREEK '	'N.J.ROUTE 47 '	46	Elevated
0209150	'I-95 US 1 9&46 & NJ 4 '	'US 9W '	46	Elevated

0917150	'US1&9 PATERSON PLANK RD '	'NJ 495 '	48	Elevated
1922150	'BEAVER RUN(PAULINS KILL)'	'ROUTE NJ 15 '	46	Elevated
1309150	'GRAVELLY BROOK '	'NJ 34 '	46	Elevated
1904153	'BRANCH OF WALLKILL RIVER'	'ROUTE NJ 23 '	46	Elevated
0418163	'NEWTON CREEK '	'ROUTE I-676 SB '	46	Elevated
0905150	'ROUTE US 1&9T '	'CENTRAL AVE CR 659'	48	Elevated
1516152	'NO.CHANNEL OF TOMS RIVER'	'ROUTE NJ 166 '	50	Elevated
0731157	'NJ 21 & RAMPS C&D '	'I280 '	49	Elevated
0908153	'NORTHERN SEC. & RAMP A '	'NJ RT 3 '	49	Elevated
1122150	'DOCTOR'S CREEK '	'US 130 '	53	Elevated
3000165	'DELAWARE & RARITAN CANAL'	'AMWELL RD(CR 514) '	53	Elevated
0219151	'NJ 3 WB '	'NJ 120 SB & RAMPS '	53	Elevated
1409155	'DL&WRR W.BLKWL.ST RIVER '	'RT US46 '	55	Elevated
1513153	'WEST THOROFARE U TURN '	'ROUTE NJ 72 '	63	High
2003150	'ROUTE US 22 '	'PARK AVENUE(CR655)'	63	High
1513154	'EAST THOROFARE '	'RT 72 '	64	High
0730192	'ORANGE 1ST ST. RAMP NJT'	'I-280 WESTBOUND '	68	High
1430153	'PASSAIC RIVER '	'NJ ROUTE 159 WB '	68	High
0206169	'PALSD AV WNDSR RD&CSX RR'	'NJ 4 '	69	High
2107156	'PAULINS KILL '	'US 46 '	69	High
0510152	'TUCKAHOE RIVER '	'ROUTE NJ 50 '	69	High
0725171	'US 1&9 AND RAMP 11 '	'I78 RAMPS 5&6 '	73	High
0722157	'PASSAIC RIVER '	'US ROUTE 46 EB '	74	High
1513152	'MANAHAWKIN BAY '	'ROUTE NJ 72 '	82	Severe
0818151	'BIG TIMBER CREEK '	'ROUTE US 130 '	82	Severe
0417158	'NEWTON CK KLEMM AV&CONRL'	'I-76 '	83	Severe
0103153	'DUCK THOROFARE '	'U.S.ROUTE 30 '	97	Severe

9.3 Data Sources

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9.4 Risk Assignment Worksheet Layout

The worksheets in the prioritization excel file have tabs that are color coded based upon their function. The first three tabs are blue and contain NBI database information, the first of which contains raw data and the remaining two containing processed NBI data for the purpose of the replacement cost linear regression estimate discussed in Section 7.3 – "Replacement Cost".

Following these three worksheets are worksheets that contain the global information for the prioritization process, color coded yellow. The first three tabs are hazard, vulnerability and exposure conditions and corresponding point assignments. Following these is the failure mode matrix discussed in Section 8. Following this worksheet is a sheet titled "Risk Scale Definitions", which contains the values for the ranking scale discussed in Section 6. The final yellow worksheet contains values of various uncertainty premiums.

The next section of worksheets is color coded with red tabs and contains the data and risk calculations for each bridge, for a total of 101 worksheets (100 bridges and a blank template). The name of each worksheet is the structure number of the bridge contained therein. Although an example of this worksheet is too large to show in this report, a simplified schematic showing the overall layout and organization of this worksheet is shown in Figure 19 below. Each of these worksheets contains three tables. The general information table at the uppermost left corner of the worksheet contains the structure number, routes carried and feature spanned, and the uncertainty premium. Below this table is a second larger table titled "Data Entry". This table contains the relevant data for each structure organized by limit state and risk component. Data entered in this table is automatically assigned a risk point value based on the criteria found in the "yellow" color coded worksheets discussed in the previous paragraph. All of the logic in these worksheets for point assignment, risk scale and uncertainty premiums references these "yellow" color coded worksheets, so changing the criteria value in these worksheets will automatically update the risk assignments of every bridge in the "red" color coded worksheets. The names of each of the criteria in this table contain hyperlinks to the location of the list of typical conditions and point assignments to allow quick alteration of these values without having to sift through a large spreadsheet to find the correct cell. The results of each structure's risk analysis are shown in the top right table, titled "Results". This table contains the aggregate risk for each limit state and failure mode, the maximum limit state/failure mode value and the final combined total risk and risk level. Each of these worksheets also contains a link to the "Summary and Index" Worksheet, discussed in the flowing paragraph.

The final section of the excel document contains a table with the final risk value and risk level of each bridge. This tab for this worksheet is color-coded orange. This table also contains the sufficiency rating and critical condition rating (minimum condition rating from NBI data) for each structure. The structure numbers in this table are hyperlinks to that particular structure's worksheet.

General Information	

Data Entry				
Parameters			Risk Value	Max Risk Value
Hazards	Safety: Geo/Hydra			
	Safety: Structural			
	Serviceability & Durability			
	Operations			
Vulnerabilities	Safety: Geo/Hydra			
	Safety: Structural			
	Serviceability & Durability			
	Operations			
Exposure	Safety			
	Serviceability & Durability			
	Operations			

Results				
Failure Mode	Aggregate Risk			
	Safety: Geo/Hydra	Safety: Structural	Serviceability & Durability	Operations
Mex H*V*E				
Max H*V*E*UP				
			Total Risk:	
			Risk Level:	

Figure 19: Risk Assignment Worksheet Layout:

9.5 Replacement Cost Multivariate Linear Regression

To gather a sample population of bridges to be used in formulating a linear regression equation to estimate replacement cost, NBI data was first gathered for states in the Mid-Atlantic Region. Although this originally included Delaware, Maryland, New Jersey, and Pennsylvania, the Pennsylvania data could not be used because it does not contain any data for item 94, “Bridge Improvement Cost”.

This data was then filtered to only include bridges that were structurally deficient and coded as 31 for item 75 “Type of Work”. This code corresponds to the following condition:

> “Replacement of bridge or other structure because of substandard load carrying capacity or substantial bridge roadway geometry (FHWA, 1995).”

This resulted in a sample population of 380 bridges, each with known replacement cost. To normalize this sample population, the replacement cost per square foot of deck was calculated by multiplying NBI Item 49 “Structure Length” by NBI Item 52 “Deck Width, Out-to-Out” to obtain deck area, and dividing this by item 95 “Bridge Improvement Cost” for the bridges.

Several criteria were then identified that were hypothesized to have an effect on the overall replacement cost of the bridge. This was then confirmed by calculating Pearson’s R Value for each. The criteria that were chosen for inclusion in the Multivariate Linear Regression Calculation are as follows:

- Skew
- Number of Spans
- Max Span Length
- Setting (Rural or Urban)
- Vertical Under clearance
- ADT
- Feature Spanned
- Material Type

Although the majority of the data types used in the calculation are numerical (Skew, Number of Spans, Max Span Length, Vertical Underclearance and ADT), several are categorical (Setting, ADT and Feature Spanned). For these criteria, dummy coding was used. In dummy coding, each item in a given category is given its own column. If that condition is true for a given case, a value of 1 is entered. If it is false, it is assigned a value of 0. For example, “material type” has six categories – Precast Concrete, Cast-in-place Concrete, Wood, Masonry, Steel or Other. An example of this categories’ dummy value table is shown in Table 19.

Table 19: Categorical Value Represented as Dummy Values Example

Structure Number	Material Type Dummy Cells (Categorical Variables)					
	PC Concrete	CIP concrete	Wood	Masonry	Steel	Other
200000CL0313010	0	0	0	0	1	0
020028D	0	1	0	0	0	0
2012150	0	0	0	1	0	0
1600105	0	0	0	0	1	0
200000CL0264010	0	0	0	0	1	0

To illustrate the implementation of dummy values, structure number 200000CL0313010 in the above table is primarily constructed from steel. Structure number 020028D is cast-in-place concrete, etc… After performing multivariable linear regression on a set of data that uses dummy values, a coefficient is calculated for each column. For example, the “Material Type” category would have a total of five coefficients in the final linear regression formula, one for each entry in the category.

Table 20: Multivariable Linear Regression Results

Data		Coefficient Variable	Coefficient Value
y- intercept		β_0	
Skew		β_1	0.00245
Number of Spans		β_2	-0.00385
Max Span		β_3	0.01269
Setting		β_4	0.41469
Vertical Underclearance		β_5	0.21380
ADT		β_6	-0.00002
Spanned Feature	Highway/RR/Ped	β_7	-0.21165
	Waterway	β_8	2.02736
	Waterway +1 More	β_9	0.00000
	Other	β_{10}	1.80592
Material Type	PC Concrete	β_{11}	-1.46903
	CIP concrete	β_{12}	-0.72563
	Wood	β_{13}	-1.86886
	Masonry	β_{14}	0.00000
	Steel	β_{15}	-0.54304
	Other	β_{16}	0.03764

Table 20 shows the results of the linear regression analysis. Microsoft Excel was used to calculate these coefficients. The first row in this table shows the y-intercept (β_0). The remainder of the rows are coefficients ($\beta_1 - \beta_{16}$). The coefficients for bridges spanning waterways and one other feature and bridges with masonry as their primary material type were both calculated to be zero. To calculate the final cost per square foot of a bridge with unknown replacement cost, the coefficients in Table 20 were used in the equation 1.3.

$$\frac{Cost}{ft^2\ of\ deck} = \beta_0 + \sum_{n=1}^{16} \beta_n(x_n) \qquad 1.3$$

Having calculated the cost per square foot of deck for each bridge, the total replacement cost could be calculated by multiplying this predicted value by the total square footage of deck.

The reliability of this analysis was determined by using a percentage of error calculation. The linear regression analysis was performed on 80% of the samples in the population. 20% of the samples were withheld from the linear regression calculation to be used in this reliability analysis. For this 20%, the cost per square foot was calculated using the linear regression model. A percentage of error was then calculated for each bridge using the results of the linear regression model as the experimental value and known replacement cost as the theoretical value. The average percentage of error for this 20% sample population subset was calculated to be 21%.

9.6 Reliability Index vs. importance factor scatterplot data

Bridge No.	*Reliability Index*	*Importance Factor*
2153161	3.61	3.16
3001151	4.24	3.16
0350162	3.87	3.16
2108155	3.87	2.00
1850166	4.24	4.47
2101150	3.16	3.61
2105153	5.83	3.61
1650161	4.69	2.65
1014162	4.80	4.47
1101163	3.16	3.16
1337150	6.48	3.16
1426150	4.69	3.61
2117161	3.16	3.61
0114155	3.16	3.16
1850164	4.69	3.61
1850160	3.16	3.16
1913154	5.48	4.80
3001175	6.48	4.47
3001176	6.48	3.16
1419169	6.71	4.47
0222153	5.48	3.16
1249165	9.49	3.16
0317155	7.55	3.61
0823150	3.87	4.47
1607161	7.07	2.00
1711156	7.07	2.00
0815150	7.75	3.61
2154160	10.10	3.61
1223153	7.87	3.16
0220154	8.77	4.80
3000163	3.16	4.80
0731156	7.55	4.80
0731160	5.00	3.16
1005151	6.48	3.61
0206173	5.74	4.80

3001159	6.48	4.80
1502152	4.69	3.16
0214159	3.16	3.61
1009150	3.16	3.61
1007159	5.00	4.80
0730152	6.48	4.47
1817155	7.55	3.61
0202159	10.10	4.80
3001162	5.00	4.80
1149160	5.20	4.80
1817156	9.33	3.61
1310155	9.90	3.16
0203153	9.49	3.61
0821160	9.64	3.16
0314151	9.75	2.65
1211152	9.33	3.16
1105151	9.75	3.61
0221152	6.16	3.61
0907152	7.87	3.16
3000169	4.24	3.61
0302151	6.16	3.61
1809150	4.80	3.16
2117160	11.05	3.16
2106151	6.48	3.16
2003166	6.71	4.80
0731154	6.48	3.16
1308154	3.16	3.16
1515150	9.33	3.61
2105152	5.83	3.61
1809158	6.93	4.80
0807152	4.00	4.47
1601162	7.35	4.80
1304151	6.93	3.16
1904153	6.71	4.80
0906156	5.20	4.80
1253164	7.55	4.80
0209150	6.93	3.61
1231168	9.75	3.61

0601152	8.06	3.61
1922150	7.75	3.16
1309150	7.07	4.80
0418163	11.58	3.16
0917150	10.25	3.61
0905150	8.49	4.80
0908153	7.28	4.47
0731157	9.43	4.80
1516152	9.33	4.47
3000165	9.33	4.47
0219151	6.71	4.80
1122150	9.49	3.61
1409155	9.49	4.47
1513153	9.49	4.80
2003150	10.25	4.47
1513154	9.49	3.61
1430153	10.49	3.61
0730192	9.90	4.80
0206169	12.92	3.61
2107156	11.58	4.80
0510152	11.05	4.80
0725171	11.18	4.80
0722157	11.18	3.16
0818151	3.16	4.47
1513152	7.55	3.16
0417158	12.92	4.47
0103153	4.24	3.16

9.7 A Risk-Based Process to Asign Priority for Bridge Replacement

A Risk-based Process to Assign Priority for Bridge Replacement

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ABSTRACT

The objective of this paper is to present a 'straw-person' framework that appears to be a practical first step towards a more transparent, objective, quantitative and risk-based approach to bridge assessment and prioritization. While the framework presented is qualitative in nature it has distinct advantages over the current approach in that (a) it explicitly recognizes key performance limit states, (b) directly addresses bridge hazards, vulnerabilities, and exposures, (c) incorporates the uncertainty associated with various assessment techniques and provides flexibility for their implementation, and (d) provides a means to capture (in a useable format) expert knowledge and heuristics from top bridge engineers. In addition to the straw-person framework, the paper presents a rudimentary classification system to illustrate one approach to implementation. A series of case studies are then presented to demonstrate the potential value of this approach in distinguishing between bridges that are essentially "equivalent" based on the current assessment approach. In addition, these case studies also serve to illustrate that how the proposed approach may be utilized with existing inspection data. The paper concludes with some observations and comments regarding the straw-person framework presented.

Keywords: Bridge assessment, bridge prioritization, risk, uncertainty

1. INTRODUCTION

Current bridge owners and stewards are facing unprecedented challenges related to aging bridge inventories and budget shortfalls as well as pressures resulting from increased public scrutiny. Fueled by the tragic collapse of the I-35W Bridge, the U.S. citizenry and the federal legislature are becoming increasingly interested in the manner in which bridges are assessed and prioritized. This interest and attention highlighted significant shortcomings in current practice, which was driven by the collapse of the Silver Bridge over the Ohio River in 1967 and has shaped the current practice of bridge inspection and condition evaluation. For example, the current approach focuses on assessing condition and implicitly ignores any issues related to the inherent vulnerable of the bridge, such as those that incorporate poorly performing details such as pin and hangers. Further, this approach does not comprehensively consider hazards (or threats) to bridges and how these hazards may mobilize the vulnerabilities, and it completely ignores the consequences associated with the resulting failure. There has been significant progress in bridge engineering in the last several decades that allow a much better evaluation of the actual risks, related to several performance limit states, associated with the large populations of "Structurally Deficient" and "Functionally Obsolete" bridges.

We note that the 110th U.S. Congress considered a "National Highway Bridge Reconstruction and Inspection Act" which among other things would have required states to "assign a risk-based priority for… bridge[s] after consideration of safety, serviceability, and essentiality for public use." Similar provisions are expected in Bills for transportation funding by the 111th Congress. In addition, a September 2008 report by the Government Accountability Office (GAO) found that the Nation Bridge Program (NBP) lacks clear goals, performance measures, and does not properly define the federal interest in bridges. While this is indirectly related to bridge assessment, overcoming these identified shortcomings requires a rational and consistent approach to prioritizing bridges. In addition, it is important that any advances related to bridge assessment explicitly recognize that Asset Management (AM) principles are becoming increasingly important to the management of public infrastructures, which was first highlighted by the US General Accounting Office in 1997 (Thompson 2004). To ensure compatibility with current and future AM developments, it is important that information developed through the envisioned bridge assessment procedure: (1) recognize the diverse set of performance limit states relevant to management decisions, and (2) be readily incorporated within risk-based decision-support tools.

Given these drivers, the need for a more transparent, objective, quantitative and risk-based approach to bridge assessment and prioritization is needed. While this will likely take some time to fully mature, the authors believe there is great merit in discussing a pragmatic approach to initiating this transition in the near-term. Towards that end, the objective of this paper is to present what the authors believe may be a pragmatic first step towards improved assessment and prioritization of bridges. The risk-based assessment framework discussed herein is intended to serve as no more than a straw-person to initiate a dialog as to what form the initial stages of this transformation may take. While this framework remains highly qualitative and subjective in nature, it has the advantages of requiring very limited changes on the actual practice of bridge inspections and thus is can be implemented for most bridges using current inspection data. In addition, the proposed approach was formulated with the following four key elements:

1. Inclusive of Relevant Bridge Performances: Bridge Performance in the 21st Century is a far more complex concept than just structural safety and serviceability. This is particularly true when trying to inform an AM system, where operational or maintenance cost-related performances are key aspects of valuing trade-offs among various asset groups. The framework discussed herein adopts four key performance limit states: (1) Safety: Geotechnical/Hydraulic, (2) Safety: Structural, (3) Durability, Serviceability and Maintenance, and (4) Functionality – including the Costs associated with operation, evaluation, maintenance and repair. It is expected that as this assessment procedure matures, additional performance limit-states may be included and some of these performance limit states may be sub-divided to allow for a higher resolution assessment.

2. Risk-Based: While the use of reliability theory (aimed at quantifying the probability that a bridge will fail to perform) is important, it fails to explicitly recognize the various levels of exposure (or consequences of not performing) for various bridges. In a risk-based approach the reliability concept is extended to explicitly include the consequences associated with a lack of performance of various bridges (route criticality, network redundancy, ADTT, replacement costs, historic nature, etc.), which is a key factor that should be explicitly included when selecting appropriate inspection techniques/intervals and allocating funds. This approach is in line with the views of law-makers, and may help alleviate some of the criticisms the GAO had of the HBP. In addition, the ability to assess the risk of not- performing adequately for each of the performance limit states discussed in (1) can be directly used by the risk-based decision-support tools that will no doubt be a part of future AM systems.

3. Explicit Consideration of Uncertainty: Assessment procedures have wide-ranging accuracies, resolutions, and reliabilities. Even individual assessment approaches, depending on how they are applied, have varying levels of uncertainty. For example, some states require at least one PE and E.I.T. for in-depth inspections; while other states require only minimal training and no formal education (see Phares et al. (2004) for examples of highly variable inspection results). We also note that some bridge owners have more effectively adopted analytical (FEM) and experimental (NDE, load testing, structural monitoring, etc) technologies that lead to more objective data which they incorporate in bridge management. Given the different cultures, histories and budget short-falls of states, it may not be realistic or effective to mandate highly-restrictive standards. Instead, the vastly different levels of uncertainty associated with different assessment techniques should be explicitly considered. For example, different in-depth inspection procedures (and requirements, such as the use of nondestructive evaluation techniques) should be treated differently with, for example, a set of weighing factors that directly reflect the underlying uncertainty. This approach not only provides decision-makers with a more complete picture of the uncertainty associated with various assessment procedures, it promotes (and rewards) the use of more reliable approaches while still providing states some freedom regarding implementation.

4. Based on Expert Knowledge: Although it would be ideal to be able to reliably estimate actual risks (in probability of losses per year) associated with various bridges, such an approach requires a far better understanding of bridge performance and far better data than is currently available. The federal government has recognized this and has recently funded the Long-Term Bridge Performance (LTBP) Program that specifically aims to acquire this type of quantitative data over the next 20 years. In the interim, the authors believe there is great merit in developing a rational approach to estimate "relative" risk and thus greatly enhance our ability to identify efficient and effective inspection technique/intervals and allocations of funds. Fortunately, the LTBP Program, through their contacts with the AASHTO T-18 Committee on Bridge Management, Evaluation, and Rehabilitation, and other experienced bridge engineers around the country, has identified a number of highly experienced engineers that are implicitly using some form of risk-based assessment and prioritization. For example, see the New York State DOT (2008) bridge inspection

manuals. It is the aim of the proposed assessment framework to be a vehicle that can eventually reflect the wisdom and experiences of top bridge engineers and eventually merge this expert knowledge with quantitative and detailed performance data that is currently being generated (e.g. LTBP Program).

2. RISK-BASED ASSESSMENT METHODOLOGY

The envisioned bridge assessment standard will be based on the concept of risk, which extends the reliability-based assessment approach to explicitly consider the consequences of not performing (i.e. exposure). The inclusion of consequences is a necessary and fundamental consideration for making rational decisions, and thus it is imperative that it be included within the assessment procedure. Consider that, on average, approximately 500 bridges fail in the U.S. each decade (Wardhana and Hadipriono 2003). Recognizing that it is not possible to eliminate all such failures, the goal has to be to ensure that the most unacceptable failures do not occur. Such 'unacceptable' failures no doubt result in the loss of human life and other undesirable impacts on commerce and the economy as a whole. This line of argument leads directly to the need to explicitly incorporate exposure, and thus to adopt a risk-based assessment and prioritization approach. In most cases risk is defined as the product of probability of an event occurring and the consequences associated with the event. For the proposed framework a more 'partitioned' definition is desirable:

Risk (H) = (Hazard) (Vulnerability) (Exposure) (Uncertainty Premium)

Hazard = the probability of a hazard, H occurring; p(H)

Vulnerability = the probability of failure (to perform adequately), given hazard, H; p(f |H)

Exposure = consequences associated with a failure to perform adequately

Uncertainty Premium = a factor to account for the level of uncertainty associated with the selected assessment approach, including the quality control measures employed.

Table 1 outlines some example hazards, vulnerabilities and exposures for the four performance limit states to be considered by the envisioned risk-based assessment approach. To prioritize relative risks, it is useful to develop a discrete scale. At this point it is envisioned that the five-level risk scale used by the Department of Homeland Security may serve, as it is not overly complicated and is intuitively understood by engineers as well as the US citizenry (Figure
1). In addition, this type of scale provides a much larger resolution than the current Structurally Deficient and Functionally Obsolete designations. However, given the lack of technical understanding by the public and media related to risk assessment, there is a potential danger that this scale may raise unfounded concerns (as with the term Structurally Deficient). As a result, it may be more prudent to adopt a generic (Level I through Level V) scale. Regardless of which scale is ultimately adopted (with input from all stake-holders), the proposed risk-based assessment approach, by its inclusion of vulnerabilities, hazards, exposures and uncertainties, has the ability to convey a transparent and rational appraisal of performance to both non-technical (public, media, government) and technical audiences.

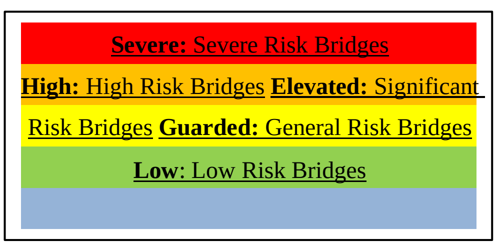

Figure. 1. Envisioned Risk Scale

3. RISK-BASED ASSESSMENT FRAMEWORK

The purpose of this section is to describe the envisioned framework and put forth a 'straw-person' version to illustrate the potential value of the proposed approach. It is stressed that this framework is very rudimentary and needs to be refined based on expert elicitation, input from the many relevant professional organizations and committees, as well as the on-going data collection efforts such as the LTBP Program prior to being appropriate for use. Figure 2 shows a diagram that illustrates the proposed framework.

Table. 1. Summary of relevant performance limit states, hazards, vulnerabilities and exposures for bridges.

Performance Limit States	Hazards	Vulnerabilities	Exposures

Safety: Geotechnical/ Hydraulic	• Flowing water
	• Seismic
	• Vessel Collision
	• Flood
		• Scour/Undermining
		• Loss of support
		• Soil liquefaction
		• Unseating of superstructure
		• Settlement
		• Overtopping
			• Loss of human life
			• Replacement and repair costs
			• Impact of removal from service related to:
			  • Safety – life line,
			  • Economic
			  • Social – mobility
			  • Defense

Safety: Structural	• Seismic • Repeated loads • Trucks and overloads • Vehicle collision • Fire	• Lack of ductility and redundancy • Fatigue and fracture • Overloads • Details and bearings	

Serviceability, Durability and Maintenance	• Climate
	• Intrinsic Loads
	• Impact (Vertical)
	• Environment
		• Corrosion
		• Cracking/spalling
		• Excessive deflections/ vibrations
		• Chemical attacks/reactions
		• Difficulty of maintenance
			• User costs
			• Maintenance costs
			  • Direct
			  • Indirect – delays, congestion, etc.

Functionality and Cost	• Traffic • Special traffic and freight demands	• Network redundancy and adequacy • Geometry and roadway alignment	• Loss of human life and property (accidents) • Economic and social impacts of congestion

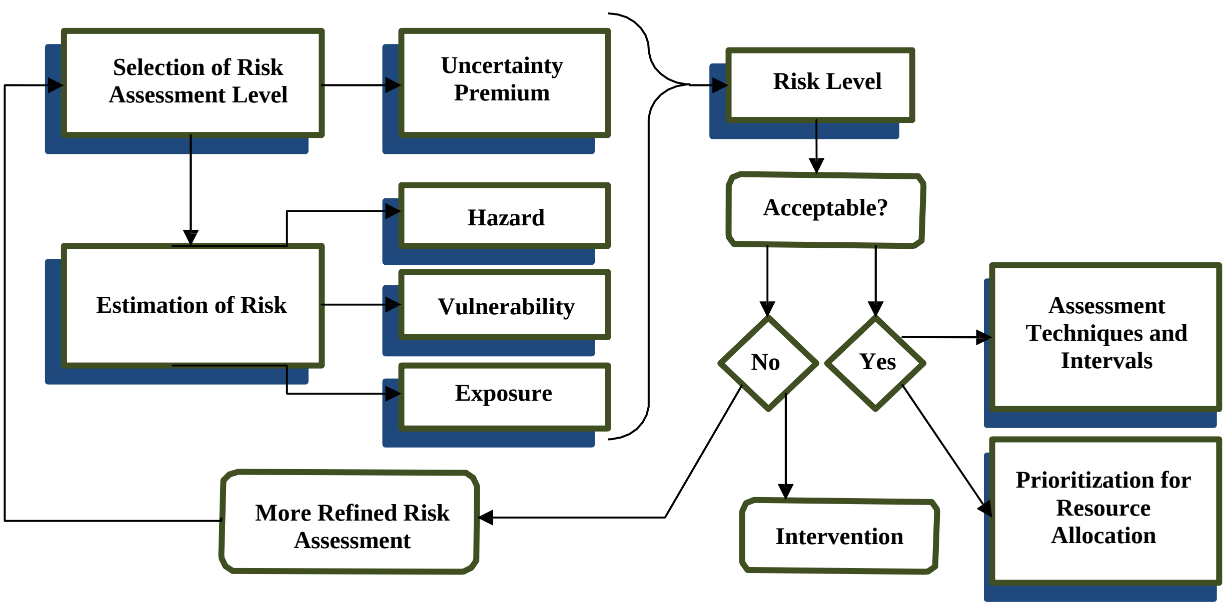

Figure. 2. Envisioned risk-based assessment framework

The overall process begins with the selection of the depth of risk assessment and the specific approaches to be utilized. Table 2 shows five example assessment levels and their potential uncertainty premiums, which reflect their accuracy and variability. For Levels 1 and 2, the risk would be computed in an aggregate manner, i.e., the aggregate risk associated with each performance limit state is computed. While this makes for a very efficient application, in some cases it may drastically over-estimate the actual risk level. Consider the case where a bridge has a high seismic vulnerability, low scour vulnerability and is exposed to a high scour hazard and a low seismic hazard. The resulting probability of failure would be low since a scour hazard would not "mobilize" the seismic vulnerability. However, when risks are computed in an aggregate sense, this mismatch between hazard and vulnerability is ignored and the risk associated with this bridge would be over-estimated. As a result, more refined risk assessment approaches (Levels 3-5) would have to be developed where the risks associated with each of the specific hazards given in Table 1 are assessed separately.

Table. 2. Risk assessment levels

Level	Example Approaches	Resolution	Quality Assurance	Uncertainty Premium
1	Visual Insp., Doc. Review	Aggregate Risks	Min Standards	2.5
2	Visual Insp., Doc. Review	Aggregate Risks	Best Practices	2.0
3	Visual Insp., Doc. Review, Anal. Tech.	Individual Risks	Min Standards	1.5
4	Visual Insp., Doc. Review, Anal. Tech.	Individual Risks	Best Practices	1.25
5	Visual Insp., Doc. Review, Anal. and NDE Tech.	Individual Risks	Best Practices	1.0

In addition, the assessment levels also reflect the specific approaches and technologies employed. Since the NBIS was initially developed, a series of analytical and experimental technologies that can reduce the uncertainty associated with assessment activities have become available. Further, there are a wide range of successful quality assurance programs that have been developed. To recognize their influence and benefits, assessment levels that take advantage of these developments will have a lower uncertainty premium associated with them. In this manner, states will retain freedom to choose from a wide range of assessment approaches, but the standards will explicitly recognize the inherent differences in the resulting uncertainty, and thus will promote the use of best practices and proven technology.

Tables 3-5 provide an illustration of how hazard, vulnerability and exposure may be quantified for Level 1 and 2 assessments. In this case, the risks are aggregated in four categories: Safety – Geotechnical/Hydraulic; Safety – Structural; Serviceability, Durability, and Maintenance; and Operational and Functional. For each of these categories, the hazard, vulnerability and exposure is given a value of 1-3 based on location, structural and operation attributes, age, etc. In the case of Individual Risk Assessments (Levels 3-5), these categories would be further divided to allow the risks associated with each individual hazard to be assessed independently. The aggregate risks are then computed as shown in Equation 1; and combined by square-root-sum-of-squares to develop the Risk Level. A preliminary scale of Risk Levels is shown in Table 6.

We note that there is a special need to clarify how various structural systems and details may impact the risk due to structural failure under live loads. While the public may be more forgiving of bridge failures during natural hazards such as earthquakes and floods, casualties due to the collapse of a highway bridge under routine operational loads become "focusing events" creating significant societal reaction. The bridge engineering community should therefore become especially careful and precise in identifying those structural systems and details that are susceptible to sudden system- level failure and prioritizing the correction of such deficiencies. The ambiguities in the current practice related to identifying which bridge systems are fatigue-sensitive and which systems are fracture-critical has been discussed in NCHRP Synthesis 354 (2005). There is a need for reliable analytical modeling and nonlinear analysis capabilities for steel and concrete bridges to reliably predict their failure modes and post-failure response. We should be able to understand the differences between various failure modes and post-failure responses - and especially the stability of failure - of various bridge systems and details, so that we can prioritize bridge funds to replace those bridges which have a higher risk of failing in a highly objectionable, sudden, system-wide collapse. As the 2005 failure of a PC overpass bridge over I70 in Washington CO, PA, and the 2008 failure of the I-35 Bridge in Minneapolis, MN revealed, highly objectionable bridge failures under operational loads are not limited to just steel or just concrete bridges. Meanwhile, the writers have tested many aged and deteriorated reinforced concrete bridges which proved to have immense reserves of load capacity in spite of extensive deterioration and damage. Obviously, we need to improve the education and practice of structural engineering of bridges to be able to better classify and prioritize bridges based on risk.

Table. 3. Preliminary hazard values for Level 1 and 2 risk assessments

Hazards Considered		Hazard Values		
		1	**2**	**3**
Safety: Geo/Hydraulic	Scour; Vessel Collision; Seismic - Liquefaction; Settlement; Flood	•Outside of a 500 yr flood plain; •Seismic Design Category A; •Over a non-navigable channel; •Located more than 500 miles from coast; •No underwater substructure; •No records of significant earthquake, floods or storm surge;…	•Outside of a 100 yr flood plain •Seismic Design Category B, C •Navigable channel for mid-sized vessels •Located more than 50 miles from coast •Substructure subjected to low to moderate flow rates • Records of moderate earthquake, floods or storm surge;…	•Within of a 100 yr flood plain; •Observed drift and debris at piers/abutment; history of ice flows in waterway; •Seismic Design Category D, E, F; •Navigable channel for large vessels; •Located within 50 miles from coast; •Substructure subjected to significant flow rates; •Records of significant earthquake, floods or storm surge;…
Safety: Structural	Seismic; Fatigue; Vehicle Collision; Overload; Fire	•Seismic Design Category A; •ADTT less than 500; •Not spanning over a roadway; •Located more than 10 miles from heavy industry; •No history of overloads, collision, earthquake;…	•Seismic Design Category B, C; •ADTT less than 10,000; •Spanning over a roadway with ADTT less than 1,000; •Located more than mile from heavy industry; •History of isolated overloads, collision, and moderate earthquakes;…	•Seismic Design Category D, E, F; • ADTT more than 10,000; •Spanning over a roadway with ADTT more than 1,000; •Spanning a rail line; •Located less than mile from heavy industry; •History of repeated overloads, collision, and significant earthquakes;…
Serviceability and Durability		•No routine use of deicing salts; •Located more than 100 miles from the coast; •Low number of freeze-thaw cycles; •No history of overloads; …	•Moderate usage of deicing salts; •Located more than 25 miles from the coast; •Moderate number of freeze-thaw cycles; •History of overloads; …	•Moderate usage of deicing salts; •Located more than 25 miles from the coast; •Moderate number of freeze-thaw cycles; •History of overloads; …
Operations		•ADTT less than 1,000 and ADT less than 10,000; •No history of fatal accidents; •No history of congestion; …	•ADTT less than 10,000 and ADT less than 50,000; •History of isolated fatal accidents; •History of moderate congestion; …	•ADTT less than 10,000 and ADT less than 50,000; •History of isolated fatal accidents; •History of moderate congestion; …

Table. 4. Preliminary vulnerability values for Level 1 and 2 risk assessments

Vulnerabilities Considered	Vulnerability Values		
	1	2	3
Safety: Geo/Hydraulic	•Founded on deep foundations or bedrock; •Meets current pier impact and scour protection standards; •No history and no evidence of scour or settlement; …	•Founded on shallow foundations on cohesive soil; •Evidence of minor scour/undermining during past/present underwater inspections; •Pier protection system in good condition; •Superstructure above 100 yr flood level; •Minor tilt of substructure elements; …	•Founded on shallow foundations on non-cohesive soil; •Evidence of moderate to significant scour/undermining during past/present underwater inspections; •Pier protection system missing or in poor condition; •Superstructure below 100 yr flood level; •Significant tilt of substructure elements; …
Safety: Structural	•Meets all current design specs; •Structure displays bi-directional redundancy; •20 years or less since construction or major renewal; •A and B fatigue details; •No evidence of structural damage; •No history of excessive displacements or vibrations; …	•Simply-supported constructed with transverse distribution capabilities; •50 years or less since construction or major renewal; C and D fatigue details; •Minor evidence of structural damage within the critical load path; •Clearance within 6 in of current standard; •History of significant displacements or vibrations; •Substructure elements within 10% of plumb…	•Non-composite construction; •Simply-supported construction with minimal transverse distribution capabilities; •50 years or more since construction or major renewal; •E and E' fatigue details; •Rocker bearings; •Intrinsic force dependency; •Exposed prestressing strands; •Pin and hanger details; •Evidence of structural damage within the critical load path; •Clearance below current standards; •History of excessive displacements or vibrations; …
Serviceability and Durability	•No visible cracks; •No evidence of reinforcement corrosion; •Elastomeric bearing; •Joints in good operating condition; •Paint in good condition; •Suppers are less than 10% clogged …	•Minor local cracking; some evidence of reinforcement and structural steel corrosion; •Joints with minor evidence of leaking; •Approach displays minor rutting; •Scuppers are between 10-50% clogged …	•Extensive cracking and spalling; •Evidence of wide-spread reinforcement and structural steel corrosion; •Exposed prestressing strands; •Frozen bearings; •Failed expansion joints; •Approach displays significant rutting; •Scuppers are between 50-100% clogged…

Operations	•Roadway geometry up to current standards; •Guard rail and road paint in good condition; •Breakdown lane/ shoulders; …	•Lane width within 1 ft of current standards; •Guard rail and road paint in fair condition; •Posted for more than 90% of legal truck weight; •Breakdown lane/ shoulders not present; •Minor rutting of pavement …	•Lane width more than 1 ft less than current standards; •Guard rail and road paint in poor condition; •Posted for less than 90% of legal truck load; •Breakdown lane/ shoulders not present; •Significant rutting of pavement…

Table. 5. Preliminary exposure levels for Level 1 and 2 risk assessments

Exposure Considered	Exposure Values		
	1	**2**	**3**
Safety: Geo/Hydaulic **Safety: Structural**	•ADT less than 10000; •Replacement cost less than \$2 million; •Not on a critical route (life line, evacuation route); •Detour route less than 5 miles;	•ADT less than 50000; •Replacement cost less than \$10 million; •Not on a critical, non-redundant route (life line, evacuation route); •Detour route less than 10 miles;	•ADT more than 50000; •Replacement cost more than \$10 million; •On a critical, non-redundant route; •Detour route more than 10 miles;
Serviceability and Durability	•Low maintenance costs; •ADT less than 50,000	•High maintenance and repair costs; •ADT more than 50,000	Not Applicable
Operations	•No history of congestion; •ADT less than 25,000; •ADTT less than 10,000	•Average peak hour delays of more than 10 min; •ADT more than 25,000; •ADTT more than 10,000	Not Applicable

Table. 6. Preliminary Risk Levels

Risk Level	Threshold Risk Values
Severe: Severe risk bridges	>40
High: High risk bridges	30-40
Elevated: Significant risk bridges	20-30
Guarded: General risk bridges	10-20
Low: Low risk bridges	<10

To translate the Risk Level into appropriate assessment techniques and intervals, a set of minimum requirements and optional assessment programs is needed. A preliminary estimate of this relationship is shown in Table 7. Again, it is emphasized that this proposed framework for assigning a relative risk to a specific bridge is conceptual and presented as a strawman. The levels of acceptable risk that would trigger more refined risk assessment and relative values of quantification of uncertainty would need to be 'calibrated" based on many case studies and expert solicitations.

Table. 7. Preliminary assessment programs per Risk Level

Risk Level	Mandatory	Option 1	Option 2
Severe	Level 3 / Year	Level 4 / 18 months	Level 5 / 2 years
High	Level 2 / Year	Level 4 / 2 years	Level 5 / 3 years
Elevated	Level 2 / 2 years	Level 4 / 3 years	Level 5 / 4 years
Guarded	Level 1 / 2 years	Level 4 / 4 years	Level 5 / 6 years
Low	Level 1 / 2 years	Level 4 / 4 years	Level 5 / 6 years

4. ILLUSTRATIVE CASE STUDIES

To illustrate the application of the envisioned risk-based assessment procedure, three bridges were used as case studies. The case studies are composites of existing bridges, with different elements and conditions taken from different similar bridges. In these examples, all of the bridges had condition ratings for their major elements between 5 (fair) and 7 (good) and thus would not be considered a source of concern to owners, in comparison to bridges rated 4 (poor) or worse. For all these examples, the initial risk assessments utilized an uncertainty premium of 2.0, which corresponds to a visual inspection for an agency/state that would utilize best practices.

The risk assessments shown in the following sections were carried out using inventory information and information contained in the previous inspection reports, which indicates that little impact on actual inspection procedure may be required to transition to a more rational risk-based approach. This may prove especially useful during implementation as it indicates that states will be able to develop initial risk assessments and re-prioritize bridges even before the next inspection is carried out.

4.1 Example 1 - Viaduct Bridge

This bridge was constructed in the 1960s and consists of multiple welded steel plate girders with cross-frame diaphragms (Figure 3a). The 13 continuous and simple spans carry 60,000 ADT (20% ADTT) over water and roadways with moderate flow rates (25,000 ADT, 20% ADTT). The spans are supported by hammerhead piers founded on piles. The bridge is located near heavy industry in a major metropolitan area with frequent congestion and a history of flooding. Based on a review of the inspection reports, the following condition information was found:

- Deck – 6: Moderate to heavy wear typical in the wheel paths. Spalls located throughout, mainly in center lanes, some with exposed reinforcement. Some are patched and a few have deteriorated and cracked patches with exposed reinforcement. Span 5 has a deteriorated patch with exposed reinforcement. Hairline transverse cracks are typical in all spans.
- Superstructure – 6: The girders exhibit moderate corrosion and paint failure throughout. A 20 ft. length at the 2nd diaphragm row from Pier 5 (possible previous fire damage, no evidence of sagging was observed) girders, diaphragms, and underside of deck is blackened and soot covered. Most girder ends and bearing stiffeners below the deck joints have areas of light to severe spot rust, no section loss observed.
- Substructure – 6: Piers exhibit large spalls with exposed and rusted reinforcement. Several vertical hairline cracks typical, typically full height. Moderate accumulation of debris and bird droppings observed on pier caps full width. Bearing pedestals have fine vertical cracks in the front face along anchor bolt line.
- Serviceability: Scuppers along right barrier are all 100% clogged. Expansion Joints all show deteriorated or missing seal material full width.
- Operations/Maintenance: Bituminous approach roadways exhibit large depressed and deteriorated bituminous patches. Traffic safety features are standard and in good condition.

Table. 8. Summary of relative risk for the viaduct bridge

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	2	1	3	2	12
Safety: Structural	3	2	3	2	36
Serviceability and Durability	2	2	2	2	16
Operations	3	2	2	2	24
				Total Risk	48 (SEVERE)

4.2 Example 2 – Former Bascule Bridge

This bridge was constructed in the 1930s and consists of two built-up girders that form a double-leaf bascule bridge. In the 1980s the bridge was locked in the closed position with shear locks at midspan and the girders were strengthened with cover plates and the floor beams were strengthened with web plates. The single span carries 1,850 ADT over a navigable waterway. The span is supported by hollow concrete caissons founded on piles. Based on a review of the inspection reports, the following condition information was found:

- <u>Deck – 7:</u> The open grid steel deck is in good condition.
- <u>Superstructure – 5:</u> The main girders are in fair structural condition due to the extent of the corrosion and degree of section loss noted throughout the girders. Severe delamination and section loss typical for most fastener nuts and rivet heads. Minor to moderate corrosion typical. The midspan pins on both the east and west girders were in good condition. Rust staining emanating from in between the pins and from the nuts at both pins. Cracks in the tack welds that join the east girder web and pin plates still exist.
- <u>Substructure – 5:</u> Abutments exhibit large spalls with exposed and rusted reinforcement. Several vertical hairline cracks typical, typically full height. Interior of the pier exhibits areas of cracking and honeycombing.
- <u>Serviceability:</u> Corrosion on and between members is moderate to severe. Rivet and bolt heads exhibit more than 40% section loss. North abutment roadway dam is in poor condition, with noted cracked butt weld at centerline of the roadway. Pins and shear lock are corroded
- <u>Operations/Maintenance:</u> The bridge railings were in poor condition. Noted vibrations in structure when trucks transit the bridge. Traffic safety features are not standard. Approach slabs are deteriorated, with cracking and spalls. Bituminous approach roadways are in fair condition with longitudinal and transverse cracks.

Figure. 3. Photos of (a) viaduct bridge, (b) former bascule bridge, and (c) prestressed spread box girder bridge

Table. 9. Summary of relative risk for the former bascule bridge

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	2	2	1	2	8
Safety: Structural	2	3	1	2	12
Serviceability and Durability	2	2	1	2	8
Operations	1	2	1	2	4
				Total Risk	17 (GUARDED)

4.3 Example 3 – Prestressed Spread Box Girder Bridge

This bridge was constructed in the 1987 and consists of a series of three-span continuous prestressed spread box girders and a composite concrete deck. This bridge carries 250 ADT over a non-navigable waterway. The spans are supported by wall piers and spill through abutments founded on piles. Based on a review of the inspection reports, the following condition information was found:

- Deck – 6: The deck underside was generally in good condition.
- Superstructure – 7: The prestressed box girders are in very good condition with no defects noted.
- Substructure – 7: The concrete pier walls were typically in good condition. The concrete abutments were in good condition, with areas of minor spalling noted along the top corner of the bearing seats and minor random hairline cracking with efflorescence noted in the backwalls.
- Serviceability: The expansion joints between the bridge deck, abutment backwalls, and approach slabs were typically clogged with dirt and gravel. Several of the scuppers were slightly clogged. Both the east and west approach slabs exhibited longitudinal 1/16” wide cracks along the center of each lane.
- Operations/Maintenance: The bridge roadway striping was typically in fair condition along the bridge deck, with minor cracking and some section loss of the reflective material. The concrete parapets were in good condition, with only some minor areas of light impact damage.

Table. 10. Summary of relative risk for the prestressed spread box girder bridge

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	1	2	1	2	4
Safety: Structural	1	1	1	2	2
Serviceability and Durability	1	2	1	2	4
Operations	1	1	1	2	2
				Total Risk	6 (LOW)

5. CONCLUDING REMARKS

The objective of this paper is to present a ‘straw-person’ framework to serve as a practical first step towards a more transparent, objective, quantitative and risk-based approach to bridge assessment and prioritization. While the framework presented is qualitative in nature it has distinct advantages over the current practice in that: (a) it explicitly recognizes key performance limit states, (b) directly addresses bridge hazards, vulnerabilities, and exposures, (c) incorporates the uncertainty associated with various assessment techniques and provides flexibility for their implementation, and (d) provides a means to capture (in a useable format) expert knowledge and heuristics from top bridge engineers. Based on the study reported herein, the following conclusions can are drawn:

1. The proposed approach provides a qualitative framework to estimate the vulnerabilities, hazards and exposures of bridges along with the uncertainty associated with the specific assessment approach used. This has distinct advantages over the current condition assessment procedures in that it can readily incorporate expert knowledge and that it provides a more complete picture of the risk associated with a specific bridge, albeit in a relative sense.
2. Although the data needed to develop a quantitative risk-based assessment and prioritization approach is not currently available, there appears to merit in developing a pragmatic approach to estimating relative risks (even if these estimates are carried out in a qualitative and subjective manner).
3. Each case study evaluation required approximately 15 minutes to develop, which demonstrates the simplicity of the proposed rudimentary risk assessment approach. While more refined risk assessment approaches (Levels 3-5) will have to be developed and will naturally be more involved, the proposed framework would only promote the use of these methods for bridges with significant risk, for which the more refined procedures are appropriate.
4. Each evaluation was performed utilizing a moderately detailed to in-depth inspection report and the Structure Inventory and Appraisal (SI&A) data. It is the intent that such lower level assessments would have little impact on the inspector and little impact on the inspection process. No additional time onsite or data collection would be required than what is normally collected in a routine or in-depth inspection.
5. As demonstrated by the case studies, the method provides a means to distinguish between bridges that currently fall within the same general condition (5- (fair) to 7- (good)). The framework is also capable of indicating whether the risk for a particular bridge is predominately hazard-related, vulnerability-related, or exposure- related, which could direct the recommended action toward reducing risk.

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Intelligent Infrastructure Systems

APPENDIX 2ABC
HPC PERFORMANCE REPORT

RU435056-3

Performance of High Performance Concrete in New Jersey
A Comprehensive Study

Technical Memorandum
April, 2014

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16. Abstract

In New Jersey, high performance concrete (HPC) has become a standard in bridge construction and rehabilitation projects. However, bridge decks built with HPC exhibit cracking, particularly at an early age, which may affect their durability. The current research project was aimed at finding possible causes for cracks as well as providing recommendations to limit them. Overall, 10 bridge decks, 8 existing and 2 new, were selected based on their exposure, geometry, age, composition and current condition for a comprehensive investigation. Internal degradation and potential corrosion activity measurements were performed on existing bridge decks by means of different NDE technologies (impact echo, electrical resistivity, ultrasonic surface waves, half-cell potential). Materials characterization testing was carried out on all 10 bridges and featured the determination of mechanical properties (compressive and tensile strength, modulus of elasticity), physical properties (shrinkage, thermal expansion, spacing factor), transport properties (diffusion coefficient, volume of permeable voids, permeability), chloride contamination, and petrographic examinations. Also, both new decks were instrumented to measure internal temperature and strains during the first months after casting. Generally, the NDE results and materials characterization indicated that the existing bridges were in good condition and showed little signs of widespread deterioration or corrosion activity. At the time of the investigation, the chloride contamination has rarely exceeded the critical value for corrosion initiation and cracks do not have a consistent influence on chloride diffusion. On new decks, calculations based on measured concrete properties and instrumentation data have shown that tensile stresses could exceed the tensile strength. Numerical calculations were performed to determine chloride exposure and compare the durability of decks built with uncracked HPC, cracked HPC (worst-case) and Class A concrete. As expected uncracked HPC decks exhibit the best durability. In some cases Class A concrete can perform as well or better than cracked HPC in the first decades after construction. This highlights the fact that HPC specifications for bridge decks must be driven by the need for accrued durability instead of relying on high mechanical properties. Improving the transport properties and reducing the cracking tendency would achieve this objective.

17. Key Words	18. Distribution Statement
HPC, performance, durability, instrumentation, deck performance, NDE, cracking, transport properties, chloride exposure	

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1. Introduction

Higher concrete strengths often lead to increased cracking in bridge decks. According to several reports, high performance concrete (HPC) decks tend to develop full depth, transverse cracks and partial depth longitudinal cracks within a few months of the concrete being placed. The primary reason is the fact that the creep of higher strength concrete is lower than of a moderate strength concrete, as tensile stresses develop due to restrained drying shrinkage and thermal contraction. The developed cracks facilitate penetration of moisture and salts, which can in some cases locally accelerate rebar corrosion and concrete deterioration.

Over the last twenty years, the improvement of mixture design methods, the widespread use of supplementary cementing materials, and the development of highly effective plasticizing admixtures have contributed to increase the overall performance of cement-based materials and led to the customary use of HPC. In New Jersey, as in many other jurisdictions, a large number of bridge decks has been built or rebuilt using HPC instead of ordinary concrete (or Class A concrete according to NJDOT classification) during that period.

Despite their superior mechanical properties and improved durability, HPC mixtures can be more sensitive to early-age cracking. The characteristically low water-binder ratio and high cement content of HPC tend at the same time to increase the rate and the intensity of early-age shrinkage, to increase the elastic modulus and reduce the creep potential, making the material more susceptible to the development of harmful internal stresses during the early stages of its life. The problem can be exacerbated by the simultaneous occurrence of thermal stresses.

Actually, NJDOT officials report that numerous HPC decks have exhibited cracking not long after casting. The NJDOT developed a remedial procedure, which involves the application of a methyl methacrylate (MMA) sealer on the surface of the deck. One of the main expected advantages in using HPC is the increased durability and lower resulting maintenance cost. It remains to be demonstrated that the MMA application restores the initial durability of the concrete and the resulting durability of a cracked and sealed HPC deck might not justify the premium related to the use of HPC and sealer application.

Problem Statement and Research Objectives

NJDOT expressed interest in research focused on evaluating the performance of HPC. The following report documents three research tasks that were synthesized to provide the current state of performance of HPC in New Jersey. The report includes nondestructive evaluation of ten bridge decks; durability analysis of ten bridge decks; and modeling, instrumentation and analysis of two bridge decks during construction. Ultimately, results of this study will be used to develop recommendations on the use of HPC, propose methods to avoid premature cracking of bridge decks and, if required, identify ways to extend the service-life of cracked HPC decks.

The following paragraphs describe the objectives of each task.

Nondestructive evaluation of bridge decks

A more objective condition assessment of bridge decks, than one relying solely on visual inspection, can be made by a complementary use of nondestructive evaluation (NDE) techniques. The condition assessments in this study are based on three main components: assessment of corrosive environment and corrosion processes, concrete degradation assessment, and assessment with respect to deck delamination. The NDE technologies used in the assessment included: half-cell potential (HCP), electrical resistivity (ER), ultrasonic surface waves (USW), ground penetrating radar (GPR), and impact echo (IE) method. Each of the five techniques has its advantages and limitations. However, each of them can contribute to a more comprehensive assessment of the condition of a deck. In addition, since the data obtained from NDE surveys are quantitative, a more objective condition rating of bridge decks was made. Different condition-rating schemes were applied in the study.

Durability analysis

The objectives of the durability analysis are to compare the costs and benefits of using HPC for the construction of durable bridge decks exposed to de-icing salts; to determine the causes of the early-age cracking in HPC bridge decks; and to evaluate the impact of the observed cracking on the long-term performance of decks. This part of the study included the following:

- On-site documentation of the cracking patterns
- Core sampling in existing decks;
- Concrete sampling in new decks;
- A complete materials characterization
- Numerical calculations to determine the exposure conditions on each deck;
- Numerical calculations to determine the time to corrosion initiation of various materials-exposure combinations.

Modeling, instrumentation and analysis of bridge decks under construction

The objectives of this portion of the study included: identifying potential causes for early age cracking from the analysis of strains and temperatures measured in two newly poured HPC bridge decks and drawing conclusions regarding the performance of the concrete used on each structure. To measure the strains and temperatures in two new HPC bridge decks, vibrating wire embedment gages were attached to the rebar cage at both top and bottom mats. The gages were oriented both parallel and perpendicular to the roadway and gages were also placed between and over the steel girders. It was hypothesized the restraint of the deck would vary depending on if the gage was installed over a girder or at the top or bottom mat of reinforcement. The measured strains required a reference point for interpretation and the point in time used for reference was approximately 1 hour after the concrete was placed. The strains interpreted for this report include the following types of strain:

- **Total Free Strain –** this is the theoretical strain of the concrete due to shrinkage and temperature changes assuming the concrete was unrestrained.
- **Free Thermal Strain –** this is the theoretical strain of concrete due to thermal loads only
- **Strain that would be Measured by an External Device –** this is the strain that would be measured by an external strain gage attached to the surface of the concrete. It is a measure of the change in unit length of the concrete.
- **Corrected Strain –** is the strain measured by the embedded gages and corrected for the difference in the coefficient of thermal expansion between steel and concrete. This is the actual measured unrestrained strain of the concrete due to shrinkage and temperature
- **Stress Producing Strain –** this is a theoretical upper bound of the amount of strain that if restrained would produce stress in the concrete.

2. Approach and methodology

Literature Review

NCHRP recently released a new synthesis report titled “High Performance Concrete specifications and practices for bridges”, which documents a survey taken of state DOTs and is intended to help bridge owners, designers, contractors and material suppliers determine the appropriate specification requirements for HPC in bridges. The team secured in advance a privileged document for internal review and reporting to NJDOT.

In general, the report indicates that states vary in their means of specifying HPC and provides a list of changes in specifications and practices that have improved performance. Some of the recommendations included are as follows:

- Providing multiple options for concrete constituent materials (Supplemental Cementing Materials or SCMs – fly ash, silica fume and slag cement)
- Specifying a limit for drying shrinkage
- Specifying permeability limits
- Starting wet curing immediately after concrete placement
- Specifying and ensuring a longer wet curing period than used previously
- Using lower cement contents
- Controlling evaporation rates

In addition, the report outlined various suggestions for future research, including:

- Identify causes of cracks in concrete bridge decks
- Study several types of structures to see if their design can be improved to reduce deck cracking

- Define how to achieve a low-permeability concrete deck without shrinkage cracks and determine if expansive additives or polypropylene fibers would be effective
- Develop effective means, using non-destructive or other tests, to ensure that concrete meets the required performance criteria for the intended environment. Tests that can be performed on fresh concrete would be particularly useful.
- Identify the most cost-effective methods of sealing cracks in decks to reduce future maintenance.
- Evaluate the effect of concrete compressive strength, modulus of elasticity, drying shrinkage, and creep on cracking.
- Investigate the use of internal curing to reduce deck cracking.

Initial selection of bridge decks

Early in the process, the team developed a bridge-selection document to aid the department in selecting bridge decks to be studied. The approach was to subdivide the time period of HPC use in New Jersey into periodic segments in which bridge decks could be assigned for testing. The periods included new decks, 2 to 5 year old decks, 5-10 year old decks and decks older than 10 years. In addition, the team solicited decks within each period that exhibited early cracking as well as decks that did not exhibit cracking. The last criterion was to identify bridges in salt environments. Given that New Jersey experiences a wide range of winter exposures, from harsh (2013-2014) to mild (2001-2002)[1], the team sought saline exposures related to marine environments. The following is the initial criteria list:

Span 1. New construction HPC (instrumented, durability, NDE tested)

Span 2. New construction HPC with skew (instrumented, durability, NDE tested)

Span 3. 2-5 year-old HPC (durability and NDE Tested)

Span 4. 2-5 year-old HPC (durability and NDE Tested)

Span 5. 2-5 year-old HPC (durability and NDE Tested) – salt environment and early-age cracking

Span 6. 2-5 year-old HPC (durability and NDE Tested) – salt environment but no early-age cracking

Span 7. 5-10 year-old HPC (durability and NDE Tested)

Span 8. 5-10 year-old HPC (durability and NDE Tested)

Span 9. 5-10 year-old HPC (durability and NDE Tested) – salt environment and early-age cracking

[1] **Tom Stavola (2012) - http://www.lightinthestorm.com/nj-snowfall**

Span 10. 5-10 year-old HPC (durability and NDE Tested) – salt environment but no early-age cracking

In addition, the team provided further general guidance:

- ADT: 10,000 to 30,000
- ADTT: 8-15%
- Span Length: 50-100-ft
- Skew: normal or nearly normal unless noted otherwise.
- In the list below, age is used to differentiate between samples. As an alternate to age, consider reviewing bridge decks following changes to the HPC material specification. These changes may have occurred over the years, resulting in different formulation in the material and hence differing performance characteristics. (Note: Per NJDOT, HPC specs have varied from bridge to bridge since beginning its use in the early 2000s)
- The structures selected should have ample historical information available about the decks selected, including information about the mix design, environmental conditions during placement and curing.
- Ideally the structural systems supporting the bridge decks would be similar.
- Overlay: bare decks to be studied in year 1 and overlays to be investigated in subsequent years.
- Sealants: for sealed decks, the structures selected should have ample historical information, including date of application, material specifications, and environmental conditions during placement and curing.

In the early process of reviewing the initially proposed criteria it was discovered that some categories could not be filled. For example, NJDOT indicated that there were no bridges in the inventory that met the criteria of "5-10 year-old HPC (durability and NDE Tested) – salt environment but no early-age cracking".

Figure 1 presents a geographical location of the selected bridges. Table 1 presents the resulting bridges selected in the program. Overall, 10 bridge decks were selected for the current investigation. Among these, 8 were on existing structures and 2 were built during the course of the study. The selected structures have different characteristics to provide a representative sample of bridge decks under the jurisdiction of the NJDOT. Most of these bridges were made of HPC to provide as much information as possible about the recurrent cracking problems affecting these decks. However, some decks made with conventional Class A concrete (i.e, without supplementary cementitious materials) were also included in the investigation. The main characteristics of the investigated decks are given in Table 2 and the theoretical concrete mixture proportions are given in Table 3.

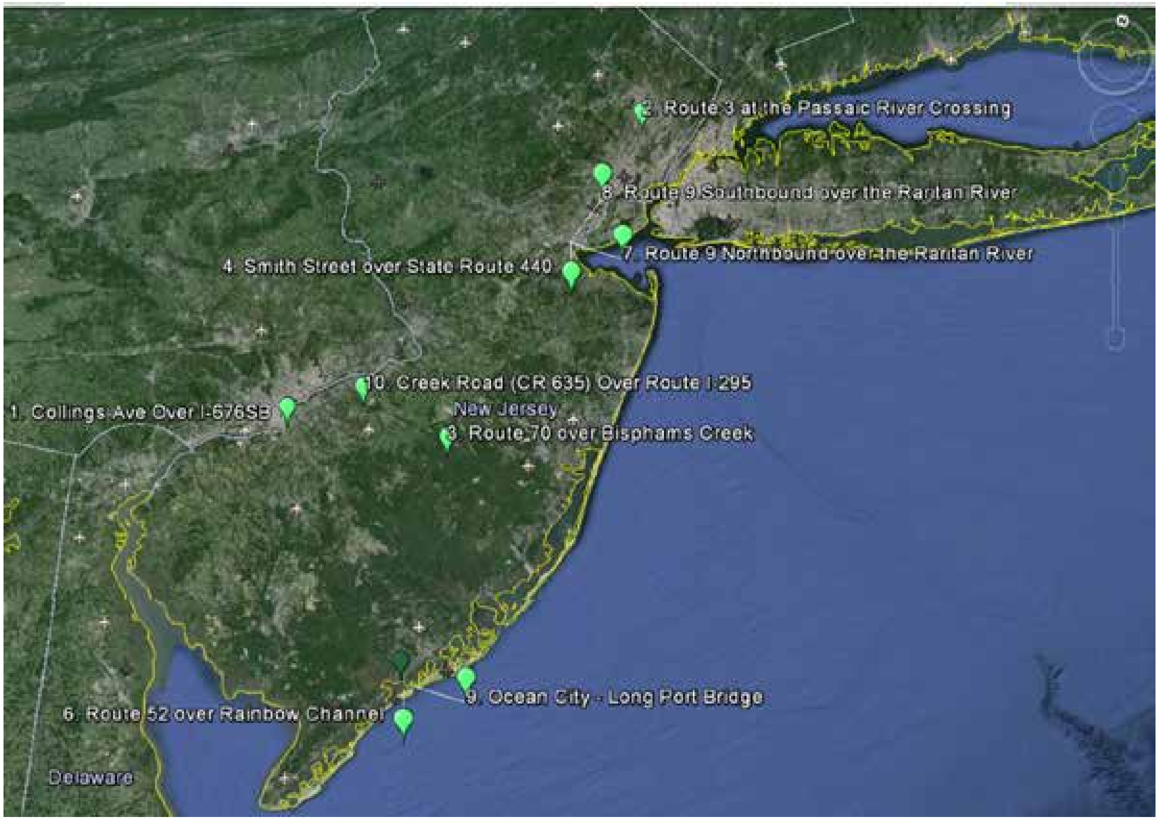

Figure 1- Location of bridges selected for the study

Table 1- Bridges selected for the study

Span	Struct. ID	Description	Initial Selection Criteria	Deck Cond. SI&A
1	0418151	Collings Ave over Route I-676 (SB)	New construction HPC (instrumented, durability, NDE)	(Reconstructed in 2013)
2	1601162	Route 3 over NJ Transit	New construction HPC with skew (instrumented, durability, NDE)	(Reconstructed in 2013)
3	0311150	Route 70 over Bisphams creek	2-5 yr old HPC (durability and NDE)	8 - Very Good
4	1234-509	Smith Street (CR 656) over State Route 440	2-5 yr old HPC (durability and NDE)	(Reconstructed in 2010)
5 & 6	0511156 - 0511157	RT 52 over Rainbow and Elbow Channels	2-5 yr old HPC (durability and NDE) – salt environment and early-age cracking	7 - Good
6	Not Chosen	Not Chosen	2-5 yr old HPC (durability and NDE) – salt environment/no early-age cracking	Not Chosen
7	1209155	Route 9 Edison (Northbound)	5-10 yr old HPC (durability and NDE)	7 - Good
8	1209156	Route 9 Edison (Southbound)	5-10 yr old HPC (durability and NDE Tested)	7 - Good
9	3100-001	Ocean City – Longport Bridge	5-10 yr old HPC (durability and NDE) – salt environment/early-age cracking	6 - Satisfactory
10	0327-166	Creek Road Over I-295	Class A concrete deck with condition rating at or under 5.	5 - Fair

Table 2 – Main bridge characteristics

Bridge name	Structure number	Type of deck and supporting element	Built in	Bridge length† (ft)	Bridge width (ft)	Deck thickness° (in.)	Total number of lanes
Route 70 over Bispham's Mill Creek	0311-150	Concrete bridge deck on precast concrete caissons	2005	23.17	47.00	8.5	2
Smith Street bridge over I-440	1234-509	Concrete bridge deck on permanent steel formwork supported by steel beams	2010*	84.85	83.00	9	4
Route 52 NB over Elbow Channel‡	0511-152	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2008	8,126	98.83	6, 9.5**	4
Route 52 NB over Rainbow Channel‡	0511-151	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2009	2,569	98.83	6, 9.5**	4
Creek Road over I-295	0327-166	Concrete bridge deck on permanent steel formwork supported by steel beams	1970[2]	259	88.00	8	5
Route 9 (Edison Bridge) Northbound	1209-155	Concrete bridge deck on permanent steel formwork supported by steel beams	2003*	4,452	52.49	10.24	3
Route 9 (Edison Bridge) Southbound	1209-156	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2003	4,452	52.49	10.24	3
Ocean City-Longport Bridge	3100-003	Concrete bridge deck on 3.5 in precast concrete slab supporting panels	2002	3,450	75.83	8.5	2
Collings Avenue	0418-151	Concrete bridge deck on permanent steel formwork panels supported by steel beams	1954, 2013*	167.3	52.0	8.5	4
Route 3	1601-162	Concrete bridge deck on permanent steel formwork panels supported by steel beams	1949, 2013*	178.0	164.5	9	3

† Free span length from end abutments.
‡ Visitor center CL taken as border line between Elbow and Rainbow Channel spans.
* Deck rebuilt.
** Variable deck slab thickness of 6 in over beams and 9.5 in between beams.
° The plans do not indicate whether the required concrete cover takes into account the presence of grooves present on all decks.

[2] At the time of the investigation, the deck was considered to have been built in 1970 and never repaired. Information provided to CAIT on March 6 suggests the deck would have been repaired and possibly rebuilt. The exact date remains unknown at the time of writing this report and the conclusions about this bridge are subject to change.

Table 3 – Concrete mixture proportions (based on mix designs[3])

Bridge	W/B ratio	Cement type I/II (lbs/cy)	Slag (lbs/cy)	Class F fly ash (lbs/cy)	Silica fume (lbs/cy)	Sand (lbs/cy)	Coarse aggregate (lbs/cy)	Air content (%)
Route 70	0.40	395	263	-	-	1,199	1,700	5.8
Smith Street	0.40	395	263	-	-	1,242	1,850	6.5
Route 52 Elbow Channel	0.37	353	247	106	-	1,208	1,625	6.2
Route 52 Rainbow Channel	0.37	395	263	-	-	1,247	1,850	6.3
Creek Road	NA	NA	NA	NA	NA	NA	NA	NA
Route 9 northbound	0.37	394	263	-	-	1,250	1,850	5.5
Route 9 southbound	0.37	394	263	-	-	1,250	1,850	5.5
Ocean City Longport Bridge	0.37	658	-	-	-	1,220	1,770	7.0
Collings Avenue	0.37	353	247	106	-	1,208	1,625	5.1
Route 3	0.40	570	-	130	25	1,083	1,773	5.0

Condition Assessment of HPC Bridge Decks Using NDE – Data Collection

Decks of eight existing and two recently constructed bridges were evaluated using a suite of nondestructive evaluation (NDE) technologies. Seven of eight existing bridge decks had HPC, while one had Class A concrete. The condition assessment concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection for the three NDE technologies is illustrated in Figure 2. The impact echo testing was conducted, in the greatest part, using an IE "cane." IE surveys on three of the eight existing bridges were conducted using Stepper, described and illustrated in Appendix A. The electrical resistivity measurements were conducted using a four-electrode Wenner probe. Finally, the concrete modulus measurements were conducted using a portable seismic property analyzer (PSPA). All the data collection was conducted on a two by two foot grid. More detailed descriptions, including principles of operation, of the three technologies, are provided in Appendix A.

[3] Information on mix designs was provided to CAIT by the NJDOT

Figure 2 - Delamination detection using IE (left), assessment of corrosive environment using ER (middle), and concrete quality assessment using USW (right).

In addition to the above three NDE technologies, several surveys were conducted using half-cell potential (HCP) and ground penetrating radar (GPR). The objective of a HCP survey is to assess corrosion activity. The GPR surveys can provide information about the placement of rebars and concrete cover. The information can then be used in a durability analysis to predict the evolution of degradation with time. In addition, GPR provides a qualitative assessment of bridge decks, with respect to possible presence of delamination and corrosive environment. HCP and GPR surveys are illustrated in Figure 3.

Figure 3 - Corrosion activity assessment using HCP (left) and GPR survey (right).

The HCP surveys were successfully completed on two bridges. The decks of the remaining six bridges were not surveyed by HCP because of the lack of electrical continuity of rebars. This was at the same time an indication of still isolating effect of epoxy coating on the top rebar mesh. GPR surveys were conducted on two bridges. Review of results indicated that GPR would provide little benefit because of a good condition of the surveyed bridges. And the reason is that thresholds for deterioration in GPR results are made based on correlations/ground truth with other data. When deterioration is only minor, a qualitative assessment does not add much value.

Model, instrument and validate stresses in curing HPC during bridge deck construction

Theory of Embedded Concrete Bridge Deck Strain Measurements

To properly interpret readings from strain gages embedded in the concrete, an understanding of stress producing strains is required. The strain of concrete decks occurs due to external forces, shrinkage, creep, and temperature. The external forces are generally related to the dead load, live loads, and also the forces arising from the restraint placed on the deck. The dead loads on the structure would consist of the superimposed dead loads from parapets and any other attachments to the structure since the deck itself does not carry any self-weight dead load. The addition of the parapets and attachments will not cause significant dead load strains within the concrete deck. The live loads on the structure are dynamic in nature and would not be captured by the slow speed vibrating strain gages used in this study. Therefore the only significant strains undergone from the concrete are those arising from the restraint, from the structural system (i.e. girders, rebar, bearings, etc.), shrinkage, creep, and thermal effects.

Concrete shrinkage will generally produce compressive force in reinforcement that is balanced by a tensile force in the concrete (Nilson 2004). Early age concrete shrinkage is comprised of three types including plastic, autogenous, and drying. Plastic shrinkage occurs during the first few hours of curing and is characterized by the loss of water from exposed concrete surfaces. During the first few hours of curing, water content is lost from exposed surfaces at a faster rate than it is replaced by bleed water from lower layers of concrete (Ganesh 2006). Autogenous shrinkage occurs during the hydration of concrete and is a phenomenon characterized as heating or boiling the water content out of the concrete. Drying shrinkage occurs following the initial setting of concrete and is described by the volume change of the concrete due to water absorption and is generally called swelling. Drying shrinkage is affected by the mix design, ambient conditions, and the deck reinforcement details. Since concrete decks generally do not have symmetrical reinforcement, the shrinkage will be restrained differently across the section and result in curvature and deflections of the section (Nilson 2004). In the study reported here, the stress producing strains were identified as those arising from the restraint of shrinkage and thermal loads. Since the different types of shrinkage are difficult to separate they are assumed to be lumped into one parameter reflecting the total shrinkage due to plastic, autogenous, and drying shrinkage.

Measured, Theoretical and Corrected Strain Definitions

Along with temperatures, the concrete strain was measured using strain gages cast in the deck. The strains in the included plots follow the sign convention that a positive strain is a tensile strain and a negative strain is a compressive strain. The data plots included in this report show the following types of strains referenced to the baseline strain approximately one hour following the deck pour:

- **Total Free Strain –** this is the theoretical strain of the concrete due to shrinkage and thermal movements assuming the concrete was unrestrained. While there is strain in this case there will be zero stress since the concrete is free to expand and contract. If this movement was fully restrained, there would be zero strain in the member since it would be unable to expand and contract. However, there will be stress due to the restraint.
- **Free Thermal Strain –** this is the strain that would occur if the deck was unrestrained and subjected to the measured temperature changes
- **Actual Strain Measured by an External Device –** this is the strain that would be measured by an external strain gage attached to the surface of the concrete and is a measure of the actual free expansion and contraction of the concrete.
- **Corrected Strain –** is the strain measured by the embedded gages and corrected for the difference in the coefficient of thermal expansion between steel and concrete. Positive change in temperature results in a buildup of compressive strain in the concrete and a negative change in temperature will result in a buildup of tensile strain in the concrete. Both of these mechanisms are measured accurately by the strain gages. While the concrete is partially restrained by different mechanisms both internal and external, the embedded gage is not restrained. A positive temperature change in the concrete will cause the wire between the end blocks to expand and go slack indicating a compressive strain. This strain is balanced slightly by the expansion of the concrete. This same behavior is true for negative temperature changes, as the wire in the gage gets shorter indicating a tensile strain. This tensile strain is balanced to some degree by the contraction of the concrete. It should be noted that the total strain of concrete for a positive change in temperature will be tensile while a negative change in temperature will indicate a compressive strain. These strains are a composite of load, restraint, creep, and shrinkage but not the free contraction of temperature. In the absence of load, creep, or shrinkage strains these readings would equal the stress producing strains due to restraint of thermal movements.
- **Stress Producing Strain –** the strain associated with the development of tensile stresses in the concrete deck. In Figure 4 when the deck cools and contracts, the concrete imparts a compressive stress on the gage end blocks. In addition, the cooling effect causes the wire used in the gage to tighten and indicate a tensile strain, slightly altering the measured compressive strain. Shrinkage effects are similar to contractions due to cooling except there is not a temperature component affecting the gage wire. To identify the stress producing strains from restrained temperature and shrinkage, the theoretical free strains are subtracted from the measured and corrected strains as shown in Figure 4 and Figure 5.

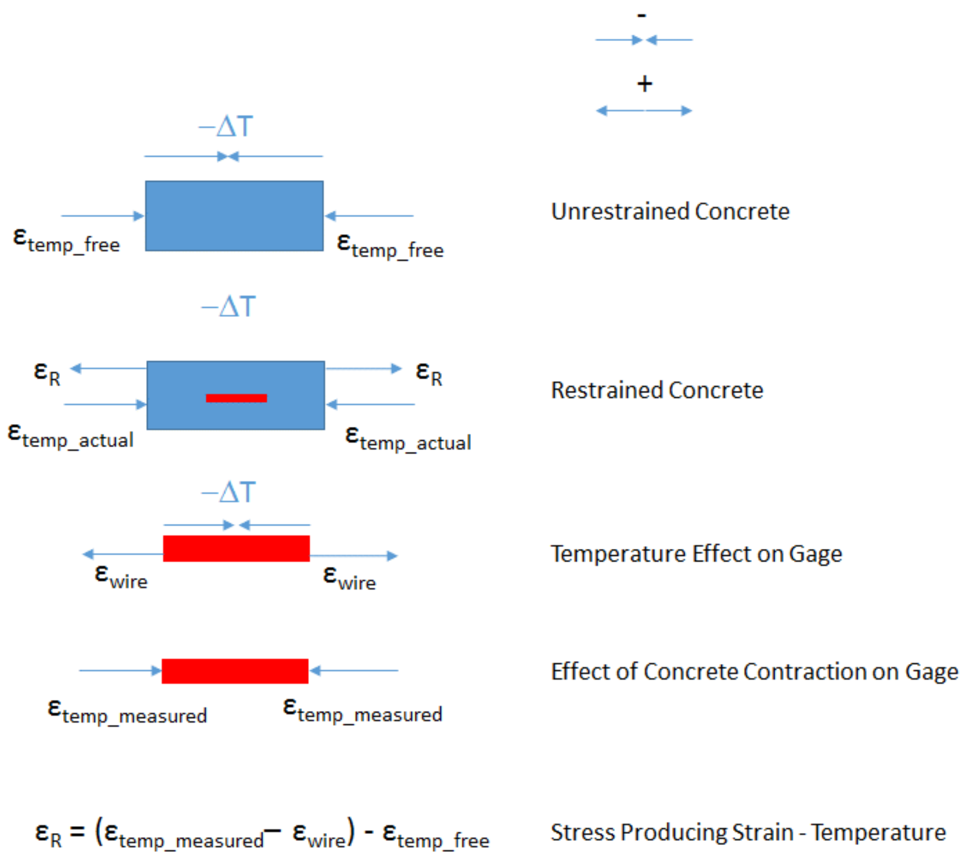

Figure 4: Stress Producing Strain due to Temperature

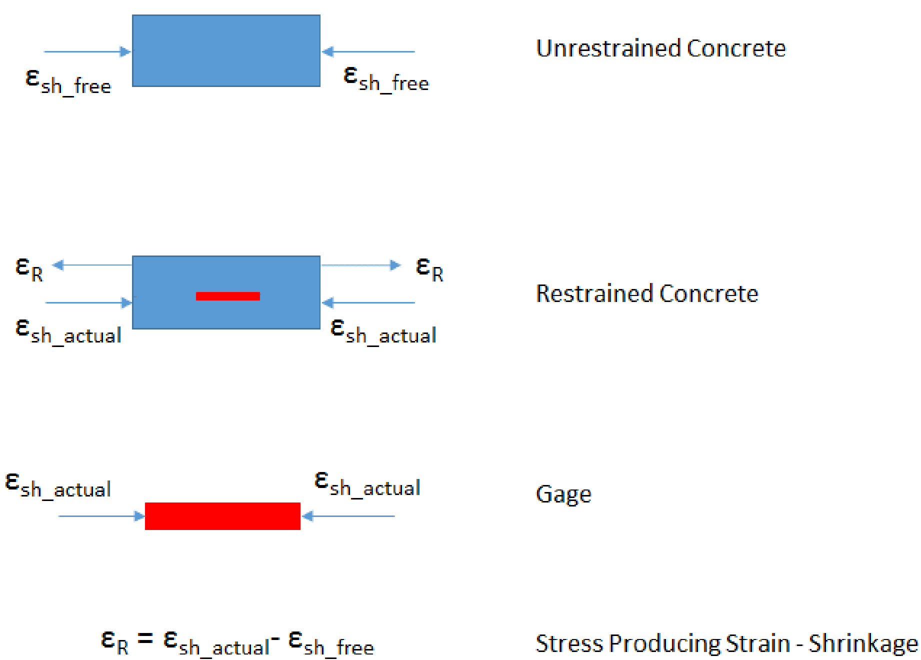

Figure 5: Stress Producing Strain due to Shrinkage

The following examples are used to illustrate the measured strains. First, assume a concrete block is unrestrained and undergoes a temperature change of -20 °F. Using an assumed concrete coefficient of thermal expansion the following corrected strains and referencing the future strains to an initial strain, would result in the following:

$$\varepsilon_{corrected} = (R_1 - R_0) + (\alpha_s - \alpha_c) * (T_1 - T_0)$$

$$\varepsilon_{corrected} = (20 * 10^{-6}) + (6.7 - 5.7)(10^{-6}) * (-20) = 0$$

R_1 = strain reading at a future time

R_0 = strain at reference point

α_s = coefficient of thermal expansion for steel

α_c = coefficient of thermal expansion for concrete

T_1 = temperature at a future point in time

T_0 = temperature at reference point

$\varepsilon_{corrected}$ = Corrected strain

ε_{ext} = strain that would be measured by an external gage

The strains including the free contraction of the concrete are calculated as follows:

$$\varepsilon_{ext} = (R_1 - R_0) + (\alpha_s) * (T_1 - T_0)$$

$$\varepsilon_{ext} = (20 * 10^{-6}) + (6.7)(10^{-6}) * (-20) = -107 * 10^{-6} = (\alpha_c) * (T_1 - T_0)$$

If the assumed concrete block is fully restrained the corrected strains would be equal to:

$$\varepsilon_{corrected} = (R_1 - R_0) + (\alpha_s - \alpha_c) * (T_1 - T_0)$$

$$\varepsilon_{corrected} = (134 * 10^{-6}) + (6.7 - 5.7)(10^{-6}) * (-20) = 114 * 10^{-6}$$

The strains including the free contraction of the concrete that is fully restrained are calculated as follows:

$$\varepsilon_{ext} = (R_1 - R_0) + (\alpha_s) * (T_1 - T_0)$$

$$\varepsilon_{ext} = (134 * 10^{-6}) + (6.7)(10^{-6}) * (-20) = 0$$

If the assumed concrete block is unrestrained and the gage reads -80*10^{-6} the corrected strains would be:

$$\varepsilon_{corrected} = (R_1 - R_0) + (\alpha_s - \alpha_c) * (T_1 - T_0)$$

$$\varepsilon_{corrected} = (-80 * 10^{-6}) + (6.7 - 5.7)(10^{-6}) * (-20) = -100 * 10^{-6}$$

The strains including free contraction of the concrete that is assumed to be unrestrained are given as follows:

$$\varepsilon_{ext} = (R_1 - R_0) + (\alpha_s) * (T_1 - T_0)$$

$$\varepsilon_{ext} = (-80 * 10^{-6}) + (6.7)(10^{-6}) * (-20) = -214 * 10^{-6}$$

Since the block is unrestrained the corrected strain should be equal to zero if temperature is the only mechanism affecting the concrete. Since this is not the case, other behaviors are producing strain within the concrete such as load, creep, or shrinkage.

Comparative durability assessment of bridge decks

The tasks performed as part of the durability assessment are summarized in Table 4. A detailed description of each task and activity of the proposed program is given in the next paragraphs.

Table 4 – Durability assessment

Task	Activity and deliverable
S1–Review of existing documentation	· Review and analyze documentation provided by NJDOT on the characteristics of HPC structures · Review and analyze data generated by CAIT for each structures
S2–Field work on existing structures	· Complementary inspection of cracked HPC bridge decks · Extraction of a limited number of concrete cores · Additional corrosion measurements (if needed) · Characterization of concrete cores
S3-Field work on new bridge decks	· Sampling of concrete cylinders during bridge deck construction · Characterization of concrete samples
S4-Service-life analysis (Class A v. HPC)	· Comparative service-life analysis for uncracked decks using STADIUMÒ

Task S1 - Review of existing documentation and analysis of available data

This portion of the study consisted in reviewing available information and data such as:

- Drawings
- Inspection reports
- NDE data

Task S2 - Field investigation – Existing structures

This task consisted in performing a visual inspection on 8 existing structures identified by CAIT and approved by NJDOT focusing on the presence of cracks. This task also included core extraction in selected areas to perform concrete physical and transport property testing. Cores were tested according to an experimental program designed to determine the properties of the in-situ concrete, assess the extent of chloride contamination and generate input data for the service-life calculations. The testing protocol is summarized in Table 5.

It is noteworthy to mention that on the basis of initial non-destructive testing results, additional measurements could be performed to clearly establish the impact of existing cracks on the durability of the reinforced concrete deck.

Table 5 – Experimental protocol for cores extracted from existing structures

Test description	Test method
Core examination and measurement	ASTM C 1542
Petrographic examination	ASTM C 856
Compressive strength	ASTM C 42
Chloride contamination	ASTM C 1152
Air-void network characteristics	ASTM C 457
Thermal expansion coefficient	USACE CRD–C 39–81
Volume of permeable voids	ASTM C 642
Migration	ASTM C 1202 modified
Drying	ASTM C 1585 modified

Task S3 - Field investigation – new structures

This task consisted in sampling concrete for testing on new structures, monitoring crack formation, and collecting temperature and relative humidity data, both inside and outside the concrete. This part of the work was coordinated with other BRP partners.

Samples were characterized according to the experimental protocol presented in Table 6. The objective of the protocol was to characterize the evolution of concrete properties and generate input data for the early-age cracking analysis and service-life calculations.

Table 6 – Experimental protocol for concrete cylinders sampled during construction

Test description	Test method
Petrographic examination	ASTM C 856
Compressive strength	ASTM C 42
Splitting-tensile strength	ASTM C496
Elastic modulus	ASTM C 469
Shrinkage	ASTM C 157
Air-void network characteristics	ASTM C 457
Thermal expansion coefficient	USACE CRD–C 39–81
Volume of permeable voids	ASTM C 642
Migration	ASTM C 1202 modified
Drying	ASTM C 1585 modified

Task S4 – Comparative durability analysis

Results from the concrete characterization program were used as input parameters in STADIUM® simulations to compare the service-life of HPC and Class A concrete decks. The simulation program included:

- The determination of representative exposure conditions on the existing HPC decks;
- Durability analysis of HPC decks (cracked and uncracked conditions), based on the concrete properties determined from the investigated structures;
- Durability analysis of Class A concrete decks, based on the properties of such concrete mixtures determined in the course of the current study.

The results of these simulations were analyzed in view of the expected durability and service life expectations for both types of concrete.

3. Summary of analyses

NDE Results

The targeted results of the NDE surveys were twofold. The first were condition maps with respect to delamination, corrosion and concrete quality. The second results were condition ratings (indices) of the surveyed bridges with respect to delamination and corrosion, and analysis of concrete quality variability. As it is described and illustrated by figures and tables, condition maps enable assessment of each individual bridge with respect to identification of areas exhibiting more pronounced signs of deterioration. On the other hand, the condition ratings enable both the assessment of a condition of a particular bridge deck, and more objective comparison of conditions between bridges.

Condition ratings with respect to corrosion and delamination are summarized for the eight bridges in Table 7. The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 kΩ-cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

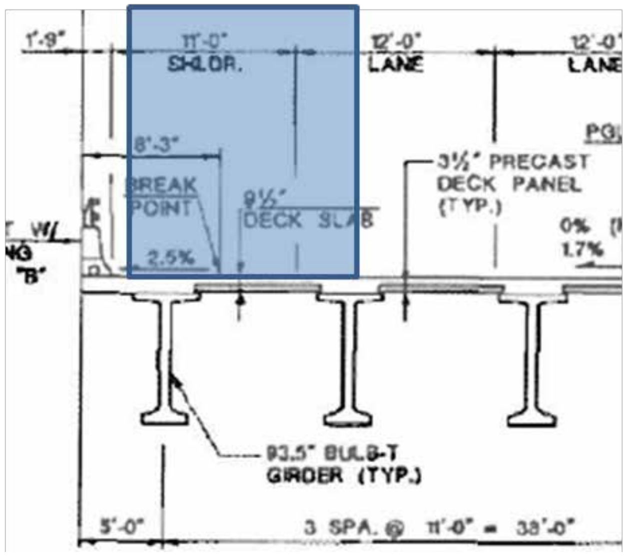

Figure 6 - Typical cross section of Route 52 Bridge with the survey area marked in blue

As shown in Table 7, there are little signs of corrosive environment in all but two bridges: Smith Street over Rt. 440 and Rt. 70 over Bisphams Creek. Similarly, there were very few signs of delamination, except for two bridges: Ocean City - Longport, and Rt. 52 Bridge. For the two Route 52 bridge sections, there is a possibility that some of the identified points of delamination actually represent resonances coming from either relatively thin (3.5 inch) precast deck panels or reflections from the bottoms of wide and deep top flanges of girders, as illustrated in Figure 6. This could be confirmed by conducting an IE measurement using a source with a higher center frequency than the one used.

Table 7 – Condition Ratings with Respect to Delamination and Corrosion for Eight Bridges

Bridge	Corrosion Rating	Delamination Rating
Route 9 Northbound	97.3	90.1
Route 9 Southbound	92.8	90.6
Ocean City - Longport	96.3	81.6
Route 52, Ocean City (Elbow)	93.2	74.8
Route 52, Ocean City (Rainbow)	99.8	77.7
Smith Street over Route 440	74.1	89.7
Route 70 over Bisphams Creek	58.9	95.5
Creek Road over I-295	99.7	94.0

In addition to condition ratings, average elastic modulus and standard deviation of the moduli were calculated for four bridges and presented in Table 8. The average modulus for the four bridge decks varies between 4,700 and 5,500 ksi, while the standard deviation varies between 420 and 1,280 ksi. It should be emphasized that the variability of concrete modulus, including lower values of moduli, are not a sign of deterioration. It is more likely a result of concrete material variability and placement procedures used during construction.

Table 8 – Average Concrete Modulus and Modulus Variability for Four Bridges

Bridge	Average Modulus (ksi)	Standard Deviation (ksi)
Ocean City Longport	5235	421
Route 9 Northbound	4718	540
Route 9 Southbound	5196	787
Creek Road over I-295	5492	1275

Figure 7 through Figure 10 contain condition maps for five bridges: Route 9 (northbound and southbound), Ocean City - Longport, and Route 52 (Elbow and Rainbow Channels). Included in the maps is also the latest National Bridge Inventory (NBI) rating for each of the bridge decks. A visual review of the condition maps supports the above findings regarding the corrosion and delamination. A review of ER maps for Route 9 and Longport bridges in Figure 7 to Figure 9 points to more corrosive environment along the parapets/curbs. Transverse coordinate 0 indicates the first survey line, which in all three cases was one foot from the parapet/curb. This is a commonly obtained result on highway bridges and an indication of likely snow, salt and debris deposition along the parapet/curb facilitating accelerated corrosive environment development and thus, higher corrosion rates.

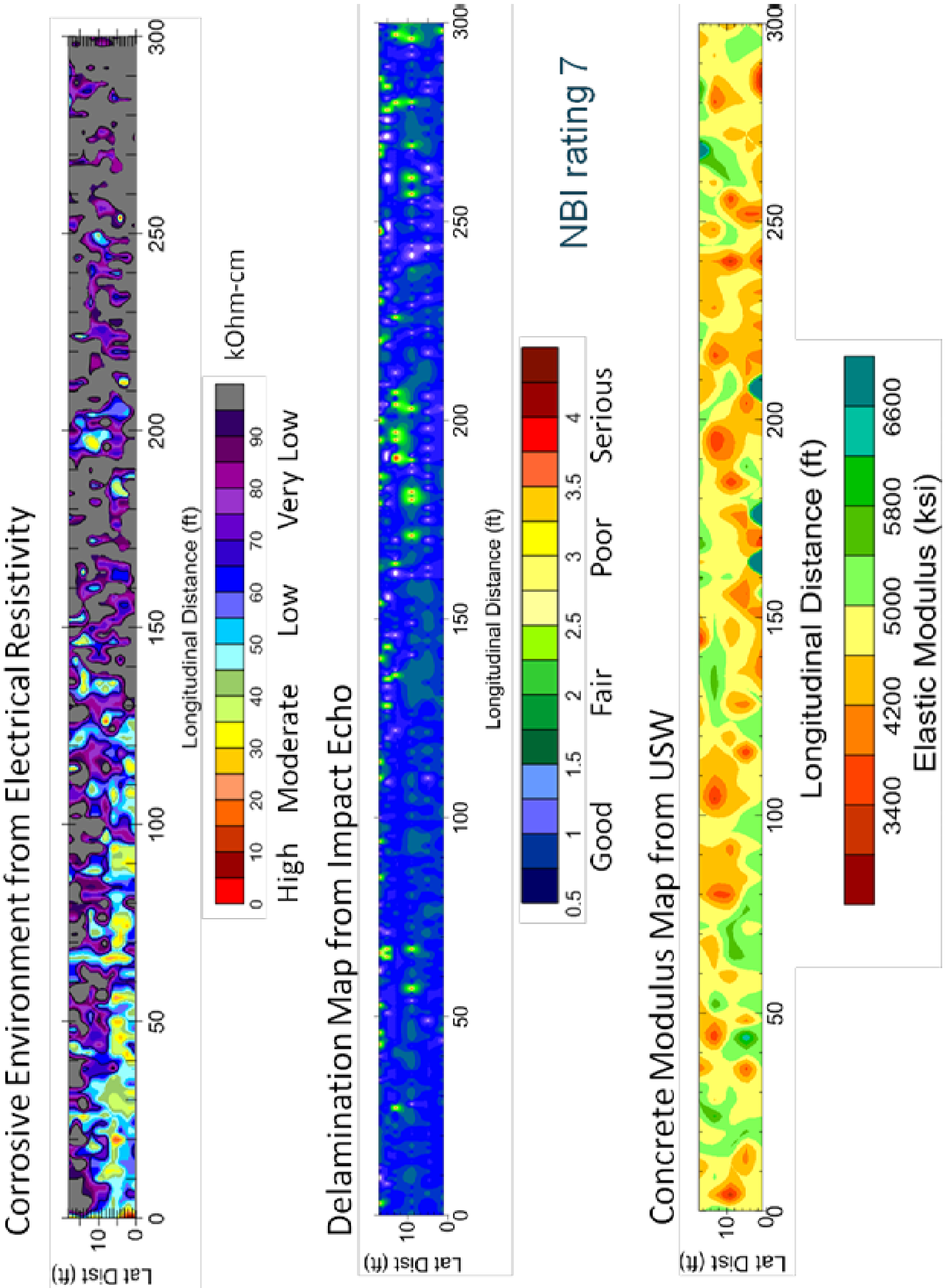

Figure 7 - Condition maps for Route 9 northbound

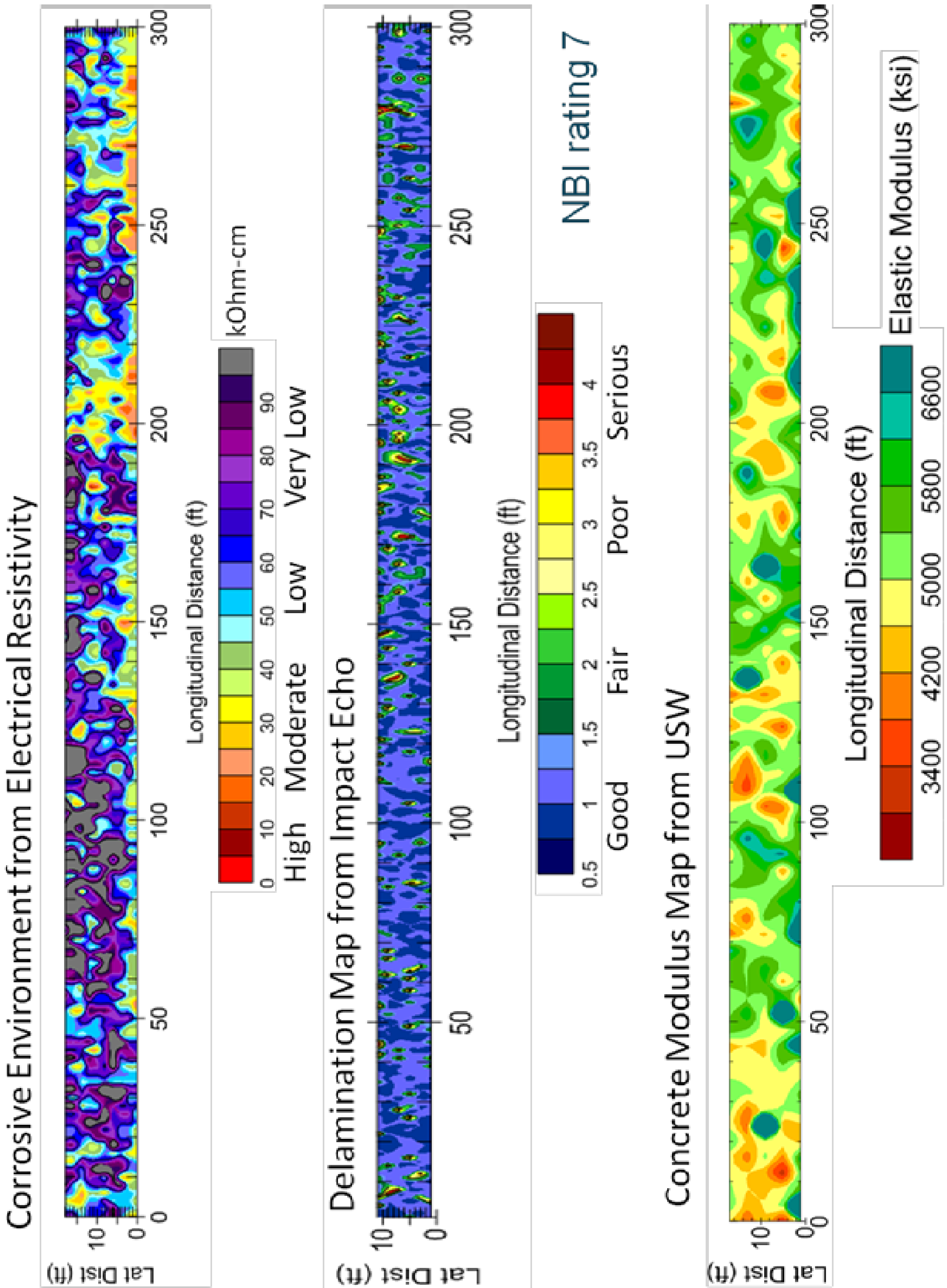

Figure 8 - Condition maps for Route 9 southbound

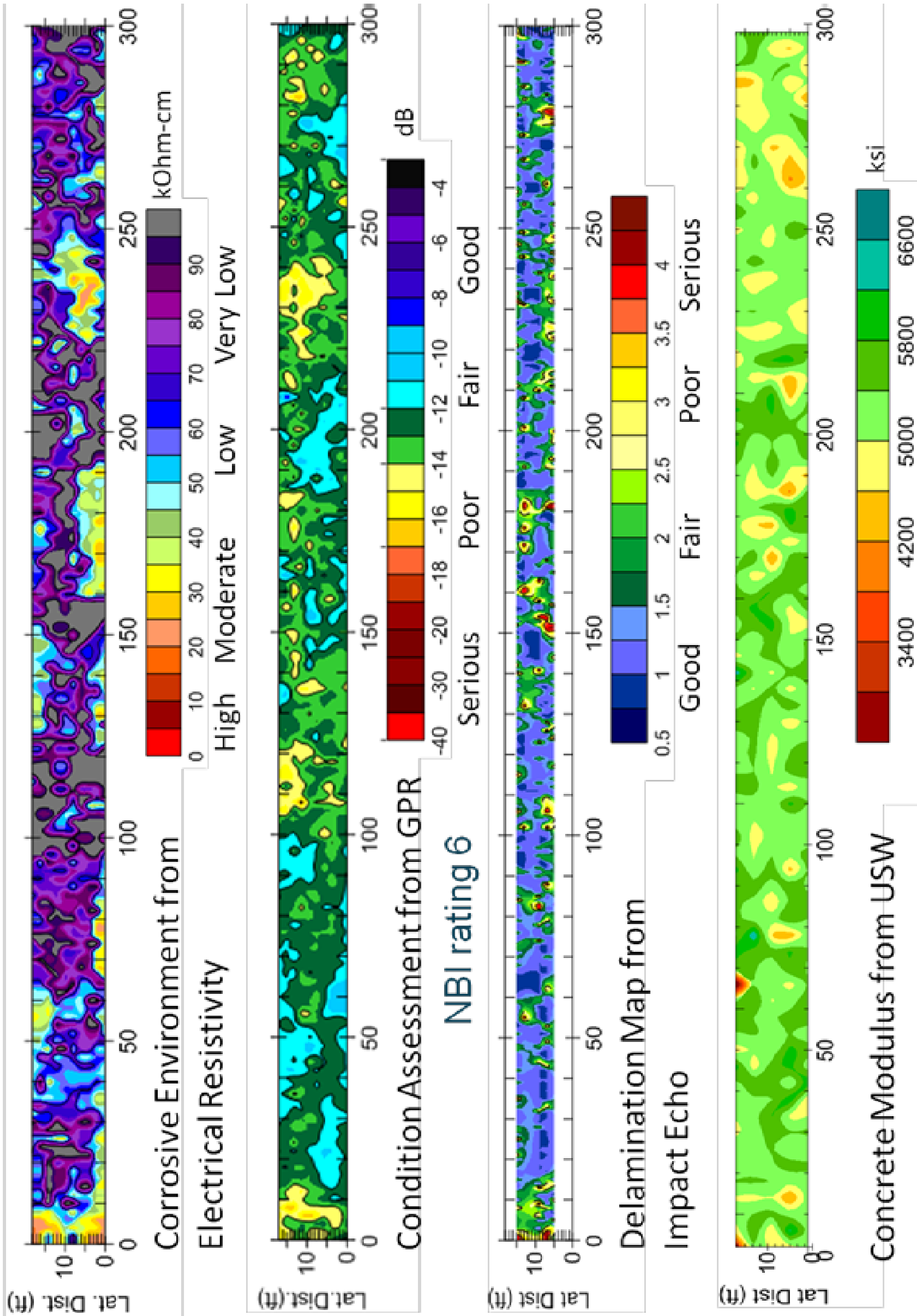

Figure 9 - Condition maps for Longport Bridge, Ocean City

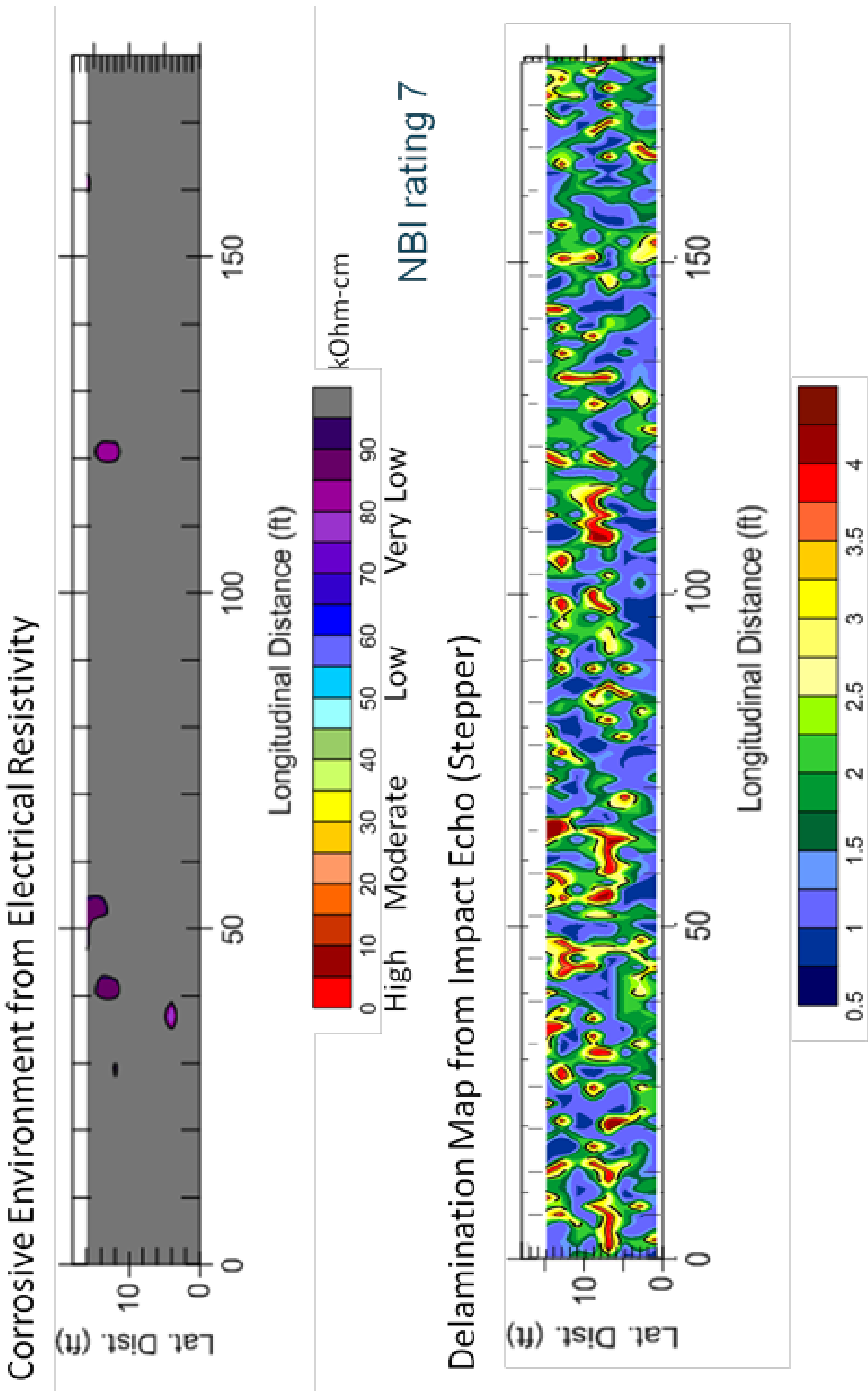

Figure 10 - Electrical resistivity and delamination maps for Route 52 causeway (Elbow Creek)

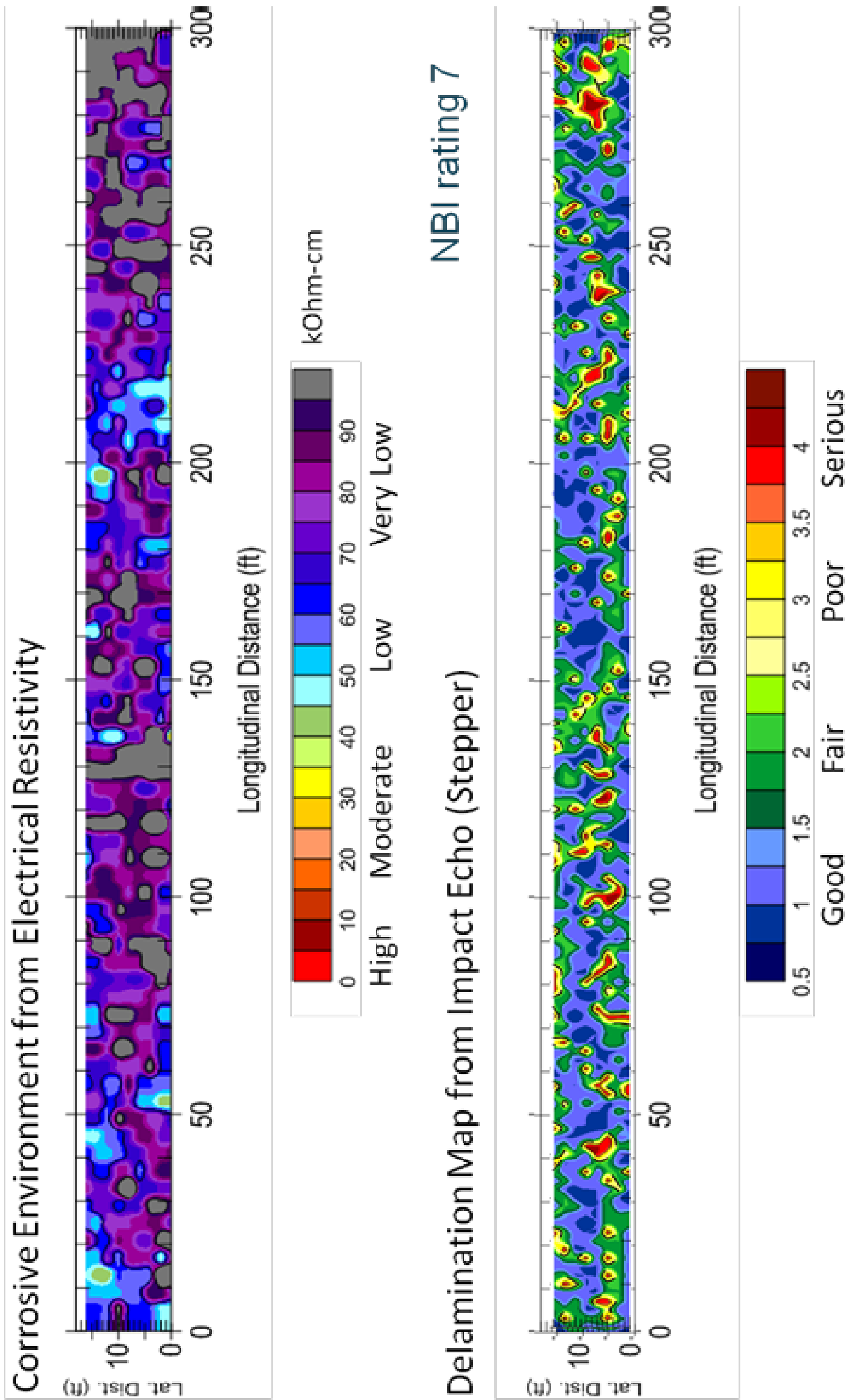

Figure 11 - Electrical resistivity and delamination maps for Route 52 causeway (Rainbow Creek)

Bridge decks of Smith Street over Route 440 and Route 70 over Bisphams Creek exhibit clear signs of the development of corrosive environment. In both cases there are deck sections with electrical resistivity at or below 15 kΩ-cm. Those values are indications of highly corrosive environments that promote high corrosion rates. In the case of the Route 70 deck, the resistivity results are matched by half-cell potential results. Rebar mesh continuity and HCP readings in the range -600 to -400 mV indicate damaged epoxy coating and high corrosion activity.

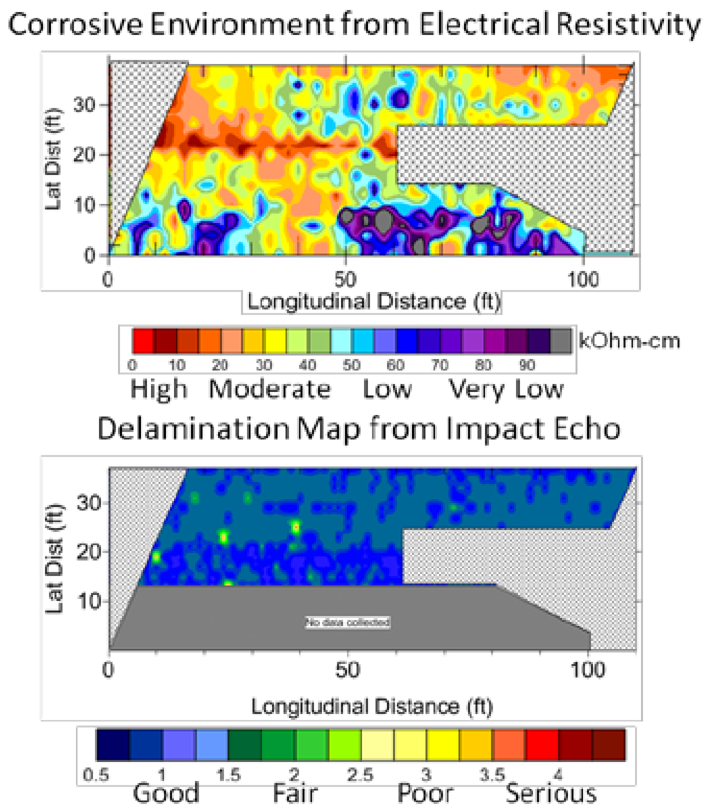

Figure 12 - Electrical resistivity and delamination map for Smith Street over SR-440 Bridge

One initially surprising result was the condition of the deck of Creek Road over I-295 Bridge. The deck was constructed with Class A concrete and uncoated rebars. The initial information about the year the bridge deck was constructed was 1971. In addition, the latest NBI rating of the deck was 5, which would not be a surprising condition for a more than forty-year old concrete deck. However, all the results shown in Figure 14 are describing an excellent condition of the deck. There are no signs of corrosive environment and HCP readings are either very low negative values or positive, indicating very unlikely corrosion activity. Similarly, there were only a few points indicating potential signs of delamination. At the review meeting held March 6, 2014, it was indicated that this bridge may have been reconstructed. However, the year of reconstruction remains undetermined at the time of writing this report.

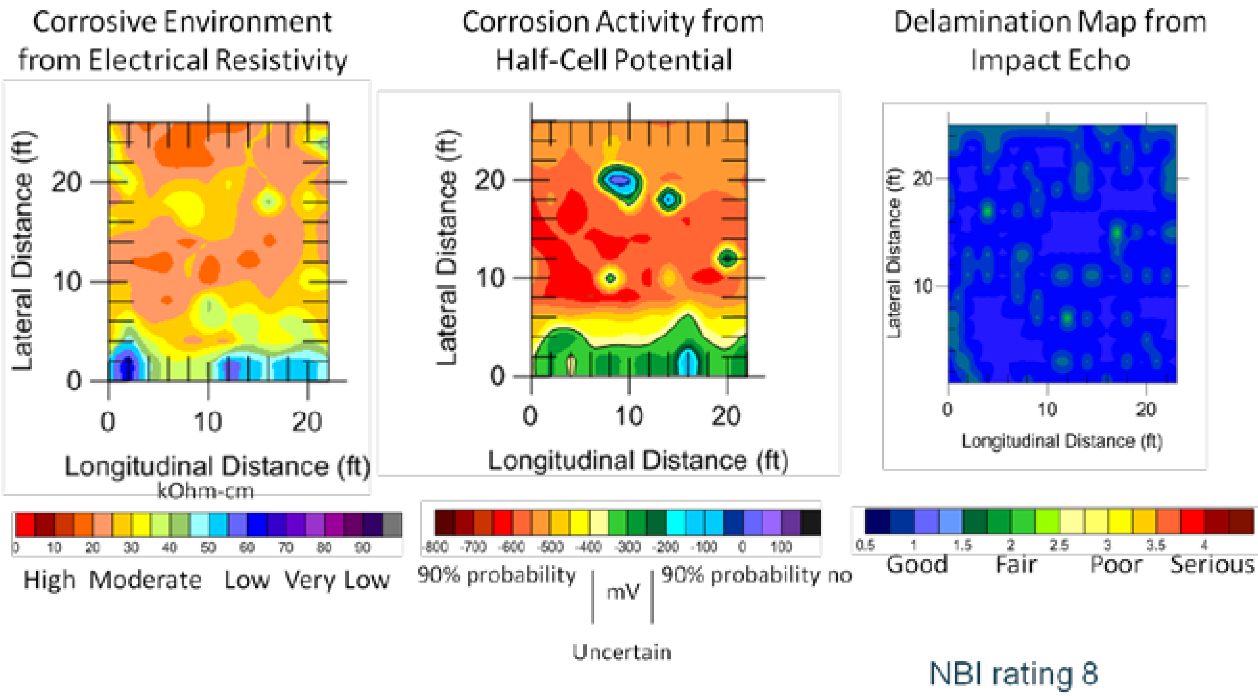

Figure 13 - Condition maps for Route 70 over Bisphams Creek Bridge

In summary, the results from NDE surveys describe an overall very good condition of the surveyed bridge decks. Only a couple of decks have signs of development of more corrosive environment and corrosion activity, and three exhibit possible delamination.

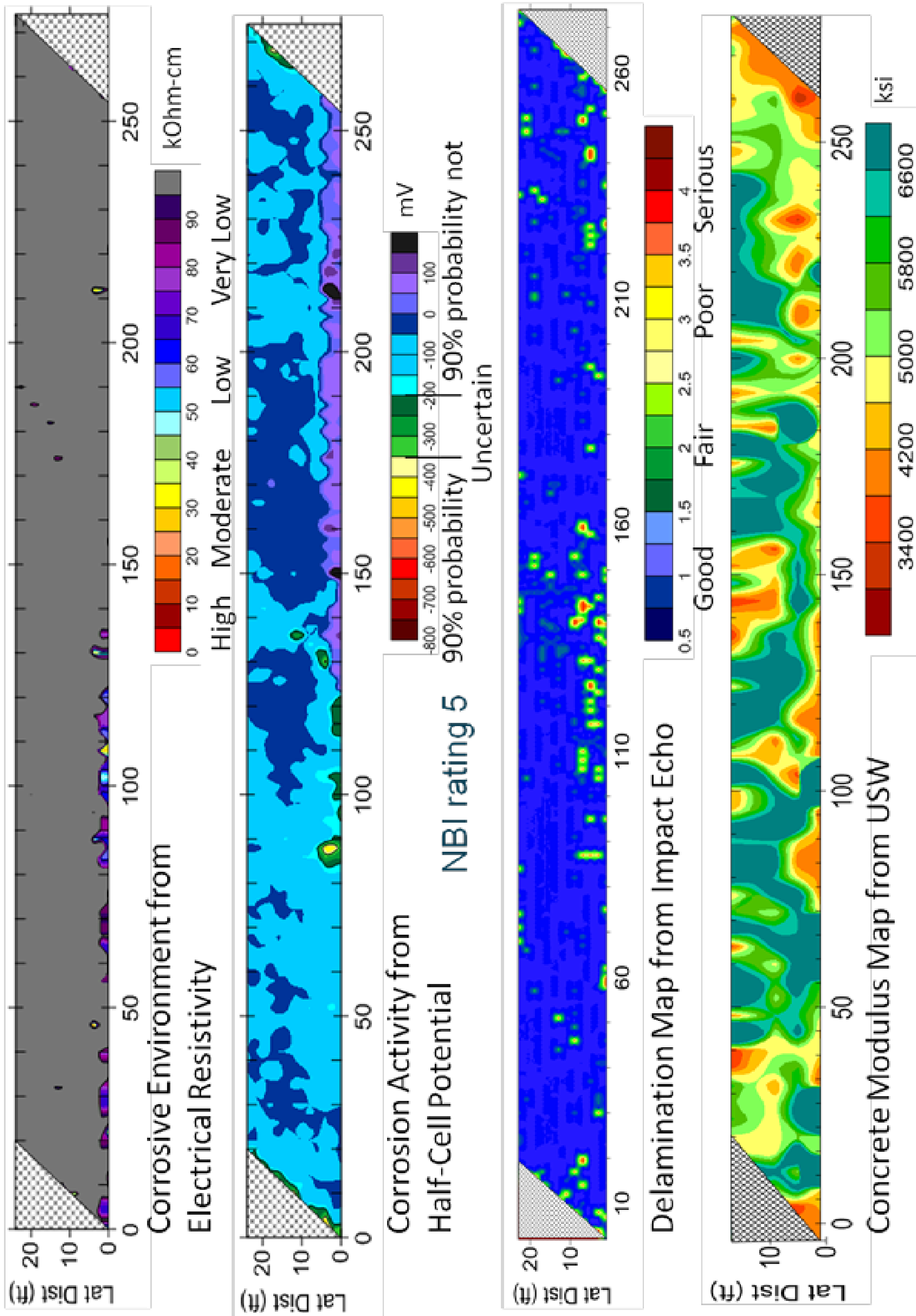

Figure 14 - Condition maps for Creek Road Bridge over I-295

Measured Strains observations

The following are the observations developed from a comparison of the mix design, curing procedures, and monitoring data from the two new bridge decks

Mix Design

- The two mixes were designed and approved to meet NJDOT specifications. Table 9 provides a comparison of the mix designs.

Table 9: Mix Design Comparison

	Route 3 (% Total)	Collings Ave (% Total)
Cement 1 (lb)	570 (15%)	353 (9%)
Slag Cement (lb)		247 (6%)
Class F Flyash (lb)	130 (3%)	106 (3%)
Silica Fume (lb)	25 (1%)	
Fine Aggregate (lb)	1083 (28%)	1208 (32%)
Coarse Aggregate (lb)	1773 (46%)	1625 (43%)
Water (lb)	292 (7%)	263 (7%)
w/c ratio	0.4	0.37
% Air	6	6
Slump (in)	6 +/- 2	6 +/- 2
Admixtures	High range water reducing admixture, Retarder, Air Entrainment, Microsilica	High and normal range water reducing admixtures, Accelerator, Retarder, Air entrainment
Total Cementitious Material (lb)	725	750
Total weight per cubic yard (lb)	3873	3801
NJDOT Design f'c (psi)	5400	5400
Mix f'c(psi)	6370	Not specified
Average Measured f'c(psi)	7020	6425

- The two mix designs are of similar proportion with respect to cement versus aggregate content.
- The total cement content of the mixes is similar, however, the makeup of the cementitious material between the two mixes differs. The Route 3 Bridge utilized Type 1 Portland cement, fly ash, and microsilica while Collings Ave used Type 1 Portland cement, slag cement and fly ash.
    - In general, concrete using slag cement as a replacement for Portland cement will have a lower permeability than concrete made with only class F fly ash as a replacement.
    - Slag cement will generally decrease the time it takes for the concrete to set.
    - Class F fly ash tends to reduce the heat of hydration as the concrete cures

and also reduces the early age strength of the concrete

- Silica fume was used in the Route 3 mix and will generally improve the compressive strength and reduce the permeability of the concrete

Curing Process

- The two new HPC decks were cured using similar operations with one significant difference. Immediately following the finishing of the concrete, the freshly machined concrete was topped with wet burlap. The wet burlap was subsequently covered with plastic sheeting weighted down to protect it against the wind. At the Route 3 structure the wet burlap placing process followed behind the concrete placement by approximately one hour, while at Collings Ave the wet burlap was applied approximately 3 hours after placement of the fresh concrete. The time lag between placement of fresh concrete and the application of wet burlap at Collings Ave could have resulted in plastic shrinkage cracking of the concrete due to evaporation of water from the concrete before it sets. An evaluation of the Collings Ave bridge deck did not reveal any plastic shrinkage cracking.
- Wet curing was utilized on both bridges and implemented by placing water hoses at varying positions along the bridge deck. The hoses were used to keep the burlap wet and the process continued for a minimum of 14 days in accordance with NJDOT specifications.

Material Tests

- The traditional drying shrinkage tests measure drying shrinkage from the point when the specimens are able to be de-molded. The drying shrinkage tests were performed by SIMCO in accordance with the specifications of ASTM C 157 –Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete.
  - The drying shrinkage values at 56 days for the two bridges is 525 microstrain as shown in Figure 15. This value is considered high and indicates the potential for cracking due to shrinkage.
- The thermal expansion coefficients of the two concrete mixes were provided by SIMCO and measured using USACE CRD-C 39-91 – Test Method for Coefficient of Linear Thermal Expansion of Concrete.
  - The average value at the Route 3 Bridge was 9x10-6/°F and falls outside the range (4.1-7.3 x10-6/°F) of expected values specified by FHWA.
  - The average value for the Collings Ave Bridge was $6.8\text{x}10^{-6}$/°F which falls at the top end of the expected range for concrete specified by FHWA.
- The compressive strength the concrete was measured at 28 days.
  - The Route 3 mix had an average compressive strength of 7020 psi at the pump and 8158 psi at the delivery truck
  - The Collings Ave mix had an average compressive strength of 6425 psi
  - Both mixes exceeded the NJDOT specified 56 day compressive strength of 5400 psi
- The splitting tensile strength of the concrete was measured at 28 days.
  - The Route 3 mix had an average tensile strength of 602 psi
  - The Collings Ave mix had an average tensile strength of 585 psi

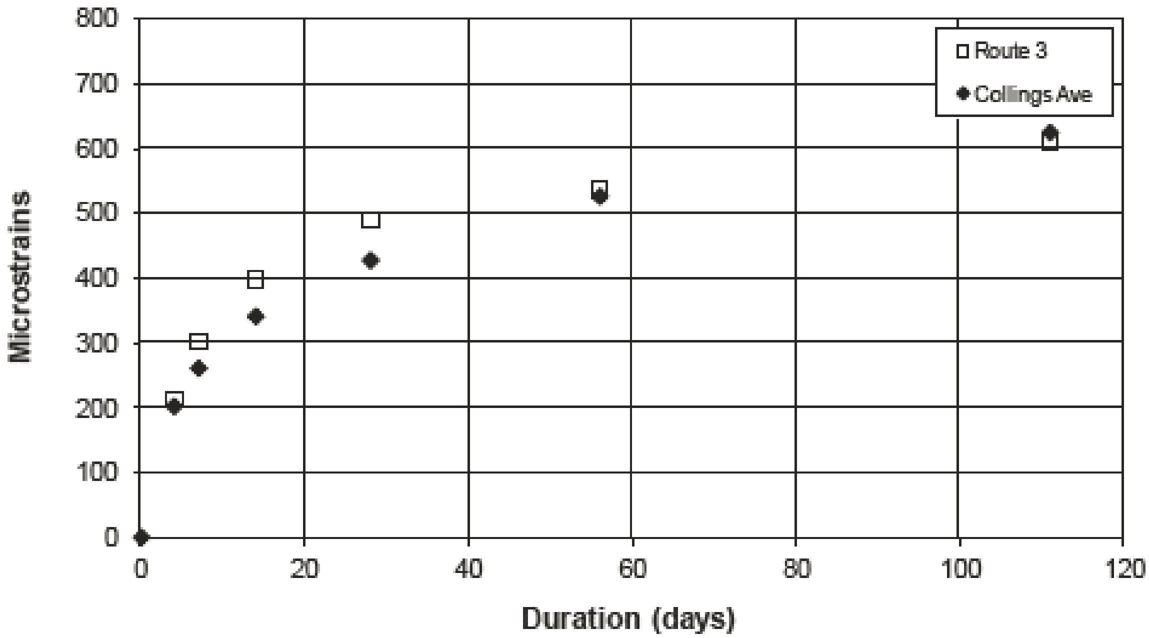

Figure 15: Measured Free Shrinkage Strains - Courtesy of SIMCO

Concrete Temperatures

- The ambient temperature during the Route 3 Bridge deck pour was between 50 and 60 °F. The concrete temperature at placement varied between 65 °F and 70 °F and the maximum temperature of the concrete in the 24 hours following the pour reached approximately 100 °F. During the subsequent monitoring period, the temperature of the deck due to temperature changes and solar radiation exceeded the reference temperature on numerous occasions and the maximum temperature that occurred was 118 °F.
- The ambient temperature at the Collings Ave Bridge during the pour varied from 78 to 95 °F. The concrete at placement varied between 81 °F and 83 °F and the maximum temperature of the concrete reached approximately 135 °F. During the subsequent monitoring period, the temperature of the deck did not exceed the maximum temperature during heat of hydration.
- Once the decks reached the peak temperature due to hydration they begin to cool and equalize with ambient temperatures. The initial cooling following peak heat of hydration varied between 20 and 35 °F. The concrete temperatures during the first few days after concrete placement are shown in Figure 16 and
- A comparison of the temperatures at locations between the girders, directly above the girders, and on the top and bottom flange of the supporting steel girder of the Route 3 Bridge is given in Figure 17 for the period after two days after deck casting and in Figure 18 for 7 days after deck casting.
    - The temperature gradient in the concrete at locations between girders is larger than the gradient at locations above the girders.
    - The girder top flange is at a lower temperature than the concrete and as the top of the concrete cools due to low ambient temperatures, the steel will not cool as quickly since it is insulated by the warm concrete. This differential cooling can

cause restraint and induce tensile stress in the concrete

  - The concrete temperatures at the bottom of the slab and the temperatures at the top flange of the steel girder have a lag associated with when their maxima and minima occur in comparison with the temperatures at the top of the slab.
  - The rate of cooling of the top of the deck compared with the rate of cooling of the steel girder can introduce tensile stress into the concrete as the girder restrains the cooling of the deck

- A comparison of the temperatures at locations between the girders and directly above the girders of the Collings Ave Bridge is given in Figure 20 for the period spanning from the casting of the deck to two days into curing.
  - The temperature gradient in the concrete at peak heat of hydration above the girders is larger than the gradient at locations between girders. This is opposite of the behavior observed at the Route 3 Bridge
  - The concrete temperatures at the bottom of the slab and the temperatures at the top flange of the steel girder have a lag associated with when their maxima and minima occur in comparison with the temperatures at the top of the slab.
  - The rate of cooling of the top of the deck compared with the rate of cooling of the steel girder can introduce tensile stress into the concrete as the girder restrains the cooling of the deck

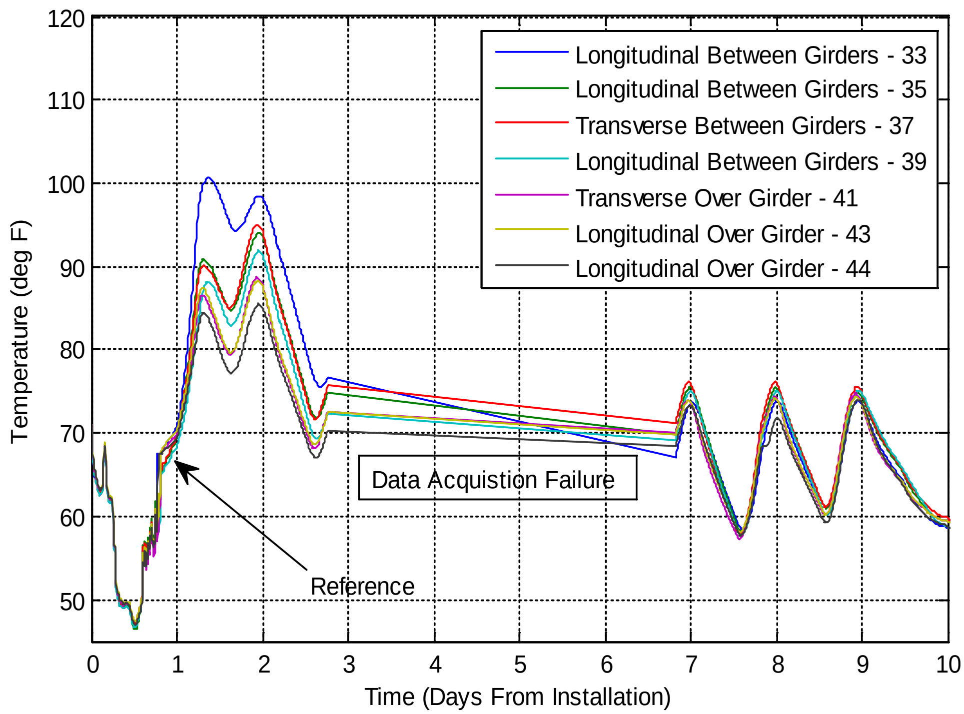

Figure 16: Peak Concrete Temperatures after Deck Pour – Route 3 over NJ Transit

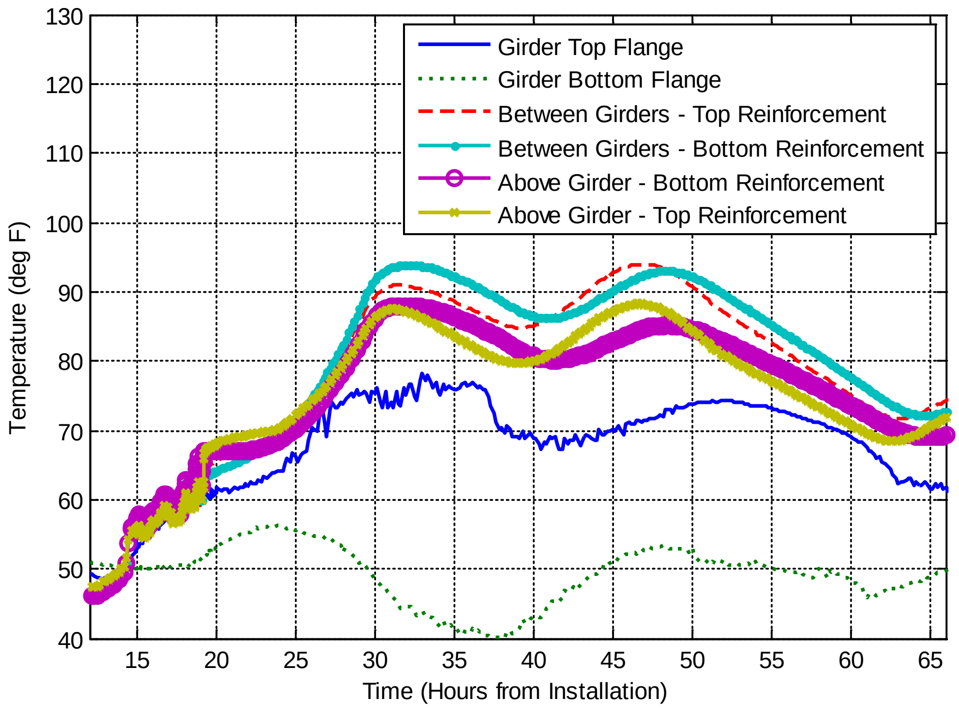

Figure 17: Comparison of Deck and Girder Temperatures - Route 3 over NJ Transit

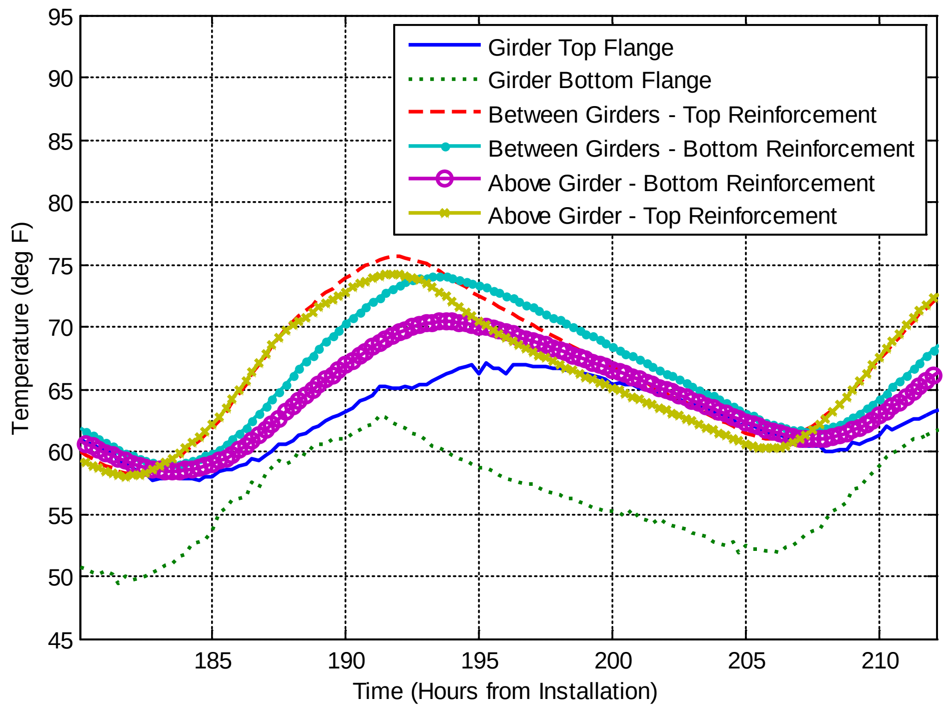

Figure 18: Comparison of Deck and Girder Temperatures – Daily Temperature Cycle - Route 3 over NJ Transit

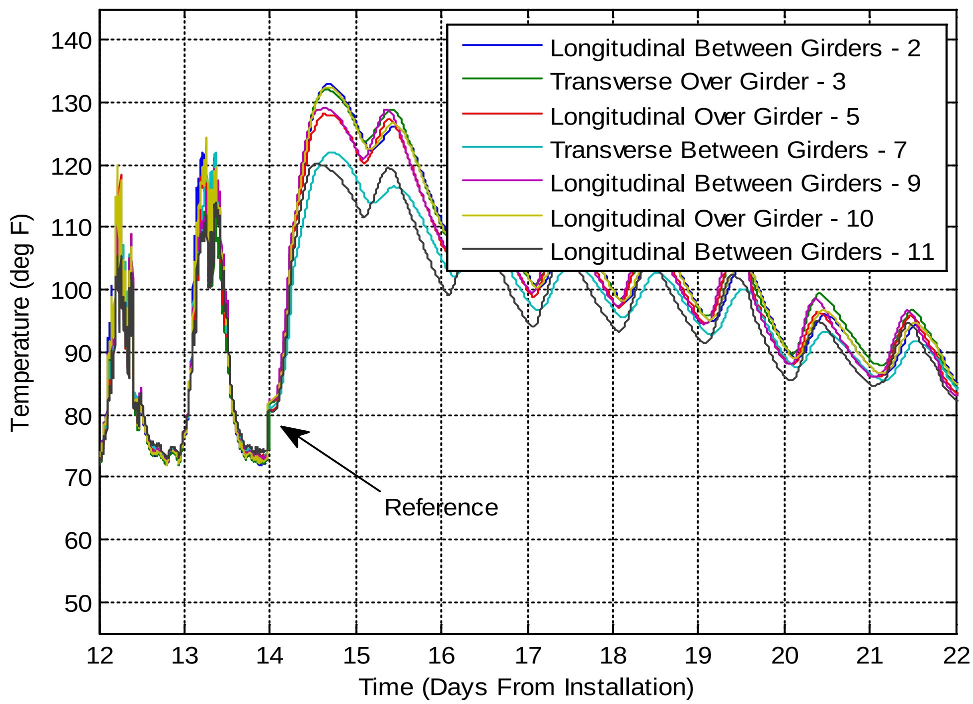

Figure 19: Peak Concrete Temperature after Deck Pour – Collings Ave

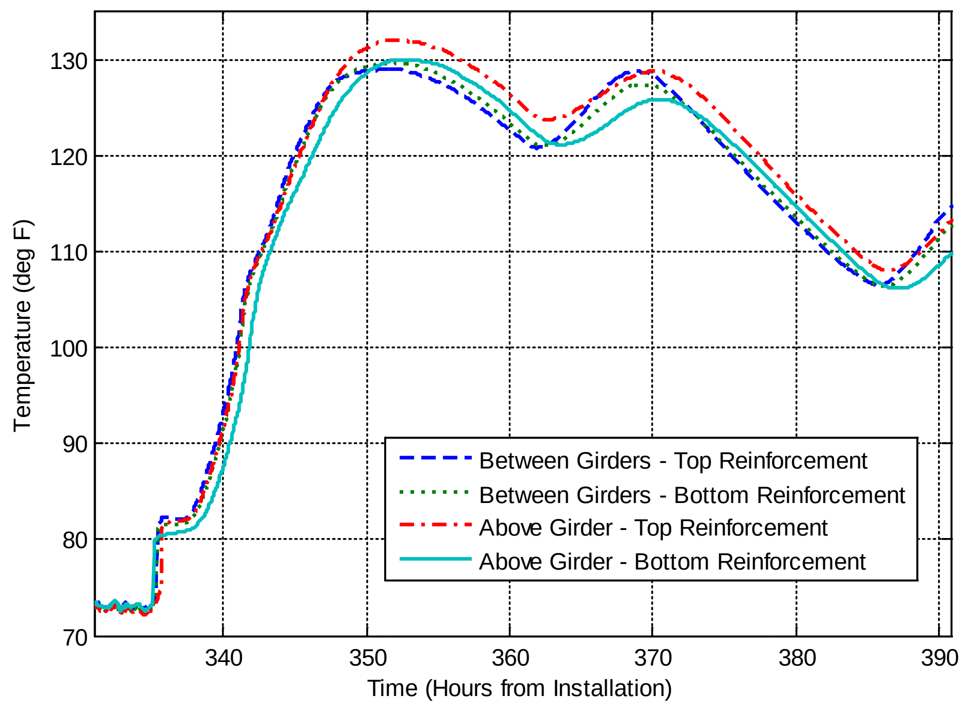

Figure 20: Comparison of Deck and Girder Temperatures – Deck Pour – Collings Ave

Total Free Strains

- The total free strain as shown in Figure 21 and Figure 22 represents a theoretical quantity and is computed from the laboratory measured free shrinkage strains, the measured temperatures, and the measured coefficient of thermal expansion of the concrete. These strains are the theoretical strains that would occur in the concrete in the absence of restraint. The total free strains provide an estimate of the shrinkage potential for the particular concrete deck. Restraint of shrinkage and thermal contractions can result in the buildup of tensile stress in the concrete. It should be noted that the free shrinkage strains presented in this report are upper bounds of the actual field free shrinkage since the specimens are cured under different ambient conditions (50% RH and 73 °F) and the measurements begin once the specimens are de-molded.
    - The shrinkage potential of both bridges due to shrinkage and temperature contraction indicates cracking is likely. Route 3 exhibited a 900 microstrain shrinkage potential while Collings Ave showed an 1100 microstrain shrinkage potential.
    - The laboratory measured free shrinkage strains are similar for each bridge deck (525 microstrain at 56 days). The additional shrinkage potential observed at Collings Avenue can be attributed to the increased temperature due to hydration that resulted in a higher peak concrete temperature from which the deck would then cool and contract to equalize with ambient temperatures.

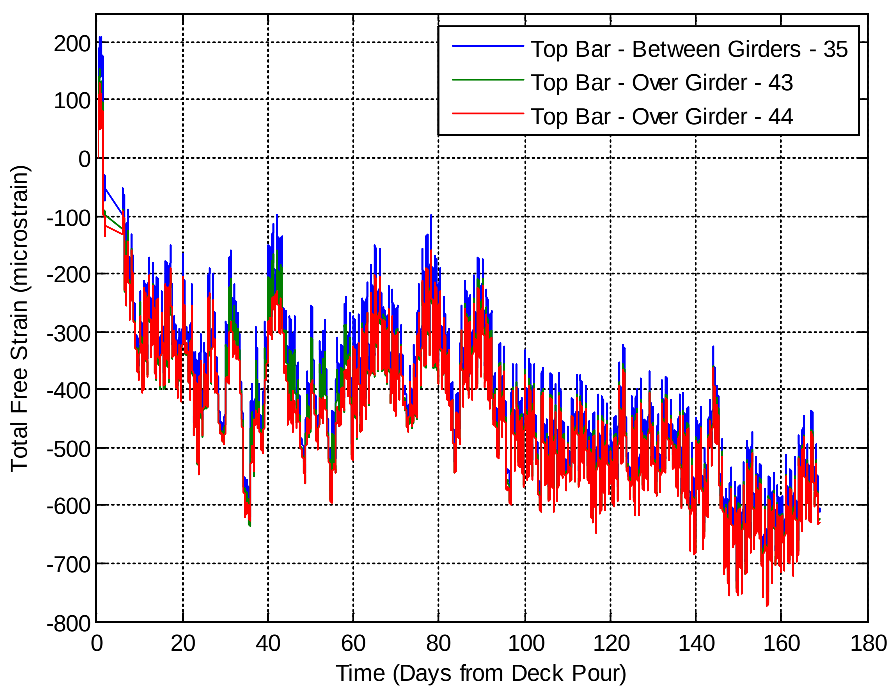

Figure 21: Route 3 - Total Free Strain at Top Reinforcement – Longitudinal

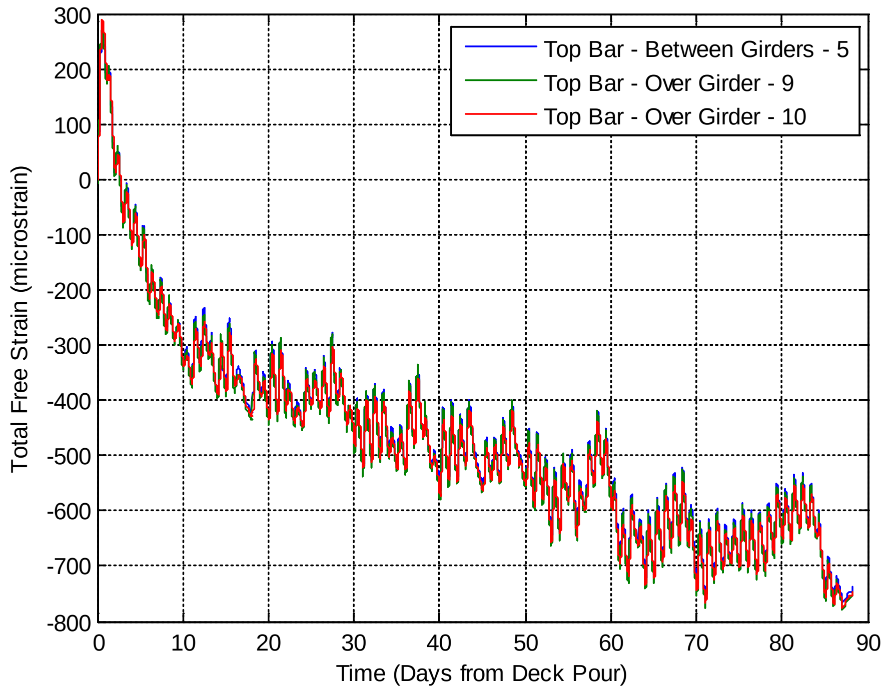

Figure 22: Collings Ave - Total Free Strain at Top Reinforcement – Longitudinal

Strain that would be measured by An External Device

- The strain that would be measured by an external gage, as shown in Figure 23 and Figure 24 is the actual strain that would be measured by an external device placed on the deck. This strain is equivalent to measured internal strains plus the strain due to changes in temperature.
    - The total strains for Route 3 show a maximum expansion of 240 microstrain and the maximum contraction of concrete is 170 microstrain.
    - The total strains for Collings Ave show that the maximum expansion of concrete is 220 microstrain and the maximum contraction is 400 microstrain.

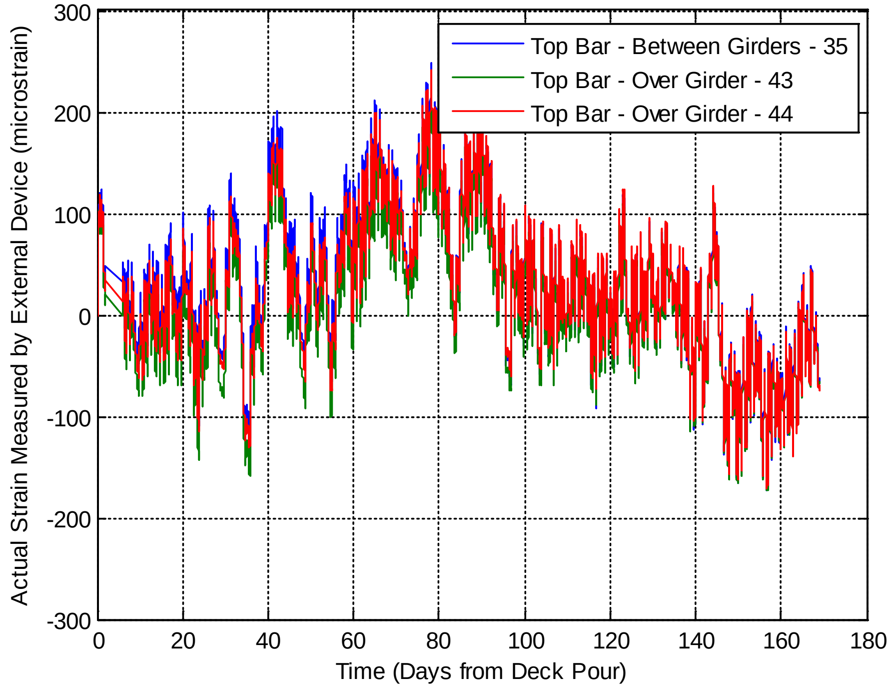

Figure 23: Route 3 - Strain That Would Be Measured by an External Device – Longitudinal Top Reinforcement

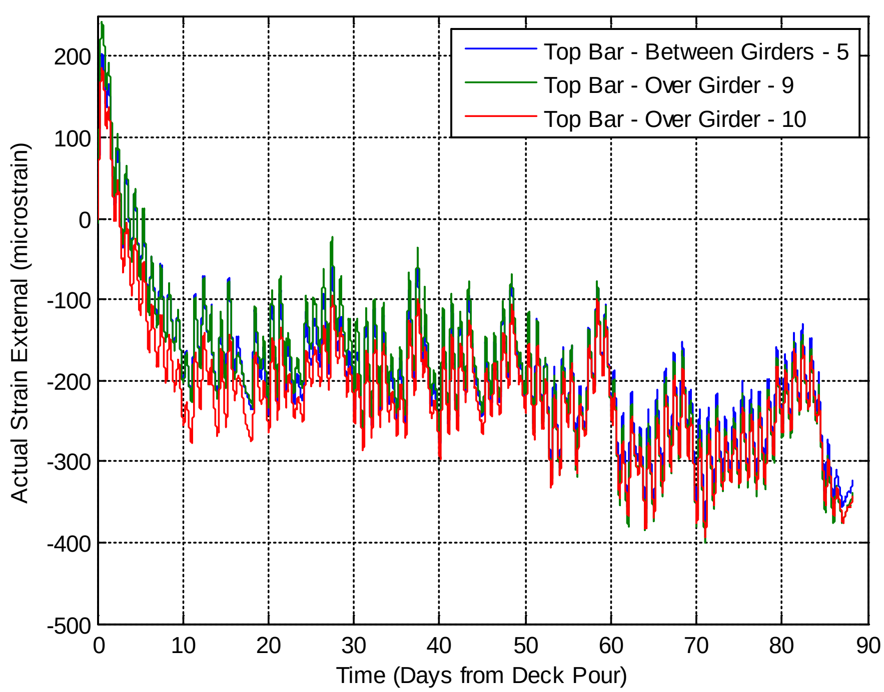

Figure 24: Collings Ave - Strain That Would Be Measured by External Device – Longitudinal Top Reinforcement

Corrected Strain

- The corrected strain of the concrete as shown in Figure 25 and Figure 26 is the measured strain corrected for the difference in the thermal expansion coefficients between the gage and the concrete. This strain is the portion of the total free strain that is not restrained, since the member is restrained from expansion and contraction but the wire inside the gage is not.
  - The corrected strains for Route 3 show three main trends, a buildup of compressive strain immediately after the pour due to the increase in temperature from heat of hydration resulting in the wire in the gage losing some tension, which decreases the vibrational frequency of the vibrating wire gage, thus indicating a compressive strain. This is followed by a buildup of tensile strain as the deck cools and contracts causing the gage to register a tensile strain as the wire contracts. This effect is somewhat balanced by the compressive strain applied to the end blocks by the concrete as it shrinks. The tensile strain is then alleviated by an expansion of the deck due to an increase in temperature of the deck during the spring and summer. Once the deck begins to cool during the fall, the compressive strain buildup over the summer starts to subside and the trend reverses slightly. This behavior is illustrated in Figure 4.
  - The corrected strains for Collings Ave show two trends, a buildup of compressive strain occurs immediately after the placement of concrete due to the change in temperature from hydration resulting in the gage registering a compressive strain due to the wire expanding and losing tension. This rate of the compressive strain buildup begins to subside as soon as the peak temperature during hydration is reached and the deck begins to cool and the concrete begins to contract and undergo shrinkage.

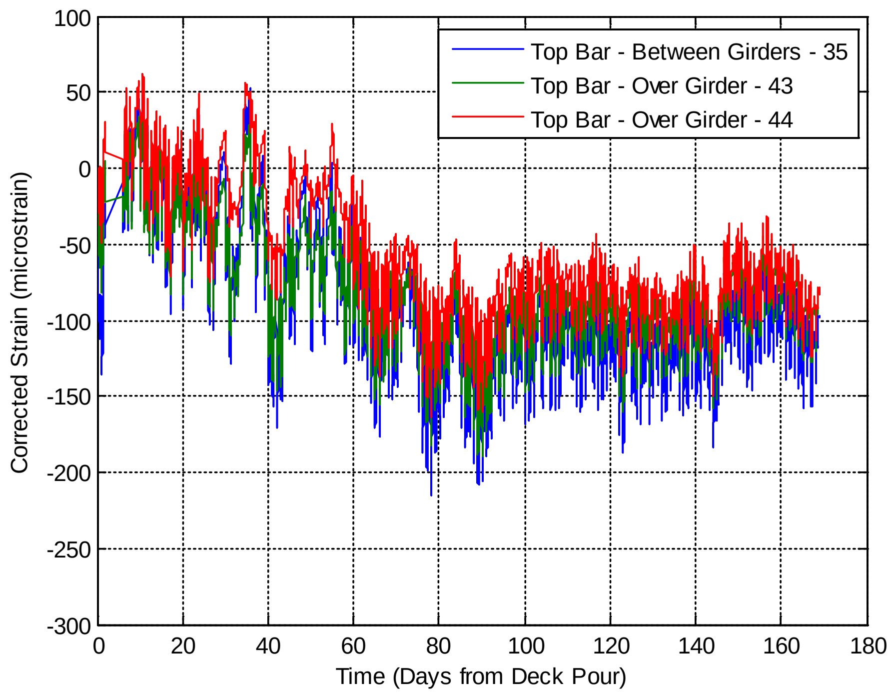

Figure 25: Route 3 - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Top Reinforcement

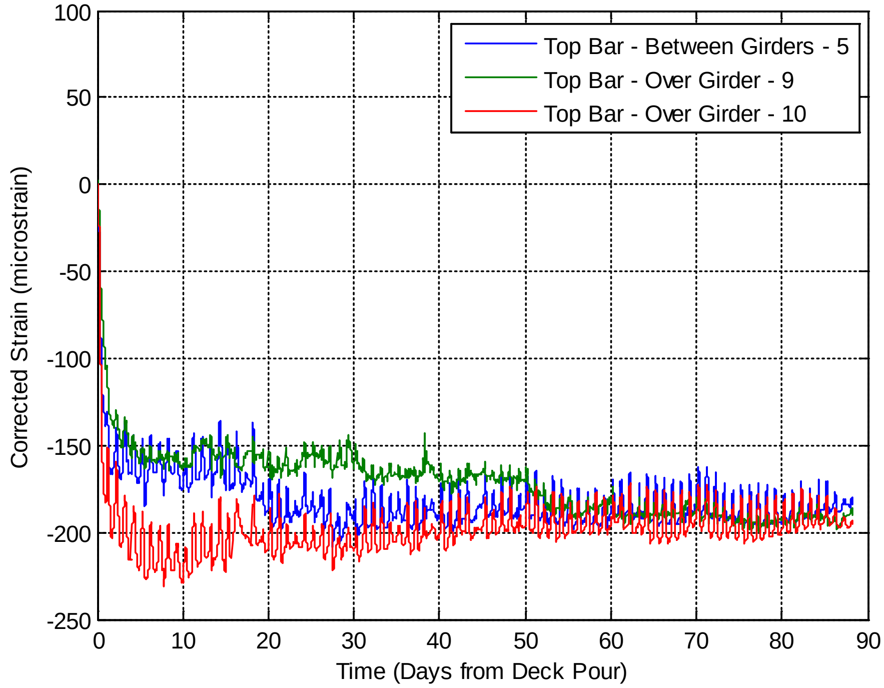

Figure 26: Collings Ave - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Top Reinforcement

Restraint of Thermal Loads for Route 3

- The magnitude of thermal restraint was estimated after 150 days of curing on the Route 3 Bridge assuming at this point in time the increase in drying shrinkage strains is minimal. More specifically, the performance of the deck following 150 days was analyzed using the following assumptions: (a) live loads were filtered due to the sampling regime, (b) additional shrinkage strains are minimal, and (c) creep is negligible. As a result, the change in strain after 150 days can be assumed to be caused by temperature only. Plotting the actual strains versus temperature results in a plot that can be fit linearly to identify a factor relating the change in strain to the change in temperature. This factor when multiplied by the change in temperature gives the actual free strain due to temperature alone. If the unrestrained thermal strain is subtracted from the actual free thermal strain, then the stress producing strain due to thermal loads is found. The thermal strain that is restrained can be compared with the theoretical free thermal strain to identify a magnitude of restraint at each gage.
- The magnitude of restraint identified for Route 3 indicates that approximately 20 to 35 percent of the temperature induced strain is restrained. If 20 to 35 percent of the theoretical free thermal strains are restrained, the stresses shown in Figure 27 would result assuming an elastic modulus versus time curve. The stresses shown indicate from the identified magnitude of restraint, the restraint of the measured temperature changes alone will not produce enough stress to exceed the tensile strength (600psi) of the concrete used on the Route 3 Bridge. Therefore, there needs to be other contributing mechanisms to cause the deck to crack. These strains would most likely arise from

shrinkage. Unfortunately a similar analysis for the Collings Ave Bridge was not possible due to vandalism and theft of the data acquisition system.

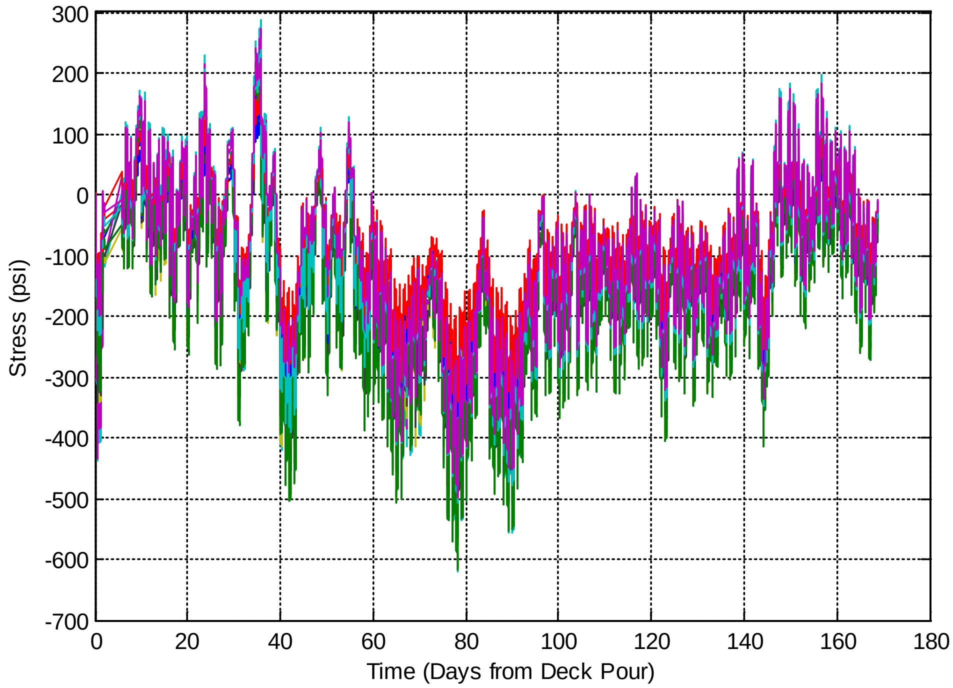

Figure 27: Stress Resulting from Restrained Thermal Strains – RT3

Stress Producing Strain

An upper bound of the stress producing strain can be estimated by subtracting the theoretical free strain from the corrected strain. If the deck were fully restrained the stress producing strain would be equal to the total free strain (including temperature and shrinkage) and if the deck were unrestrained there would be no stress producing strain The strains presented in this section provide a theoretical upper bound on the stress producing strain in the concrete since free shrinkage strains are measured under controlled ambient conditions which are more severe than those experienced in the field as discussed previously. The field free shrinkage will most likely be less than the laboratory measured free shrinkage. The total stress producing strain curves for Route 3 and Collings Ave are given in Figure 28 and Figure 29 respectively.

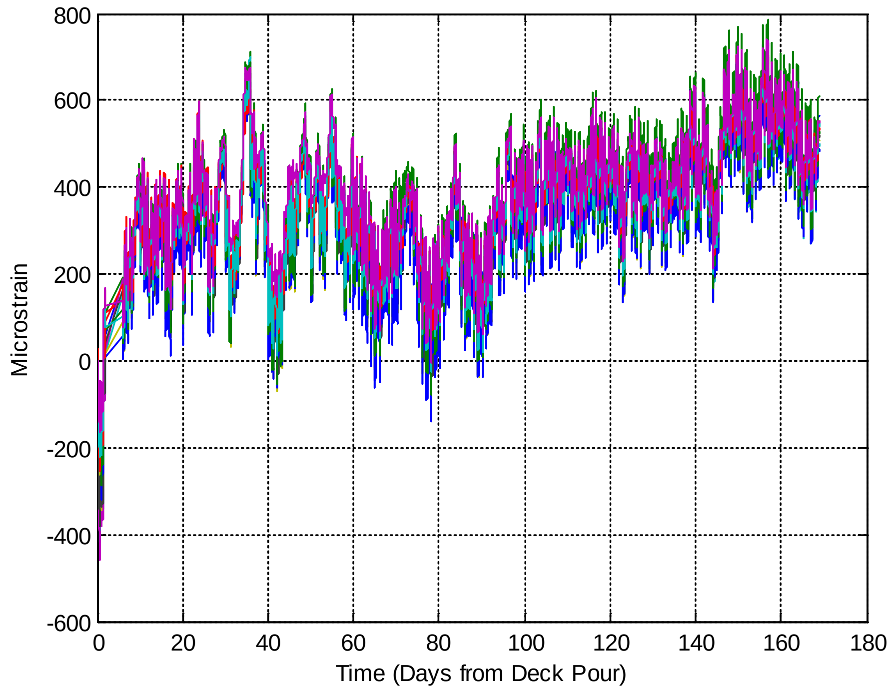

Figure 28: Theoretical Stress Producing Strain – Route 3

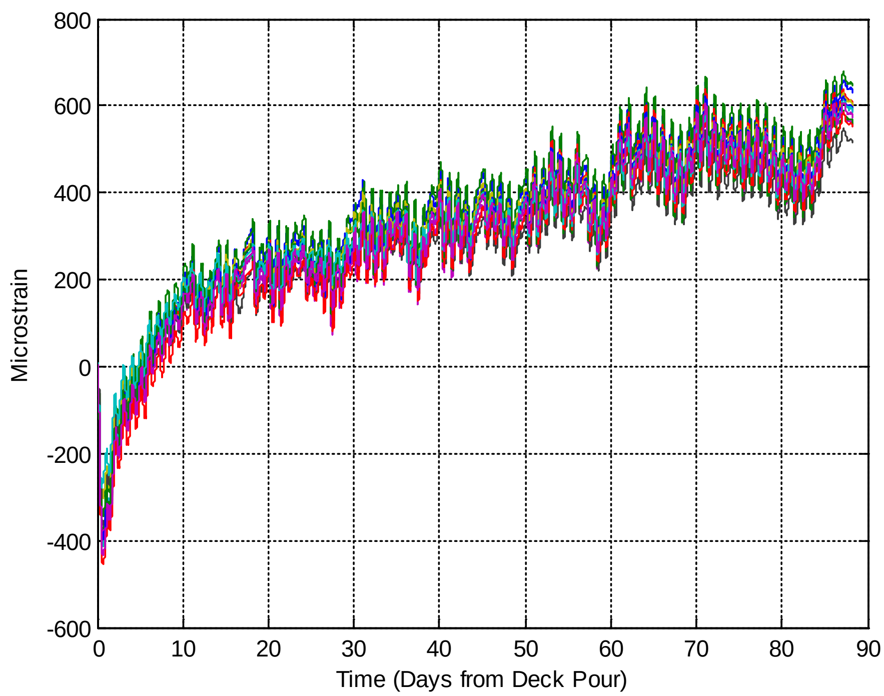

Figure 29: Theoretical Stress Producing Strain – Collings Ave

From the previous figures, the total stress producing strain for the Route 3 Bridge at 56 days after the deck pour averages 350 microstrain depending on the gage location. The total stress producing strain for the Collings Ave Bridge at 56 days after the deck pour averages approximately 400 microstrain. The maximum strain at 88 days is 650 microstrain while the minimum strain at 2 days was approximately -400 microstrain.

Observed Cracking

An evaluation of the Collings Ave bridge deck was performed to identify deck cracking. Since traffic volume is low and there are pedestrian sidewalks, the evaluation did not require traffic control to take photos and measurements of the observed cracks. Figure 30 shows two observed cracks and Figure 31 highlights the observed cracks and provides the spacing between the two cracks. The cracks are spaced between four and five feet apart along the length of the deck. The cracks are present in the sidewalk and continue into the bridge deck. There are cracks at the acute corners near the supports that propagate from the joint to the sidewalk as shown in Figure 32.

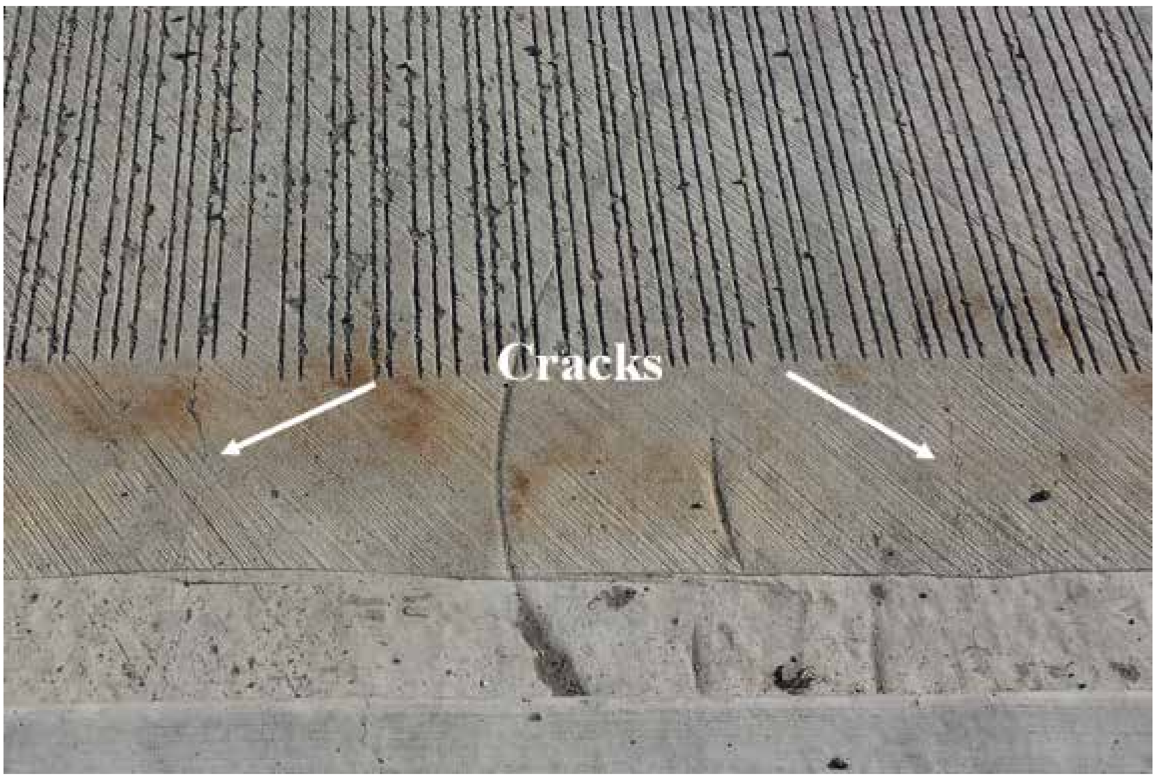

Figure 30: Collings Ave Bridge Deck Cracking

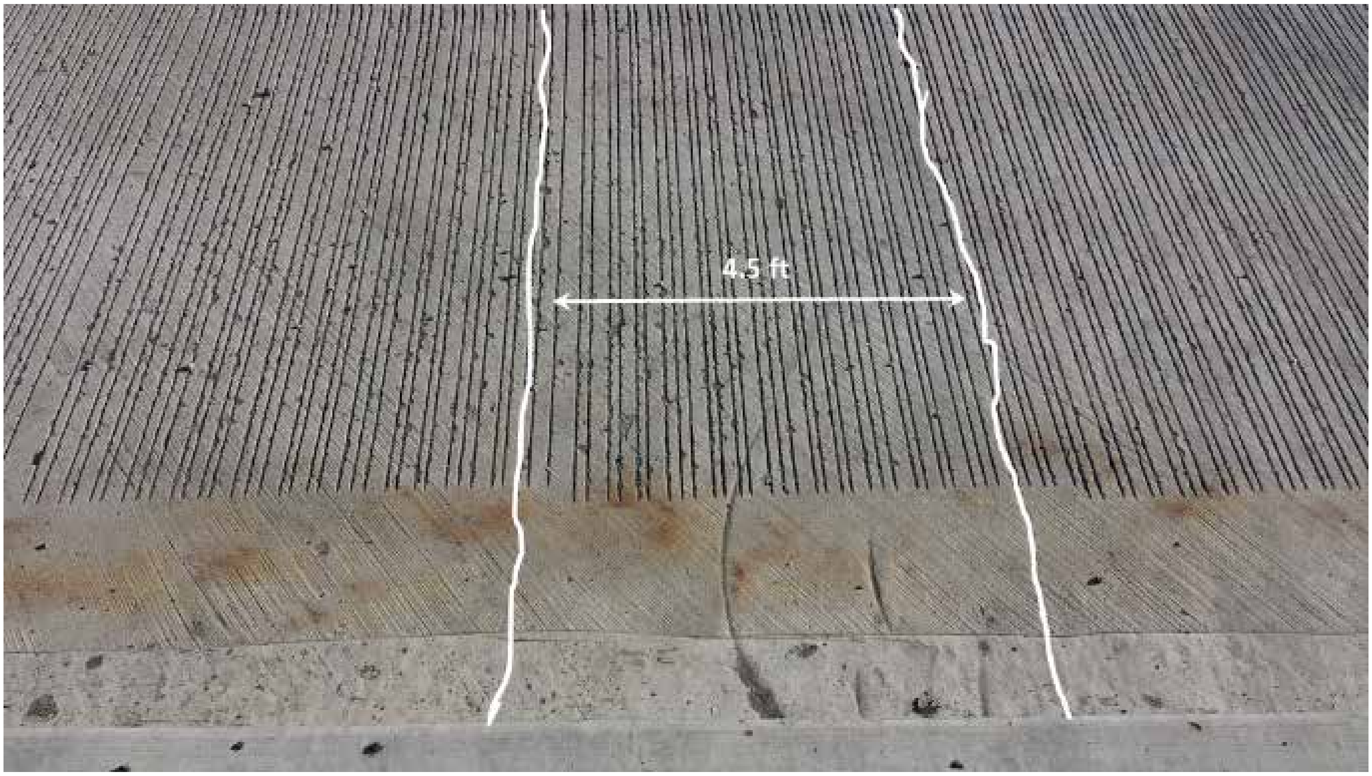

Figure 31: Collings Ave Bridge Deck Cracking - Spacing

Figure 32: Collings Ave Bridge Deck Cracking – Acute Corner

The strain measurement testing on new concrete decks provided significant data and analysis that resulted in the following recommendations:

- The laboratory measured drying shrinkage strains indicate the potential for early age cracking of concrete and the ultimate shrinkage and rate of shrinkage should be reduced if possible. There are several classifications of HPC specified by FHWA and specifying a mix that has a lower free shrinkage at 56 days may provide improved resistance to cracking due to restrained shrinkage. Many agencies place a limit on the drying shrinkage at 56 days and it is recommended NJDOT consider implementing such a provision.
- The ambient temperature and the temperatures of the concrete during pours should be monitored closely.
    - High temperatures in the concrete early in the curing process may lead to autogenous shrinkage.
    - A large temperature differential between the ambient and peak temperature in the deck due to the heat of hydration will induce compressive strains in the concrete. Typically a portion of this contraction is restrained creating tensile stresses in the deck. Therefore, limiting the differential between the ambient temperature and the peak concrete temperature can reduce the amount of theoretical thermal strain that can occur and be restrained.
        - Limiting the peak heat of hydration can be accomplished through changes in mix design and also ensuring the concrete temperature at placement is controlled.
    - The rate at which the deck cools after the concrete pour versus the rate of cooling of the supporting steel girders is different. The top flange of the girder is insulated from the top by concrete which causes the flange to expand and contract at a slower rate than the concrete near the surface of the deck. Therefore, the steel girder provides restraint to the expansion and contraction of the concrete near the top of the deck. This restraint will induce tensile strain as the deck cools.
- The stress producing strains identified from the two decks indicate the theoretical shrinkage potential (a combination of both shrinkage and contraction due to temperature) for both mixes would be enough to produce deck cracking
- The structure including the girders, rebar, bearings, etc can influence the restraint of the bridge and it is theorized that more flexible bearings (elastomeric, etc) will allow more movement of the deck system due to thermal loads and reduce the amount of restraint of thermal loading.
    - It is important that bearing performance is maintained to ensure cracking does not occur due to unintended restraint of the bearings. Due to deterioration older bearings will likely induce more restraint than new

ones and should be evaluated prior to any deck replacement project.
  - It has been shown in previous studies (French et al. 1999, Rogalla 1996) that concrete decks supported by steel girders tend to have a higher risk of transverse cracking and increased girder spacing can reduce cracking tendency.
    - Rogalla et al. (1995) showed that smaller girders coupled with a wider spacing will reduce the tendency of the concrete decks to crack. In this study, the Route 3 Bridge has a narrower girder spacing coupled with larger girders (top flange 24” x 1.5”, 7’ 8” O.C, 54” deep webs) versus the Collings Ave Bridge (top flange 18” x approx. 1”, 8’ 1” O.C, 36” deep webs). It is hypothesized the girders at Route 3 would provide more restraint than those at Collings Ave due to the girder size and number of shear studs (4 per row at Route 3 and 3 per row at Collings Ave). The minimum number of shear studs and the maximum shear stud spacing required to meet strength and fatigue requirements for shear stud design should be implemented.

- NCHRP synthesis report 333 has shown that bar size, type, spacing and distribution have a significant effect on the tendency for cracking. Limiting transverse bar size and or maximizing transverse bar spacing is recommended. French et al. (1999) also recommended limiting transverse bar size and/or maximize transverse bar spacing to reduce cracking in bridge decks. Krauss and Rogalla (1996) recommend use of No. 4 bars with maximum spacing of 6 inches.

Comparative Durability Analysis Results

Field work – Observations

All 10 bridge decks, the 8 existing and the 2 new, were inspected between April and October 2013 by Michel Plante, Eng., and Patrick Power, Jr. Eng., from SIMCO. The existing bridge decks were variably affected by early-age cracking. For the existing decks, the field work consisted in an assessment of the bridge deck current condition and the supervision of the coring operations. The crack patterns, width and density were documented. In the case of new decks, the field work consisted in witnessing the deck casting operations and collecting fresh concrete samples. The observations on each bridge are summarized in Table 10.

Table 10 – Summary of observations on the decks

Bridge	Crack width	Crack depth (in.)	Crack length (ft)	Crack type	Remarks
Route 70	Hairline to 0.006 in. (0.2 mm)	Up to 6	1 to 5	Transverse	· Section investigated was 23 ft long by 25 ft wide. · Very few cracks on the bridge. Few cracks on the east approached slab. · Only 4 transverse fine cracks, 2 to 5 ft long. · 3 to 4 cracks on the east approach slab. · Hairline cracks 6 to 12 in. long originate from deck abutment, parallel to longitudinal axis.

Smith Street	Average of 0.006 in. (0.2 mm)	Up to 6	5 to 17	Transverse	· Section investigated was 95 ft long by 17 ft wide. · Most of the cracks initiate from construction joint. · From 5 to 17 ft long. · Deck vibrates with passage of trucks.
Route 52 Elbow	Average of 0.009 in. (0.25 mm)	2 to 5	2 to 30	Random, transverse, longitudinal	· Over the 300 ft long section, multiple cracks. · A section between the 180 and 300 marks was overlaid. · Cracks follow no distinctive pattern. · Deck vibrates with passage of trucks.
Route 52 Rainbow	Average of 0.009 in. (0.25 mm)	4 to 6	2 to 30	Random, transverse, longitudinal	· Over the 300 ft long section, numerous cracks of different shapes. · Cracks follow no distinctive pattern. · Deck vibrates with passage of trucks.
Creek Rd	Hairline to 0.009 in. (0.8 mm)	0.4 to 7	5 to 9	Transverse	• Section investigated was 260 ft long by 25 ft wide. • Cracks initiate from or near the curb. • Overall, the cracks appeared to taper to a width of less than 0.004 in. at approximately 9 ft from the curb. • The east span presented a lower crack recurrence than the west span of the investigated section.
Route 9 northbound	Average of 0.006 in. (0.2 mm)	3 to 6	5 to > 20	Transverse	· Over the 300 ft long section, about 55 fine transverse cracks. · Cracks initiate from parapet and expand up to full width of investigated section. · This section of the deck is made with three different concrete pours. · Concentration of crack is similar from one pour to the other. Deck vibrates with passage of trucks.
Route 9 southbound	Average of 0.006 in. (0.2 mm)	2 to 6	2 to > 20	Transverse, plastic shrinkage, random	· Over the 300 ft long section, about 30 fine transverse cracks. · Cracks initiate ± 6 ft from parapet and expand up to full width of investigated section. · This section of the deck is made with three different concrete pours. · Concentration of crack is different from one pour to the other. · Deck vibrates with passage of trucks.
OCLP	Hairline to 0.006 in. (0.20 mm)	0.25 to 5	2 to > 20	Transverse	· Over the 300 ft long section, about 15 very narrow transverse cracks from parapet through opposite side of the deck · 4 to 5 longitudinal cracks up to 10 ft long. · Cracks initiate close to parapet and expand up to full width of investigated section. · Other cracks (plastic shrinkage, crazing) were revealed only by wetting and drying.

Sampling

Cores samples were extracted from the existing bridge decks while for the new decks, concrete was sampled at the time of casting.

The concrete cores from the existing bridge decks were sampled in conformity with ASTM C42. Most cores were taken in uncracked concrete to characterize the physical properties, the transport properties (i.e., ionic diffusion and moisture coefficients and volume of permeable voids) and the concrete condition of the bridge decks. A few cores were also extracted over cracks considered representative of the observed patterns. Table 11 presents a summary of the core sampling on the existing bridge decks.

For the new bridge decks, the concrete cylinders used in the laboratory investigation were cast from concrete delivered onsite during deck pouring operations. All cylinders were produced in conformity with Section 7 of ASTM C31 and all rectangular prisms were produced in conformity with ASTM C157. Samples were obtained from more than one truck for both bridge deck pours. Table 12 presents a summary of the fresh concrete sampling for laboratory testing.

Table 11 – Core sampling in existing bridges

Bridge name	Number of cores	Area of deck investigated		Number of pours
		Length (ft)	Width (ft)	
Route 70	17	70	27	2
Smith Street	17	100	17	1
Route 52 Elbow Channel	15	300	20	3
Route 52 Rainbow Channel	15	300	20	3
Creek Road	17	260	25	2
Route 9 northbound	17	300	20	3
Route 9 southbound	17	300	23	3
Ocean City Longport Bridge	17	300	20	2

Table 12 – Concrete sampling on new bridges

Bridge name	Number of trucks sampled	Number of cylinders produced	Number of beams produced*
Collings Avenue	2	12	3
Route 3	3	18	3

*11.75 x 3 x 3 in. beams produced in conformity with section 7 of ASTM C157.

Laboratory test results

The main conclusions of the laboratory testing results are summarized hereafter. Detailed results are given in the individual report prepared for each bridge, given in the Durability analysis report, Appendix C.

Existing bridges

The compressive strength is significantly higher than the verification strength[4], although in the expected range of long-term values for the mixtures tested. It is interesting to note that the strength of HPCs is not significantly higher than that of Class A concretes. This may be explained by the fact that all mixes had a similar low water-binder ratio and that the only difference between both categories is that HPCs are made with SCMs.

The air-void system is not consistently adequate to resist damage induced by freezing and thawing. Only 2 of the 8 existing bridges have a spacing factor below 0.008 in. (200 μm), the recommended value according to ACI 201.1R – *Guide to Durable Concrete* for normal weight concrete. This value represents a general recommendation to ensure the concrete is resistant to freezing and thawing damage. In recent research, it was observed that each concrete has a critical spacing factor value, which can be as high as 400 μm in certain cases, most particularly for high

[4] The verification strength specified by NJDOT is 5,400 psi. This strength is the requirement that applies to concrete sampled during a verification batch..

performance concrete.[5] All measured spacing factors are lower than 400 µm, which could explain the fact that the investigated decks do not exhibit severe scaling or other forms of damage due to poor frost resistance.

Thermal expansion coefficients are in the order of 11 to 14 microstrains/°C. This is in the upper part of the commonly reported range (approximately 6 to 15 microstrains/°C[6,7]). The investigated concretes are made with trap rock, which is not known to yield high thermal expansion coefficients. The high thermal expansion coefficients could be related in part to the paste content and poor granular packing density (i.e., combination of sand and coarse aggregates).

The chloride contamination is overall consistent across each investigated bridge deck, except for the Route 9 northbound bridge, for which variable conditions were recorded. Older decks tend to be more contaminated, although the contamination is not always proportional to the age of the decks, which is an indication of variable exposure conditions and/or transport properties. Interestingly, no clear influence of the cracks on chloride contamination could be noted. Cracked cores tend to be more contaminated – although not always the case – but the higher contamination may either be present near the surface, deep from the surface, or on the entire depth. Table 13 summarizes the influence of cracks on chloride penetration in existing decks.

The diffusion coefficients of HPCs are indicative of a good resistance to chloride penetration, which was expected due to the presence of supplementary cementitious materials (SCM). SCMs increase the tortuosity and decrease the connectivity of the porous network. This results in lower ionic diffusion coefficients, which translates into a better resistance to chloride penetration. Similar to the contamination, the influence of cracks on the diffusion coefficient is not constant. Table 14 gives the influence of cracks on the diffusion coefficient.

It is interesting to note that the Route 52 Elbow Channel samples present the lowest diffusion coefficients along with the lowest compressive strengths, while the Route 9 southbound samples present some of the highest diffusion coefficients along with the highest compressive strengths. This is an indication that high compressive strengths do not necessarily lead to improved durability.

Table 13 – Influence of cracks on chloride penetration

Bridge	Influence of cracks
Route 70	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core not the most contaminated, even at higher depth

[5] M.Pigeon and R.Pleau, *Durability of Concrete in Cold Climates*, E & FN Spon, 1995
[6] A.M. Neville, Properties of Concrete, Fourth Edition, Longman Group Limited, 1995
[7] ACI 224R – Control of Cracking in Concrete Structures

Smith Street	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core more contaminated deeper than 0.5 in.
Route 52 Elbow Channel	· Influence of crack mainly at a shallower depth (< 0.5 in.) · Cracked core more contaminated in first 0.5 in. only
Route 52 Rainbow Channel	· Cracked core more contaminated over all depth · Not a significant effect though
Creek Road	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core more contaminated deeper than 0.5 in.
Route 9 northbound	· No cracked core · Variable contamination
Route 9 southbound	· Cracked core more contaminated over all depth · Significant effect of crack
Ocean City Longport Bridge	· No influence of crack

Table 14 – Influence of cracks on the diffusion coefficient

Bridge	Influence of cracks
Route 70	· Cracked core lower than 3 uncracked and higher than 2
Smith Street	· Cracked core approximately twice the diffusion coefficient of uncracked (11.6 vs. 5-6)
Route 52 Elbow	· Cracked cores (2) identical and lower than 1 uncracked and higher than 3
Route 52 Rainbow	· Cracked core lower that 4 uncracked and higher than 1
Creek Road	· Cracked core lower than 1 uncracked and higher than 4
Route 9 northbound	· Cracked core higher than all 5 uncracked cores (5.6 vs. 5.4 for the second)
Route 9 southbound	· Cracked core is higher than all 5 uncracked cores
OCLP	· Cracked core higher than all 5 uncracked cores (11.0 vs. 10.1 for the second) · Variable values

The volume of permeable voids is higher for HPCs than for conventional mixtures. In this study, it was found that the volume of permeable voids is roughly 14% for HPC while it is 11.5% for Class A concrete. Volume of permeable voids varies proportionally with the initial quantity of water in the concrete mix. Thus, the explanation could either be related to a higher water content or the aggregate porosity.

Petrographic examinations carried out on cores extracted from the existing decks suggest that the cracking observed on the decks would be caused by early-age drying shrinkage. The cracks had a maximum width of 0.02 to 0.04 in. tapering to 0.01 in. away from the surface. The cracks visible on the surface extend to various depths, from fractions of inches to the entire depth of the core. In addition to the cracks that are visible on the decks, micro-cracking was observed on the majority of the cores. Micro-cracking is either in the form of thin surface cracks extending down to several inches into the core or distributed cracking within the paste, consequent with autogeneous shrinkage. The estimated concrete characteristics correlate with the theoretical mixture proportions except in two cases. The Smith Street concrete has an estimated water-binder ratio between 0.35 to 0.45, which could be higher than the theoretical 0.40 value. Also, the Route 9 southbound bridge concrete is only made with Portland cement. No SCM was observed in the sample analyzed (Core SB-06) and explains while this deck has a high diffusion coefficient.

New decks

The compressive strengths recorded at 28 days are much higher than the verification strength at 56 days. This is in agreement with the results on existing bridges and can be related to the durability requirements that imply low water-binder ratios and the use of supplementary cementitious materials. The other mechanical properties are also high. However, the elastic modulus recorded on the Route 3 concrete is lower while the compressive and tensile strengths are higher. With respect to cracking, higher elastic moduli are not necessarily desirable because they limit the deformability, which can result in more severe cracking under imposed deformations. In this perspective, the Route 3 concrete would have a lesser tendency to crack.

The air-void network is adequate, with a spacing factor below the recommended value by ACI 201.2R of 0.008 in.

The drying shrinkage recorded on both concretes is higher than 500 microstrains at 56 days, and still increasing afterwards to exceed 600 microstrains after 112 days of drying. The NJDOT has no current standard specification for drying shrinkage but other jurisdictions usually set limits between 400 and 500 microstrains. For instance, the NJTA limits the drying shrinkage to 450 microstrain for HPCs. FHWA also provides limits for different classes of HPC, but in this case, the values are higher and are based on measurements at 180 days.[8] The values recorded on both decks are considered high and this can be a significant contributing factor explaining the presence of cracks on bridge decks.

The volume of permeable voids is higher than expected, and this is similar to what was observed on existing decks made of HPC. As mentioned previously, this can be explained in part by the type of aggregate used or by excessive water addition. Given the good mechanical properties, the high volume of permeable voids is more likely related to the aggregate used. However, variability was noted for the Collings Avenue concrete, some individual results being lower and in the expected range. This suggests either some heterogeneity in the concrete itself or it may be related to the sampling process. The diffusion coefficients are at the upper end of the expected range for the type of concrete used to build both decks. Interestingly, the values at 90 days are not much lower than those at 28 days. Some improvement was noted though for the Collings Avenue concrete, but none for the Route 3 concrete. Usually, with Class F fly ash and slag, a more significant improvement of the transport properties is expected due to prolonged hydration

[8] Based on SHRP C/FR-91-103, p. 3.25

of these SCMs. The lack of improvement of the diffusion coefficient between 28 and 90 days for the Route 3 concrete was not clearly explained but can be related to the hydration kinetics of the binder or to a dispersion problem with the fly ash, which would have caused the tested samples to have a low fly ash content.

Petrographic examinations have indicated the presence of silica fume agglomerations in the Route 3 concrete. Silica fume agglomerations indicate insufficient blending and are to be avoided because they can result in alkali-silica reaction in a short period of time. In the current case, ASR was observed in the agglomerations on concrete samples that were only a few weeks old, but damage is not expected on the Route 3 deck because agglomerations only observed in small quantities. Furthermore, poor silica fume dispersion does not allow to fully benefit from its effect on pore size distribution and transport properties. In the case of the Collings Avenue sample, the examination revealed that it was made of Portland cement with moderate amount of slag and minor amount of Class F fly ash. This is in agreement with the mix design. The degree of hydration of the new concrete samples are generally low to moderate, but this was expected considering the age of the concrete at the time the examinations were performed.

Bridge deck durability

Corrosion initiation

For steel bars embedded in concrete, corrosion can be initiated when the chloride threshold for corrosion initiation is reached or exceeded at the rebar depth, which corresponds to the concrete cover. Cover values measured on the investigated decks varied from one deck to another, even though the specified cover was the same.

For the durability analysis, a probabilistic analysis was performed for each deck. In Appendix C, a description of the critical chloride concentration required to initiate steel reinforcement corrosion in concrete is presented. For the probabilistic analysis, a value of 0.50 ± 0.05 % per mass of binder was used.

In addition, most bridge decks were made with epoxy-coated reinforcement. Epoxy coating can protect the embedded steel and prevent corrosion, but this method is not always efficient, since the coating may delaminate or exhibit flaws[9]. Corrosion was indeed observed on a core from the Route 9 northbound bridge, under the epoxy coating. Thus, as a conservative assumption, SIMCO did not consider the presence of the epoxy coating for the durability evaluation.

Concrete contamination

Based on the chloride profiles, the calculated corrosion initiation threshold and the reinforcement depth, extensive corrosion should not be ongoing at the time of the investigation for the largest

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[9] Bertolini et al. (2000) "*Corrosion of Steel in Concrete – Prevention, Diagnosis and Repair*", Wiley-VCH

part of the bridge decks, since the chloride concentration at mean reinforcement depth does not exceed the corrosion initiation threshold in 41 cases out of 43.

Service life analysis

Numerical calculations were performed for all existing and new bridge decks with the STADIUM® software. This tool can be used to:

- Determine past exposure conditions;
- Calculate future chloride ingress;
- Determine the time and/or risk of corrosion initiation;
- Compare the performance of different materials;
- Evaluate concrete degradation.

Calculation assumptions and model validation are presented in Appendix C. The durability analysis results are presented hereafter.

Calculation results

The calculation results are expressed as degradation curves, used to assess the risk of corrosion initiation as a function of time. The results can also be expressed as chloride content evolution as a function of time at a given depth. An example of each curve is given in Figure 33 and Figure 34, on which the time of the investigation is shown with the blue dotted line. For the purpose of the current analysis, the corrosion initiation risk has been classified as shown in Table 15. The summary of the calculations is given in

Table 16.

Table 15 – Classification of the corrosion initiation risk

Risk	Zone	Interval	Repairs
Mild	Green	< 15 %	No repairs expected
Moderate	Yellow	15 to 50 %	Minor repairs (patching on a small proportion of the deck)
Severe	Red	> 50 %	Major repairs (patching on a large proportion of the deck or resurfacing)

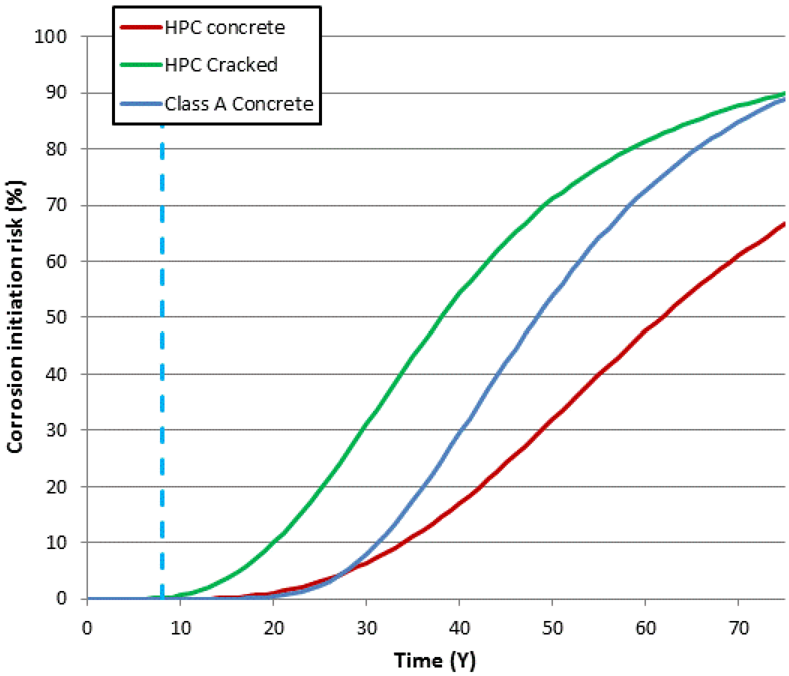

Figure 33 – Example of a degradation curve – Route 70

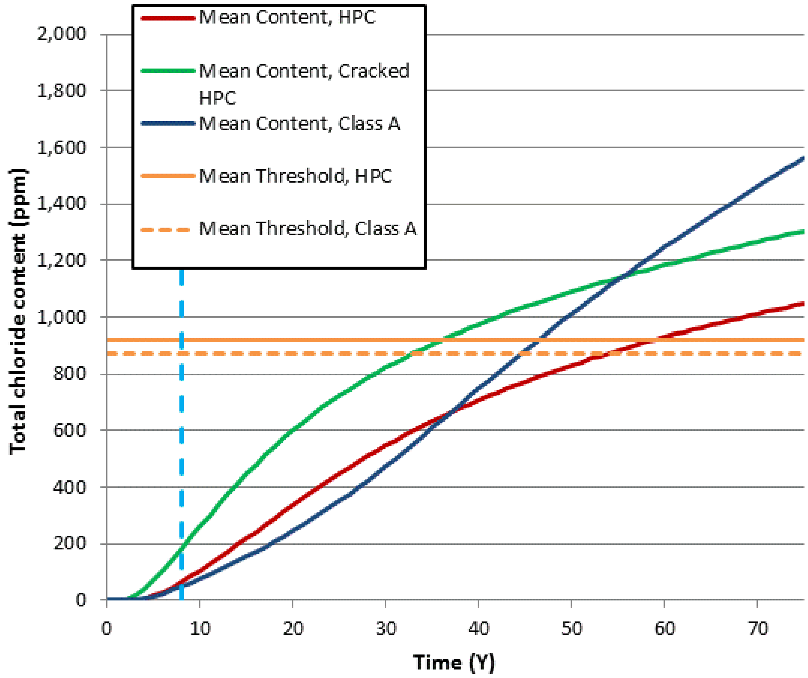

Figure 34 – Example of a chloride content evolution curve – Route 70

Table 16 – Summary of the calculations

Bridge	Condition	Corrosion risk at time intervals after construction			
		25 years	50 years	65 years	75 years
Route 70	HPC uncracked	3.1 %	31.9 %	54.7 %	66.6 %
	HPC cracked	19.5 %	71.1 %	84.9 %	89.8 %
	Class A	2.4 %	54.0 %	79.5 %	88.9 %
Smith Street	HPC uncracked	0.3 %	4.3 %	8.8 %	12.6 %
	HPC cracked	2.9 %	15.7 %	26.1 %	33.4 %
	Class A	0.0 %	0.6 %	2.7 %	5.1 %
Route 52 Elbow Channel	HPC uncracked	0.0 %	0.0 %	0.0 %	0.0 %
	HPC cracked	0.1 %	0.1 %	0.2 %	0.2 %
	Class A	0.0 %	0.0 %	0.1 %	0.4 %
Route 52 Rainbow Channel	HPC uncracked	0.0 %	0.7 %	2.1 %	3.3 %
	HPC cracked	0.5 %	5.4 %	10.6 %	15.1 %
	Class A	0.0 %	3.5 %	12.6 %	21.4 %
Creek Road*	Class A	0.2 %	7.6 %	17.1 %	24.3 %
Route 9 northbound	HPC uncracked	0.0 %	0.0 %	0.0 %	0.1 %
	HPC cracked	0.0 %	0.4 %	1.0 %	1.7 %
	Class A	0.0 %	0.0 %	0.0 %	0.0 %
Route 9 southbound	Class A	0.4 %	7.0 %	13.5 %	18.3 %
Ocean City Longport Bridge	Class A	0.2 %	22.4 %	48.4 %	64.6 %
Collings Avenue	HPC uncracked	0.8 %	15.8 %	32.3 %	43.1 %
	HPC cracked	5.0 %	36.8 %	55.4 %	65.1 %
	Class A	9.6 %	75.3 %	93.3 %	97.6 %
Route 3	HPC uncracked	0.0 %	2.3 %	6.2 %	9.9 %
	HPC cracked	0.8 %	10.7 %	20.6 %	27.9 %
	Class A	0.0 %	1.8 %	6.4 %	10.9 %

* Calculation results will be reviewed based on actual rehabilitation date

The calculations indicate that in most cases, the corrosion initiation risk will remain in the green zone, i.e., below 15 %, and this, even for cracked HPC or Class A concrete. This is particularly true for the first 25 years after construction. The only bridge for which deck repairs due to corrosion damage are not expected in the next 75 years are the Elbow Channel and Route 9 northbound bridges.

The calculations also show that, as could be expected, HPC decks are generally the most durable option, as long as they remain uncracked. However, Class A concrete also provides good durability performance for the investigated decks, especially in the first decades after construction. Based on the calculation parameters used, Class A concrete can perform better than *cracked* HPC over an horizon of 50 years after construction. At 65 or 75 years, the advantage is not as clear but the performance of Class A concrete still compares to *cracked* HPC for many of the investigated decks.

It is interesting to note that major repairs are not expected before 50 to 65 years in uncracked HPC decks. If decks were made of Class A mixtures, major repairs would have been expected before 50 years for only two of the investigated bridge decks (Route 70 and Collings Avenue).

One of the most striking conclusions that could be drawn from the calculations was the evidence of a combined influence of concrete properties, exposure conditions and concrete cover to cost-effectively design durable structures. This type of calculations may therefore be used to evaluate the benefit of using HPC as a function of the salt loading on a bridge or the actual cover depth. Furthermore, the service-life calculations can also be used, in combination with destructive or non-destructive testing, to prioritize maintenance interventions by identifying bridge decks that are likely to deteriorate faster.

4. Correlations between data

Although the testing program was not planned especially for that purpose, some interesting correlations could be identified between the different data sets collected by BRP partners. It has to be mentioned that site logistics required a minimum closing time on the bridge decks, which implies that coring and NDE were performed in a short time interval, on a two-day basis. One preferred way to maximize the benefit of a deck investigation program would be to select coring locations based on the NDE results, in hot spots and areas that are in good condition.

Both the NDE and the durability analysis indicate the decks are, at the time of inspection, in a generally good condition. The results do not indicate the presence of extensive deterioration in the investigated decks. With the NDE techniques, a corrosive environment was detected for the Route 70 and Smith Street Bridge decks while signs of delamination were observed on the OCLP and Route 52 Bridge decks. In the latter case, the results would have to be verified because the particular geometry of the deck could have influenced the measurements. The cores only revealed the presence of delamination on the Creek Road Bridge deck. As mentioned previously, cores were not systematically taken in areas where NDE indicated the presence of delamination, and it explains the apparent absence of correlation between the results. In general, the corrosive environment and presence of delamination on the Route 70, Smith Street and OCLP Bridge decks is in agreement with the durability analysis that indicated higher corrosion risks for these bridges. In addition, the durability analysis indicated a higher corrosion risk for the Creek Road Bridge. In the case of the Route 52 Bridges, particularly the Elbow Channel Bridge, the presence of delamination is not in agreement with the measured contamination and properties and with the calculated corrosion risk. This suggests that the NDE results are likely to have been influenced by the bridge geometry, as noted in the NDE analysis section of this report.

Unfortunately, the presence of epoxy-coated bars has prevented the performance of half-cell potential measurements on most bridges. However, resistivity measurements could be performed on all bridge decks and were found, wherever comparisons were possible, to correlate well with half-cell potential measurements. These measurements were used to determine possible correlations with the contamination severity or the materials properties. The main observations are summarized in Table 17.

Furthermore, correlations were noted between the NDE and the durability analysis in the case of the Creek Road Bridge. The NDE provided low resistivity results that were not expected for a bridge of this age. The durability calculations indicated low exposure conditions, much different than those found on the other bridges. At the time of writing this report, indications suggest that the deck could have indeed been repaired or rebuilt. This would mean that the rather unusual results obtained on this bridge through the NDE and durability analysis could be explained by the fact it is much younger than originally estimated when it was selected as part of the study.

Table 17 – Correlations between NDE and materials characterization

Bridge	Core locations vs NDE results	Correlations
Route 70	Cores 2,5,12 in higher resistivity areas Others in low-resistivity	No correlation, properties in higher resistivity are not significantly better.
Smith St	Cores 1,2,4,12, 16, 17 in lower resistivity areas	Good correlation: Core 1: cracked, high diffusion coefficient Core 4: high surface chloride contamination Core 16: areas of localized high porosity, paste content and higher estimated water-binder Core 17: high diffusion coefficient, although still close to others
	Cores 6,10,13,14 in higher resistivity areas	Core 6: high volume of permeable voids (no correlation) Good correlation: Core 10: low diffusion coefficient Core 14: lowest chloride contamination observed
Elbow	All cores in high-resistivity area	No correlation possible
Rainbow	All cores in high-resistivity area	No correlation possible
Creek Rd	All cores in high-resistivity area	No correlation possible
Rte 9 NB	All cores in high-resistivity area	No correlation possible
Rte 9 SB	All cores in high-resistivity area, but Core 15 close to a low-resistivity area	Good correlation with core 15: cracked and more contaminated
OCLP	All cores in high-resistivity area	No correlation possible

In addition, correlations could be found between instrumentation and materials testing. The tests performed on concrete samples indicated high shrinkage and thermal expansion coefficients, which correlate with the high strains recorded. The strain calculations used materials test results to evaluate the cracking tendency. Although no cracking was reported on the Route 3 project, the team observed cracking on the Collings Avenue Bridge. This observation correlates with the strain analysis and predictions made.

5. Recommendations and guidance based on current testing program

The literature review and current testing program performed as part of the BRP allowed to highlight some contributing factors that could explain the presence of cracks in bridge decks. These factors include:

- A high shrinkage potential;
- High thermal expansion coefficients;
- A high binder content;
- High compressive strength and elastic modulus;
- Stay-in-place forms.

All these factors contribute in a way to create conditions favorable to the development of cracks on bridge decks. In addition, the type of binder, the casting sequence, the curing procedures and the behavior under loading can also contribute to the formation of cracks.

To reduce the risk of cracking and improve the performance of decks with respect to crack formation, the following actions could be implemented:

- Limit the binder content;
- Act on the binder type and proportions;
- Limit the strength if high strength is not required;
- Allow the use of higher water-binder ratios if durability requirements are met with the use of pozzolans;
- Increase the aggregate content;
- Use largest possible aggregates;
- Provide wet curing as soon as possible for as long as possible;
- Use removable forms;
- Limit the degree of restraint provided by the SIP forms;
- Use internal curing aggregate;
- Use shrinkage-reducing admixtures;
- Use shrinkage-compensating concrete;
- Use fibers.

It is important to note these measures need not all to be implemented. A combination of some would lead to a reduction in the risk of cracking. Moreover, all these measures may not be readily applicable, although, some are quite straightforward. The team members propose, at least initially, to add a shrinkage limitation in the NJDOT specifications for HPC. If it is estimated that conventional Class A concrete decks also exhibit cracking, this requirement could be extended to *all* bridge decks. Conformance with this shrinkage specification would have to be verified on a periodical basis, not necessarily on a project-specific basis, similar to the aggregate reactivity verification. The addition of a shrinkage limitation could be completed by guidance on mix designs, such as the maximum binder content, the total aggregate content and the sand/aggregate ratio.

The compressive strength may also be limited to a range instead of a minimum. In addition, and specifically for HPC made with SCM, the strength and durability requirements should not explicitly be mentioned to be possibly attained at 28 days instead of 56 days since this leads to overdesigning mixtures, mainly by using a cement content higher than required, which results in an increased cracking tendency.

In summary, possible improvements to the specifications could be as indicated in Table 18. These specifications would only apply to bridge decks.

Table 18 – Recommendations for HPC bridge decks specifications to limit cracking

Parameter	Recommendation
Shrinkage	Limit shrinkage at 56 days after 14 days of wet curing to 450 microstrains
Binder content	Limit the binder content to 650 lb/cy Limit cement content to 550 lb/cy
Water-binder ratio	Limit water-binder ratio to 0.40 to 0.45
Paste content	Limit the volume of water and binder to 28 % of the total volume
Aggregate content	Use at least 65 % of aggregate per volume
Sand/aggregate ratio	Limit the sand proportion to 45 % per volume of total aggregate
Strength	Limit the strength gain at 28 days to 5,000 psi
Chloride permeability	Limit the average permeability at 28 days to 1,500 coulombs with no individual values higher than 2,000 coulombs
Acceptance interval	Require testing to be done at 56 days instead of accepting 28-day results

In addition to these recommendations, other measures could be implemented. The most practical would involve internal curing and the use of shrinkage-reducing admixtures. It is recommended to test these measures as part of the BRP Year-2 program, in the laboratory and on pilot projects. Pilot projects could also involve the use of new forming techniques to reduce restraint. It is considered that the current curing procedure that requires 14 days of wet curing is acceptable. However, it would be interesting to collect more data to verify if the relative humidity remains at 100 % for the full interval. Preliminary measurements performed during the current study period indicate it might not be the case. The use of shrinkage-compensating concrete and fibers are not considered as viable short-term alternatives.

Finally, since the bridge deck specifications are mostly driven by a durability criteria, and not strength, it is recommended to perform durability evaluations of the proposed mixes as part of the validation process. This would involve testing and calculations similar to what was performed during the course of the current study. Such calculations are the only way to evaluate the actual durability of a bridge deck taking into account the concrete durability properties, the cover and the exposure conditions. With such calculations, it would be possible to optimize mix proportions to reduce the risk of overdesigning the concrete properties, thus increasing the risk of cracking.

For example, the NAVFAC implemented in their specifications a mixture validation procedure for the service life of concrete structures exposed to harsh environments. Briefly, the program requires concrete producers to validate their mixes to be used on NAVFAC projects. Using past history or new concrete batches, the mixture are evaluated after 28 and 90 days of curing. The following tests are preformed:

- Measurement of the volume of permeable voids following ASTM C 642
- Determination of the ionic diffusion coefficient following the modified AASHTO T277 (ASTM C 1202) procedure
- Determination of the moisture coefficient following the modified ASTM C 1585 procedure

Using the results from 5 different concrete batches prepared with the same materials and proportions, it is possible to evaluate the variability of the concrete production. Taking this variability into account, STADIUM® calculations are performed to evaluate if the concrete meets the durability requirements. If it is the case, the mix is approved and QA/QC target values are set. With those values, it is also possible to monitor the production during a specific project and assess the durability of the structure for each sampled lots. If one lot is found to be non-compliant, preventative measures can be implemented at the time of construction to correct the situation and prevent early degradation. More details can be found in the UFGS section 03 31 29 available online.

From a nondestructive evaluation of bridge decks, there are a variety of data collection programs that can assist the department in quantifying bridge deck deterioration at an early age, and track progression to determine the ideal period of intervention. From this perspective, NDE allows the department to make informed decisions about deterioration through a quantifiable perspective.

The following activities comprise a comprehensive NDE program:

1) Build a database of HPC bridge deck performance. Develop a logical sequence of NDE survey to predict deterioration. For example, in most cases deterioration results from rebar corrosion, which is greatly dependent on the corrosive environment developed in the deck. Therefore, periodically conducting resistivity measurements in a nonlinear interval (CAIT can develop and recommend an interval) to assess corrosion potential. Results of resistivity data in quantifying corrosion potential can be used to determine future evaluation, including the assessment of active corrosion and delamination.
2) Use Smith Street and Route 70 Bridges for a pilot study. In comparing with other bridges, the two bridges exhibit a propensity for accelerated corrosion. Thus, periodic testing can be very beneficial for predicting the remaining service life of bridges of similar corrosion potential.
3) Collect baseline measurements for all new bridges, including concrete modulus mapping, electrical resistivity mapping, and concrete cover mapping.

6. Conclusions and future work

The objective of the current study was to investigate the durability and performance of bridge decks made of high performance concrete (HPC). HPC definitions vary from one instance to the other but it can be broadly defined as concrete that achieves certain performance requirements for a given application that otherwise cannot be obtained from normal concrete. Ideally, the HPC mix must also be easily placed and consolidated.

HPC requirements may involve enhancements of the following characteristics:

- Placement and compaction without segregation;
- Enhanced long-term mechanical properties, including the toughness;
- High early-age strength;
- Reduced cracking tendency;
- High resistance to contaminants and chemical attack;
- High abrasion resistance;

Clearly, for a bridge deck, the desired property is enhanced service life of the concrete in order to reduce the cost of maintenance and push as far back as possible the rehabilitation of the deck. Less maintenance reduces cost but also improves the network accessibility.

It is important for the owner to define objectives for the use of HPC because mix proportioning has to be based on the specific objectives to achieve. Ideally, HPC specifications should be adapted to the different intended uses because not all HPC concretes are good for all applications.

The work accomplished during the current study program has yielded interesting conclusions and recommendations that were used to establish 2014 objectives. The original 2014 program consisted of the following activities related to the study of concrete (bullet designations refer to subtask within the proposed 2014 work plan):

2a. Use Robot Assisted Bridge Inspection Tool (RABIT), as well as other appropriate technologies, to provide a comprehensive multimodal NDE scanning of up to ten (10) bridge decks, or up to an equivalent 60,000sf of deck, to provide guidance on deck performance based on specific coatings, sealants and or overlays.

2b. Develop a testing program and perform concrete testing to determine the cause of early age cracking of reinforced concrete and its consequence on the long-term performance of bridge decks.

2c. Develop synthesis reports on the performance of reinforcement steel in concrete decks throughout the nation and perform a comparative analysis of reinforcement steel in concrete decks in New Jersey; and on the use of fiber reinforced polymer composites for the repair of concrete structures.

4a. Develop a synthesis report on precast concrete deck systems as a means of deck replacement, including guidance on shear stud and other construction details. Consideration will be given to materials used, such as high early strength concrete.

Through these studies, the team anticipates furthering the knowledge of bridge deck performance beyond HPC. Through these studies, there will also be a focus on responding to findings from the current program. Issues such as elevated microstrains induced during curing are expected to be addressed via task 2b. However, additional findings of the current program can be evaluated for immediate or short-term implementation, and some have been dismissed. The following describes how each action was catalogued:

Immediate actions

- Select pilot bridge decks to validate internal curing of HPC. Standard specification developed in a separate activity under the bridge resource program.

Short term actions to be investigated

- Limit free shrinkage via revisions to the current standard specifications for road and bridge construction.
- Provide further guidance on possible revisions of the current standard specifications for road and bridge construction.
- Consider lab testing the effectiveness of shrinkage reducing admixtures.
- Provide guidance on optimization of concrete mixture specifications:
    - Maximum binder content, and other binder related options.
    - Consider increasing coarse aggregate content.
    - Consider specifying a coarse/sand aggregates optimum ratio.

- Consider the modification of the curing procedure (to be validated in laboratory testing):
  - Wet curing.
  - Curing blankets.
  - Curing compounds.
  - Combination of different techniques mentioned above.
  - Consider mitigating high evaporation rates.
  - Consider longer wet curing such as 21-day curing with a 7-day weaning period.
- Consider the potential effects of reducing or eliminating the use of #8 stone in concrete mixes.
- Consider allowing the use of higher water-binder ratios if durability requirements are met with the use of pozzolans.
- Limit the degree of restraint provided by stay-in-place forms.

Recommendations that were dismissed

The department expressed concerns over suppliers' willingness to embrace an upper strength boundary. Currently, suppliers focus on providing contractors with concrete that rapidly set, achieving target criteria well within the required period. Thus, HPC hydration rates result in increasingly high strength. Suppliers would resist balancing this high-reaction. The department has suggested further review of target criteria that could facilitate high hydration rates such that concrete can cure properly.

The department has also expressed concerns over limiting the use of stay-in-place (SIP) forms. This is the primary method contractors in New Jersey use as formwork for concrete decks. The ability to rapidly construct bridge decks and eliminate the need to remove forms post-construction results in cost-savings to the department and contractor. Since the effects of restraint by the SIP on concrete bridge decks is limited, the department has suggested that this recommendation be withheld until further understanding of SIPs effects on concrete restraint is better understood, or until a better understanding of other techniques to limit shrinkage can be fully quantified.

Potential revisions to 2014 program

In addition to the proposed scope of work related to concrete performance, the team has identified various activities that can further inform the department on the performance of HPC in New Jersey. The proposed 2014 program could include the following aspects:

<u>Part 1 - Testing on concrete mixes</u>

Mixes can be produced in the laboratory using materials from a local NJ concrete producer or directly at the batch plant with the same concrete producer. The testing program should include the following:

- Based on concrete properties, the HPC tested during the current study program have shown to have high volume of permeable voids, which usually indicates a higher water content in the mix and a propensity for drying shrinkage, and increased risk of cracking. Optimizing the packing density of a HPC mixture could be an interesting avenue for future investigation in order to reduce the volume of permeable voids

- Internal curing is a promising solution to reduce shrinkage in HPC, which is more prone to self-desiccation. Internal curing would provide a supply of internal water to compensate self-desiccation and reduce the shrinkage potential. Internal curing should be used in trial mixes to determine a preferred lightweight fine aggregate proportion to use in HPC.
- Recent experience by CAIT partners indicate shrinkage-reducing admixtures are effective in reducing shrinkage in high-performance concrete. When properly dosed, SRA can reduce drying shrinkage by the order of 30 % in HPC. SRA acts on the capillary water tension to reduce the forces caused by the menisci on the concrete pore surfaces upon drying, which is the root cause of shrinkage.

These solutions can be used individually or in combination to create concrete mixes that exhibit a much lower shrinkage tendency.

<u>Part 2 – Testing on pilot projects</u>

Pilot projects could be selected to implement new generation HPC mixes and also to evaluate alternate construction practices. This part of the program would involve:

- Casting decks with low-cracking HPC developed in the lab during Part 1 and/or in collaboration with local producers and monitor crack development and internal strains.
- Measuring surface and internal temperature and moisture development during the curing period with probes installed on and inside the concrete deck.
- Identifying alternative construction practices to reduce the restraint currently provided by the ribbed stay-in-place forms that are commonly used on most bridge decks.

Appendix A – Description of NDE Technologies

Assessment of Rebar Corrosion

Deterioration in concrete bridge decks is in the greatest part a result of steel (rebar) corrosion (Figure A1). During the process, concrete allows electrolytic conduction and hence, the flow of ions from anodes to cathodes on rebars. Once the oxide film is destroyed, an electric cell is formed along the rebar or between rebars and the electrochemical process or corrosion begins. In addition, chloride ions typically penetrate from the surface into a bridge deck resulting in a higher salt concentration, more corrosive environment, and more negative electrical potential at the top reinforcing steel layer than at the bottom layer. The two elements, corrosion activity and corrosive environment, can be evaluated by HCP and ER methods. Rebar corrosion leads to concrete deterioration, delamination, contamination and loss of rebar section. If the corrosion involves large areas, it will cause large cracking and delamination, and ultimately concrete spalling.

Figure A1. Rebar corrosion.

Half-cell potential (HCP) and electrical resistivity (ER) are the most commonly used NDE methods to define the corrosive environment of concrete decks. HCP involves the measurement of the electrical potential between the reinforcement and a reference electrode (usually copper electrode in a copper sulphate solution) coupled to the concrete surface. By moving the electrode from one point to another, or by using a wheel electrode (Figure A2), a potential map can be created. The principle of HCP is also illustrated in Figure A3, where the wheel electrode i connected to a rebar through a high impedance voltmeter. The measured potential values are influenced by both the corrosion activity and by the concrete cover and concrete resistance (Elsner, 2003). In general, regions with a more negative potential indicate a higher probability of corrosion. The ASTM C876 gives general guidelines for evaluating corrosion probability in concrete structures. HCP measurements cannot give quantitative information about the actual corrosion rate of the reinforcement.

The corrosive environment of concrete and thus potential for corrosion of reinforcing steel can be well evaluated through measurement of electrical resistivity of concrete. The higher the electrical resistivity of the concrete, the lower will be the corrosion current passing between the anodic and cathodic areas of rebars. Dry concrete will pose a high resistance to the passage of current, and thus will be unable to support ionic flow. On the other hand, presence of water and chlorides in concrete, and increased porosity due to damage and cracks, will increase ion flow, and thus reduce resistivity. It has been observed that a resistivity less than 5 kΩcm can support very rapid corrosion of steel (Brown, 1980). In contrast, dry concrete may have resistivity can exceed 100 kΩcm. Whiting and Nagi (2003) have related the electrical resistivity of concrete to the corrosion rates for reinforcing steel. Measurement of electrical resistivity using the Wenner probe is shown in Figure A3 and schematically presented in Figure A4. The current is induced through the two outer electrodes and the potential of generated electrical field measured between the two inner electrodes, enabling the calculation of resistivity.

Figure A2. HCP survey using a wheel HCP probe (left) and ER survey using Wenner probe (right).

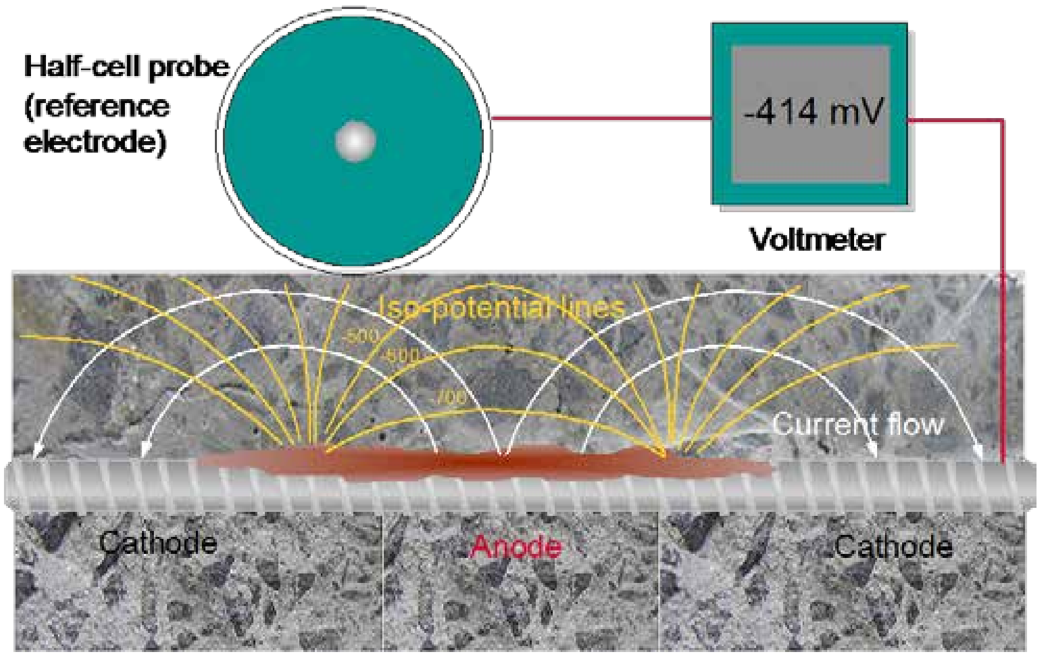

Figure A3. Schematic of corrosion activity evaluation using HCP.

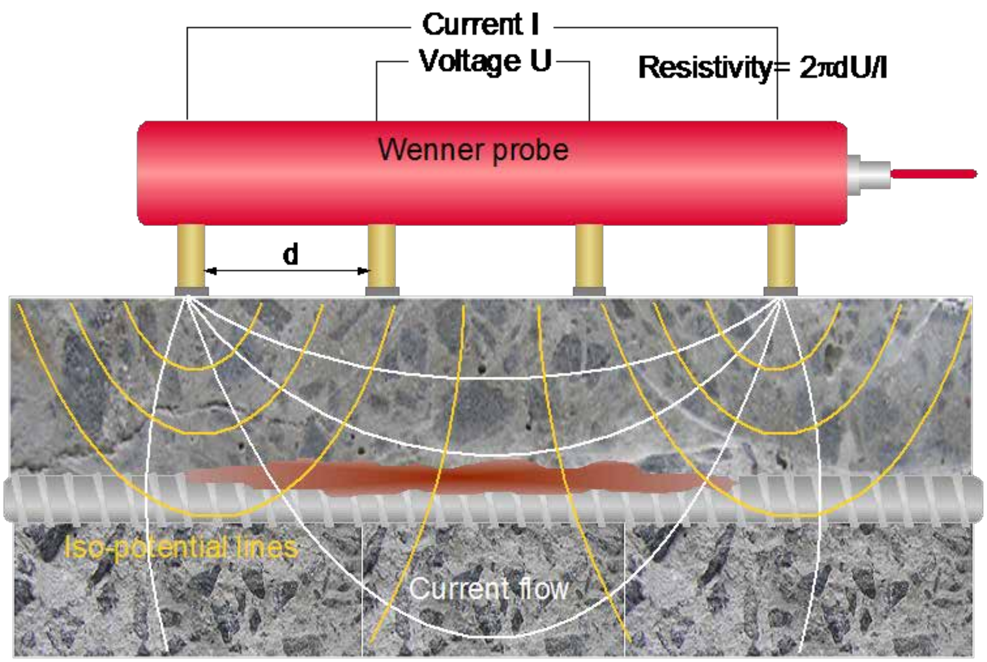

Figure 4A. Schematic of assessment of corrosive environment using a Wenner resistivity probe.

Concrete Delamination

Concrete delamination is most often a result of rebar corrosion. During the corrosion, the buildup of corrosive products on a rebar causes a significant increase in the rebar volume over the original one. The pressure of the increased volume induces cracking and further delamination of concrete. However, delamination can be also a result of other types of concrete deterioration or repeated overloading, or a combination of those. An example of what can be described as initial delamination, a thin crack propagating from one to another rebar, is shown on the left side of Figure A5. A progressed delamination is shown on the right side of the same.

Figure A5. Initial (left) and progressed delamination (right).

Impact echo (IE) has been successfully implemented in detection and characterization of delamination in bridge decks (Sansalone, 1993 and 1997, Gucunski, 2000 and 2008, Algernon and Wiggenhauser, 2006). The primary objective of IE testing is to determine dominant reflectors in the deck, which are in most cases the bottom of the deck or a delamination. IE is a frequency response method. Thus, the position of reflectors is obtained from resonant frequencies of „standing waves" between the surface and the reflector. IE testing using Stepper, developed at BAM (German Federal Institute for Material Research and Testing), is shown in Figure 4. The impact ball and transducer are shown on the right side of the figure. The Stepper allows automated data collection at a prescribed spacing between data points.

Four grades are typically assigned in the condition assessment with respect to delamination, as illustrated in Figure A7. In the case of strong reflections from the bottom of the deck (truly oscillations of standing Love waves), the deck is described as good. In the case of a delaminated deck, reflections of the compression wave occur at shallower depths, causing a shift in the response spectrum towards higher frequencies. Depending on the extent and continuity of the delamination, the partitioning of energy of waves being reflected from the bottom of the deck and delamination may vary. Initial delamination (fair condition) is described as occasional separations between the two deck zones. Thus it will have two distinct peaks corresponding to reflections from the bottom of the deck and the delamination. Progressed delamination (poor condition) is characterized by a single peak at a frequency corresponding to the depth of the delamination. Finally, in cases of wide or shallow delamination, the dominant response of the deck to an impact is characterized by a low frequency response of flexural mode oscillations of the upper delaminated portion of the deck. This condition is graded as a serious condition and is always in the audible frequency range.

Figure A6. Delamination assessment using Stepper.

Concrete Quality Assessment

While chlorides are usually considered to be the biggest concern for salt induced corrosion, a number of deterioration processes in decks are associated with other chemicals and actions. Those, for example, include penetration of sulfates, which can attack concrete chemically, altering the microstructure of concrete and pore size distribution of the matrix. The by-products of these reactions are volumetrically larger than the original materials, thereby causing expansive stresses (cracks) within the concrete. Similar deterioration can be caused by repeated freeze and thaw, alkali-silica-reaction (ASR), mechanical stressing, overloading, etc. In all the cases deterioration leads to either reduced mechanical properties or altered electrical/dielectric properties, or both. Ultrasonic surface waves (USW) method will be effective in detecting and measuring changes in mechanical properties, while ground penetrating radar (GPR) will be effective in detecting changes in dielectric properties.

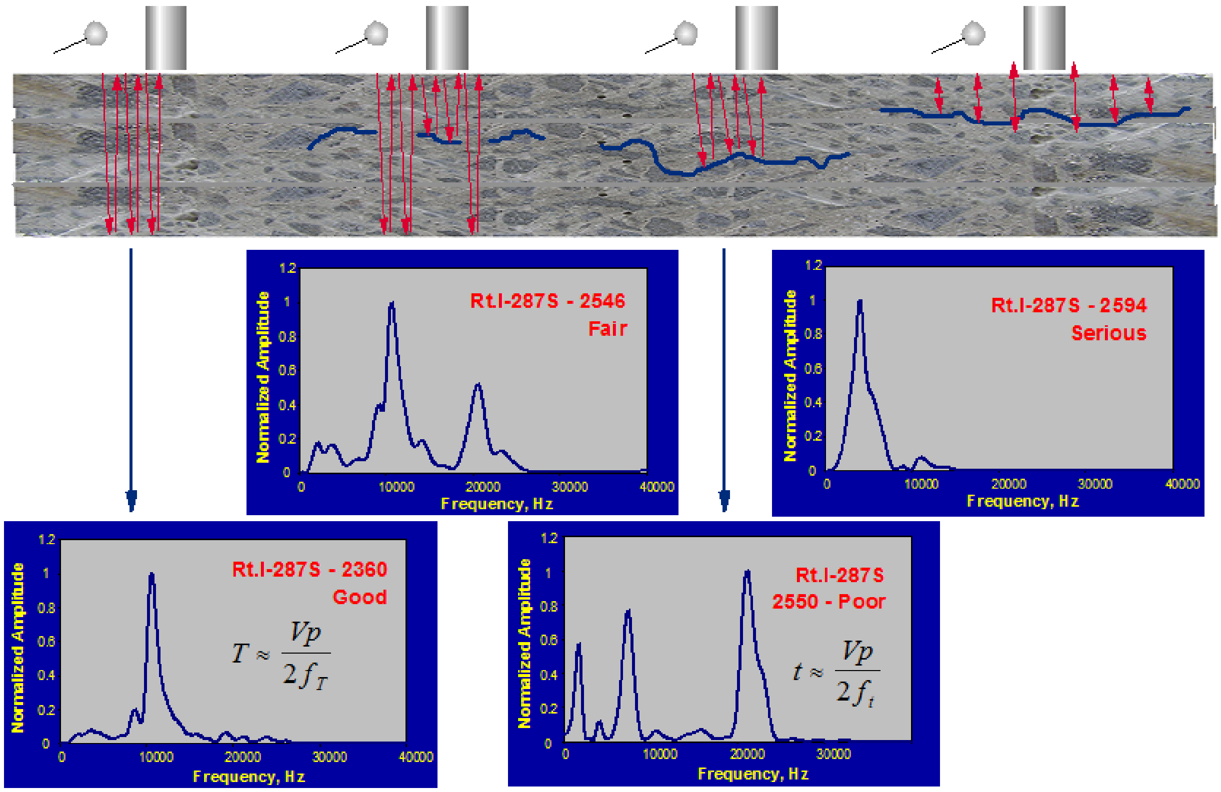

Figure A7. Principle of IE test and delamination grade assignment.

The objective of the USW test is to measure the velocity of surface waves that can be linked to the concrete elastic modulus. In cases of mostly uniform materials, like concrete in bridge decks, the velocity is fairly constant for a limited range of wavelengths (Nazarian et al., 1993). Therefore, the modulus is obtained from the average surface wave velocity for wavelengths not exceeding the thickness of the deck. Variation in the phase velocity would be an indication of the variation of material properties with depth. Devices like the portable seismic property analyzer (PSPA), shown on the right side of Figure A8, can be used in the evaluation of concrete modulus by the USW method. The principle of USW testing is illustrated in Figure A9. An impact is applied on the surface of a deck and the resulting propagating elastic waves are detected by a pair of nearby receivers. The information from the two receivers is used to obtain the dispersion curve (velocity of propagation versus frequency relationship) and from there a modulus profile or an average modulus. Variation in concrete modulus in the deck does not necessarily mean deterioration. Such variations can often be introduced at the time of construction, due to material variation and placement procedures. Therefore, only a periodical measurement of changes in the concrete modulus would lead to identification of deterioration processes.

Figure A8. GPR survey using a ground coupled antenna (left) and USW testing using PSPA (right).

GPR provides a qualitative assessment of concrete deck deterioration through measurement of attenuation of electro-magnetic waves on the rebar level. Correlations with impact echo data and ground truth measurements have also shown that GPR has potential for delamination detection in areas of highly attenuated signal. In addition, GPR surveys can provide information about deck thickness, concrete cover and rebar configuration (Romero et al., 2000; Barnes and Trottier, 2000). Concrete that is moist and high in free chloride ions, such as a deck that has undergone deterioration due to corrosion of the rebar, can significantly attenuate a GPR signal. A GPR survey of a bridge deck using a ground coupled antenna is shown on the left side of Figure A8. When the antenna is above or in proximity of a rebar, electro-magnetic waves are reflected from them. The amplitude of the reflection will be highest when the deck is in a good condition and weak when delamination and corrosion are present. Since the rebar depth can significantly influence signal attenuation, measured reflection amplitudes are corrected for variations due to the rebar depth. Once the attenuation map is completed, a unique deterioration threshold is established using ground truth, such as cores or NDE methods like impact echo (Barnes and Trottier, 2000; Gucunski et al., 2005) to provide the results interpretation.

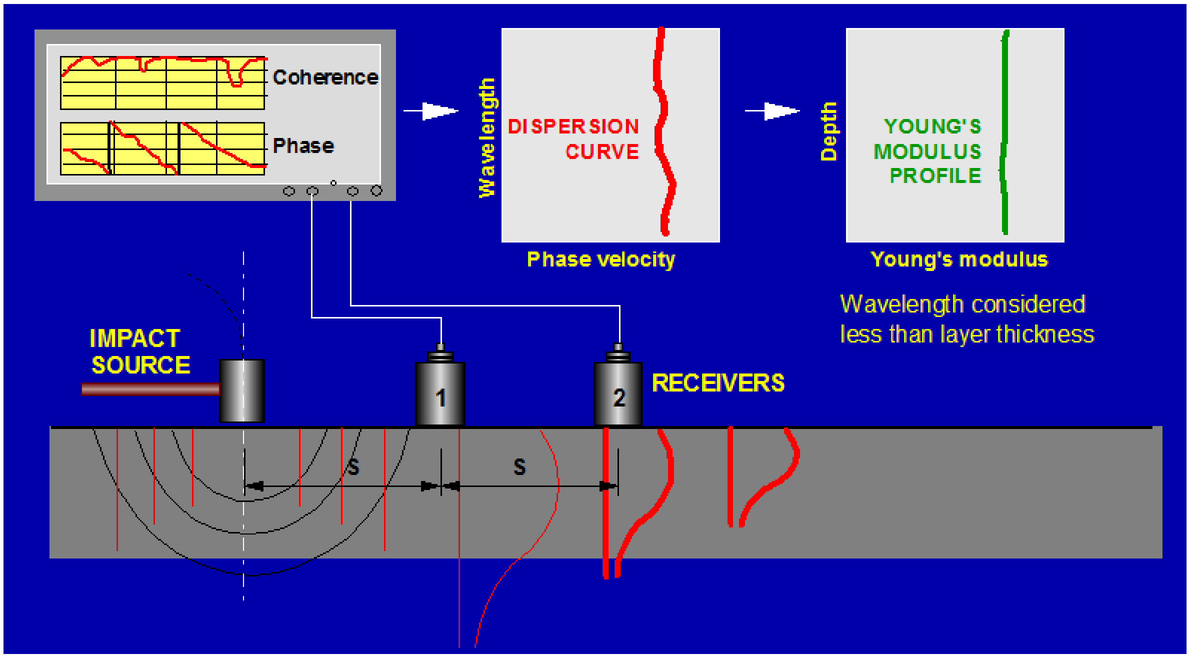

Figure A9. Concrete modulus measurement using USW method.

NDE Result Presentation

Condition assessment of bridge decks using such a multiple NDE technology approach is illustrated by the results of evaluation of a bridge deck in Virginia in Figure A10. The bridge has an eight-inch thick deck on steel girders. The condition assessment from four technologies, namely HCP, ER, IE and GPR, is shown in Figure A10. For all NDE technology results, the hot colors (reds and yellows) represent high level of deterioration and the cool colors (blues and greens) low level of deterioration or a good condition. Qualitatively, HCP and ER point to about the same areas as having active corrosion and corrosive environment. This points to a somewhat expected relationship between the corrosive environment and active corrosion. Qualitative similarities to HCP and ER results can be also observed in the GPR results, and to a lesser extent IE results. This should be explained that the likely primary cause of deterioration is corrosion. Still, there are also differences. For example, some IE identified delamination is not identified by the GPR as zones of high attenuation. All of those are an illustration of how results from different NDE technologies complement each other in building a complete picture of bridge deck deterioration.

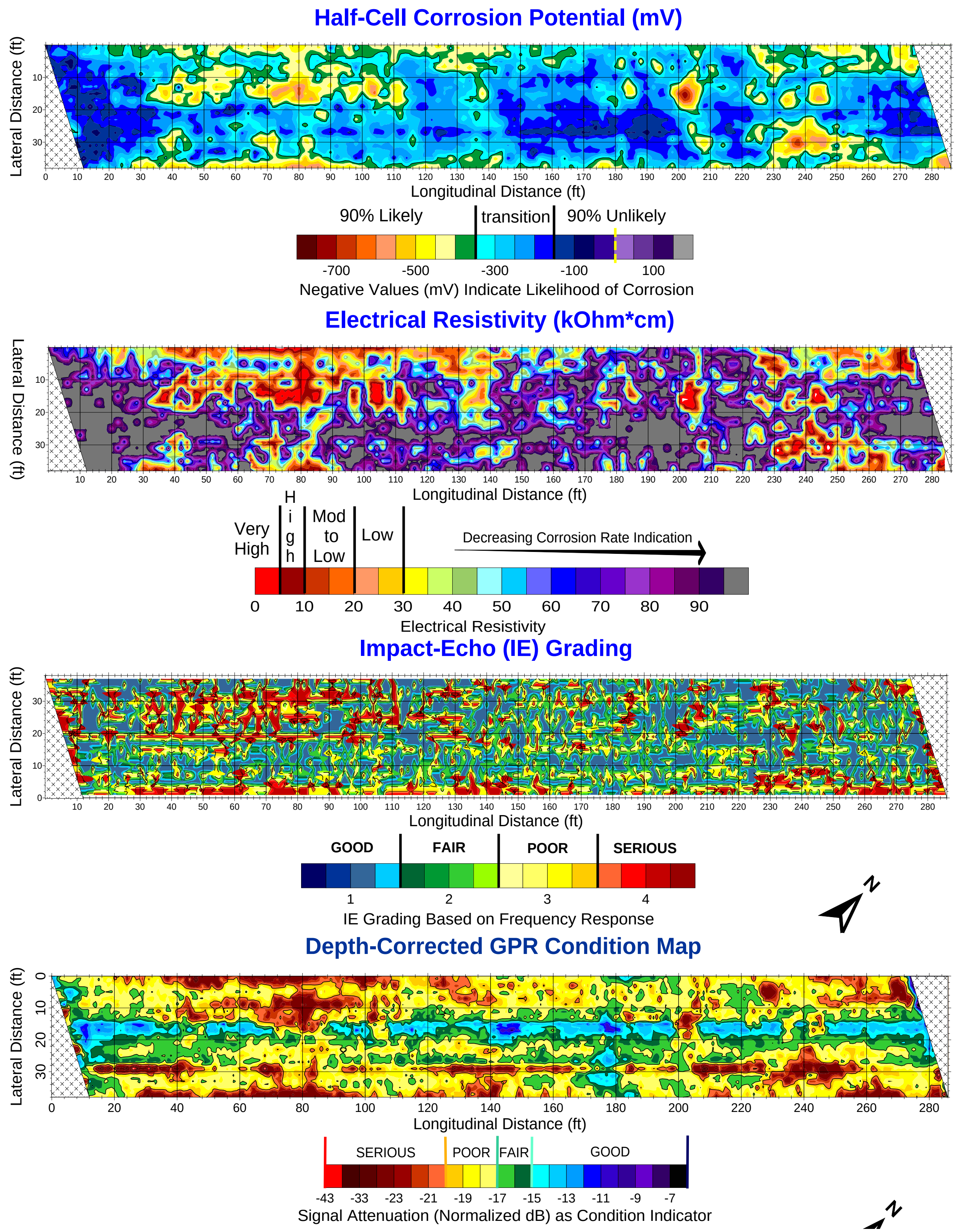

Figure A9. Condition assessment of the deck of using HCP, ER, IE and GPR.

Appendix B – Results of NDE testing on bridge decks

Span	Struct. ID	Description	Initial Selection Criteria	Deck Cond. SI&A
1	0311150	Route 70 over Bisphams creek	2-5 yr old HPC (durability and NDE)	8 - Very Good
2	1234-509	Smith Street (CR 656) over State Route 440	2-5 yr old HPC (durability and NDE)	(Reconstructed in 2010)
3	0511156	RT 52 over Rainbow Channel	2-5 yr old HPC (durability and NDE) – salt environment and early-age cracking	7 - Good
4	0511157	RT 52 over Elbow Channel	2-5 yr old HPC (durability and NDE) – salt environment and early-age cracking	7 - Good
5	1209155	Route 9 Edison (Northbound)	5-10 yr old HPC (durability and NDE)	7 - Good
6	1209156	Route 9 Edison (Southbound)	5-10 yr old HPC (durability and NDE Tested)	7 - Good
7	3100-001	Ocean City – Longport Bridge	5-10 yr old HPC (durability and NDE) – salt environment/early-age cracking	6 - Satisfactory
8	0327-166	Creek Road Over I-295	Class A concrete deck with condition rating at or under 5.	5 - Fair

Condition Assessment of the Route 70 over Bisphams Creek Bridge Deck (Pemberton Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Corrosion activity assessment using half-cell potential (HCP) survey.

The data collection was conducted on July 19, 2013 on about 22 feet long and 26 feet wide, covering the entire bridge deck. The survey was conducted on a two by two feet grid. The first line of the testing grid was matching one foot away from the right curb. Three NDE technologies were deployed: impact echo, electrical resistivity and half-cell potential. The impact echo testing was conducted using an impact echo "cane". The electrical resistivity measurements were conducted using a four-electrode Wenner probe. Finally, the half-cell potential measurement was conducted using a rolling HCP probe, as illustrated in Figure 1.

Figure 1. Half-Cell Potential measurement on the Route 70 over Biphams Creek Bridge.

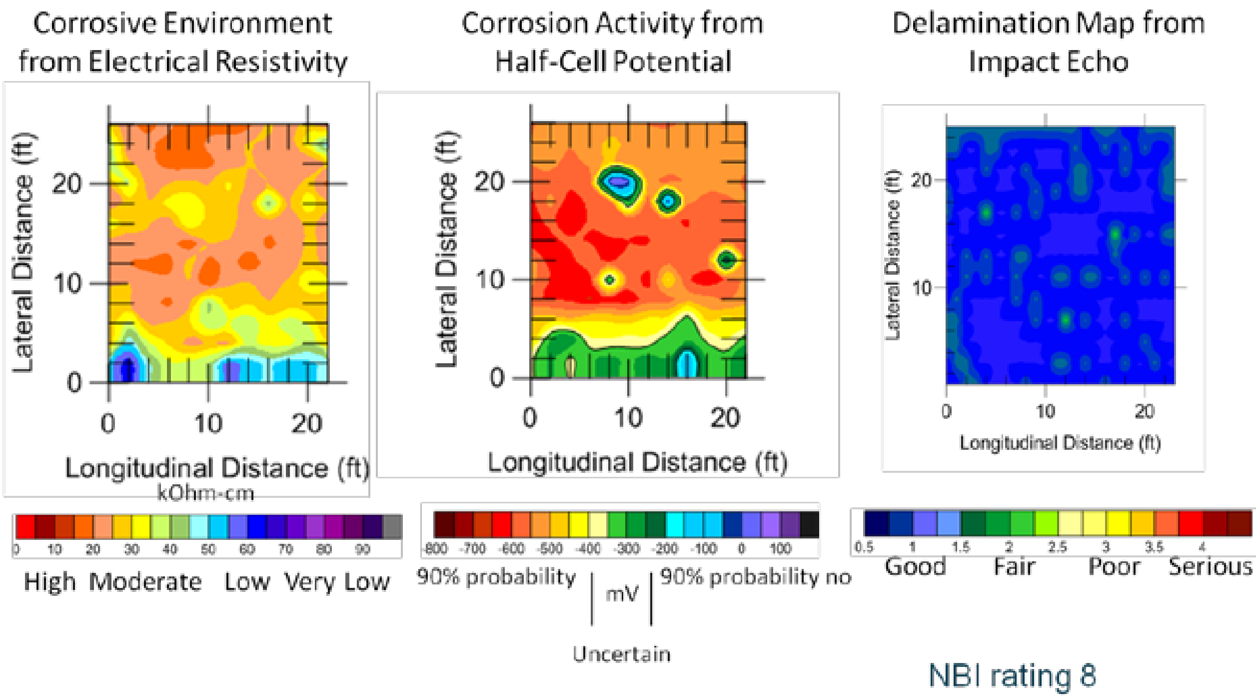

Figure 2. Condition maps for Route 70 over Bisphams Creek bridge deck: corrosive environment (left), corrosion activity (middle) and delamination (right).

The results of the three surveys are represented by the condition maps in Figure 2. The electrical resistivity in the top map is linked to the corrosive environment of concrete and to them related anticipated corrosion rates. Almost entire of the deck had electrical resistivity below 40 kOhm-cm, and in a significant part the resistivity was below 25 kOhm-cm. These are indication of already moderate to highly corrosive environment that is likely producing moderate to high rebar corrosion rates. These findings are almost perfectly matched by the results from the half-cell potential survey. Based on the potential measured, almost ninety percent of the deck is, according to the ASTM C876-09, has 90 percent probability of being in the state active corrosion. On the other hand, there are no signs of delamination, only minimal signs of very early stage delamination. However, it is expected that the high corrosion rates and activity will soon lead to more progressed delamination. The delamination map grades in the map are described in the main body of the report.

Condition ratings with respect to delamination and corrosion for the section of the bridge surveyed were calculated to be 95.5 and 58.9, respectively. The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k -cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

Condition Assessment of Smith Street over SR 440 Bridge Deck (Woodbridge Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on July 15, 2013 on about 100 feet long and 38 feet wide section of the bridge deck. As shown in Figure 1, the surveyed area had a bit odd shape due to maintenance activities on a section of the deck. The survey was conducted on a two by two feet grid. The first line of the testing grid was matching one foot away from the right curb. Three NDE technologies were deployed: impact echo, electrical resistivity. The impact echo testing was conducted using an impact echo "cane". The electrical resistivity measurements were conducted using a four-electrode Wenner probe. A few concrete modulus measurements were taken using a portable seismic property analyzer (PSPA).

Figure 1. View on the area of Smith Street Bridge surveyed.

The results of the of the electrical resistivity and impact echo surveys are represented by the condition maps in Figure 2. The electrical resistivity in the top map is linked to the corrosive environment of concrete and to them related anticipated corrosion rates. Significant areas of the deck had electrical resistivity below 40 kohm-cm, and in some the resistivity dropped to only 10 kOhm-cm. These are indication of already moderate to highly corrosive environment that is likely producing moderate to high rebar corrosion rates. On the other hand, there are no signs of delamination, only minimal signs of very early stage delamination. However, it is expected that the high corrosion rates will soon lead to more progressed delamination. The delamination map grades in the map are described in the main body of the report.

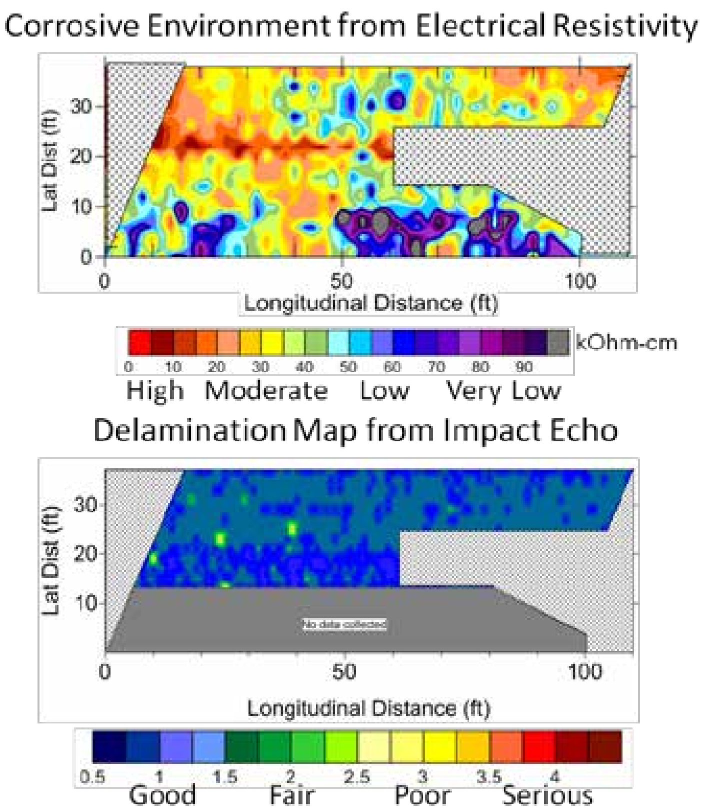

Figure 2. Electrical resistivity (top) and delamination from impact echo (bottom) maps for Smith Street over SR 440 bridge deck.

Condition ratings with respect to delamination and corrosion for the section of the bridge surveyed were calculated to be 89.7 and 74.1, respectively The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k -cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

Condition Assessment of the Route 52 Rainbow and Elbow creek Bridge Decks, (Ocean City Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on two sections of the bridge: Elbow and Rainbow Creek, was conducted on April 22 and 23, 2013, respectively. At both locations a 300 feet long and 15 feet wide sections of the bridge deck were surveyed (Figure 1). The survey was conducted on a two by two feet grid. The first line of the testing grid was one foot away from the parapet in the shoulder area. Three NDE technologies were deployed: impact echo, electrical resistivity and half-cell potential. The impact echo testing was conducted using a Stepper, as shown in Figure 2 The electrical resistivity measurements were conducted using a four-electrode Wenner probe. Finally, the half-cell potential (HCP) measurement was attempted, but it could not be conducted because of the lack of electrical continuity of the rebar mesh need to perform the measurement. Identification of a rebar to make a HCP probe connection is shown in Figure 2.

Figure 1. NDE surveys on Route 52 Elbow Creek bridge deck.

Figure 2. Pachometer measurement to identify a rebar location for half-cell potential probe connection (left) and IE testing using Stepper (right) on Route 52 bridge.

The results of the of the NDE surveys using the impact echo and electrical resistivity for the two sections of the Route 52 bridge are represented by the condition maps in Figures 3 (Rainbow Creek) and 4 (Elbow Creek). The electrical resistivity in the top map is linked to the anticipated corrosion rates. Similarly, the delamination map grades are linked to the different stages of delamination progression, as described in the main body of the report.

Condition ratings with respect to delamination and corrosion for the Rainbow Creek bridge section surveyed were calculated to be 74.8 and 99.9, respectively. Condition ratings with respect to delamination and corrosion for the Elbow Creek bridge deck were calculated as 71.5 and 100. The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

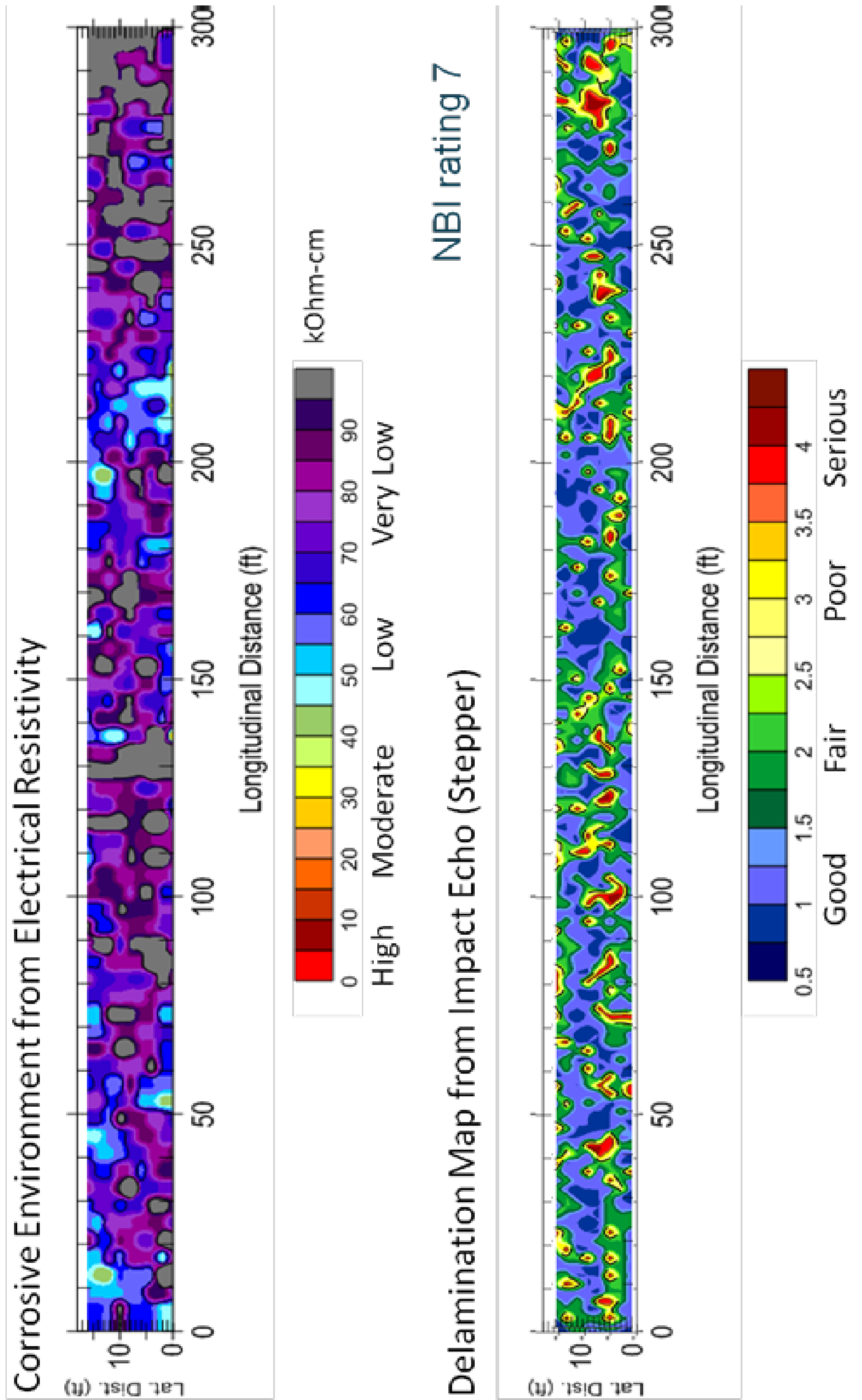

Figure 3. Electrical resistivity and delamination maps for Route 52 causeway (Rainbow Creek).

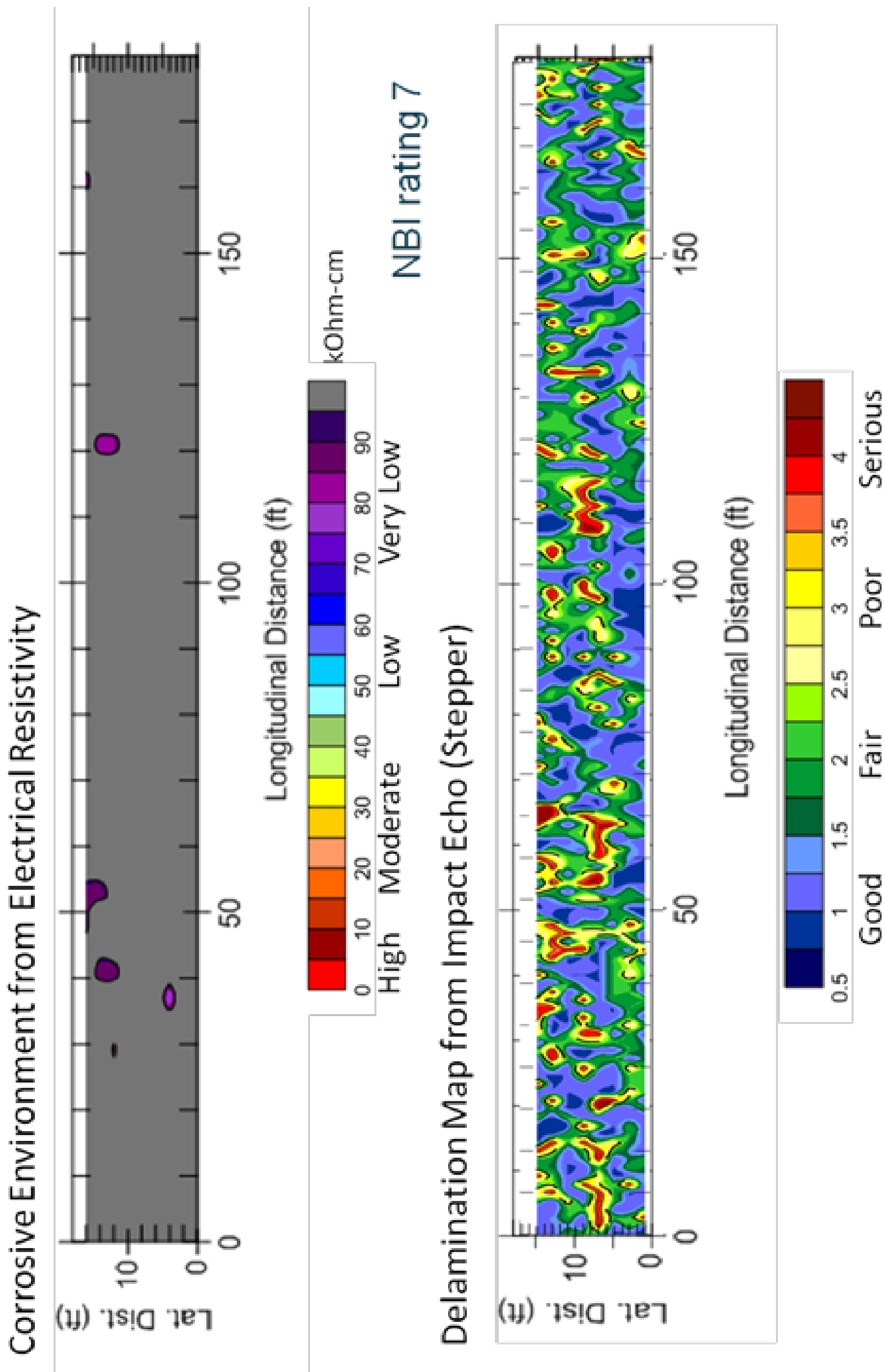

Figure 4. Electrical resistivity and delamination maps for Route 52 causeway (Elbow Creek).

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. For the two Route 52 bridge sections, there is a possibility that some of the identified points of delamination actually represent resonances coming from either relatively thin (3.5 inch) precast deck panels or reflections from the bottoms of wide and deep top flanges of girders, as illustrated in Figure 5. This could be confirmed by conducting an IE measurement using a source with a higher center frequency than the one used.

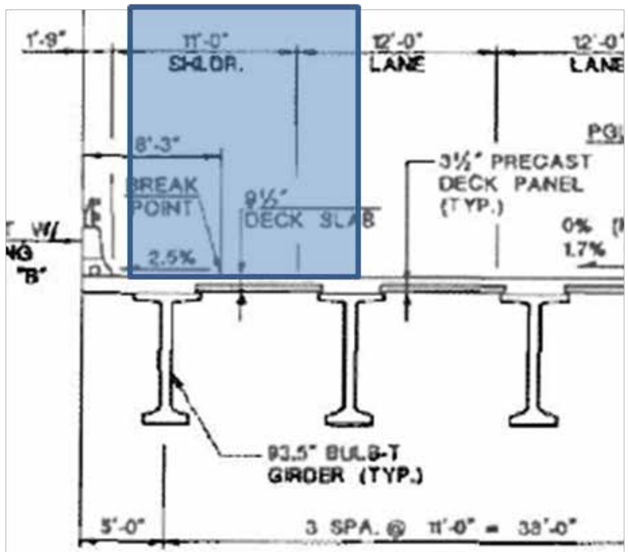

Figure 5. Typical cross section of Route 52 Bridge with the survey area marked in blue.

Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 kΩcm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

Perfect corrosion rating scores of 100 for both Rainbow and Elbow Creek bridges indicate that concrete is at the moment having very low contamination with moisture and chlorides.

Condition Assessment of the Route 9 Northbound Bridge Deck (Woodbridge Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on June 20, 2013 on a 300 feet long and 18 feet wide section of the bridge deck. The survey was conducted on a two by two feet grid. The first line of the testing grid was one foot away from the parapet in the shoulder area. Three NDE technologies were deployed. The impact echo testing was conducted using an IE "cane." The electrical resistivity measurements were conducted using a four-electrode Wenner probe, as shown in Figure 1. Finally, the concrete modulus measurements were conducted using a portable seismic property analyzer (PSPA).

Figure 1. Electrical resistivity measurement on Route 9 Northbound Bridge deck.

The results of the NDE surveys using the three technologies are represented by the condition maps in Figure 2. The electrical resistivity in the top map is linked to the anticipated corrosion rates. Similarly, the delamination map grades are linked to the different stages of delamination progression, as described in the main body of the report.

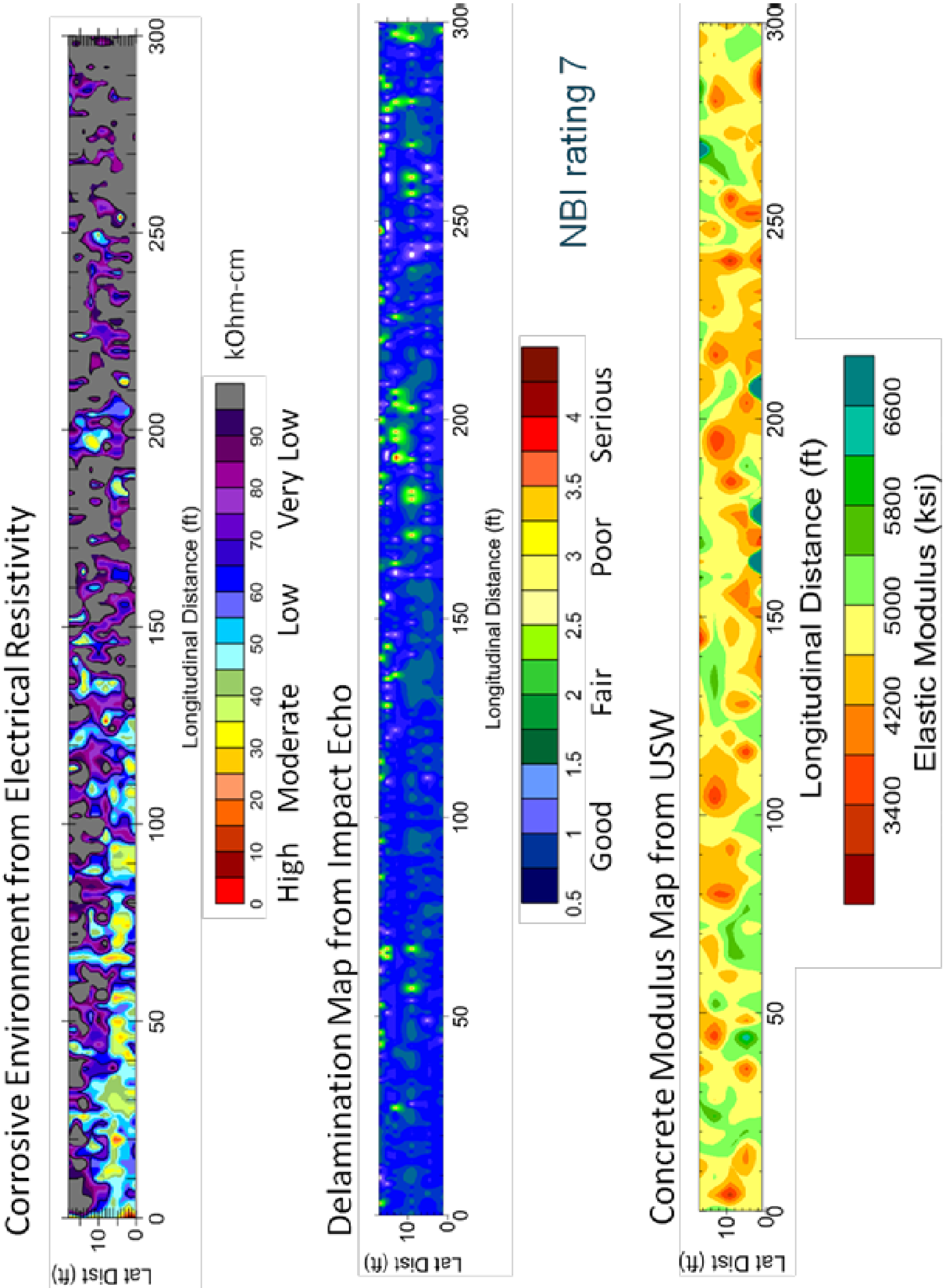

Figure 2. Condition maps for Route 9 northbound

Condition ratings with respect to delamination and corrosion for the bridge section surveyed were calculated to be 90.1 and 97.3 The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k -cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

In addition to condition ratings, average elastic modulus and standard deviation of the modulus was calculated for the bridge deck section. The average concrete modulus4718 ksi, while the standard deviation 540 ksi.

The calculated condition ratings with respect to delamination and corrosion describe a bridge deck in a good condition. Also, the calculated standard deviation of concrete modulus describes a lower variability of concrete modulus.

Condition Assessment of the Route 9 Southbound Bridge Deck (Woodbridge Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on June 21, 2013 on a 300 feet long and up to an 18 feet wide section of the bridge deck (Figure 1). The survey was conducted on a two by two feet grid. The first line of the testing grid was one foot away from the parapet in the shoulder area. Three NDE technologies were deployed. The impact echo testing was conducted using an IE "cane.", as shown in Figure 2 The electrical resistivity measurements were conducted using a four-electrode Wenner probe. Finally, the concrete modulus measurements were conducted using a portable seismic property analyzer (PSPA) shown in Figure 2.

Figure 1. NDE surveys on Route 9 southbound bridge deck.

Figure 2. IE "Cane" (left) and PSPA testing (right) on Route 9 southbound bridge.

The results of the NDE surveys using the three technologies are represented by the condition maps in Figure 3. The electrical resistivity in the top map is linked to the anticipated corrosion rates. Similarly, the delamination map grades are linked to the different stages of delamination progression, as described in the main body of the report.

Condition ratings with respect to delamination and corrosion for the bridge section surveyed were calculated to be 90.6 and 92.8 The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

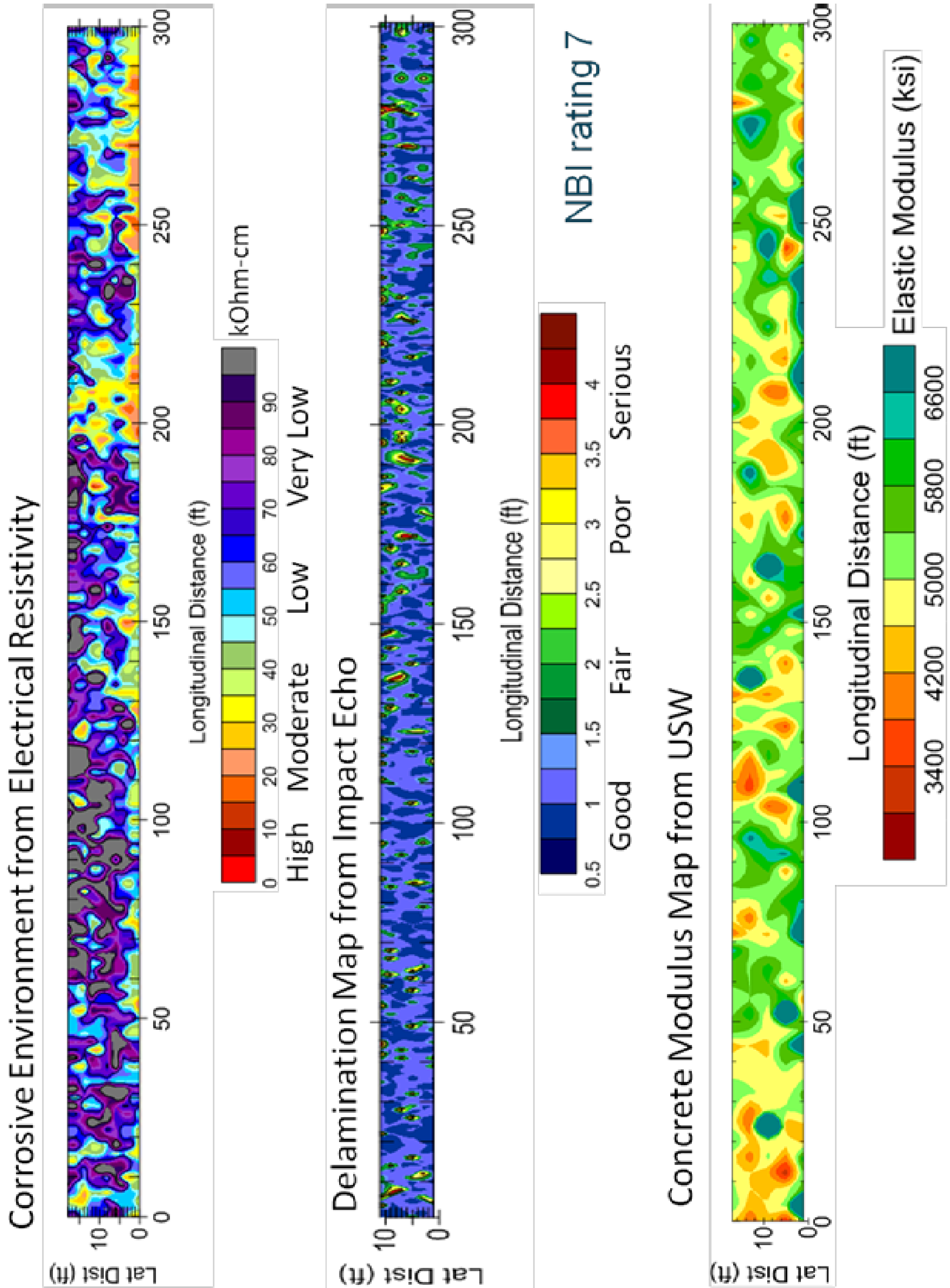

Figure 3. Condition maps for Route 9 southbound bridge deck.

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k -cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

In addition to condition ratings, average elastic modulus and standard deviation of the modulus was calculated for the bridge deck section. The average concrete modulus is 5196 ksi, while the standard deviation 787 ksi.

The calculated condition ratings with respect to delamination and corrosion describe the bridge deck to be in a good condition. Also, the calculated standard deviation of concrete modulus describes a moderate variability of concrete modulus.

Condition Assessment of the Ocean City-Longport Bridge Deck (Ocean City Township) using NDE

The condition assessment of this HPC deck concentrated on three evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement, and
3. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on June 6, 2013 on a 300 feet long and 18 feet wide section of the bridge deck (Figure 1). The survey was conducted on a two by two feet grid. The first line of the testing grid was one foot away from the parapet in the shoulder area. Four NDE technologies were deployed. The impact echo testing was conducted using the Stepper and impact echo cane (Figure 2). .The electrical resistivity measurements were conducted using a four-electrode Wenner probe. The concrete modulus measurements were conducted using a portable seismic property analyzer (PSPA). Finally, a ground penetrating radar (GPR) was deployed to provide a qualitative assessment of the condition of the bridge deck.

Figure 1. Preparation for NDE surveys on Ocean City Longport Bridge.

Figure 2. Impact echo testing using a single probe (left) and Stepper.

The results of the NDE surveys using the four technologies are represented by the condition maps in Figure 3. The electrical resistivity in the top map is linked to the corrosive environment of concrete and to them related anticipated corrosion rates. Areas of electrical resistivity below 40 kohm-cm can be observed at several locations of the surveyed section, indicating likelihood of moderate rebar corrosion rates at the same. The GPR obtained condition map points similar to the ER map to worse conditions in proximity 0 and 240 feet longitudinal distance locations. The delamination map grades are linked to the different stages of delamination progression, as described in the main body of the report. In general, the deck is sound with respect to the delamination. However, there are several points on the deck that provide signs of both incipient and progressed delamination. Finally, the USW survey map describes concrete of a generally uniform modulus, mostly in a range 5000 to 5500 ksi.

Condition ratings with respect to delamination and corrosion for the section of the bridge surveyed were calculated to be 81.6 and 96.3, respectively The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

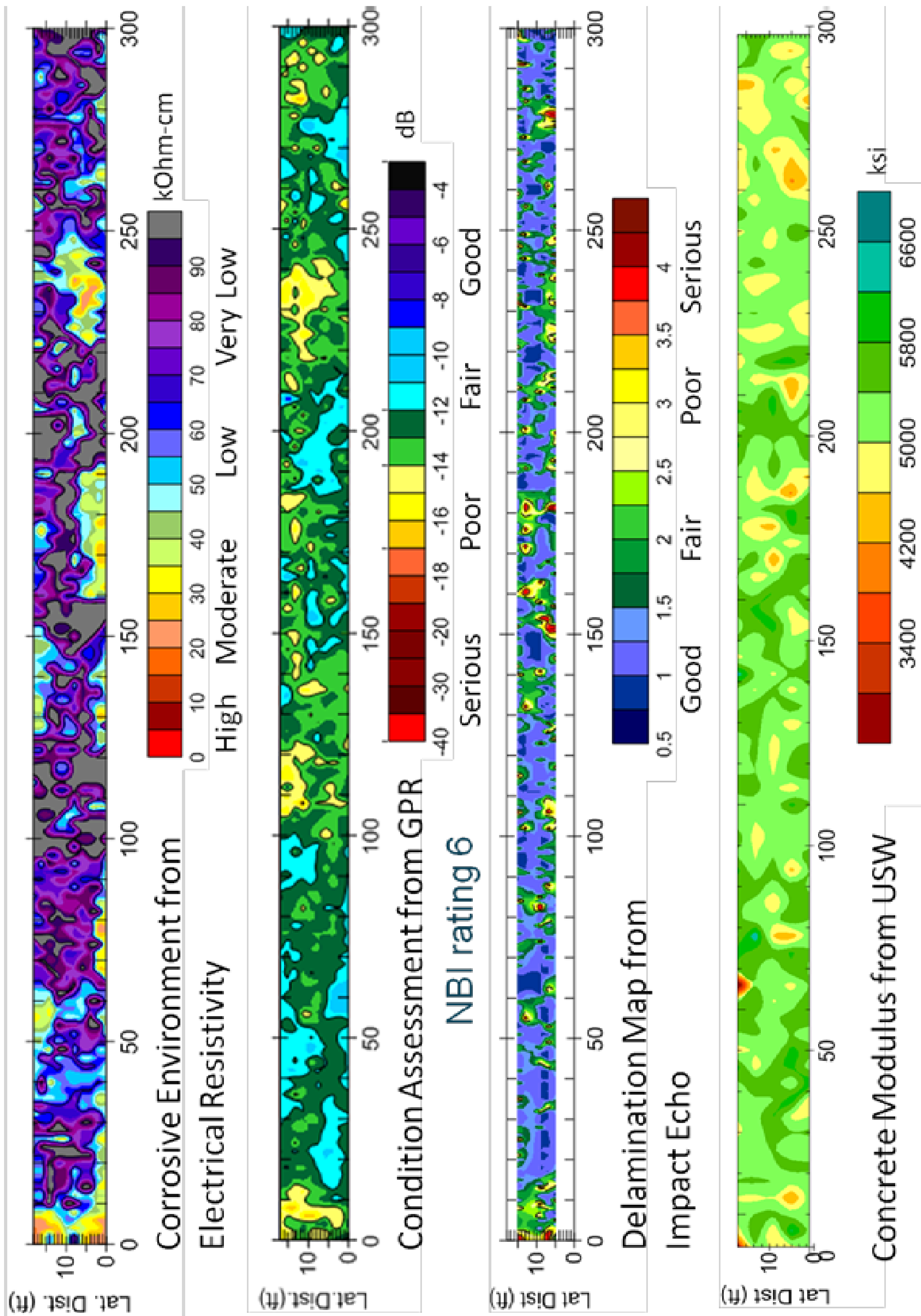

Figure 3. Condition maps for Ocean City Longport bridge deck.

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k -cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

In addition to condition ratings, average elastic modulus and standard deviation of the modulus was calculated for the bridge deck section. The average concrete modulus is 5235 ksi, while the standard deviation 431 ksi.

The calculated condition ratings with respect to delamination and corrosion describe the bridge deck to be in a good condition. Also, the calculated standard deviation of concrete modulus describes a low to moderate variability of concrete modulus.

Condition Assessment of the Creek Road over I-295 Bridge Deck using NDE

The condition assessment of this deck with Class A concrete and uncoated rebars concentrated on four evaluations:

1. Delamination assessment using impact echo (IE),
2. Corrosive environment assessment using electrical resistivity (ER) measurement,
3. Corrosion activity evaluation using half-cell potential (HPC), and
4. Concrete quality assessment through the measurement of concrete modulus using ultrasonic surface waves (USW) method.

The data collection was conducted on November 21, 2013 on a 255 feet long and 24 feet wide (two right lanes) eastbound section of the bridge deck (Figure 1). The survey was conducted on a two by two feet grid. The first line of the testing grid was one foot away from the curb of the sidewalk. Four NDE technologies were deployed. The impact echo testing was conducted using an impact echo cane. The electrical resistivity measurements were conducted using a four-electrode Wenner probe. The concrete modulus measurements were conducted using a portable seismic property analyzer (PSPA). Finally, corrosion activity was measured using a rolling HCP probe.

Figure 1. Image of the NDE surveyed area of Creek Road over I-295 Bridge.

The results of the NDE surveys using the four technologies are represented by the condition maps in Figure 2. The initial information about the year the bridge deck was constructed was 1972. In addition, the latest NBI rating of the deck was 5, which would not be a surprising condition for a more than forty years old concrete deck. However, all the results shown in Figure 2 are describing a deck in an excellent condition. There are no signs of corrosive environment, electrical resistivity is throughout the deck above 100 kOhm-cm. Also, half-cell potential readings have either very low negative or positive values, indicating very unlikely corrosion activity. Similarly, there were only a few points indicating potential signs of delamination. Finally, the USW survey map describes concrete generally above 5000 ksi, with some sections in a range 4000 to 5000 ksi.

Condition ratings with respect to delamination and corrosion for the section of the bridge surveyed were calculated to be 94.0 and 99.7, respectively. The delamination rating, on a scale 0 (worst) to 100 (best), is calculated using the following formula:

Delamination rating = % area in severe*0 + % area in poor*40 + % area in fair*70 + % sound area *100

Different delamination levels or grades, and how they are evaluated, are described in Appendix A. Similarly, the corrosion rating is defined with respect to the severity or corrosive environment from electrical resistivity measurements using the following formula:

Corrosion rating = % area (Resistivity<10 k □cm)*0 + % area (10<Resistivity<25) *40 + % area (25<Resistivity<40)*70 + % area (Resistivity>40)*100

In addition to condition ratings, average elastic modulus and standard deviation of the modulus was calculated for the bridge deck section. The average concrete modulus is 5495 ksi, while the standard deviation 1275 ksi.

The calculated condition ratings with respect to delamination and corrosion describe the bridge deck to be in a good condition. Also, the calculated standard deviation of concrete modulus describes relatively high variability of concrete modulus.

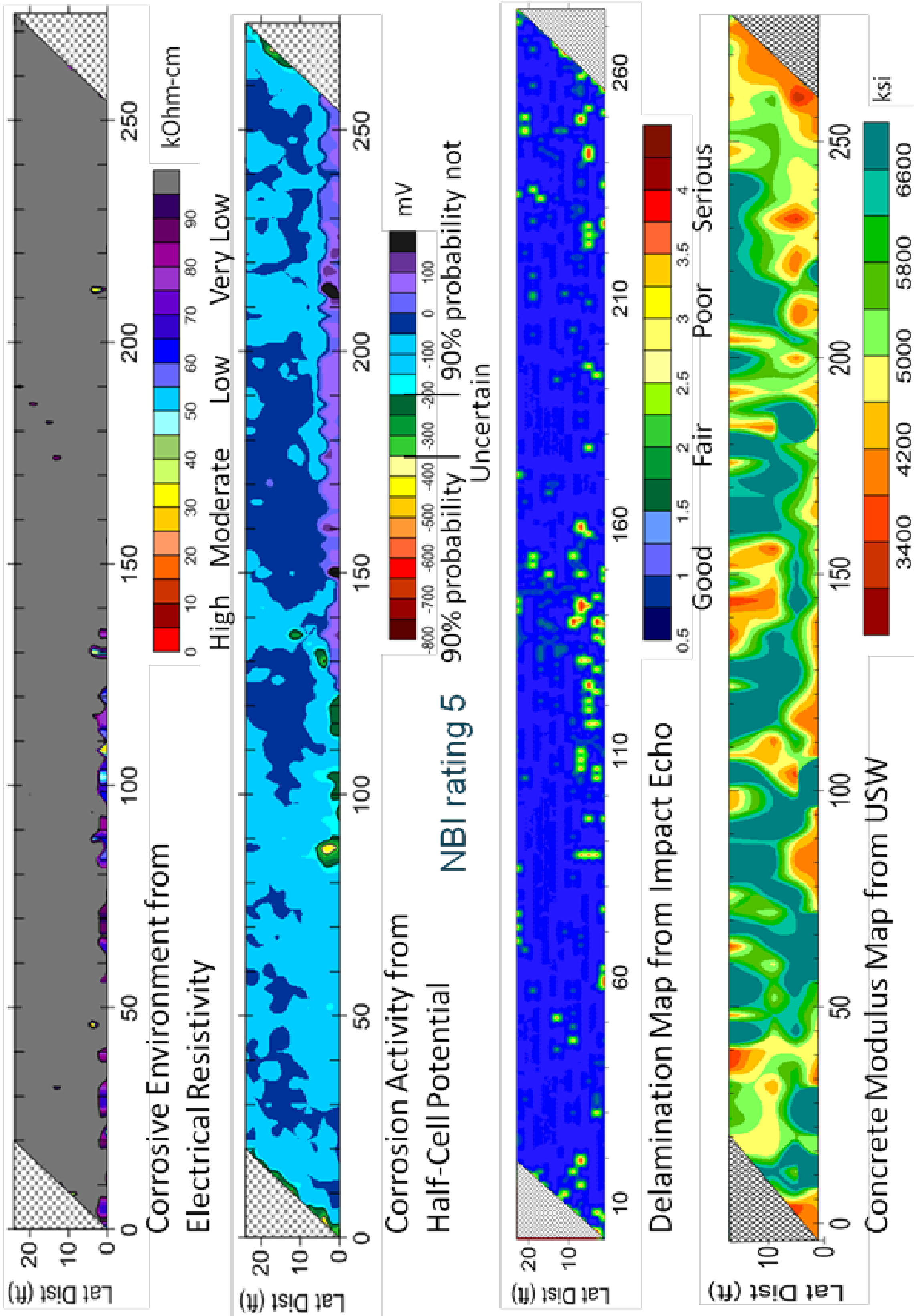

Figure 2. Condition maps for Creek Road over I-295 bridge deck.

Monitoring of concrete curing using NDE- route 3 over Passaic River and Collings Avenue over I-676 bridges

The objective of the surveys on two bridges was to both demonstrate the ability of the NDE technologies to evaluate maturing of concrete in newly constructed bridge decks, Route 3 over Passaic River, and Collings Avenue over I-676 bridges. From all the NDE technologies deployed on the existing bridges, two technologies were of specific interest: ultrasonic surface waves (USW) method for concrete modulus measurement (Figure 1), and electrical resistivity (ER) for assessment of concrete resistivity due to changes in the moisture content. The goal was to describe the maturing of concrete through periodical measurements of modulus and resistivity, at least on a weekly basis. This was not achieved due to deployment of the team and equipment to other states for a previously arranged bridge deck surveys. The team will use opportunities in 2014 to conduct multiple periodical USW and ER measurements on newly constructed HPC bridge decks. Still the limited results from the two bridges capture the maturing of concrete.

Figure 1. Concrete modulus measurement on Route 3 over Passaic River bridge (June 11, 2013).

The map in Figure 2 illustrates concrete modulus variation for a six by feet are of the deck. The testing was conducted on June 11, 2013, when the deck was 22 days old. The burlap was temporarily removed to provide access to the deck (Figure 1). While there were significant variation in the modulus, it was also on an average on a lower side of an anticipated, indicating still maturing of concrete. Even more pronounced observations can be made for the deck of the Collings Avenue Bridge that was surveyed on July 29, 2013, 14 days after the deck was poured. The concrete modulus in this case is in the greatest part below 3000 ksi.

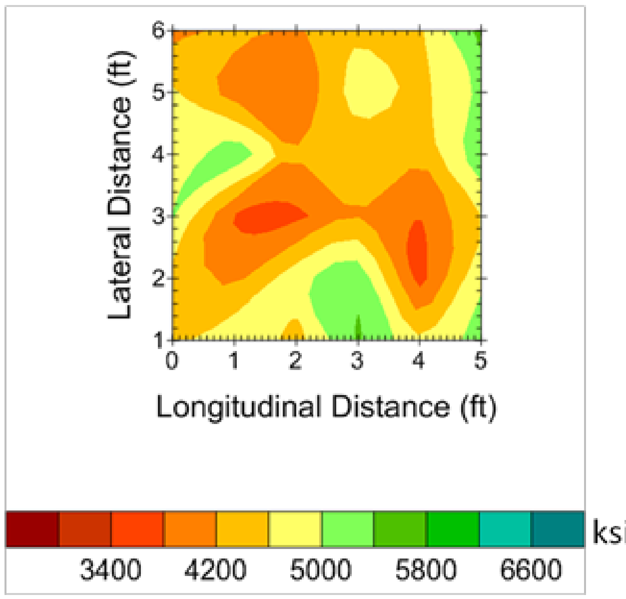

Figure 2. Concrete modulus map for a section of Route 3 over Passaic River bridge deck.

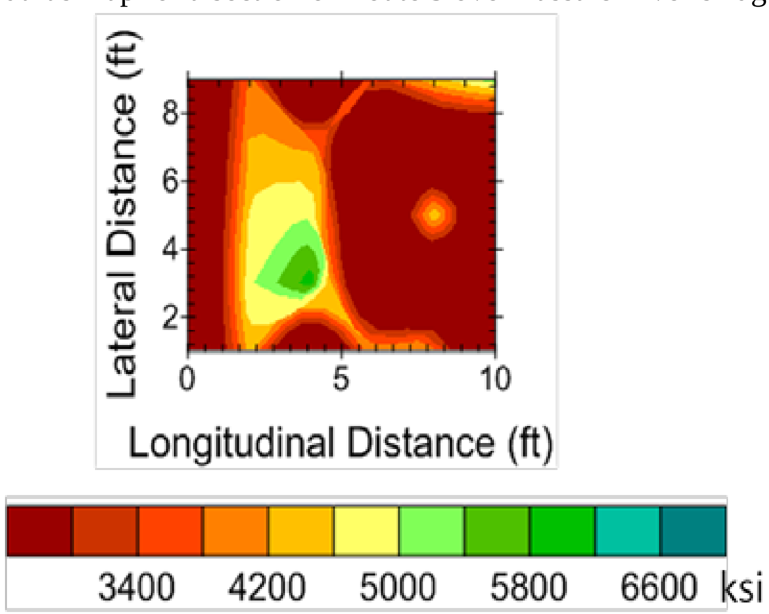

Figure 3. Concrete modulus map for a section of Collings Avenue over I-676 bridge deck.

Similar indications of a maturing concrete are coming from electrical resistivity measurements. A resistivity map of a section of the Collings Avenue bridge deck is shown in Figure 4. The measurements were also conducted on July 29, 2013, on a fourteen days old deck. The resistivity is in the entire area below 20 kOhm-cm, which is an indication of concrete with a still significant moisture content. The second ER measurement conducted on the entire bridge deck on August 22, 2013, 38 days after concrete pouring provided a map of increased resistivity, as shown in Figure 5. This increase is a result of reduced moisture or drying of concrete.

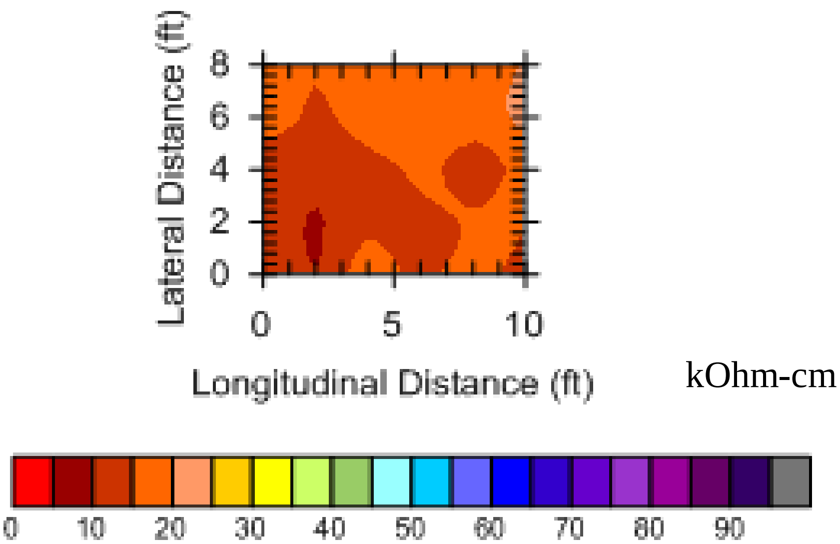

Figure 4. Electrical resistivity map for a section of Collings Avenue bridge deck (July 29, 2013).

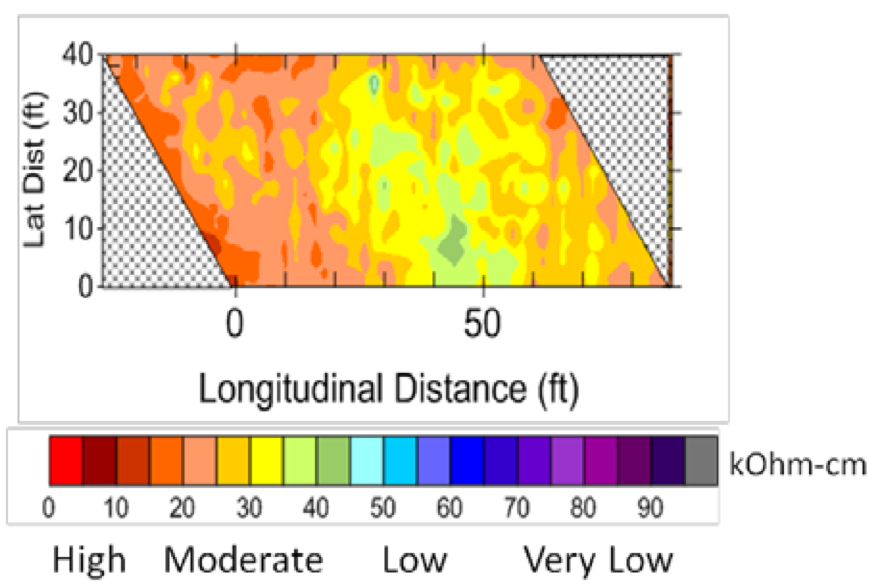

Figure 5. Electrical resistivity map for Collings Avenue bridge deck (August 22, 2013).

Appendix C – Summary report Comparative Durability Analysis on bridge decks

Introduction

SIMCO Technologies Inc. (SIMCO) performed an investigation about cracking observed on a series of high-performance concrete (HPC) bridge decks under the jurisdiction of the New Jersey Department of Transportation (NJDOT), that were built over the past few years. SIMCO's contribution was designed as a complementary effort to the more global approach developed by Rutgers University's Center for Advanced Infrastructure and Transportation (CAIT) under the auspices of the Bridge Resource Program (BRP)[10]. This program is intended to provide engineering support to the New Jersey Department of Transportation (NJDOT) to address the most important issues affecting bridge infrastructure in New Jersey, particularly in the domains of bridge evaluation, monitoring and asset management.

The objectives of the investigation were:

- To compare the costs and benefits of using HPC for the construction of durable bridge decks exposed to de-icing salts:
- To determine the causes of the early-age cracking in HPC bridge decks;
- To evaluate the impact of the observed cracking on the long-term performance of decks.

Ultimately, results of this study will be used to develop recommendations on the use of HPC, propose methods to avoid premature cracking of bridge decks and, if required, identify ways to extent the service-life of cracked HPC decks.

Background

Over the last twenty years, the improvement of mixture design methods, the widespread use of supplementary cementing materials, and the development of highly effective plasticizing admixtures have contributed to increase the overall performance of cement-based materials and led to the customary use of HPC. In New Jersey, as in many other jurisdictions, a large number of bridge decks has been built or rebuilt using HPC instead of ordinary concrete (or Class A concrete according to NJDOT classification) during that period. Despite their superior mechanical properties and improved durability, HPC mixtures can be more sensitive to early-age cracking. The characteristically low water-binder ratio and high cement content of HPC tend at the same time to increase the rate and the intensity of early-age shrinkage, to increase the elastic modulus and reduce the creep potential, making the material more susceptible to the development of harmful internal stresses during the early stages of its life. The problem can be exacerbated by the simultaneous occurrence of thermal stresses. It was mentioned to SIMCO that

[10] Szary, P., Gucunski, N., A. Roda E. Ouellet, R. Cantin and J. Prader, 2014. Performance of High Performance Concrete in New Jersey: A Comprehensive Study, Technical Memorandum, Report No. RU435056-3, Center for Advanced Infrastructure & Transportation (CAIT), Rutgers, the State University, Piscataway, NJ

NJDOT officials report that numerous HPC decks have exhibited cracking not long after casting. The NJDOT developed a remedial procedure, which involves the application of a methyl methacrylate (MMA) sealer on the surface of the deck. One of the main expected advantages in using HPC is the increased durability and lower resulting maintenance cost. It remains to be demonstrated that the MMA application restores the initial durability of the concrete and the resulting durability of a cracked and sealed HPC deck might not justify the premium related to the use of HPC and sealer application.

Scope of work

The tasks performed by SIMCO during 2013 are summarized in Table 1. A detailed description of each task and activity of the proposed program is given in the next paragraphs.

Table 1 – Activities performed by SIMCO in 2013

Task	Activity and deliverable
S1–Review of existing documentation	· Review and analyze documentation provided by NJDOT on the characteristics of HPC structures · Review and analyze data generated by CAIT for each structures
S2–Field work on existing structures	· Complementary inspection of cracked HPC bridge decks · Extraction of a limited number of concrete cores · Additional corrosion measurements (if needed) · Characterization of concrete cores
S3-Field work on new bridge decks	· Sampling of concrete cylinders during bridge deck construction · Characterization of concrete samples
S4-Service-life analysis (Class A v. HPC)	· Comparative service-life analysis for uncracked decks using STADIUM®

Task S1 - Review of existing documentation and analysis of available data

This portion of the study consisted in reviewing available information and data such as:

- Drawings
- Inspection reports
- Delamination surveys (chain drag, GPR)
- Crack surveys
- Corrosion activity measurements
- NDT data

Part of this information was generated by other members of the BRP.

Task S2 - Field investigation – Existing structures

This task consisted inperforming a visual inspection on 8 existing structures identified by CAIT and approved by NJDOT focusing on the presence of cracks. This task also included core extraction in selected areas to perform concrete physical and transport property testing. Cores were tested according to an experimental program designed to determine the properties of the in-situ concrete, assess the extent of chloride contamination and generate input data for the service-life calculations. The testing protocol is summarized in Table 2.

On the basis of initial non-destructive testing results, additional measurements could be performed to clearly establish the impact of existing cracks on the durability of the reinforced concrete deck.

Table 2 – Experimental protocol for cores extracted from existing structures

Test description	Test method
Core examination and measurement	ASTM C 1542
Petrographic examination	ASTM C 856
Compressive strength	ASTM C 42
Chloride contamination	ASTM C 1152
Air-void network characteristics	ASTM C 457
Thermal expansion coefficient	USACE CRD–C 39–81
Volume of permeable voids	ASTM C 642
Migration	ASTM C 1202 modified
Drying	ASTM C 1585 modified

<u>Task S3 - Field investigation – new structures</u>

This task consisted in sampling concrete for testing on new structures, monitoring crack formation, and collecting temperature and relative humidity data, both inside and outside the concrete. This part of the work was coordinated with other BRP partners.

Samples were characterized according to the experimental protocol presented in Table 3. The objective of the protocol was to characterize the evolution of concrete properties and generate input data for the early-age cracking analysis and service-life calculations.

Table 3 – Experimental protocol for concrete cylinders sampled during construction

Test description	Test method
Petrographic examination	ASTM C 856
Compressive strength	ASTM C 42
Splitting-tensile strength	ASTM C496
Elastic modulus	ASTM C 469
Shrinkage	ASTM C 157
Air-void network characteristics	ASTM C 457
Thermal expansion coefficient	USACE CRD–C 39–81
Volume of permeable voids	ASTM C 642
Migration	ASTM C 1202 modified
Drying (includes porosity testing)	ASTM C 1585 modified

<u>Task S4 – Comparative durability analysis</u>

The results of the characterization program were used as input parameters in STADIUM® simulations to compare the service-life of HPC and Class A concrete decks. The simulation program included:

- The determination of representative exposure conditions on the existing HPC decks;
- Durability analysis of HPC decks (cracked and uncracked), based on the concrete properties determined from the investigated structures;
- Durability analysis of Class A concrete decks, based on the properties of such concrete mixtures determined in the course of the current study.

The results of these simulations were analyzed in view of the expected durability and service life expectations for both types of concrete.

General information on selected bridges

Overall, 10 bridge decks were selected for the current investigation. Among these, 8 were on existing structures and 2 were built during the course of the study. The selected structures have different characteristics to provide a representative sample of bridge decks under the jurisdiction of the NJDOT. Most of these bridges were made of HPC to provide as much information as possible about the recurrent cracking problems affecting these decks. However, some decks made with conventional Class A concrete (i.e., made without supplementary cementitious materials), were also included in the investigation. The main characteristics of the investigated decks are given in Table 4 and the theoretical concrete mixture proportions are given in Table 5.

Table 4 – Main bridge characteristics

Bridge name	Structure number	Type of deck and supporting element	Built in	Bridge length† (ft)	Bridge width (ft)	Deck thickness° (in.)	Total number of lanes
Route 70 over Bispham's Mill Creek	0311-150	Concrete bridge deck on precast concrete caissons	2005	23.17	47.00	8.5	2
Smith Street bridge over I-440	1234-509	Concrete bridge deck on permanent steel formwork supported by steel beams	2010*	84.85	83.00	9	4
Route 52 NB over Elbow Channel‡	0511-152	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2008	8,126	98.83	6, 9.5**	4
Route 52 NB over Rainbow Channel‡	0511-151	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2009	2,569	98.83	6, 9.5**	4
Creek Road over I-295	0327-166	Concrete bridge deck on permanent steel formwork supported by steel beams	1970	259	88.00	8	5
Route 9 (Edison Bridge) Northbound	1209-155	Concrete bridge deck on permanent steel formwork supported by steel beams	2003*	4,452	52.49	10.24	3
Route 9 (Edison Bridge) Southbound	1209-156	Concrete bridge deck on permanent steel formwork supported by precast concrete beams	2003	4,452	52.49	10.24	3
Ocean City-Longport Bridge	3100-003	Concrete bridge deck on 3.5 in precast concrete slab supporting panels	2002	3,450	75.83	8.5	2
Collings Avenue	0418-151	Concrete bridge deck on permanent steel formwork panels supported by steel beams	1954, 2013*	167.3	52.0	8.5	4
Route 3	1601-162	Concrete bridge deck on permanent steel formwork panels supported by steel beams	1949, 2013*	178.0	164.5	9	3

† Free span length from end abutments.
‡ Visitor center CL taken as border line between Elbow and Rainbow Channel spans.
* Deck rebuilt.
** Variable deck slab thickness of 6 in over beams and 9.5 in between beams.
° The plans do not indicate whether the required concrete cover takes into account the presence of grooves present on all decks.

Table 5 – Concrete mixture proportions (based on mix designs)

Bridge	W/B ratio	Cement type I/II	Slag	Class F fly ash	Silica fume	Sand	Coarse aggregate	Air content (%)
		(lbs/cy)						
Route 70	0.40	395	263	-	-	1,199	1,700	5.8
Smith Street	0.40	395	263	-	-	1,242	1,850	6.5
Route 52 Elbow Channel	0.37	353	247	106	-	1,208	1,625	6.2
Route 52 Rainbow Channel	0.37	395	263	-	-	1,247	1,850	6.3
Creek Road	NA	NA	NA	NA	NA	NA	NA	NA
Route 9 northbound	0.37	394	263	-	-	1,250	1,850	5.5
Route 9 southbound	0.37	394	263	-	-	1,250	1,850	5.5
Ocean City Longport Bridge	0.37	658	-	-	-	1,220	1,770	7.0
Collings Avenue	0.37	353	247	106	-	1,208	1,625	5.1
Route 3	0.40	570	-	130	25	1,083	1,773	5.0

Field work

Observations

All 10 bridge decks, the 8 existing and the 2 new, were inspected between April and October 2013 by Michel Plante, Eng., and Patrick Power, Jr. Eng., from SIMCO. The existing bridge decks were variably affected by early-age cracking. For the existing decks, the field work consisted in assessing the bridge deck condition and supervising coring operations. The crack patterns, width and density were documented. In the case of new decks, the field work consisted in witnessing the deck casting operations and collecting fresh concrete samples. The observations on each bridge are summarized in Table 6.

Table 6 – Summary of observations on the decks

Bridge	Crack width	Crack depth (in.)	Crack length (ft)	Crack type	Remarks
Route 70	Hairline to 0.006 in. (0.2 mm)	Up to 6	1 to 5	Transverse	· Section investigated was 23 ft long by 25 ft wide. · Very few cracks on the bridge. Few cracks on the east approached slab. · Only 4 transverse fine cracks, 2 to 5 ft long. · 3 to 4 cracks on the east approach slab. · Hairline cracks 6 to 12 in. long originate from deck abutment, parallel to longitudinal axis.
Smith Street	Average of 0.006 in. (0.2 mm)	Up to 6	5 to 17	Transverse	· Section investigated was 95 ft long by 17 ft wide. · Most of the cracks initiate from construction joint. · From 5 to 17 ft long. · Deck vibrates with passage of trucks.
Route 52 Elbow	Average of 0.009 in. (0.25 mm)	2 to 5	2 to 30	Random, transverse, longitudinal	· Over the 300 ft long section, multiple cracks. · A section between the 180 and 300 marks was overlaid. · Cracks follow no distinctive pattern. · Deck vibrates with passage of trucks.
Route 52 Rainbow	Average of 0.009 in. (0.25 mm)	4 to 6	2 to 30	Random, transverse, longitudinal	· Over the 300 ft long section, numerous cracks of different shapes. · Cracks follow no distinctive pattern. · Deck vibrates with passage of trucks.
Creek Rd	Hairline to 0.009 in. (0.8 mm)	0.4 to 7	5 to 9	Transverse	• Section investigated was 260 ft long by 25 ft wide. • Cracks initiate from or near the curb. • Overall, the cracks appeared to taper to a width of less than 0.004 in. at approximately 9 ft from the curb. • The east span presented a lower crack recurrence than the west span of the investigated section.
Route 9 northbound	Average of 0.006 in. (0.2 mm)	3 to 6	5 to > 20	Transverse	· Over the 300 ft long section, about 55 fine transverse cracks. · Cracks initiate from parapet and expand up to full width of investigated section. · This section of the deck is made with three different concrete pours. · Concentration of crack is similar from one pour to the other. Deck vibrates with passage of trucks.
Route 9 southbound	Average of 0.006 in. (0.2 mm)	2 to 6	2 to > 20	Transverse, plastic shrinkage, random	· Over the 300 ft long section, about 30 fine transverse cracks. · Cracks initiate ± 6 ft from parapet and expand up to full width of investigated section. · This section of the deck is made with three different concrete pours. · Concentration of crack is different from one pour to the other. · Deck vibrates with passage of trucks.
OCLP	Hairline to 0.006 in. (0.20 mm)	0.25 to 5	2 to > 20	Transverse	· Over the 300 ft long section, about 15 very narrow transverse cracks from parapet through opposite side of the deck · 4 to 5 longitudinal cracks up to 10 ft long. · Cracks initiate close to parapet and expand up to full width of investigated section. · Other cracks (plastic shrinkage, crazing) were revealed only by wetting and drying.

Sampling

The concrete cores from the existing bridge decks were sampled in conformity with ASTM C42. The majority were selected in uncracked concrete to characterize the physical properties, the transport properties and the condition of the bridge decks. However, cores were also extracted over cracks representative of the observed patterns. Table 7 presents a summary of the core sampling on the existing bridge decks.

For the new bridge decks, the concrete cylinders used in the laborarory investigation were cast from concrete delivered onsite during deck pouring operations. All cylinders were produced in conformity with Section 7 of ASTM C31 and all rectangular prisms were produced in conformity with ASTM C157. Samples were obtained from more than one truck for both bridge deck pours. Table 8 presents a summary of the fresh concrete sampling for laboratory testing.

Table 7 – Core sampling in existing bridges

Bridge name	Number of cores	Area of deck investigated		Number of pours
		Length (ft)	Width (ft)	
Route 70	17	70	27	2
Smith Street	17	100	17	1
Route 52 Elbow Channel	15	300	20	3
Route 52 Rainbow Channel	15	300	20	3
Creek Road	17	260	25	2
Route 9 northbound	17	300	20	3
Route 9 southbound	17	300	23	3
Ocean City Longport Bridge	17	300	20	2

Table 8 – Concrete sampling on new bridges

Bridge name	Number of trucks sampled	Number of cylinders produced	Number of beams produced*
Collings Avenue	2	12	3
Route 3	3	18	3

*11.75 x 3 x 3 in. beams produced in conformity with section 7 of ASTM C157.

Laboratory test results

The main conclusions of the laboratory testing results are summarized hereafter. Details are given in the individual report prepared for each bridge, provided as attachments to the complete durability analysis report prepared by SIMCO[11].

[11] Cantin, R., Ouellet, E., 2014., Technical Memorandum, Report No. RU435056-5, Center for Advanced Infrastructure & Transportation (CAIT), Rutgers, the State University, Piscataway, NJ

Existing bridges

The compressive strength is significantly higher than the verification strength[12], although in the expected range of long-term values for the mixtures tested. It is interesting to note that the strength of HPCs is not significantly higher than that of Class A concretes. This may be explained by the fact that all mixes had a low water-binder ratio and that the only difference between both categories is that HPCs are made with SCMs.

The air-void system is not consistently adequate to resist damage induced by freezing and thawing. Only 2 of the 8 existing bridges have a spacing factor below 0.008 in. (200 µm), the recommended value according to ACI 201.1R – *Guide to Durable Concrete*. This value represents a general recommendation to ensure the concrete is resistant to freezing and thawing damage. In reality, each concrete has a critical spacing factor value, which can be as high as 400 µm in certain cases, particularly for high performance concrete.[13] All the recorded spacing factors are lower than 400 µm, which could explain the fact that the investigated decks do not exhibit severe scaling or other forms of damage that could be related to a poor frost resistance. Thermal expansion coefficients are in the order of 11 to 14 microstrains/°C. This is in the upper part of the commonly reported range (approximately 6 to 14 microstrains/°C). The investigated concretes are made with trap rock, which is not known to yield high thermal expansion coefficients. The high thermal expansion coefficients may be related in part to the high paste content of HPCs.

The chloride contamination is overall consistent across each investigated bridge deck, except for the Route 9 northbound bridge, for which variable conditions were recorded. Older decks tend to be more contaminated, although the contamination is not always proportional to the age of the decks, which is an indication of variable exposure conditions and/or transport properties. For example, the Creek Road Bridge deck is not the most contaminated, although it is 40-year old. Interestingly, no clear influence of the cracks on chloride contamination could be noted. Cracked cores tend to be more contaminated – although not always the case – but the higher contamination may either be present near the surface, deep from the surface, or on the entire depth. Table 9 summarizes the influence of cracks on chloride penetration in existing decks. The diffusion coefficients of HPCs are indicative of a good resistance to chloride penetration. As could be expected, concretes made without supplementary cementitious materials (SCM) have a higher diffusion coefficient. As for the contamination, the influence of cracks on the diffusion coefficient is not constant. Table 10 gives the influence of cracks on the diffusion coefficient.

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[12] The verification strength specified by NJDOT is 5,400 psi. This strength is the requirement that applies to concrete sampled during a verification batch.

[13] M.Pigeon and R.Pleau, *Durability of Concrete in Cold Climates*, E & FN Spon, 1995

Table 9 – Influence of cracks on chloride penetration

Bridge	Influence of cracks
Route 70	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core not the most contaminated, even at higher depth
Smith Street	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core more contaminated deeper than 0.5 in.
Route 52 Elbow Channel	· Influence of crack mainly at a shallower depth (< 0.5 in.) · Cracked core more contaminated in first 0.5 in. only
Route 52 Rainbow Channel	· Cracked core more contaminated over all depth · Not a significant effect though
Creek Road	· Influence of crack mainly at higher depth (> 0.5 in.) · Cracked core more contaminated deeper than 0.5 in.
Route 9 northbound	· No cracked core · Variable contamination
Route 9 southbound	· Cracked core more contaminated over all depth · Significant effect of crack
Ocean City Longport Bridge	· No influence of crack

Table 10 – Influence of cracks on the diffusion coefficient

Bridge	Influence of cracks
Route 70	· Cracked core lower than 3 uncracked and higher than 2
Smith Street	· Cracked core approximately twice the diffusion coefficient of uncracked (11.6 vs. 5-6)
Route 52 Elbow	· Cracked cores (2) identical and lower than 1 uncracked and higher than 3
Route 52 Rainbow	· Cracked core lower that 4 uncracked and higher than 1
Creek Road	· Cracked core lower than 1 uncracked and higher than 4
Route 9 northbound	· Cracked core higher than all 5 uncracked cores (5.6 vs. 5.4 for the second)
Route 9 southbound	· Cracked core is higher than all 5 uncracked cores
OCLP	· Cracked core higher than all 5 uncracked cores (11.0 vs. 10.1 for the second) · Variable values

The volume of permeable voids is higher for HPCs than for conventional mixtures. No clear explanation could be found for this, but it could either be related to a higher water content, the aggregate porosity or the composition of the cementitious matrix. It is interesting to note that the Route 52 Elbow Channel samples present the lowest diffusion coefficients along with the lowest compressive strengths, while the Route 9 southbound samples present some of the highest diffusion coefficients along with the highest compressive strengths. This is an indication that high compressive strengths do not necessarily lead to improved durability.

Petrographic examinations carried out on cores extracted from the existing decks suggest that the cracking observed on the decks would be caused by early-age drying shrinkage. The cracks had a maximum width of 0.02 to 0.04 in. tapering to 0.01 in. away from the surface. The cracks visible on the surface extend to various depths, from fractions of inches to the entire depth of the core. In addition to the cracks that are visible on the decks, micro-cracking was observed on the majority of the cores. Micro-cracking is either in the form of thin surface cracks extending down to several inches into the core to several inches or distributed cracking within the paste, consequent with autogeneous shrinkage. The estimated concrete characteristics correlate with the theoretical mixture proportions except in two cases. The Smith Street concrete has an estimated water-binder ratio between 0.35 to 0.45, which could be higher than the theoretical 0.40 value. Also, the Route 9 southbound bridge concrete is only made with Portland cement. No SCM was observed in the sample analyzed (Core SB-06). This would explain the high diffusion coefficient recorded on this concrete. In addition, the volume of permeable voids was lower for this concrete, which makes it very similar to the OCLP concrete, which was a low-water-binder ratio concrete made without SCMs.

New decks

The compressive strengths recorded at 28 days are much higher than the verification strength at 56 days. This is in agreement with the results on existing bridges and can be related to the durability requirements that imply low water-binder ratios and the use of supplementary cementitious materials. The other mechanical properties are also high. However, the elastic modulus recorded on the Route 3 concrete is lower while the compressive and tensile strengths are higher. With respect to cracking, higher elastic moduli are not necessarily desirable because they limit the deformability, which can result in more severe cracking under imposed deformations. In this perspective, the Route 3 concrete would have a lesser tendancy to crack.

The air-void network is adequate, with a spacing factor below the 0.008 in. value recommended by ACI 201.2R – *Guide to Durable Concrete* for resistance to freezing and thawing. The drying shrinkage recorded on both concretes is higher than 500 microstrains at 56 days, and still increasing afterwards to exceed 600 microstrains at 112 days. The NJDOT has no current standard specification for drying shrinkage but other jurisdictions usually set limits between 400 and 500 microstrains. For instance, the NJTA limits the drying shrinkage to 450 microstrain for HPCs. FHWA also provides limits for different classes of HPC, but in this case, the values are higher and are based on measurements at 180 days.[14] The values recorded on both decks are considered high and this can be a significant contributing factor explaining the presence of cracks on HPC decks.

The volume of permeable voids is higher than expected, and this is similar to what was observed on existing decks made of HPC. As mentioned previously, this can be explained in part by the

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[14] Based on SHRP C/FR-91-103, p. 3.25

type of aggregate used or by excessive water addition. Given the good mechanical properties, the high volume of permeable voids is more likely related to the aggregate used. However, variability was noted for the Collings Avenue concrete, some individual results being lower and in the expected range. This suggests either some heterogeneity in the concrete itself or it may be related to the sampling process. The diffusion coefficients are at the upper end of the expected range for the type of concrete used to build both decks. Interestingly, the values at 90 days are not much lower than those at 28 days. Some improvement was noted though for the Collings Avenue concrete, but none for the Route 3 concrete. Usually, with Class F fly ash and slag, a more significant improvement of the transport properties is expected due to prolonged hydration of these SCMs. The lack of improvement of the diffusion coefficient between 28 and 90 days for the Route 3 concrete was not clearly explained but can be related to the hydration kinetics of the binder or to a dispersion problem with the fly ash, which would have caused the analyzed samples to have a low fly ash content.

Petrographic examinations have indicated the presence of silica fume agglomerations. Silica fume agglomerations indicate insufficient blending and are to be avoided because they can result in alkali-silica reaction in a short period of time. Furthermore, poor silica fume dispersion does not allow to fully benefit from its effect on pore size distribution and transport properties. In the case of the Collings Avenue sample, the examination revealed that it was made of Portland cement with moderate amount of slag and minor amount of Class F fly ash. This is in agreement with the mix design. The degree of hydration of the new concrete samples are generally low to moderate, but this was expected considering the age of the concrete at the time the examinations were performed.

Corrosion initiation

For steel bars embedded in concrete, corrosion can be initiated when the chloride threshold for corrosion initiation is reached or exceeded at the rebar depth, which corresponds to the concrete cover. Cover values measured on the investigated decks, at the location of the cores, are given in Table 11.

Table 11 – Cover values (in.)

Bridge	Nominal*	Measured on site			Observed on the cores			Selected for analysis
		Minimum	Maximum	Average	Minimum	Maximum	Average	
Route 70	2.5	2.32	3.66	2.92	2.91	4.33	3.44	2.92
Smith Street	2.5	2.32	2.83	2.47	2.68	4.29	3.29	3.30
Route 52 Elbow	2.5	N/A	N/A	N/A	2.36	3.54	3.03	3.03
Route 52 Rainbow	2.5	N/A	N/A	N/A	2.17	3.62	3.00	3.00
Creek Road	1.5	1.22	2.20	1.60	1.57	2.48	1.93	0.00
Route 9 northbound	2.6	2.87	3.78	3.23	2.91	4.33	3.45	3.30
Route 9 southbound	2.6	N/A	N/A	N/A	2.05	3.27	2.46	2.94
OCLP	2.5	3.25	4.04	3.59	3.39	3.70	3.49	3.59
Collings Ave	2.5	N/A	N/A	N/A	N/A	N/A	N/A	2.5
Route 3	2.5	N/A	N/A	N/A	N/A	N/A	N/A	2.5

*From the drawings

The critical chloride concentration required to initiate steel reinforcement corrosion in concrete has been extensively investigated in the past decades. Chloride threshold values found in the literature generally vary between between 0.30 % and 0.70 % of the mass of cement found in a concrete mix (see for instance Figure 1, taken from ACI 222 – *Protection of Metals in Concrete Against Corrosion*, based on recommendations from the *CEB Design Guide for Durable Concrete Structures*).

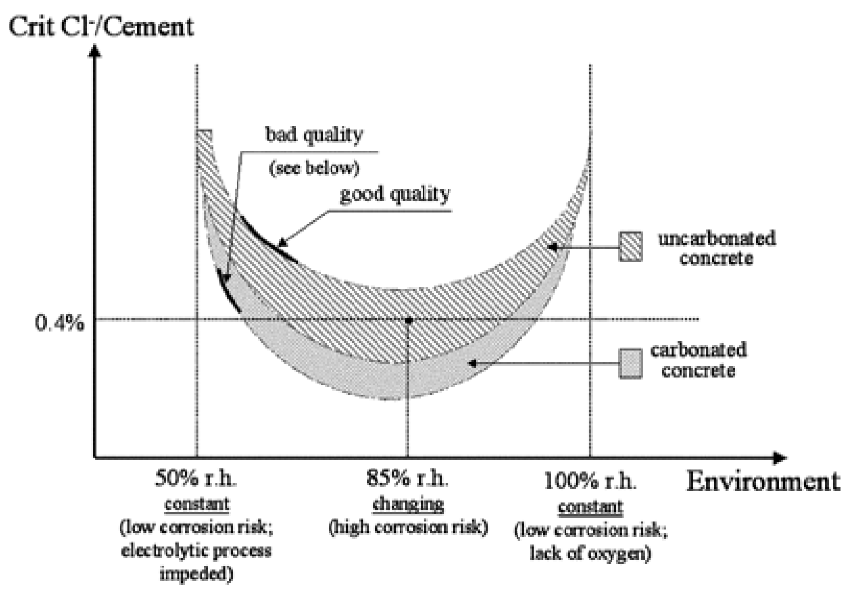

Figure 1 – Critical chloride content for different conditions

Epoxy coating can protect the embedded steel and prevent corrosion, but this method is not always efficient, since the coating may delaminate or exhibit flaws[15]. Pianca et al. (2005)[16] report that:

> *There is general agreement that the critical chloride concentration necessary to initiate corrosion at flaws in the epoxy coated rebar is the same as that for black steel and common sense would argue that it should be because the steel exposed at the flaws in the epoxy coating is black steel.*
>
> *...rapid crevice corrosion can be more prevalent in hot marine climates. The small anode-large cathode theory also suggests corrosion could be accelerated.*

Thus, as a conservative assumption, SIMCO does not consider the presence of the epoxy coating when evaluating the time to corrosion initiation. If the coating is damaged, corrosion initiates at the same chloride level, and can cause localized pitting in the exposed area, which, in the end, can be even worse than widespread corrosion. It was found in the same study that the use of epoxy coated rebar in a concrete deck does not improve noticeably the service life.

Tests recently performed by SIMCO have given a value of 0.50 ± 0.05 %.[17] When cores are extracted from an existing structure, the actual critical chloride concentration should be expressed as a ratio of the total mass of dry concrete. Such a conversion was made and results are given in Table 12, along with the number of profiles for which the threshold is exceeded at rebar depth.

Based on the chloride profiles, the calculated corrosion initiation threshold and the reinforcement depth, extensive corrosion should not be ongoing for the largest part of the bridge decks, since the chloride concentration at mean reinforcement depth does not exceed the corrosion initiation threshold in 41 cases out of 43.

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[15] Bertolini et al. (2000) "*Corrosion of Steel in Concrete – Prevention, Diagnosis and Repair*", Wiley-VCH

[16] Pianca, F., Schell, H. and Cautillo, G. (2005) *"The performance of Epoxy Coated Reinforcement: Experience of the Ontario Ministry of Transportation"*

[17] Henocq P., Samson E., Marchand J., Clark B., Determination of the chloride content threshold to initiate steel corrosion, 5th Int. Essen Workshop on Transport in Concrete (Essen, Germany), M.J. Setzer ed. Aedificatio Publishers, p. 25-39, 2007.

Table 12 – Chloride profiles currently exceeding the corrosion initiation threshold

Bridge	Chloride concentration at lowest tested depth (ppm)	Corrosion initiation threshold (ppm)	Cl profiles	
	Cracked		Exceeding threshold at selected cover	Total
Route 70	1,310 (isolated case) < 800 (cracked) < 300 (uncracked)	924	1	5
Smith Street	< 800 ppm (cracked) < 200 (uncracked)	877	0	5
Route 52 Elbow	~0	876	0	7
Route 52 Rainbow	~0	998	0	6
Creek Road	1,060 (cracked) < 530 (uncracked)	720	1	5
Route 9 northbound	< 650	875	0	4
Route 9 southbound	< 915 (cracked) < 185 (uncracked)	875	0	5
OCLP	983 at 1.3 in. (cracked) < 345 at 1.7 in. (uncracked)	901	0	6
Total			***2***	***43***

Service life analysis

Numerical calculations were performed for all existing and new bridge decks with the STADIUM® software. This tool can be used, among other things, to:

- Determine past exposure conditions;
- Calculate future chloride ingress;
- Determine the time and/or risk of corrosion initiation;
- Compare the performance of different materials;
- Evaluate concrete degradation.

Calculation assumptions, model validation and degradation evaluation are presented in the next sub-sections.

Concrete properties

The transport properties determined in the course of the study (see individual reports) were used as input parameters in the calculations. The mix proportions used in the calculations are presented in Table 13. They were based on the mixture designs given in Table 5, laboratory test results, and information provided by the petrographic examination. The only exception was Route 9 southbound, for which no slag was detected in the concrete and the properties corresponded more to what is expected for concretes made without SCMs. For this mix, the proportions used in the calculations differ from those given in Table 5 and are given in Table 13.

Table 13 – Mixture proportions used in the calculations

Bridge	W/B ratio	Cement type I/II	Slag	Class F fly ash	Silica fume	Sand	Coarse aggregate	Air content (%)
		(lbs/cy)						
Route 70	0.40	395	263	-	-	1,199	1,700	5.8
Smith Street	0.40	395	263	-	-	1,242	1,850	6.5
Route 52 Elbow	0.37	353	247	106	-	1,208	1,625	6.2
Route 52 Rainbow	0.37	395	263	-	-	1,247	1,850	6.3
Creek Road	0.35	573	-	-	-	1,193	1,888	6.5
Route 9 northbound	0.37	394	263	-	-	1,250	1,850	5.5
Route 9 southbound	0.37	657		-	-	1,250	1,850	5.5
OCLP	0.37	658	-	-	-	1,220	1,770	7.0
Collings Avenue	0.37	353	247	106	-	1,208	1,625	5.1
Route 3	0.40	570	-	130	25	1,083	1,773	5.0

As a comparison basis, for HPC decks, calculations were also performed for a conventional Class A concrete mix with proportions similar to those of the Route 9 southbound bridge deck concrete.

Exposure conditions

The exposure conditions used in STADIUM® calculations are: temperature, relative humidity and chloride exposure. The temperature and relative humidity on the top surface of each deck were taken from www.weatherbase.com for stations located closely to each structure. A de-icing salts exposure was selected to model future chloride ingress on the top surface of each deck. The boundary condition is modeled with a semi-sinusoidal chloride concentration curve centered at the coldest time of the year. The exposure period is a function of the number of days with sub-zero temperatures. In the current case, it varied generally between 20 and 24 days, with the exception of the Creek Road Bridge for which it was only 10 days. The amplitude (the chloride concentration in mmol/L) was determined from calculations performed with the STADIUM® model to reproduce the experimental chloride profiles recorded on the existing bridges. An example of calculated and experimental chloride profiles is presented in Figure 2. The chloride concentration that was found to reproduce the chloride profiles varies across locations. The variations could be related to local conditions (operator, equipment, road geometry) or the salt application frequency. Exposure conditions are summarized in Table 15 along with the location used for weather data assumptions.

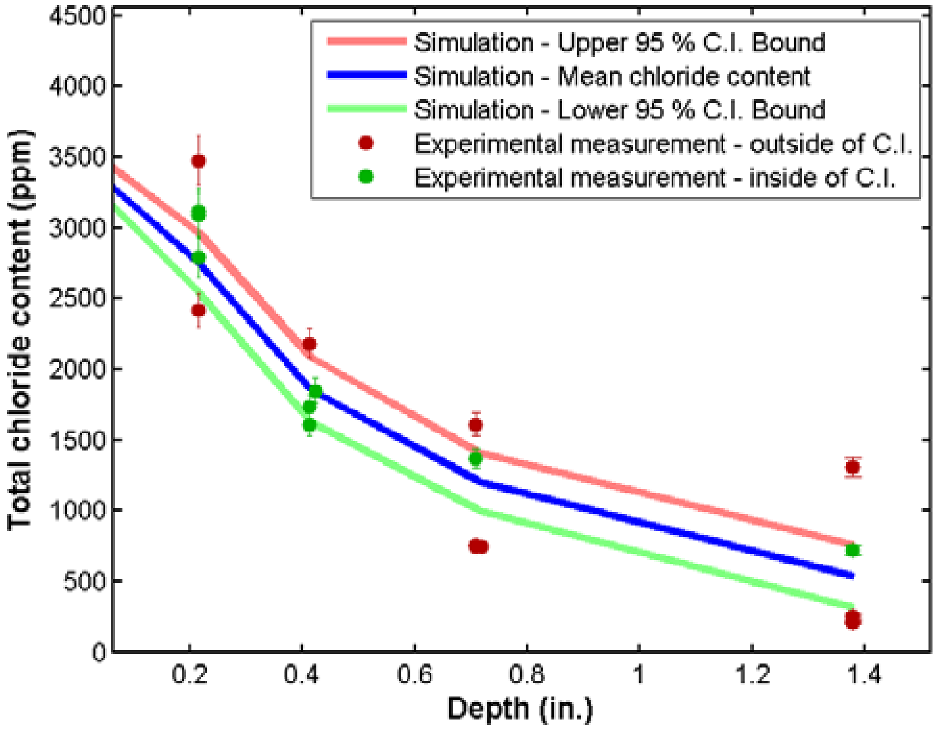

Figure 2 – Exposure conditions for the Route 70 Bridge

Table 15 – Exposure conditions

Bridge	Temperature and relative humidity				Deicing salts	
	Source	Temperature (°C)	Variation (°C)	Humidity (%)	Duration (days)	Concentration
Route 70	Toms River, NJ	52.9	21.4	70	20	Moderate
Smith Street	Adelphia, NJ	52.7	21.8	70	24	Low-moderate
Route 52 Elbow	Absecon, NJ	53.7	20.3	70	20	Low
Route 52 Rainbow	Absecon, NJ	53.7	20.3	70	20	Low-moderate
Creek Road	Toms River, NJ	52.9	21.4	70	10	Low
Route 9 northbound	Adelphia, NJ	52.7	21.8	70	23	Low
Route 9 southbound	Adelphia, NJ	52.7	21.8	70	23	Low
OCLP	Absecon, NJ	53.7	20.3	70	20	Moderate
Collings Avenue	Toms River, NJ	52.9	21.4	70	20	Moderate
Route 3	Adelphia, NJ	52.7	21.8	70	23	Low

Calculation approach

Contrary to other diffusion models, STADIUM® is a finite element software, and is not based on empirical relationships or coefficients. Therefore, it can be used to perform either deterministic or probabilistic analyses. In the current case, calculations were performed using a probabilistic approach based on Rosenblueth[18] estimations, which were used to calculate the statistical moments (mean, standard deviation and asymmetry coefficient).

This approach integrates the variability of the following parameters in the calculations:

- The exposure conditions;
- The concrete transport properties;
- The concrete cover;
- The corrosion initiation threshold.

For each parameter, the variability considered in the calculations was based on either the measured properties, past experience with similar mixes, or literature data. For all bridges, calculations were performed for the actual deck properties and for a higher diffusion coefficient, to account for the effect of cracks. Based on the measured properties, it was assumed that the cracks caused an increase of 200 % of the diffusion coefficient in the decks. For HPC decks, calculations were also performed with the properties of a reference Class A concrete to evaluate the benefit of the use of HPC, in uncracked and cracked conditions. The main calculation parameters and associated variability are given in Table 16.

Table 16 – Main calculation parameters and variability

Parameter	Description	Variability (COV)
Diffusion coefficient	Uncracked decks or uncracked sections of HPC decks: measured diffusion coefficient	15 – 25 % Based on measured values
	Cracked sections of HPC decks: two times the measured diffusion coefficient	25 % Based on measured values
	Reference Class A: Measured average on the Route 9 southbound deck	15 % Based on measured values
Permeability	Uncracked or cracked HPC decks: measured permeability	30 % in all cases Based on past experience
	Reference Class A: Measured value on the Route 9 southbound deck	
Corrosion initiation threshold	Calculated for each deck based on 0.5 % of the binder content and the measured dry bulk density, based on the proportions of the mixture for each deck or the proportions of the reference Class A mixture	15 % in all cases Based on literature data
Bar depth	Calculated based on measured values on each deck and observations on the cores	7 – 29 %, mostly between 14 – 17 % Based on measured values

[18] E. Rosenblueth. *Two-point estimates in probabilities*. Applied Mathematical Modeling, 5:329–335, 1981.

Calculation results

The calculation results are expressed as degradation curves, used to assess the risk of corrosion initiation as a function of time. The results can also be expressed as chloride content evolution as a function of time at a given depth. An example of each curve is given in Figures 3 and 4. For the purpose of the current analysis, the corrosion initiation risk has been classified as shown in Table 17. The summary of the calculations is given in Table 18.

Table 17 – Classification of the corrosion initiation risk

Risk	Zone	Interval	Repairs
Mild	Green	< 15 %	No repairs expected
Moderate	Yellow	15 to 50 %	Minor repairs (patching on a small proportion of the deck)
Severe	Red	> 50 %	Major repairs (patching on a large proportion of the deck or resurfacing)

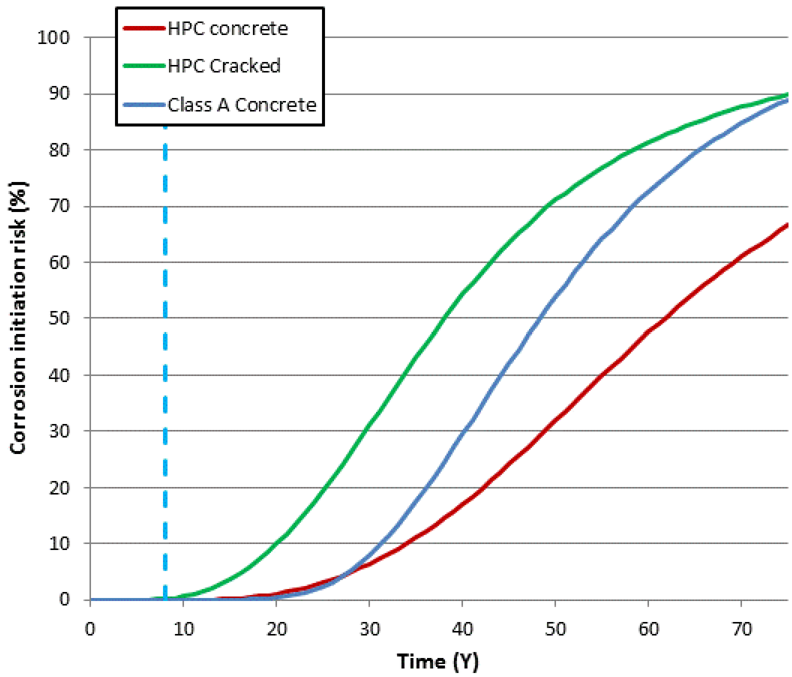

Figure 3 – Example of a degradation curve – Route 70

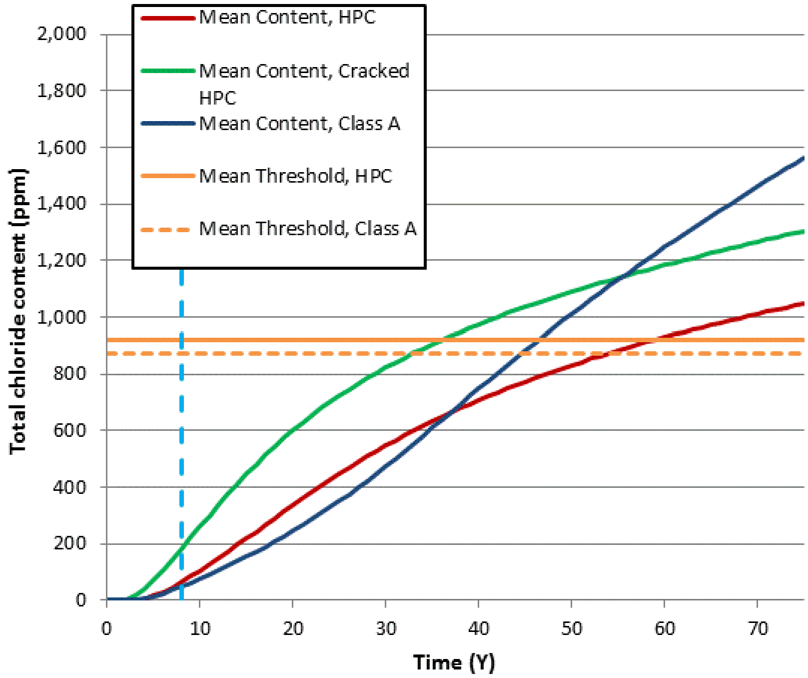

Figure 4 – Example of a chloride content evolution curve – Route 70

Table 17 – Summary of the calculations

Bridge	Condition	Corrosion risk at time intervals after construction			
		25 years	50 years	65 years	75 years
Route 70	HPC uncracked	3.1 %	31.9 %	54.7 %	66.6 %
	HPC cracked	19.5 %	71.1 %	84.9 %	89.8 %
	Class A	2.4 %	54.0 %	79.5 %	88.9 %
Smith Street	HPC uncracked	0.3 %	4.3 %	8.8 %	12.6 %
	HPC cracked	2.9 %	15.7 %	26.1 %	33.4 %
	Class A	0.0 %	0.6 %	2.7 %	5.1 %
Route 52 Elbow Channel	HPC uncracked	0.0 %	0.0 %	0.0 %	0.0 %
	HPC cracked	0.1 %	0.1 %	0.2 %	0.2 %
	Class A	0.0 %	0.0 %	0.1 %	0.4 %
Route 52 Rainbow Channel	HPC uncracked	0.0 %	0.7 %	2.1 %	3.3 %
	HPC cracked	0.5 %	5.4 %	10.6 %	15.1 %
	Class A	0.0 %	3.5 %	12.6 %	21.4 %
Creek Road	Class A	0.2 %	7.6 %	17.1 %	24.3 %
Route 9 northbound	HPC uncracked	0.0 %	0.0 %	0.0 %	0.1 %
	HPC cracked	0.0 %	0.4 %	1.0 %	1.7 %
	Class A	0.0 %	0.0 %	0.0 %	0.0 %
Route 9 southbound	Class A	0.4 %	7.0 %	13.5 %	18.3 %
Ocean City Longport Bridge	Class A	0.2 %	22.4 %	48.4 %	64.6 %
Collings Avenue	HPC uncracked	0.8 %	15.8 %	32.3 %	43.1 %
	HPC cracked	5.0 %	36.8 %	55.4 %	65.1 %
	Class A	9.6 %	75.3 %	93.3 %	97.6 %
Route 3	HPC uncracked	0.0 %	2.3 %	6.2 %	9.9 %
	HPC cracked	0.8 %	10.7 %	20.6 %	27.9 %
	Class A	0.0 %	1.8 %	6.4 %	10.9 %

The calculations indicate that in most cases, the corrosion initiation risk will remain in the green zone, i.e., below 15 %, and this, even for cracked HPC or Class A concrete. This is particularly true for the first 25 years after construction. During that period, only Route 70 Bridge is expected to exhibit a moderate corrosion risk. Between 25 and 50 years, the risk will extend to three other bridges (Smith St, OCLP and Collings Av.). Over 65 years, the risk will extend to the Creek Rd and Route 3 bridges and at 75 years, it will also affect the Route 9 southbound and Rainbow Channel bridges. The only bridge for which deck repairs due to corrosion damage are not expected in the next 75 years are the Elbow Channel and Route 9 northbound bridges.

The calculations also show that, as could be expected, HPC decks are generally the most durable option, as long as they remain uncracked. However, Class A concrete also provides good durability performance for the investigated decks, especially in the first decades after construction. Indeed, in all cases but one, Class A concrete compares or is better than HPC over the first 25 years. This is in a large part explained by a higher chloride binding capacity of Class A mixtures (due to the different binder composition), which contributes, for a limited period of time, to keep the chloride content lower at the reinforcement level. However, on a long-term basis, HPCs are better than Class A concretes because of their lower diffusion coefficients.

Based on the calculation parameters used, Class A concrete performs better than *cracked* HPC over an horizon of 50 years after construction. At 65 or 75 years, the advantage is not as clear but the performance of Class A concrete still compares to *cracked* HPC for many of the investigated decks.

It is interesting to note that major repairs are not expected before 50 to 65 years in uncracked HPC decks. If decks were made of Class A mixtures, major repairs would have been expected before 50 years for only two of the investigated bridge decks.

One of the most striking conclusions that could be drawn from the calculations is that they allow to highlight the combined influence of concrete properties, exposure conditions and concrete cover to cost-effectively design durable structures. This type of calculations may therefore be used to evaluate the benefit of using HPC as a function of the salt loading on a bridge or the actual cover depth. Furthermore, the service-life calculations can also be used, in combination with destructive or non-destructive testing, to prioritize maintenance interventions on bridge decks by identifying those that are likely to deteriorate faster.

Appendix D – Supplemental data from strain measurement activity on Route 3 and Collings Avenue

Concrete Material Property Summary

Mix Design

The two mixes were designed and approved to meet NJDOT specifications. Table 9 provides a comparison of the mix designs.

Table 19: Mix Design Comparison

	Route 3 (% Total)	Collings Ave (% Total)
Type 1 Cement (lb)	570 (15%)	353 (9%)
Slag Cement (lb)		247 (6%)
Class F Flyash (lb)	130 (3%)	106 (3%)
Silica Fume (lb)	25 (1%)	
Fine Aggregate (lb)	1083 (28%)	1208 (32%)
Coarse Aggregate (lb)	1773 (46%)	1625 (43%)
Water (lb)	292 (7%)	263 (7%)
w/c ratio	0.4	0.37
% Air	6	6
Slump (in)	6 +/- 2	6 +/- 2
Admixtures	High range water reducing admixture, Retarder, Air Entrainment, Microsilica	High and normal range water reducing admixtures, Accelerator, Retarder, Air entrainment
Total Cementitious Material (lb)	725	750
Total weight per cubic yard (lb)	3873	3801
NJDOT Design f'c (psi)	5400	5400
Mix f'c(psi)	6370	Not specified
Average Measured f'c(psi)	7020	6425

The two mix designs are of similar proportion with respect to cement versus aggregate content. The total cement content of the mixes is similar however the makeup of the cementitious material between the two mixes differs. The Route 3 Bridge utilizes Type 1 Portland cement, flyash, and microsilica while Collings Ave uses Type 1 Portland cement, slag cement and flyash. In general, concrete using slag cement as a replacement for Portland cement will have a lower permeability than concrete made with only class F flyash as a replacement. Slag cement will generally decrease the time it takes for the concrete to set. Class F fly ash tends to reduce the heat of hydration as the concrete cures and also reduces the early age strength of the concrete. Silica fume was used in the Route 3 mix and will generally improve the compressive strength and reduce the permeability of the concrete

Drying Shrinkage of Concrete

The details of the drying shrinkage tests performed by SIMCO are given in a separate report. Experimental determination of restrained shrinkage values requires implementing ASTM PP34-98 procedures to physically measure the restrained shrinkage of a particular concrete mix. To identify unrestrained shrinkage values for a particular mix design, ASTM 157-98 (AASHTO T160) procedures are used. In the case of both decks presented in this report, the AASHTO T160 tests were performed to estimate the free shrinkage strain of the concrete. ASTM restrained shrinkage tests were not performed for the bridges presented in this study. The measured free shrinkage curves are shown in

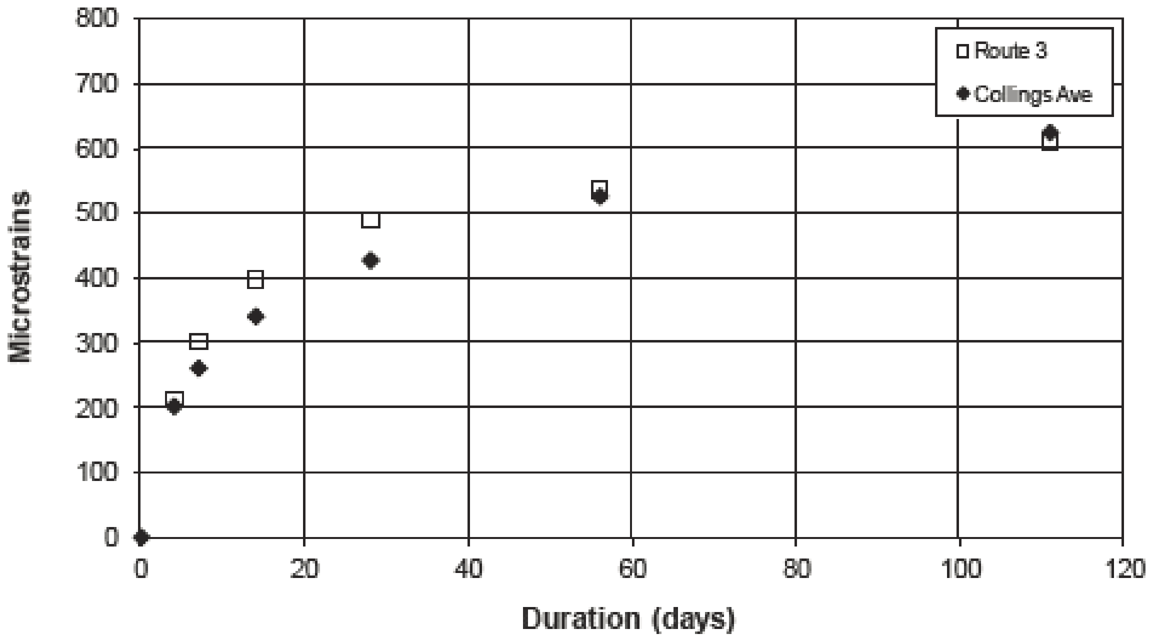

Figure 35: Measured Free Shrinkage Strains - Courtesy of SIMCO

Coefficient of Thermal Expansion

For the two decks monitored in this study, the coefficient of thermal expansion of concrete was measured in the laboratory for each concrete deck. The thermal expansion coefficients of the two concrete mixes were provided by SIMCO and measured using USACE CRD-C 39-91 – Test Method for Coefficient of Linear Thermal Expansion of Concrete. Typical values of the coefficient of thermal expansion of concrete are (4.1-7.3 x10-6/°F) according to FHWA. These values were used to correct for the differential expansion and contraction of the concrete versus the steel gage. For the Route 3 Bridge the average coefficient of thermal expansion was 9×10^{-6} mE/°F and for the Collings Ave Bridge the average coefficient of thermal expansion was 6.8×10^{-6}/°F.

Curing Process

The two new HPC decks were cured using similar operations. Immediately following the finishing of the concrete the freshly machined concrete was topped with wet burlap. The wet burlap was subsequently covered with plastic sheeting weighted down to protect it against the wind. At the Route 3 structure the wet burlap was placed approximately one hour following the placement of the concrete, while at Collings Ave the wet burlap was applied at 9am approximately 4 hours after the pour began. Wet curing was applied by placing water hoses at varying positions along the bridge deck that provided a constant supply of water to the concrete deck. The wet curing process continued for a minimum of 14 days in accordance with NJDOT specifications.

Fresh concrete samples were taken from the concrete arriving in the ready mix trucks as well as the concrete being pumped onto the bridge deck. At RT3, twelve 4” x 8” cylinders and two 3” x 11.75” prisms were produced. At Collings Ave eighteen 4” x 8” cylinders and three 3” x 11.75” prisms were produced by SIMCO staff. The fresh concrete samples were transferred to the nearby laboratory facilities for moist curing in accordance with ASTM C31 provisions. Following 14 days of moist curing, the concrete samples were covered in moist cleaning tissue and sealed in plastic for shipping to SIMCO laboratories. After arrival at the SIMCO laboratories, the samples were moist cured for the remainder of the curing period.

Route 3 over NJ Transit Monitoring Results

Bridge Description

The RT3 Bridge was constructed using a staged approach to replace an existing structure over NJ Transit rail lines in Rutherford, NJ. The geographic location of the bridge is shown in Figure 36. The structure was built in three stages and tied together with closure pours. The second construction stage of the RT3 Bridge was selected by NJDOT for instrumentation to provide quantitative information regarding the strains and temperatures within the concrete deck. This bridge is composed of a 178 ft. simple span and carries NJ Route 3 over NJ Transit rail lines near Rutherford, NJ. The bridge is characterized by a large camber, vertical curve, and skew angle due to the approach profiles and the required clearance over the rail lines. The total dead load camber is on average 18 inches, while the additional camber to account for the vertical curve is 4 inches for a total of 22 inches of camber. The skew angle of the structure is 52 degrees. The deck is supported on 54 inch deep plate girders with two splice joints. The girders were constructed using both 50 ksi and 70 ksi high performance weathering steel since the required clearance limited girder depth and resulted in the need for higher strength steel. The girders are supported on multi-rotational pot bearings at each end that allow different translations and rotations depending upon which girder they support.

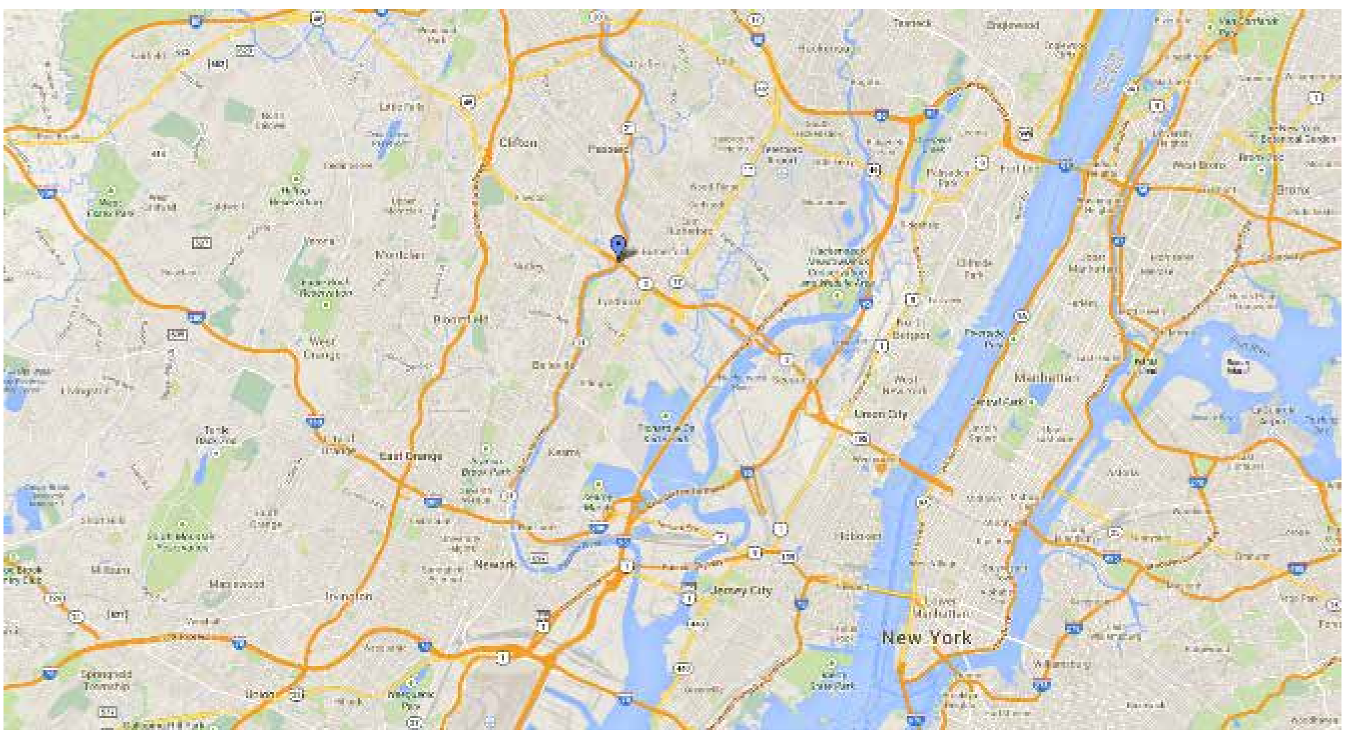

Figure 36: Geographic Location of RT3 Bridge over NJ Transit

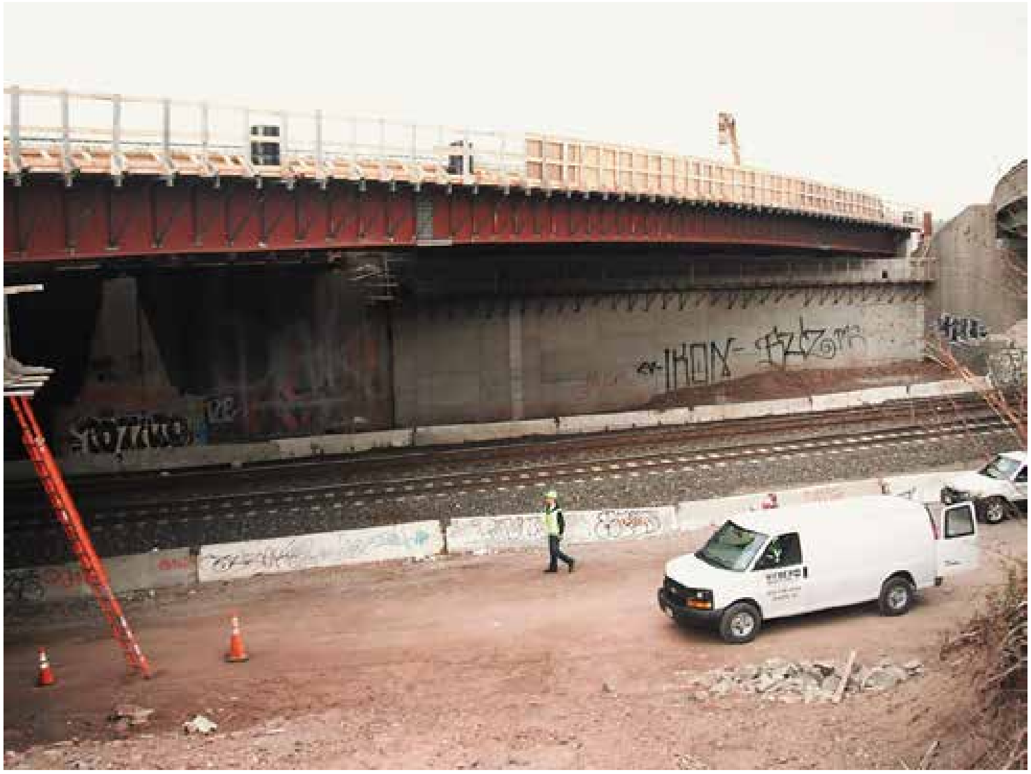

Figure 37: Route 3 over NJ Transit

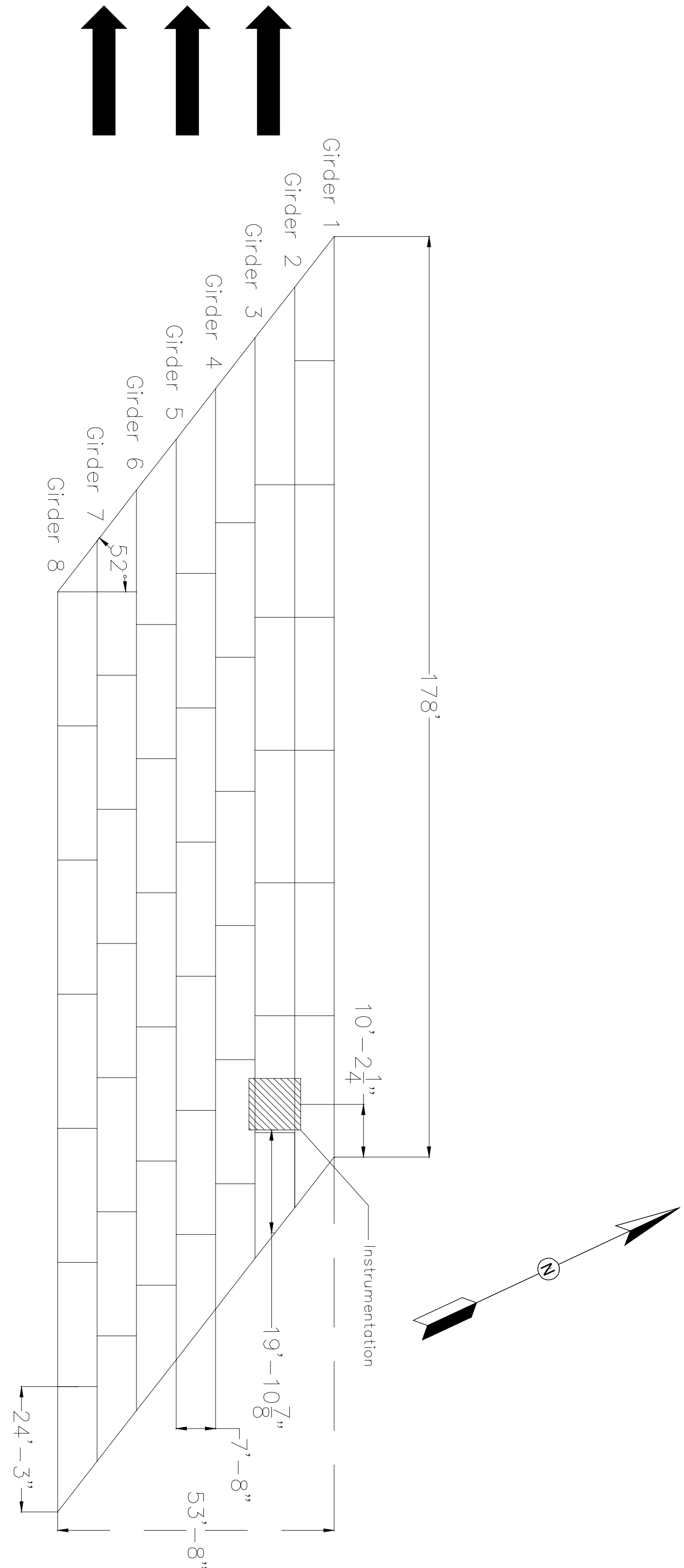

Figure 38: Route 3 over NJ Transit – Plan View

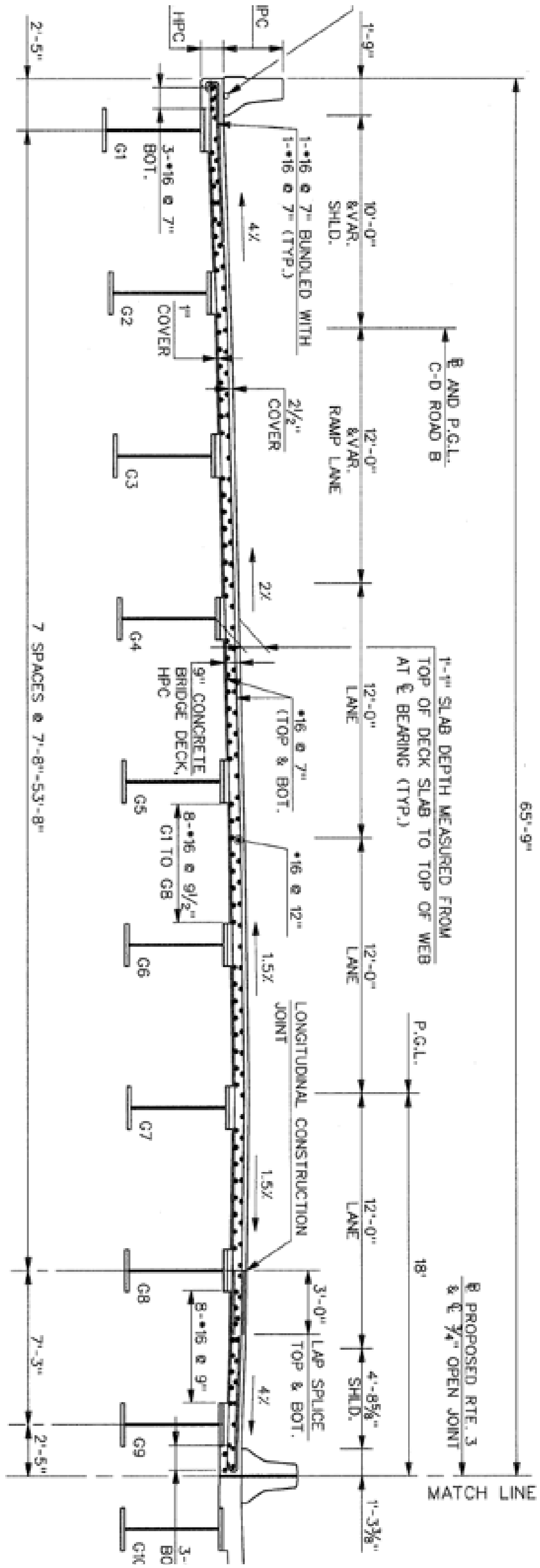

Figure 39: Route 3 over NJ Transit – Cross Section

Mix Design

The design mix specified for the concrete used in the bridge deck is given in Table 20

Table 20: Route 3 Concrete Deck Design Mix Specifications

	Route 3 (% Total)
Type 1 Cement (lb)	570 (15%)
Slag Cement (lb)	
Class F Flyash (lb)	130 (3%)
Silica Fume (lb)	25 (1%)
Fine Aggregate (lb)	1083 (28%)
Coarse Aggregate (lb)	1773 (46%)
Water (lb)	292 (7%)
w/c ratio	0.4
% Air	6
Slump (in)	6 +/- 2
Admixtures	High range water reducing admixture, Retarder, Air Entrainment, Microsilica
Total Cementitious Material (lb)	725
Total weight per cubic yard (lb)	3873
NJDOT Design f'c (psi)	5400
Mix f'c(psi)	6370
Average Measured f'c(psi)	7020

Instrumentation

The instrumentation chosen for the monitoring of concrete strains during curing consisted of model 4200 and model 4204 vibrating wire (VW) embedment strain gages. The VW gages were chosen due to their long term stability and ruggedness. The VW gage uses a tensioned wire between two blocks whose tension (and natural frequency) changes due to applied loads. This change in frequency can be related to the strain using mechanics. Each deck was instrumented using twelve vibrating VW embedment strain gages which were offset from the rebar cage using styrofoam before the concrete pour. Embedment gages were mounted on both top and bottom rebar in the longitudinal and lateral directions. Gages were also mounted at locations where the restraint is theorized to vary, including over the steel girders and between girders. The gages located between the girders represent a location of assumed lower restraint, while gages located directly over the beams represent a location with an assumed higher restraint. The mounting of gages to the rebar cage is shown in Figure 40.

Figure 40: Vibrating Wire Embedment Gage Installation at Route 3 over NJ Transit

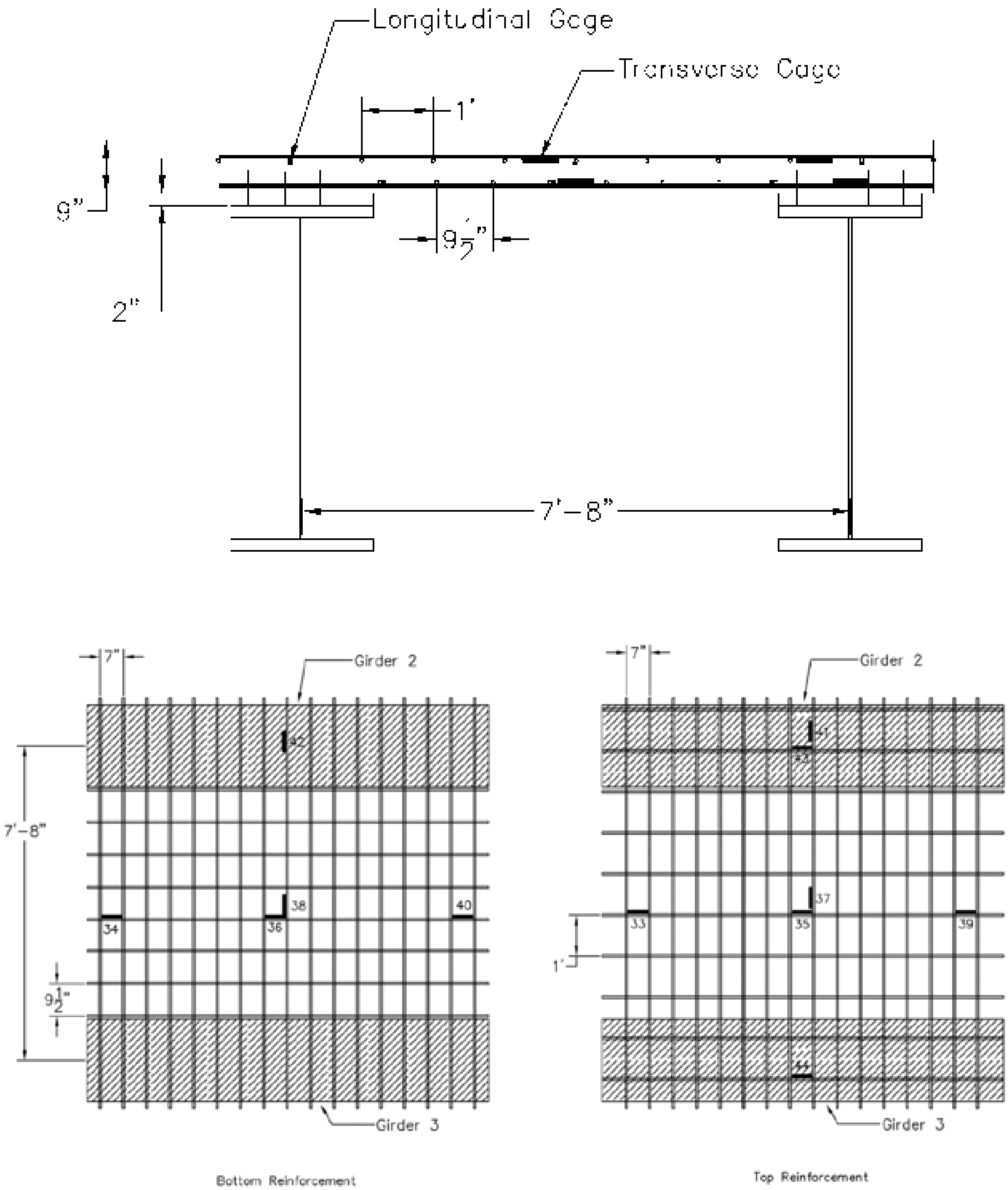

Figure 41: Route 3 over NJ Transit Instrumentation

Description of Monitoring

The Route 3 deck pour began at approximately 8:30 am on April 20th, 2013. Average ambient temperatures varied between 50 to 60 °F and the weather was mostly cloudy with few periods of sun and strong winds from the northwest. The day was dry with no precipitation. Over a period of 166 days following the casting of the concrete deck at the Route 3 Bridge, strains and temperatures were measured. During the pour, the strains and temperatures were interrogated at 2 minute intervals during the 5 hour pour which was relaxed to 15 minute intervals following the completion of concrete placement and continued at this interval for the remainder of the monitoring period. A timeline of the monitoring is provided in Figure 42.

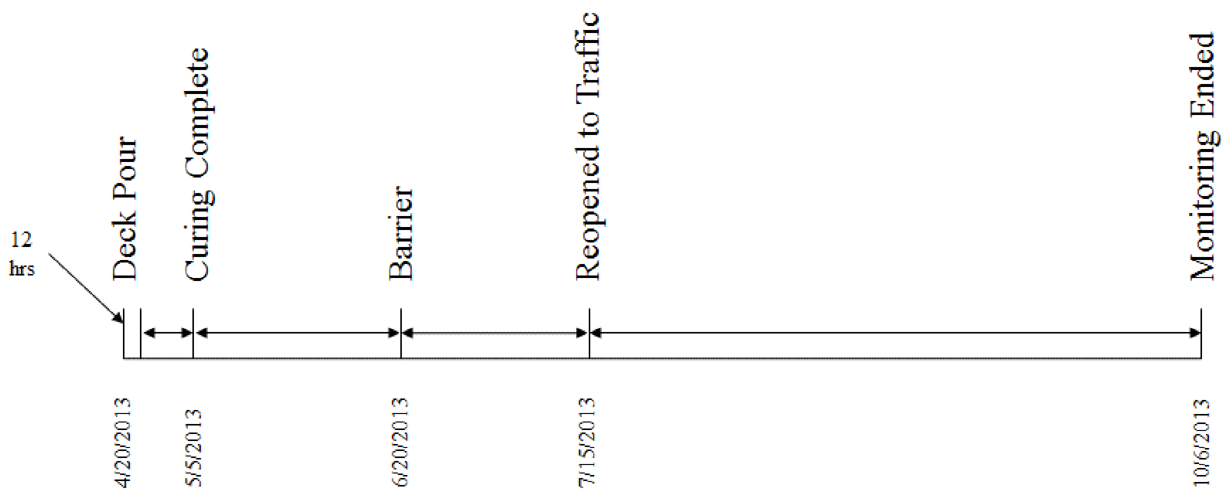

Figure 42: Monitoring Timeline – Route 3 Over NJ Transit

To properly interpret the strain readings, a reference strain is required to provide a baseline for referencing the strains measured following deck casting. The reference point was chosen as the strain/temperature that occurred approximately 1 hour after the concrete was finished over the gage locations. This point is marked in Figure 44. This strain/temperature corresponds to the point in time where the concrete begins to expand due to the heat produced by the hydration process. An overall plot of the deck temperatures versus time are shown Figure 43.

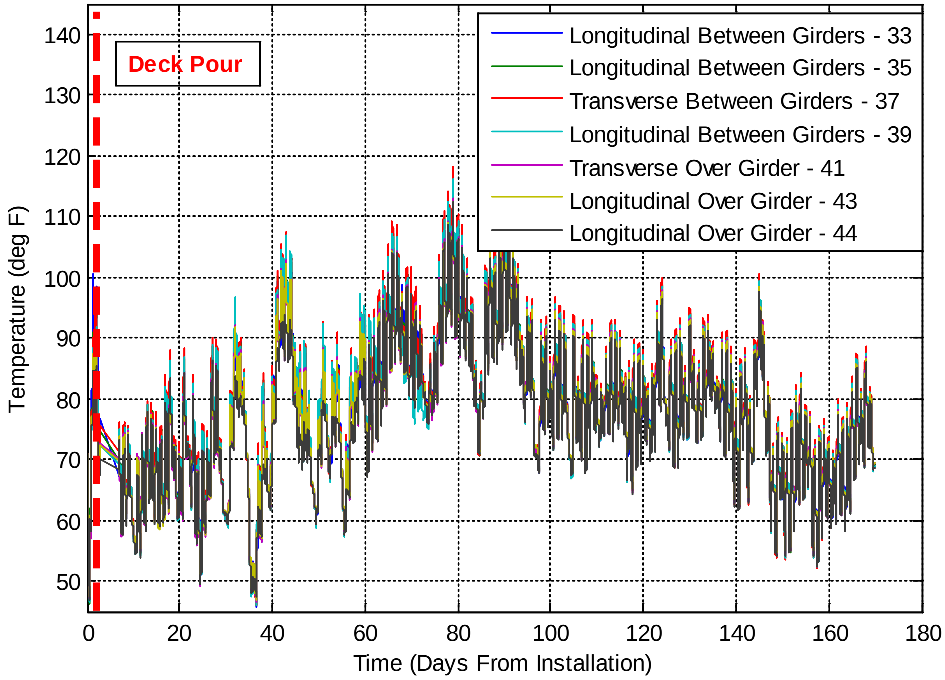

Figure 43: Measured Deck Temperatures – Route 3 over NJ Transit

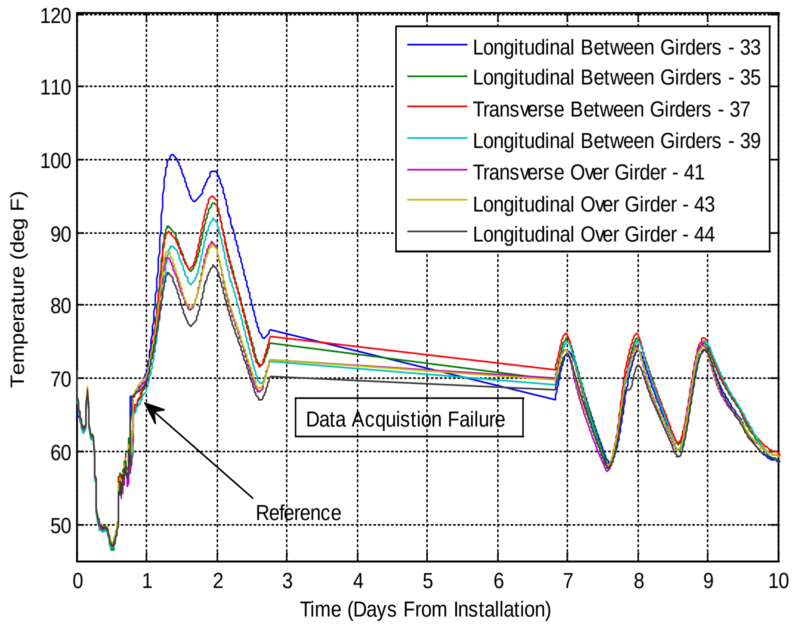

Figure 44: Peak Temperature after Deck Pour – Route 3 over NJ Transit – Top Reinforcement

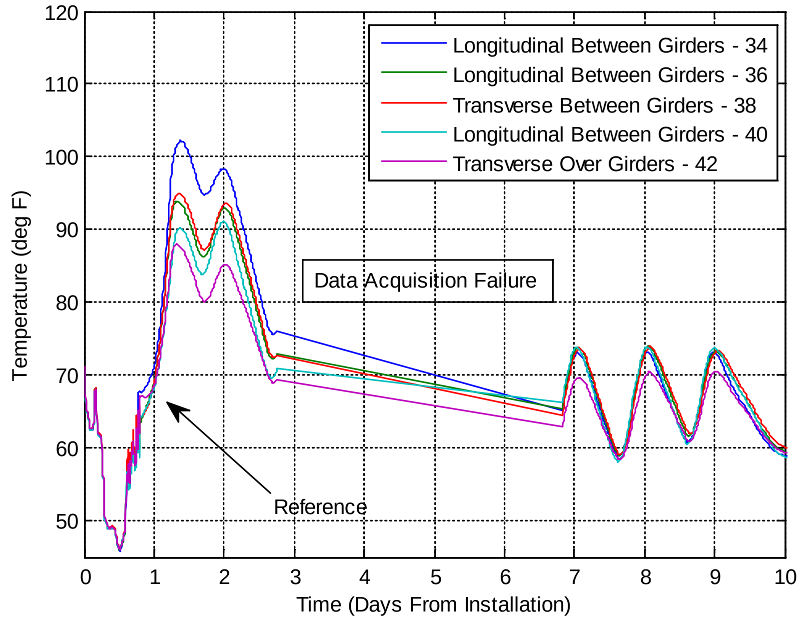

Figure 45: Peak Temperature after Deck Pour – Route 3 over NJ Transit – Bottom Reinforcement

The peak temperature recorded during initial curing of Route 3 bridge deck occurred approximately 14 hours after the concrete was placed over the gages. The peak temperatures due to the hydration process ranged from 85 °F to 102 °F depending on the gage location. During the subsequent 166 day monitoring period, the temperatures within the concrete exceeded the temperature associated with heat of hydration on several occasions. For the Route 3 Bridge, the gages located over the girders have lower peak temperatures during initial curing than the gages located between the girders. This is due to the steel girders being at a lower temperature and cooling the concrete at these locations. The gages between the girders are located over stay in place forms which do not have as much thermal mass and therefore allow the concrete to reach higher temperatures.

Measured and Theoretical Strains

The deck pour began at approximately 8:00 am on April 20, 2013. Average ambient temperatures were recorded onsite and varied between 50 and 60 °F. The weather during the deck pour was mostly cloudy with few periods of sun and strong winds from the northwest. No precipitation occurred during the day. As curing commenced the concrete expanded due to hydration products placing the deck into tension. This tensile strain is quickly removed from the deck by the shrinkage of the concrete as it cures over time. Figure 50 shows the total free strain at the top mat of reinforcement. The plots combine the measured free strain obtained using the AASHTO T160 test plus the theoretical thermal strain using the measured thermal expansion coefficient of concrete obtained from the lab studies performed by SIMCO.

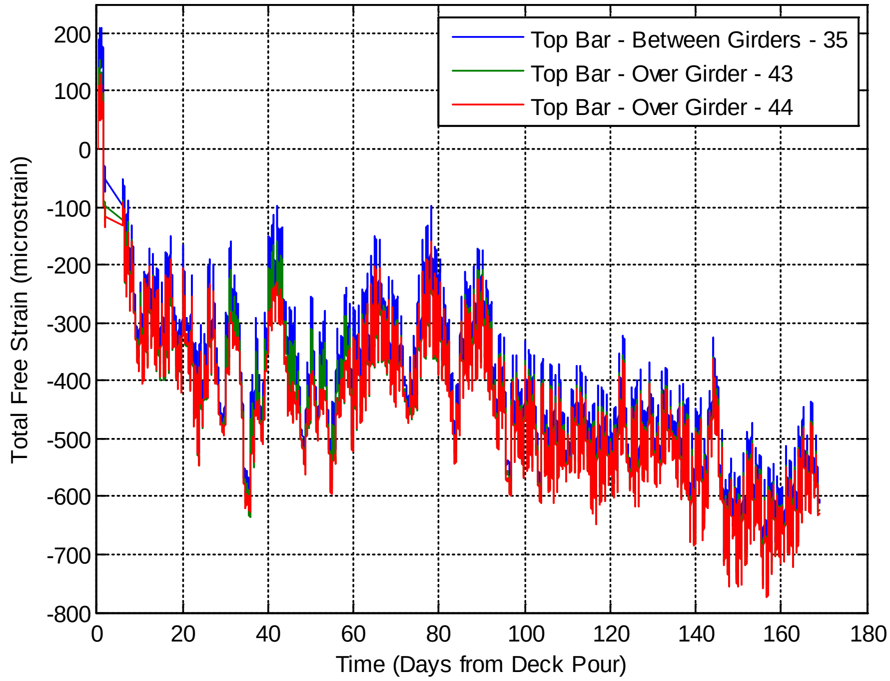

Figure 46: Route 3 - Total Free Strain at Top Reinforcement – Longitudinal

Figure 46 shows the theoretical unrestrained strain varies from a tensile strain of 200 microstrain in the days following the concrete pour to a compressive strain of almost 750 microstrain after 166 days. The general trend follows the free shrinkage curves provided by SIMCO. If a portion of this strain is restrained by internal or external restraints, the concrete in the deck is susceptible to cracking. These plots represent the total shrinkage potential due to the shrinkage of concrete and also contraction due to thermal loads. Converting these theoretical values would result in the 178' concrete deck experiencing a range of movement of 2.1 inches if the deck was allowed to freely expand and contract due to shrinkage and temperature. The free thermal strains without the theoretical free shrinkage superimposed are shown in Figure 47.

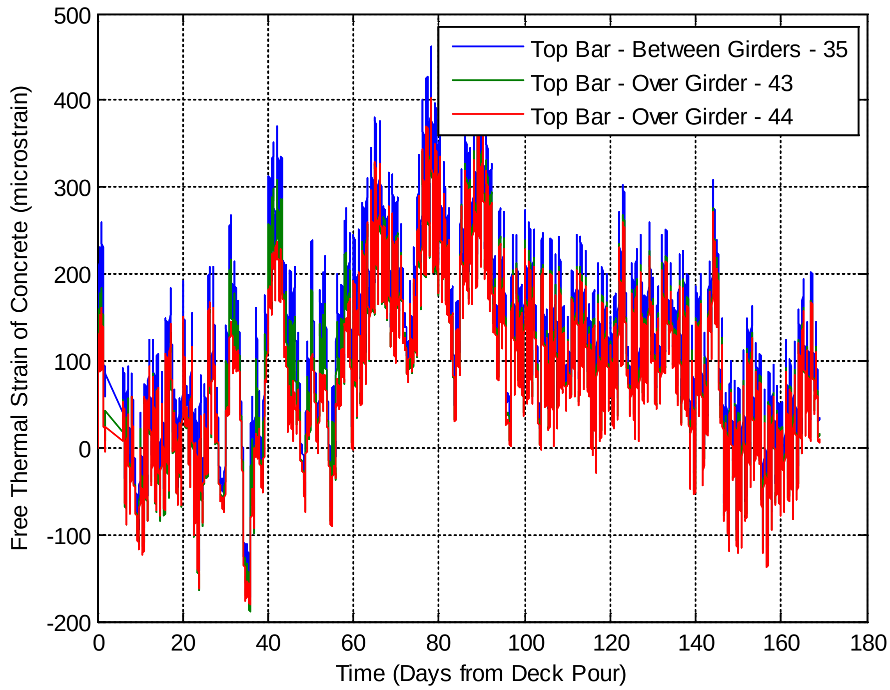

Figure 47: Route 3 - Free Thermal Strain at Top Reinforcement – Longitudinal

The theoretical unrestrained strain readings in concrete due to temperature changes follow the trend of the temperature changes as shown in Figure 43. Since the concrete was poured at a low ambient temperature, it is noted that initially the concrete experiences an expansion due to heat of hydration followed by a period of cooling where the deck contracts. Once the deck stabilizes with the ambient air temperature it begins to follow the ambient temperature fluctuations and goes through daily cycles of expansion and contraction along with longer term expansion and contraction cycles related to seasonal changes in temperature. The unrestrained strains due to thermal loading generally vary from a tensile strain of 500 microstrain to a compressive strain of 200 microstrain. Converting these theoretical values would result in the 178’ concrete deck experiencing approximately 1.5 inches of total movement if the deck was allowed to freely expand and contract due to temperature changes.

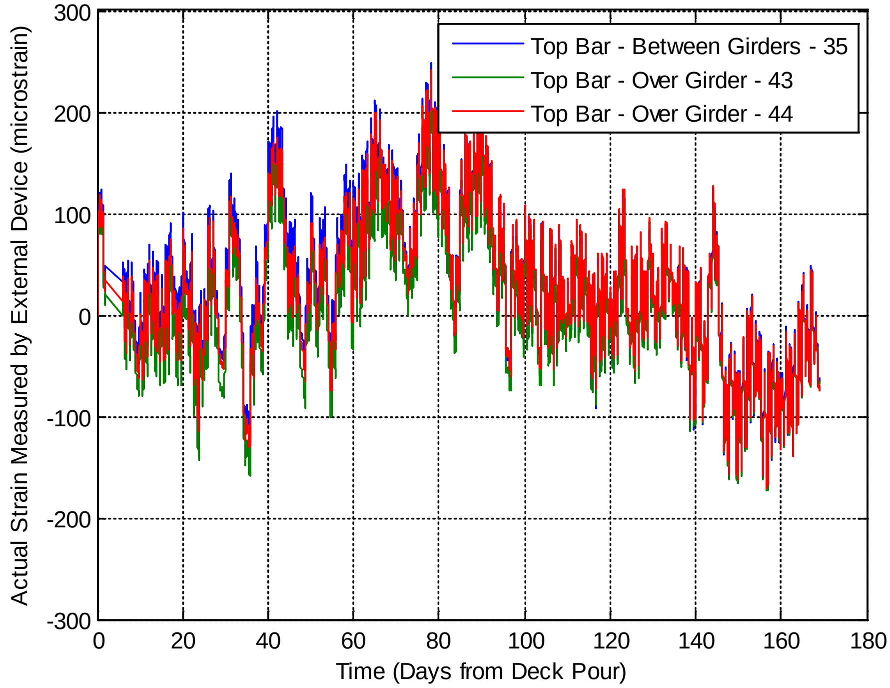

Figure 48: Route 3 - Strain That Would Be Measured by an External Device – Longitudinal Top Reinforcement

The plots shown in Figure 48 are the strains that would be measured on the concrete by an external gage that is the actual change in length of the concrete slab. These gages follow the measured temperature curves and the strains that would be measured by an external device indicate a range of strains from a tensile strain of 300 microstrain to a compressive strain of almost 200 microstrain. These strains follow the same trend as the temperature curves presented earlier.

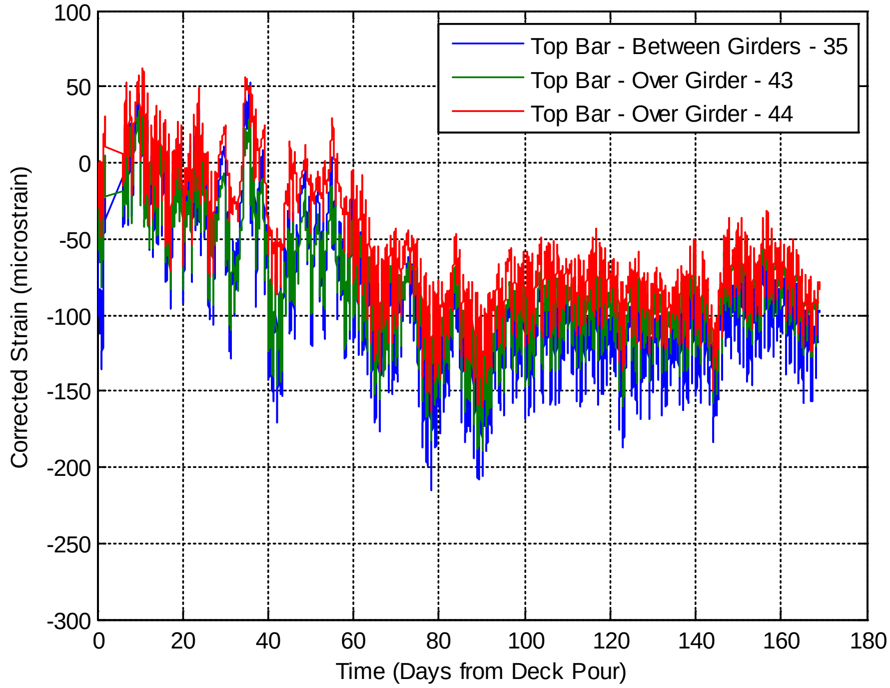

Figure 49: Route 3 - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Top Reinforcement

The corrected strains, shown in Figure 49, measured by the embedded strain gages indicate a buildup of compressive strain in the concrete as the temperatures increase during the hours immediately after the completion of the concrete pour. As the deck cools and the deck contracts and shrinkage begins to occur the strain becomes tensile. After approximately 10 days, the ambient temperatures begin to increase each day which results in a buildup of compressive strain in the concrete balancing the tensile strains from the initial deck cooling and shrinkage. This trend continues for approximately 3 months where the compressive strain buildup appears to subside and begins a buildup of tensile strain due to a cooling trend in ambient temperatures and the concrete. Since the gage is not restrained temperature affects the vibrating by causing the wire to slacken under positive temperature changes and tighten due to negative temperature changes. These phenomena are partially counteracted by the expansion of concrete due to a positive temperature change and contraction due to negative temperature change. The corrected strains take into account the laboratory measured thermal expansion of concrete for this particular concrete mix. The remainder of the data plots are presented in the following section.

Other Data Plots

Total Unrestrained Strain

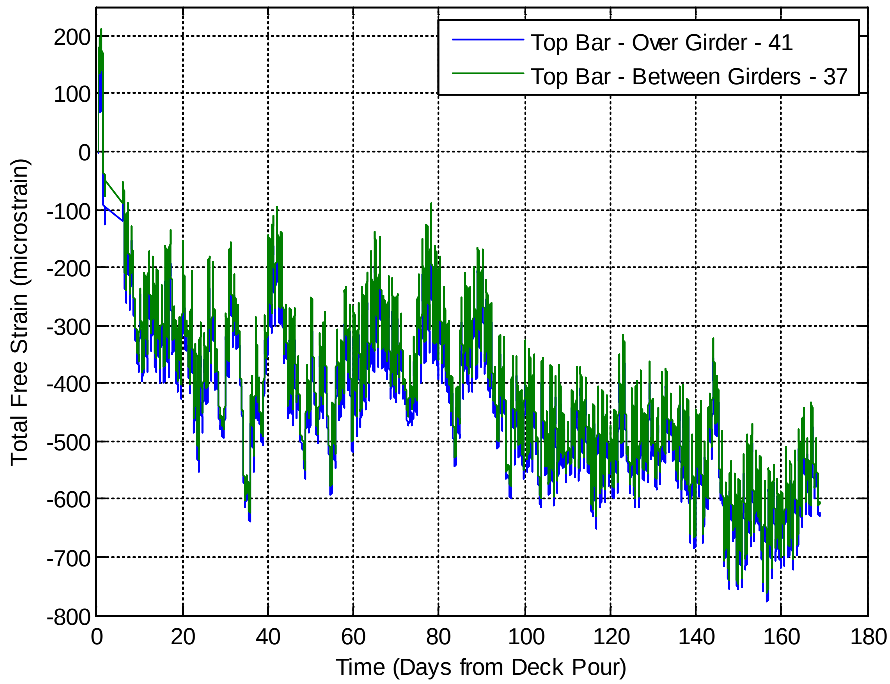

Figure 50: Route 3 - Total Free Strain at Top Reinforcement – Transverse

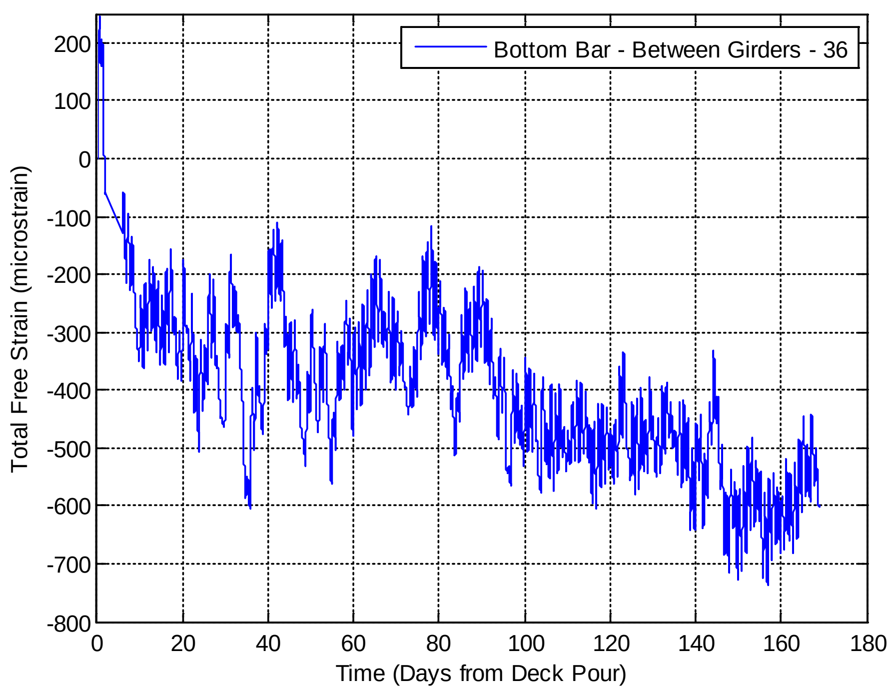

Figure 51: Route 3 - Total Free Strain at Bottom Reinforcement – Longitudinal

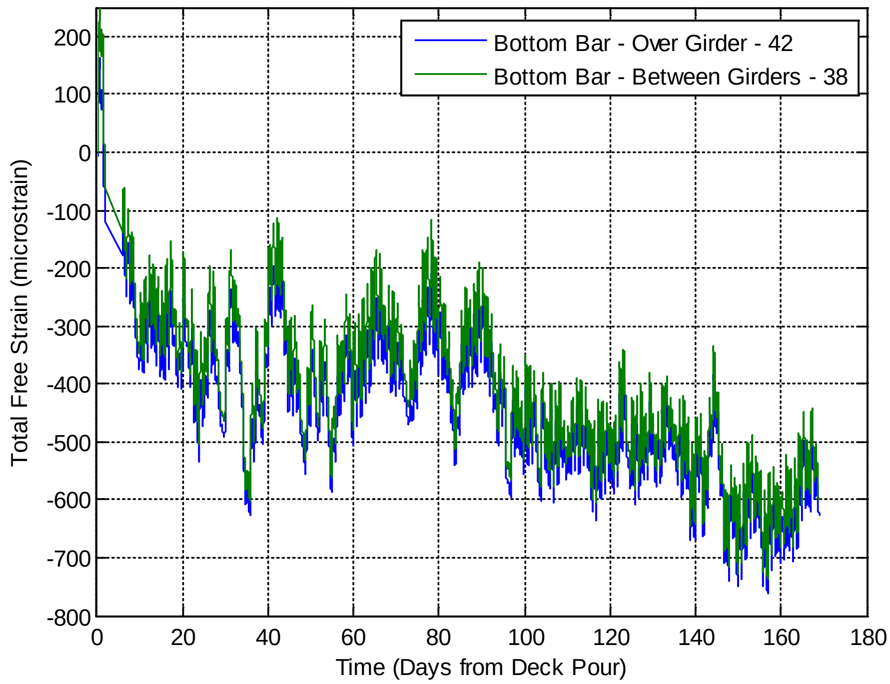

Figure 52: Route 3 - Total Free Strain at Bottom Reinforcement – Transverse

Unrestrained Thermal Strain

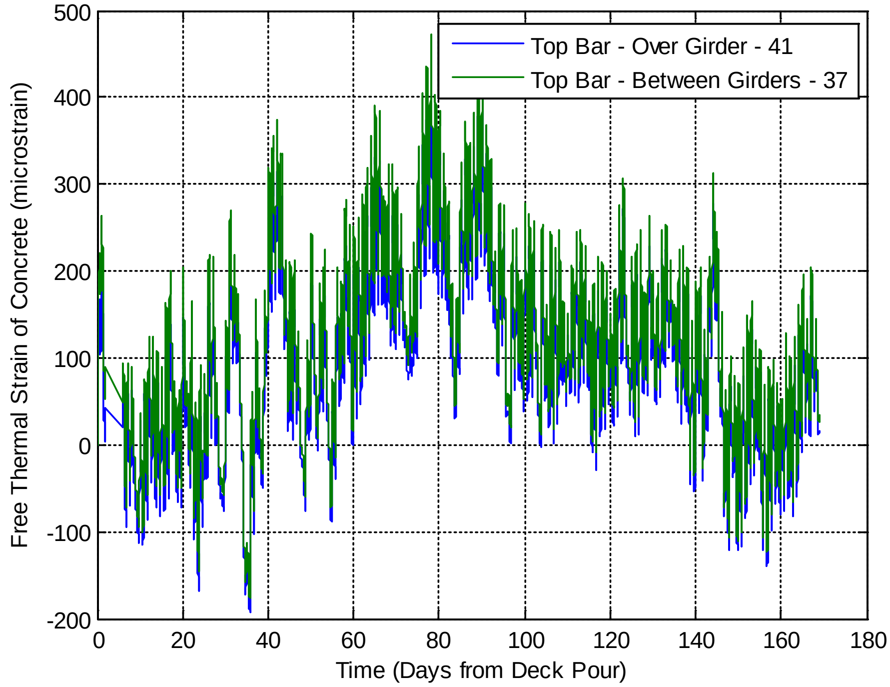

Figure 53: Route 3 - Free Thermal Strain at Top Reinforcement – Transverse

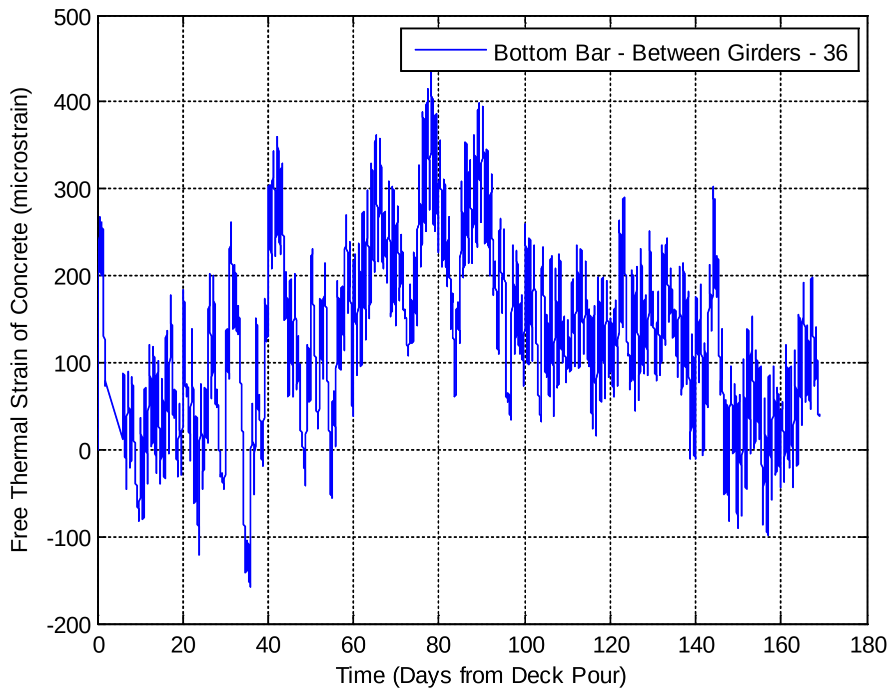

Figure 54: Route 3 - Free Thermal Strain at Bottom Reinforcement – Longitudinal

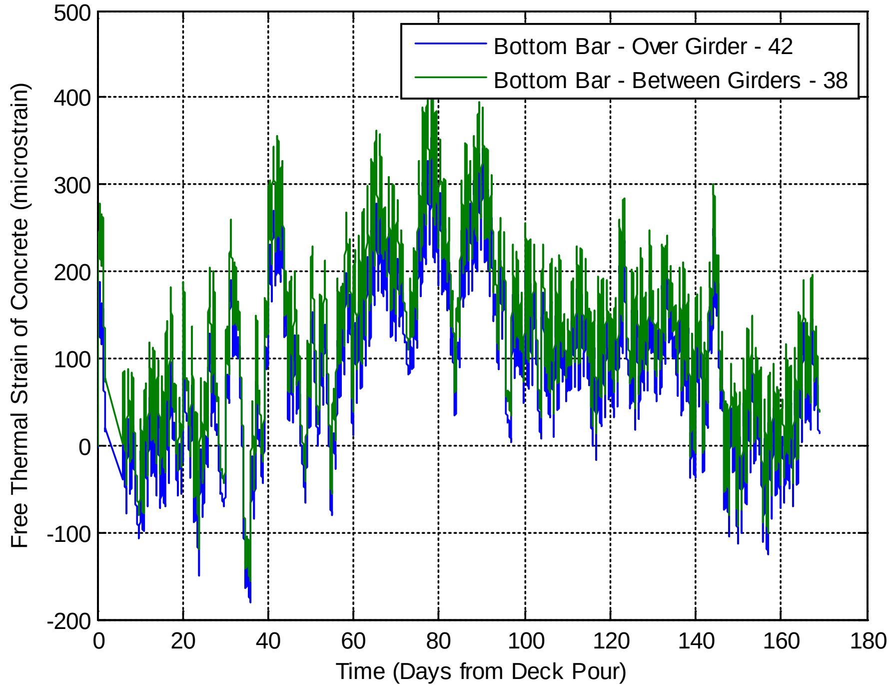

Figure 55: Route 3 - Free Thermal Strain at Bottom Reinforcement – Transverse

Strain that would be measured by an External Device

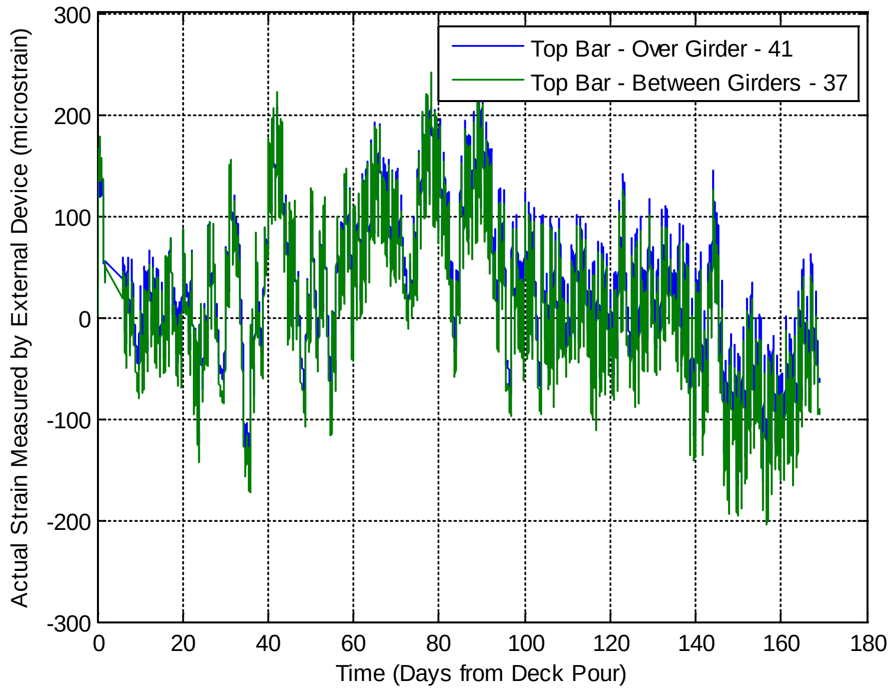

Figure 56: Route 3 - Strain That Would Be Measured by an External Device – Transverse Top Reinforcement

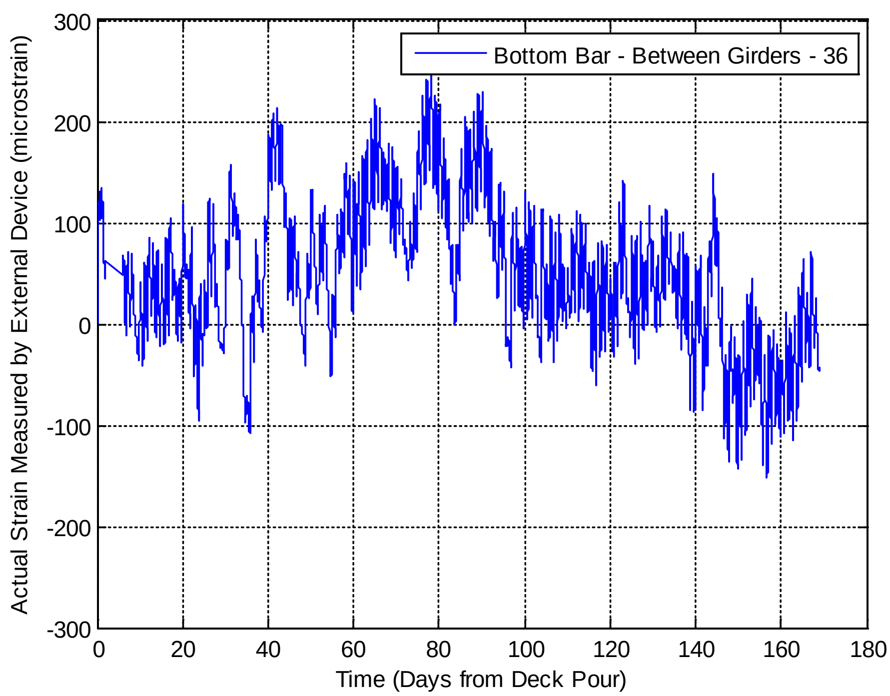

Figure 57: Route 3 - Strain That Would Be Measured by an External Device – Longitudinal Bottom Reinforcement

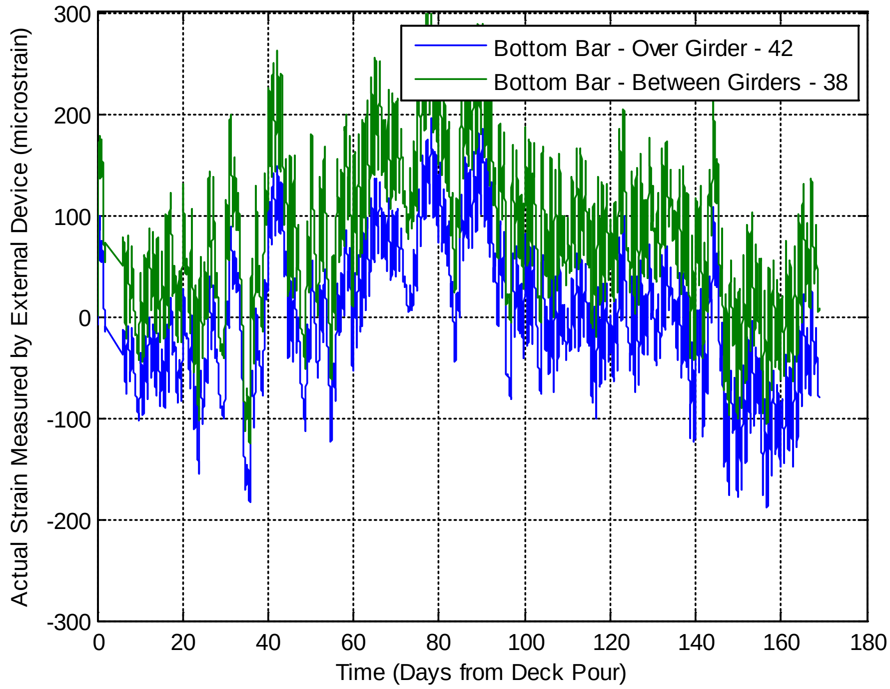

Figure 58: Route 3 - Strain That Would Be Measured by an External Device – Transverse Bottom Reinforcement

Corrected Strain

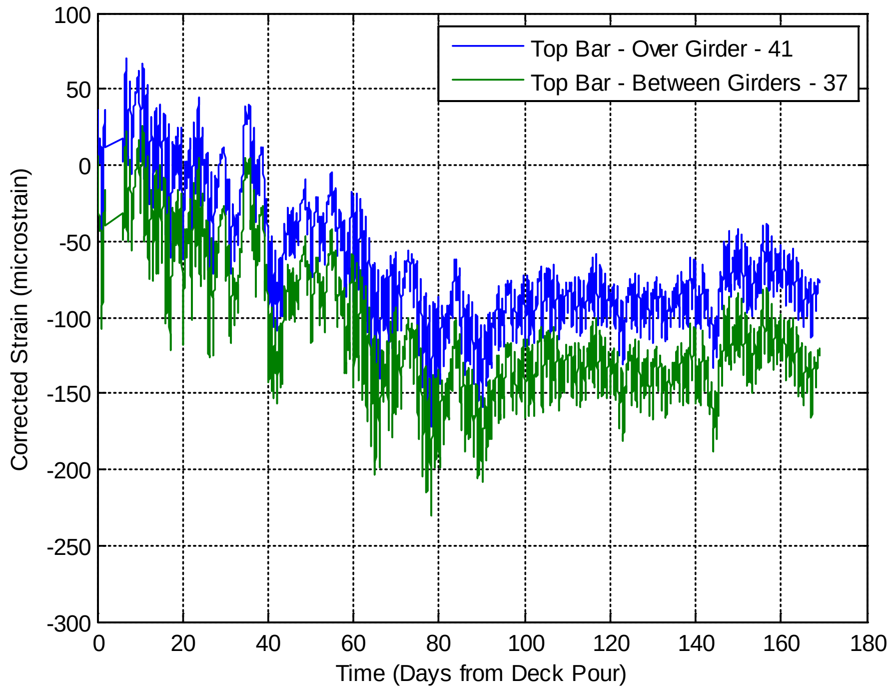

Figure 59: Route 3 - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Transverse Top Reinforcement

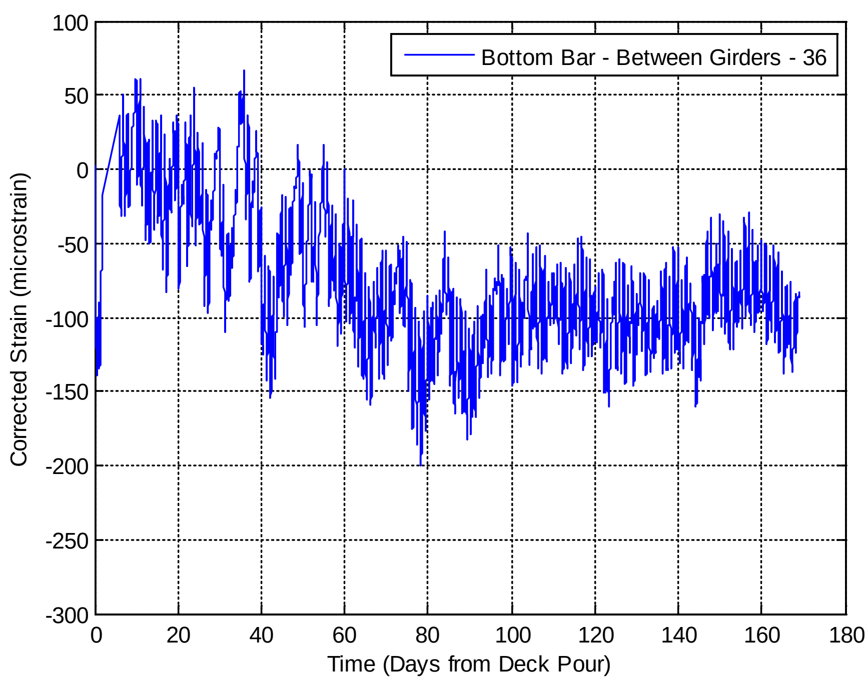

Figure 60: Route 3 - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Bottom Reinforcement

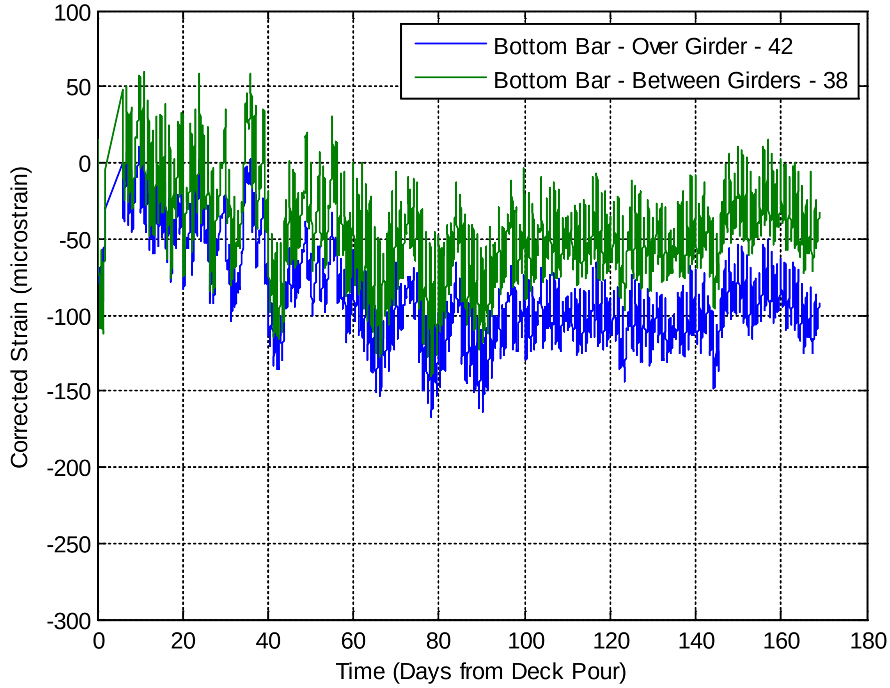

Figure 61: Route 3 - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Transverse Bottom Reinforcement

Collings Avenue Bridge Monitoring Results

Bridge Description

The Collings Ave Bridge is located near Gloucester City, NJ and the concrete deck was replaced as part of a rehabilitation project. The geographic location of the bridge is shown in Figure 62. This bridge is a three span structure with a total length of 160 feet. The skew angle of the structure is 37 degrees. The deck is supported on riveted built up girders. The girders are supported on rocker bearings that allow unidirectional translation and rotation depending on if the bearing is classified as free or fixed. An end view of the bridge is shown in Figure 63.

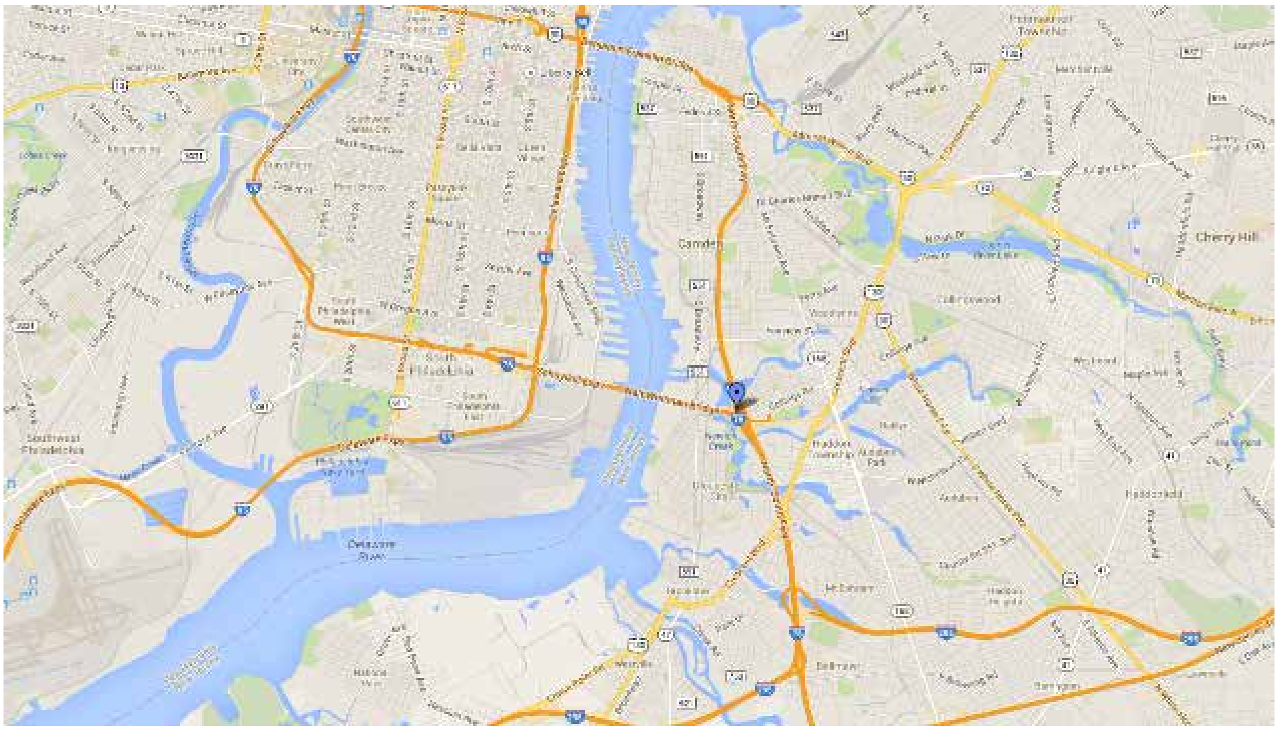

Figure 62: Geographic Location of Collings Ave Bridge

Figure 63: Collings Ave over I-676 – View from East

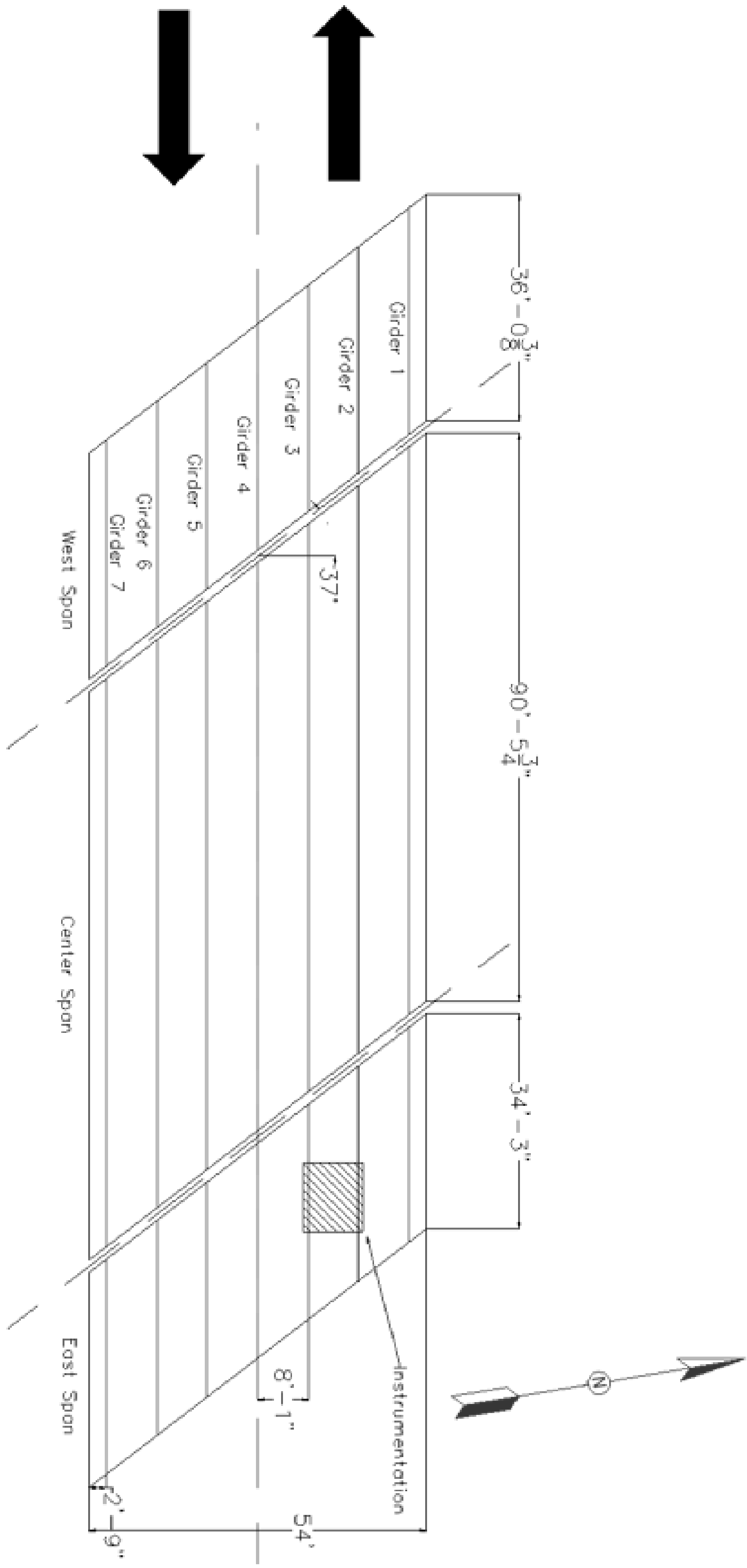

Figure 64: Collings Ave over I-676 – Plan View

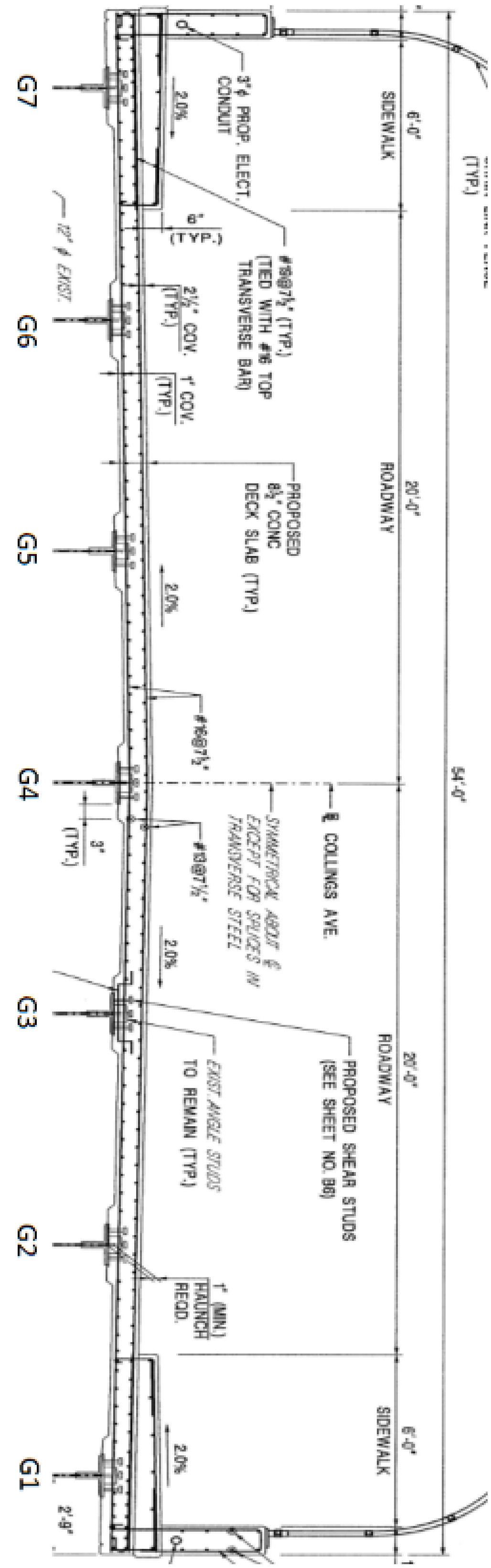

Figure 65: Collings Ave over I-676 – Section View

Mix Design

The mix design specified for the Collings Ave Bridge is detailed in Table 21.

Table 21: Collings Ave Concrete Deck Design Mix Specifications

	Collings Ave (% Total)
Type 1 Cement (lb)	353 (9%)
Slag Cement (lb)	247 (6%)
Class F Flyash (lb)	106 (3%)
Silica Fume (lb)	
Fine Aggregate (lb)	1208 (32%)
Coarse Aggregate (lb)	1625 (43%)
Water (lb)	263 (7%)
w/c ratio	0.37
% Air	6
Slump (in)	6 +/- 2
Admixtures	High and normal range water reducing admixtures, Accelerator, Retarder, Air entrainment
Total Cementitious Material (lb)	750
Total weight per cubic yard (lb)	3801
NJDOT Design f'c (psi)	5400
Mix f'c(psi)	Not specified
Average Measured f'c(psi)	6425

Instrumentation

The instrumentation chosen for the monitoring of concrete strains during curing consisted of model 4200 and model 4204 vibrating wire (VW) embedment strain gages. The VW gages were chosen due to their long term stability and ruggedness. The VW gage uses a tensioned wire between two blocks whose tension (and natural frequency) changes due to applied loads. This change in frequency can be related to the strain using mechanics. Each deck was instrumented using twelve vibrating VW embedment strain gages which were offset from the rebar cage using styrofoam before the concrete pour. Embedment gages were mounted on both top and bottom rebar in the longitudinal and lateral directions. Gages were also mounted at locations where the restraint is theorized to vary, including over the steel girders and between girders. The gages located between the girders represent a location of assumed lower restraint, while gages located directly over the beams represent a location with an assumed higher restraint. The mounting of gages to the rebar cage is shown in Figure 66.

Figure 66: Vibrating Wire Embedment Gage Installation at Collings Ave

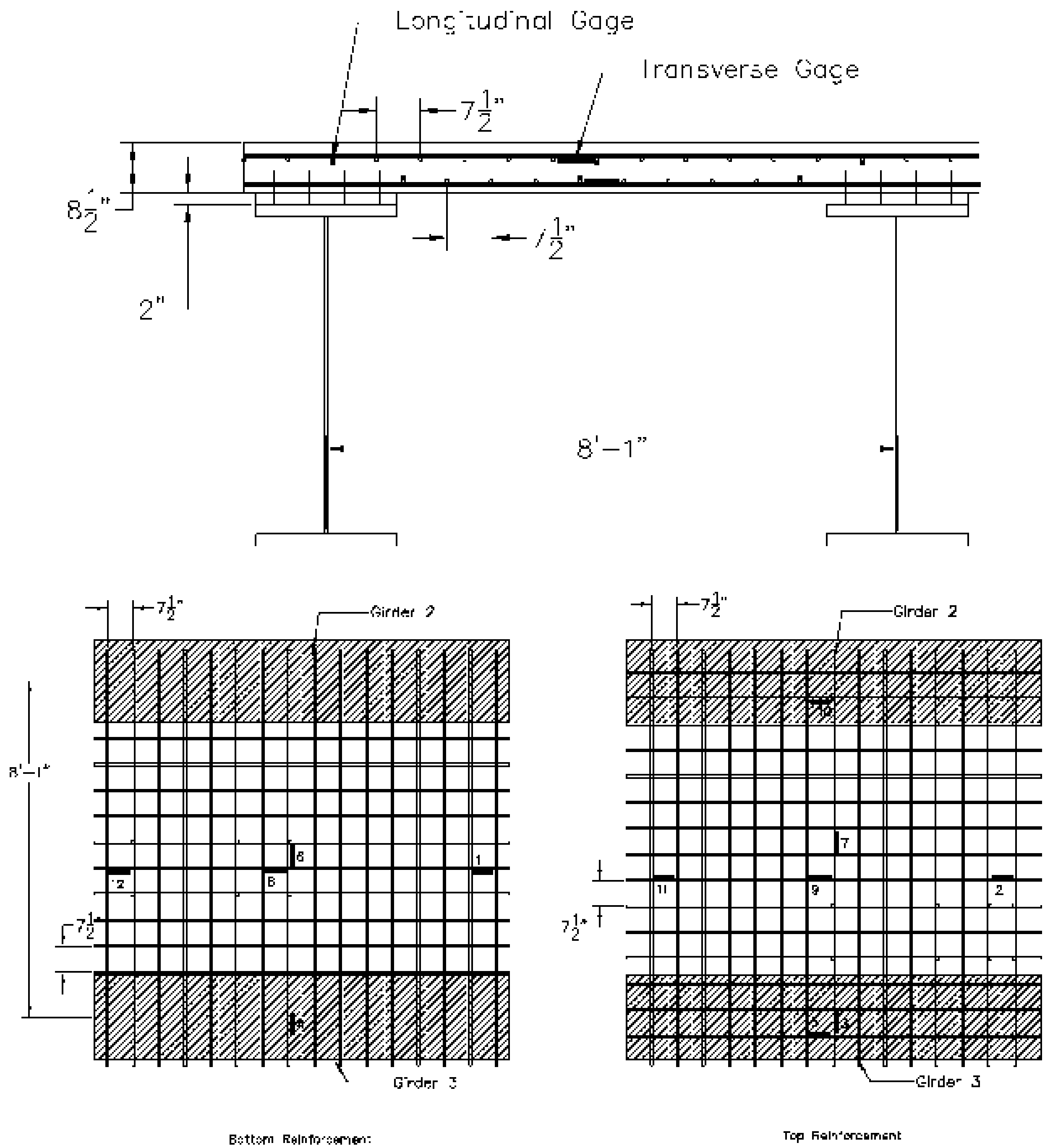

Figure 67: Collings Ave Instrumentation

Description of Monitoring

The Collings Ave Bridge deck pour began at approximately 5:00 am on July 15, 2013. Average ambient temperatures were recorded onsite and varied between 78 °F at 6:30 am to 95 °F in the afternoon. The sky was clear in the morning and partly cloudy during the day with negligible wind on the bridge deck. No precipitation occurred during the day. Over a period of 88 days following the casting of the concrete deck at the Collings Ave Bridge, strains and temperatures were measured at two minute intervals. After 50 days the interval was increased to 5 minutes. The monitoring period at the Collings Ave Bridge was shorter due to vandalism and theft of the data logging equipment. A summary of the monitoring period is given in Figure 68.

Figure 68: Monitoring Timeline – Collings Ave

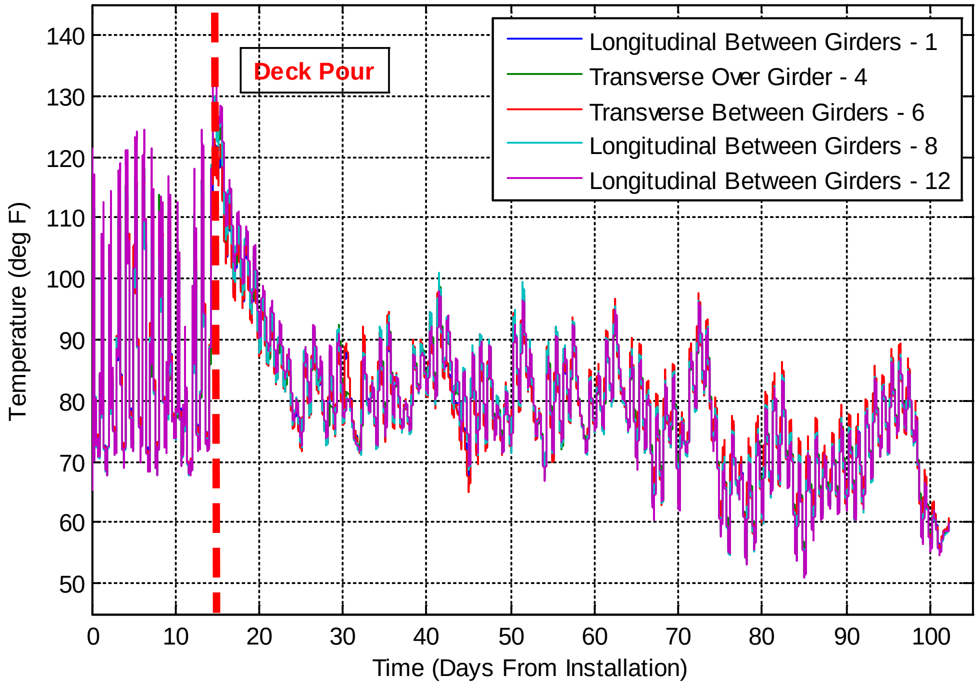

Figure 69: Measured Deck Temperatures – Collings Ave

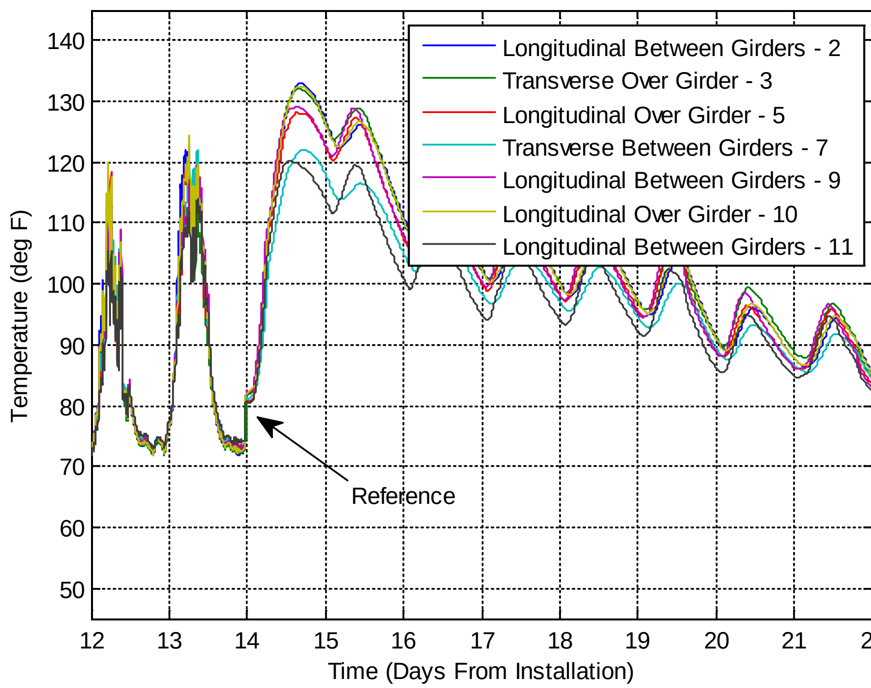

Figure 70: Peak Temperature after Deck Pour – Collings Ave – Top Reinforcement

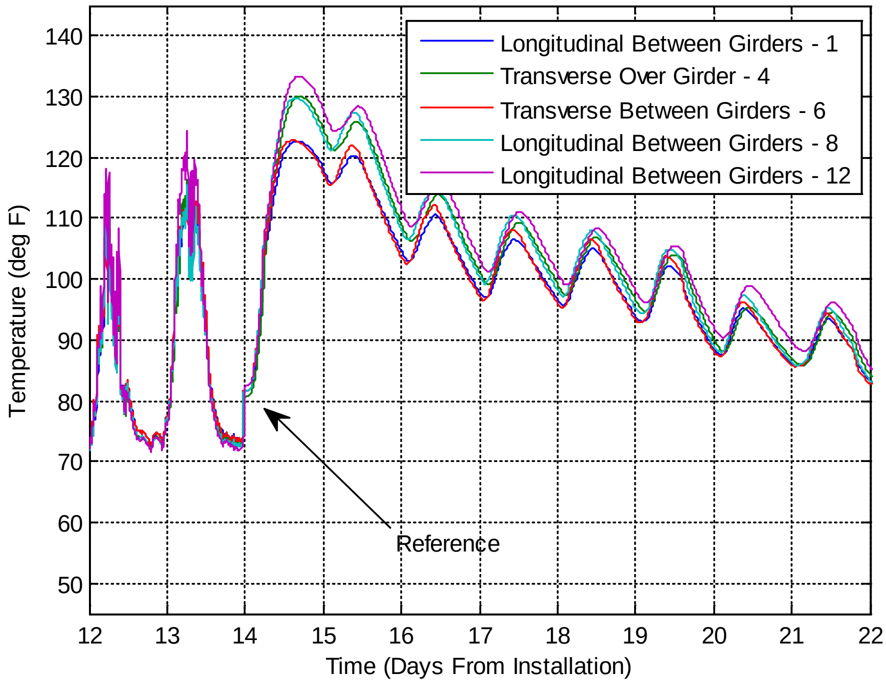

Figure 71: Peak Temperature after Deck Pour – Collings Ave – Bottom Reinforcement

The peak temperature due to the hydration process of the Collings Ave bridge deck curing occurred at approximately 17 hours from the placing of concrete and varied from 120 °F to 135 °F depending on the gage location. These temperatures were not exceeded during the subsequent 88 day monitoring period. For the Collings Ave Bridge there does not appear to be a correlation between the gage location or orientation and temperature.

Measured and Theoretical Strains

From Figure 72 it can be seen that the theoretical unrestrained strain varies from a tensile strain of 300 microstrain in the days following the concrete pour to a compressive strain of almost 775 microstrain after 85 days. The general trends follow the free shrinkage curves provided by SIMCO. The concrete used in the Collings Ave deck pour has an approximate combined shrinkage and temperature contraction potential of 1075 microstrain (300 tensile + 775 compressive). Converting these theoretical values would result in the 35’ concrete deck having a total movement of approximately 0.5 inches if the deck was allowed to freely expand and contract due to temperature and shrinkage. The free thermal strains without the superimposed theoretical free shrinkage are shown in Figure 73.

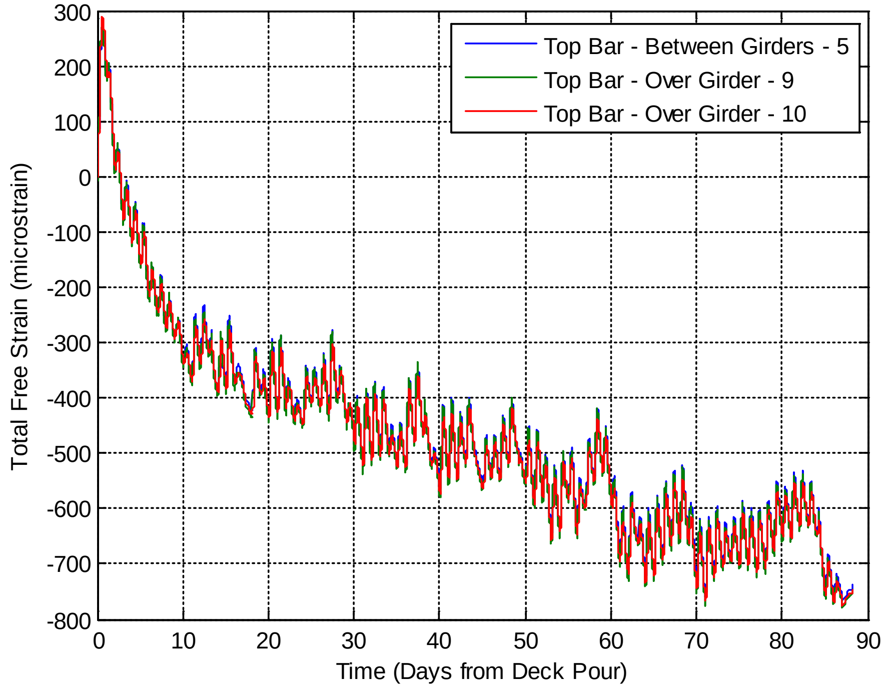

Figure 72: Collings Ave - Total Free Strain at Top Reinforcement – Longitudinal

The theoretical unrestrained strain in concrete due to temperature changes follows the general trend of the concrete temperatures as shown in Figure 69. Since the concrete was poured at a higher ambient temperature than Route 3, it is noted that initially the concrete experiences an expansion due to heat of hydration followed by a period of significant cooling where the deck contracts. Once the deck stabilizes with the ambient air temperature it begins to follow the ambient temperatures fluctuations and goes through daily cycles of expansion and contraction along with longer term seasonal expansion and contraction cycles. The unrestrained strains due to thermal loading generally vary from a tensile strain of 350 microstrain to a compressive strain of 200 microstrain. The largest expansive strains are seen during the initial stages of concrete curing since the deck was poured at a high ambient temperature. Subsequent predicted unrestrained expansions due to changes in temperature are equal to half of the initial unrestrained expansions produced by the hydration process. The contraction strain potential due to thermal loads only is approximately 550 microstrain (350 tensile + -200 compressive). Converting these theoretical values would result in the 35’ concrete deck having a total movement of 0.25 inches if the deck was allowed to freely expand and contract.

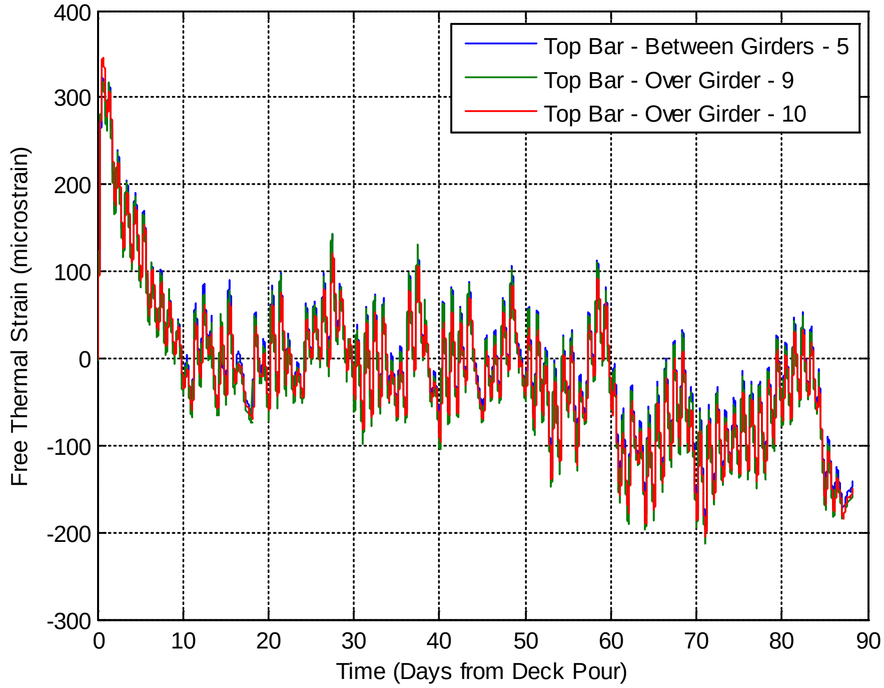

Figure 73: Collings Ave - Free Thermal Strain at Top Reinforcement – Longitudinal

The curves shown in Figure 74 are the strains that would be measured on the concrete by an external gage which would reflect the actual deformation of the concrete slab. These gages follow the measured temperature curves and the strains that would be measured by an external device indicate a range of strains from a tensile strain of 225 microstrain to a compressive strain of approximately 375 microstrain. Using the total theoretical free strains as a baseline, the external strains indicate a missing 475 microstrain of shrinkage potential.

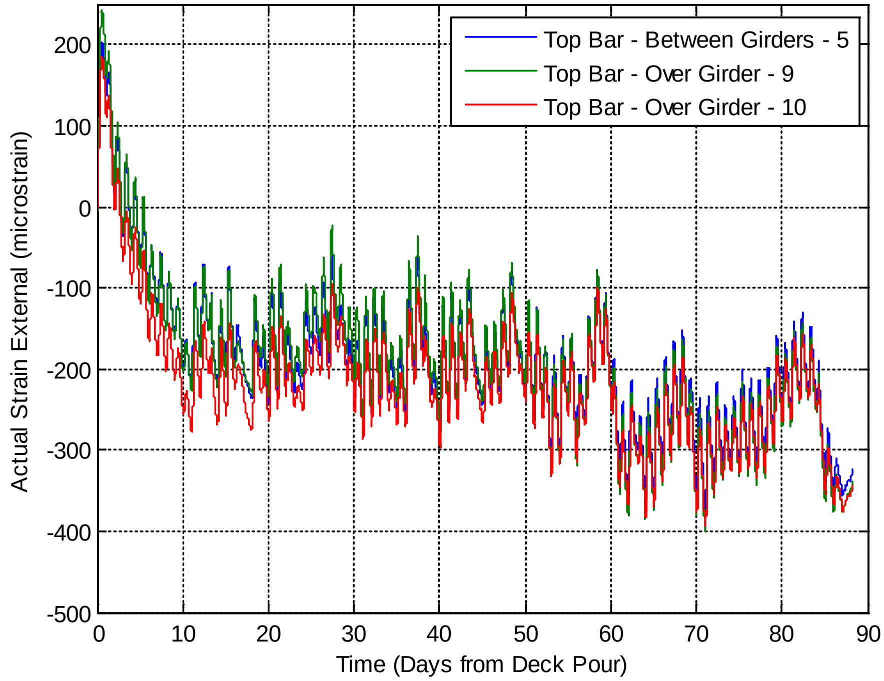

Figure 74: Collings Ave - Strain That Would Be Measured by External Device – Longitudinal Top Reinforcement

The strains measured by the embedded gages shown in Figure 75 are a composite of temperature effects and external load. The corrected strains measured by the internal gages show compressive strain varying in maximum magnitude from 150 to 250 microstrain (compression).

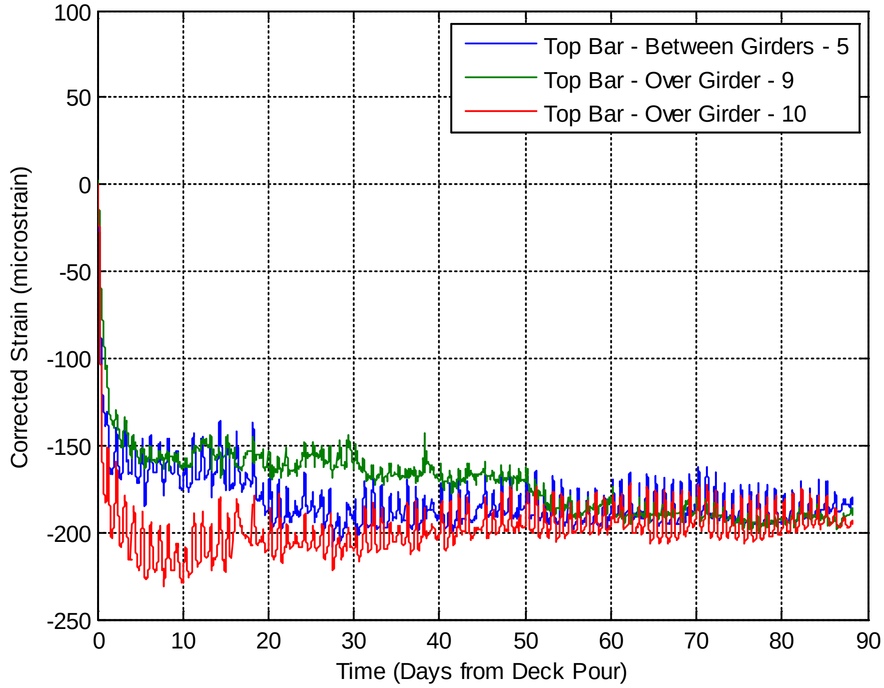

Figure 75: Collings Ave - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Top Reinforcement

Other Data Plots

Total Unrestrained Strain

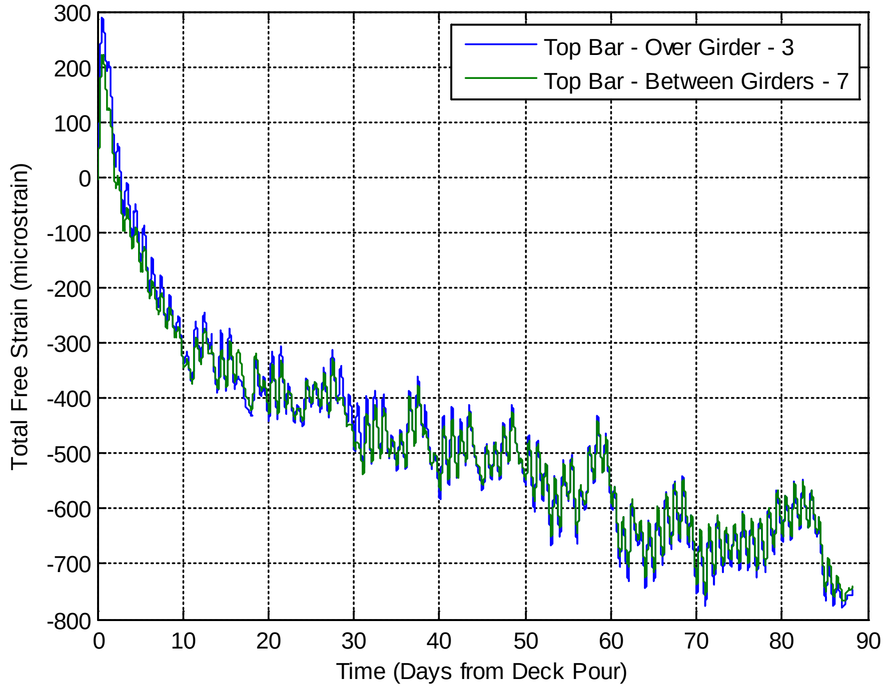

Figure 76: Collings Ave - Total Free Strain at Top Reinforcement – Transverse

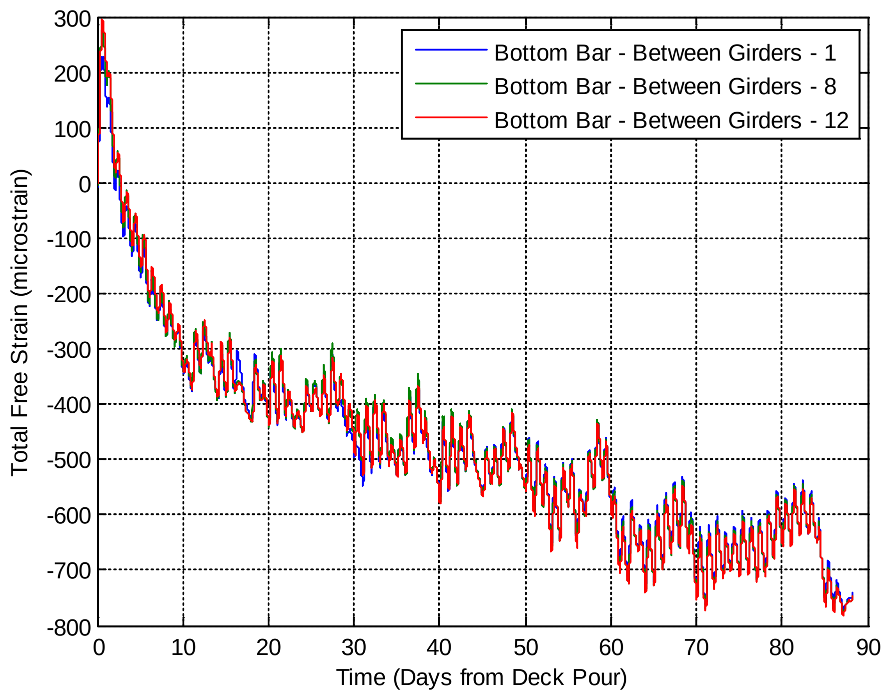

Figure 77: Collings Ave - Total Free Strain at Bottom Reinforcement – Longitudinal

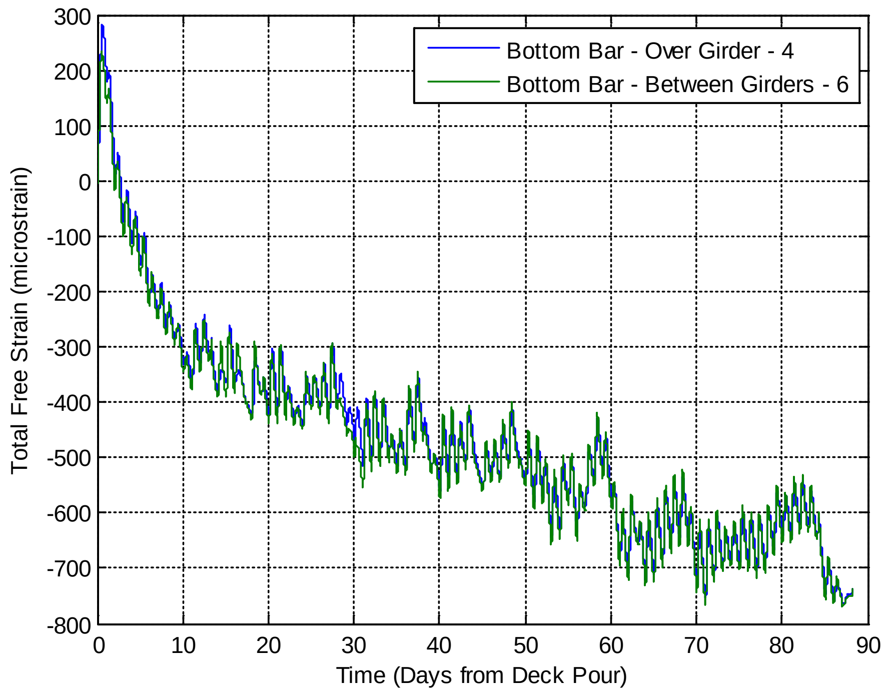

Figure 78: Collings Ave - Total Free Strain at Bottom Reinforcement – Transverse

Unrestrained Thermal Strain

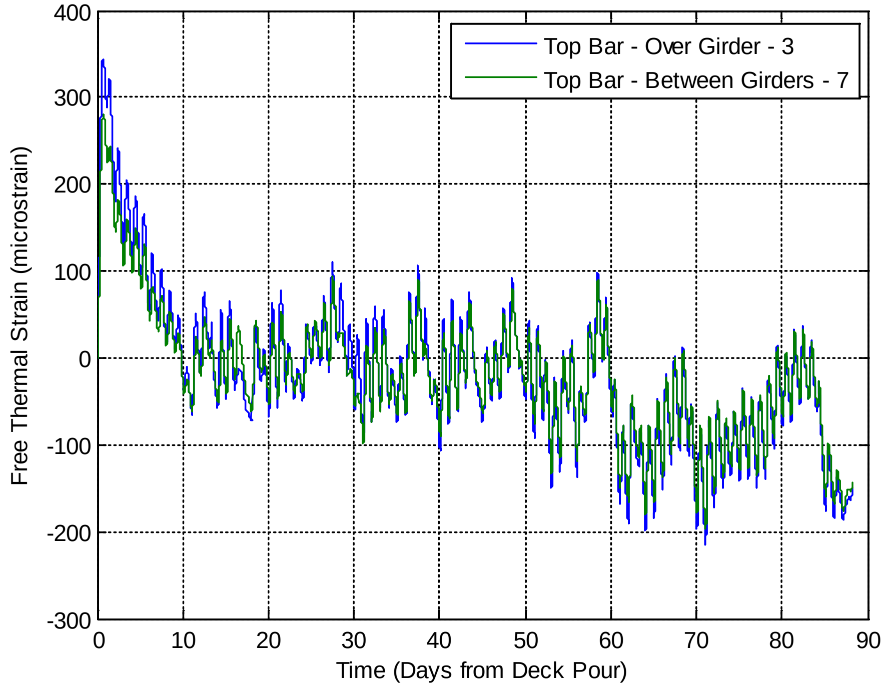

Figure 79: Collings Ave - Free Thermal Strain at Top Reinforcement – Transverse

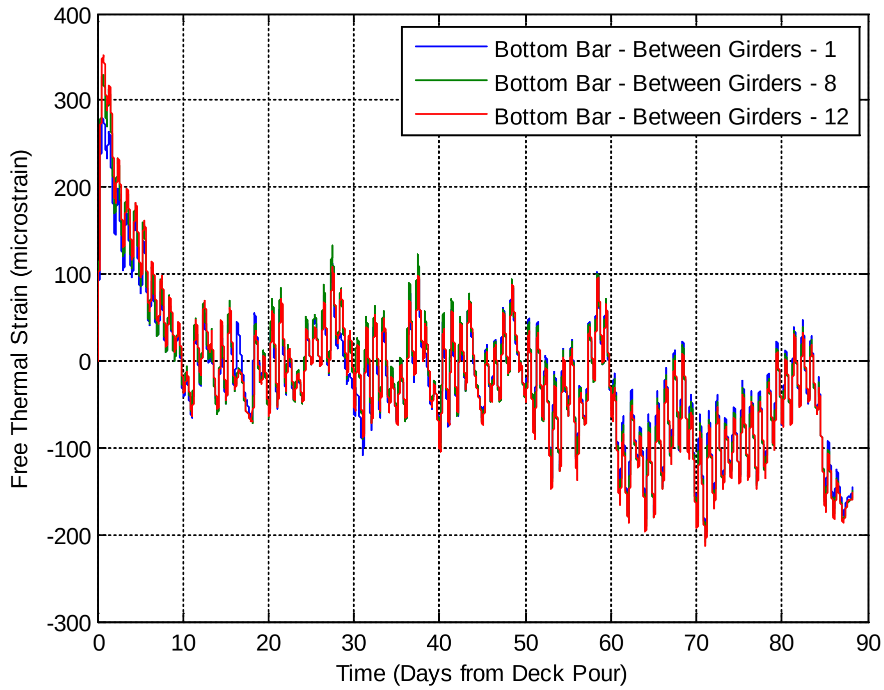

Figure 80: Collings Ave - Free Thermal Strain at Bottom Reinforcement – Longitudinal

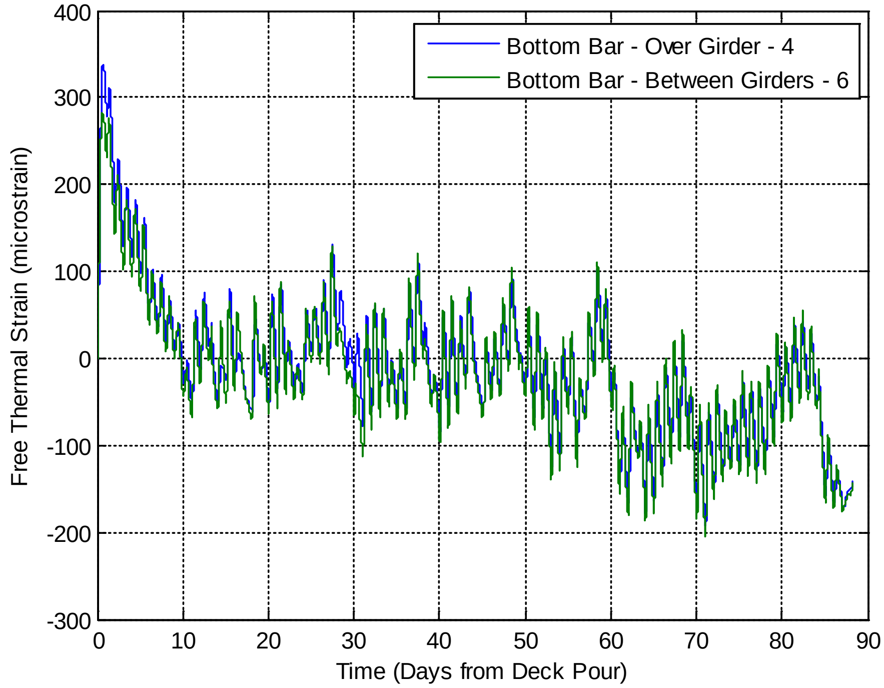

Figure 81: Collings Ave - Free Thermal Strain at Bottom Reinforcement – Transverse

Strain that would be Measured by an External Sensor

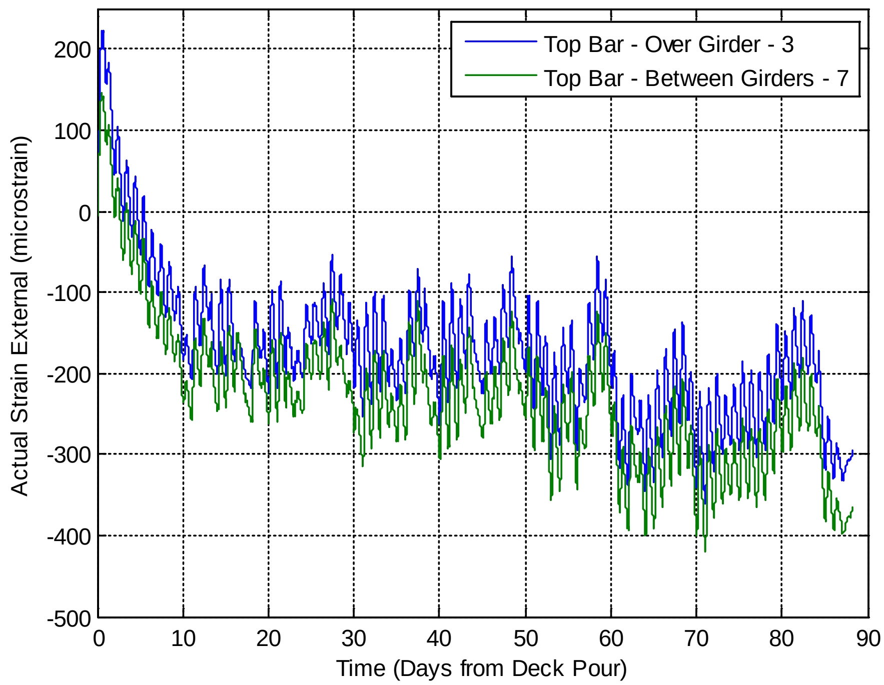

Figure 82: Collings Ave - Strain That Would Be Measured by External Device – Transverse Top Reinforcement

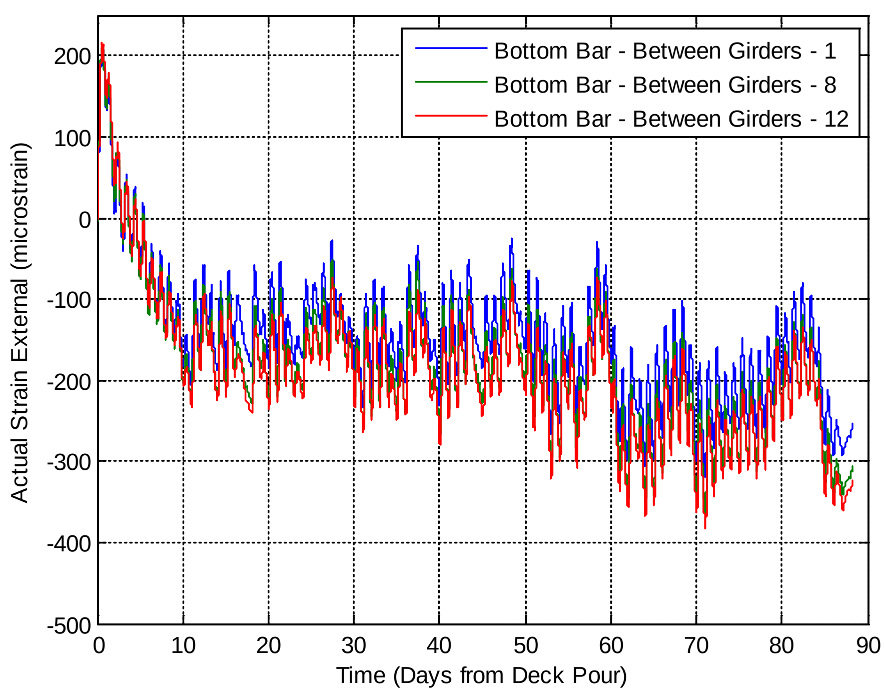

Figure 83: Collings Ave - Strain That Would Be Measured by an External Device – Longitudinal Bottom Reinforcement

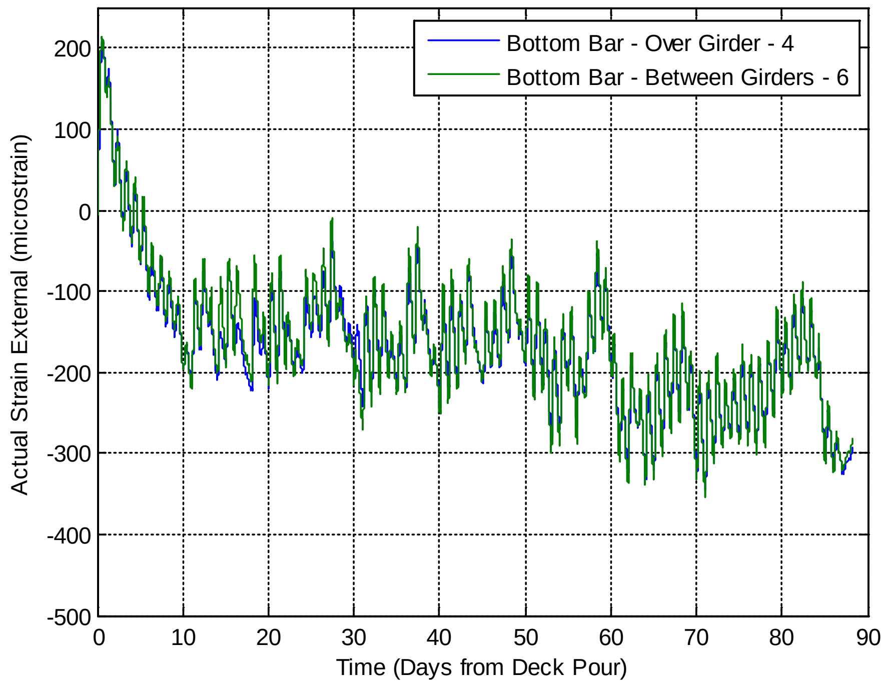

Figure 84: Collings Ave - Strain That Would Be Measured by an External Device – Transverse Bottom Reinforcement

Corrected Strain

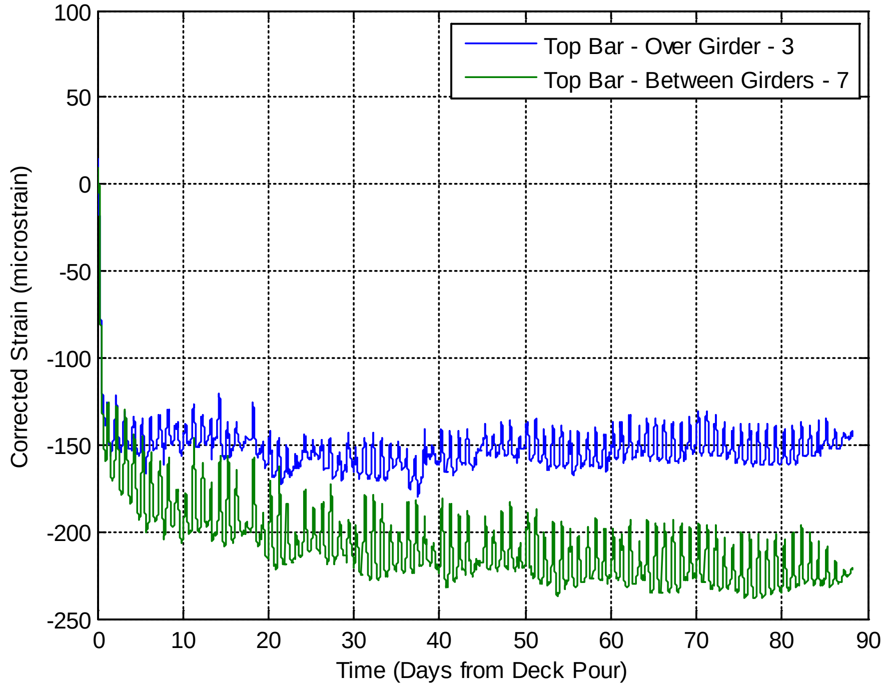

Figure 85: Collings Ave - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Transverse Top Reinforcement

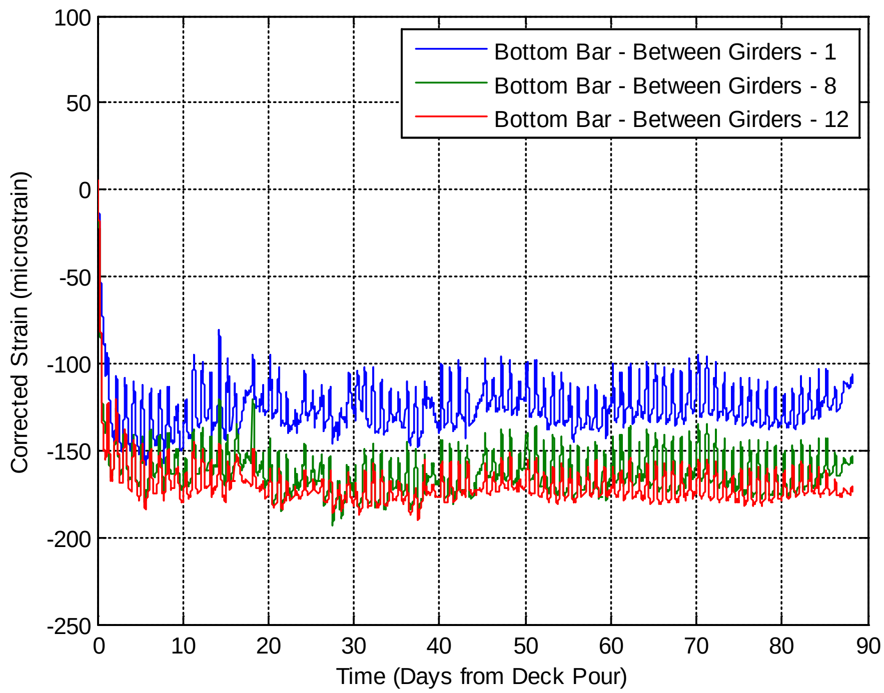

Figure 86: Collings Ave - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Longitudinal Bottom Reinforcement

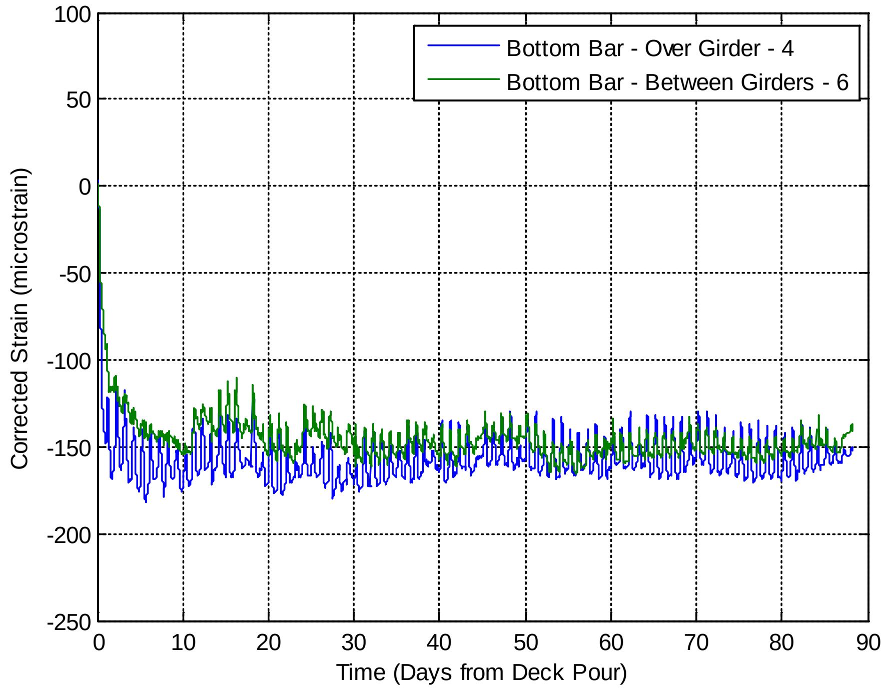

Figure 87: Collings Ave - Strain Measured by Internal Gage Corrected for Difference in Thermal Expansion Coefficient – Transverse Bottom Reinforcement

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APPENDIX 2D
REFINED LOAD RATING TASK

Intelligent Infrastructure Systems

NJDOT Refined Load Rating

Prepared For:
Rutgers University
New Jersey Department of Transportation

Draft Report Submitted:
April 14, 2014

Scope of Work

Intelligent Infrastructure Systems performed refined load rating as part of the first year of the NJDOT Bridge Resource Program. The task focused on utilizing analytical and experimental tools to provide updated load ratings for eight bridges. The tools used included finite element modeling (FEM), instrumentation, and truck load testing. The eight structures were selected by NJDOT for refined load rating and the selected group is characterized by a single overarching issue, low analytical load ratings using AASHTO prescribed methods. The selected bridges vary in size, structural form, and construction material. One of the eight bridges selected for this program was the RT18 John Lynch Memorial Bridge which is a complex structure and in consultation with Rutgers-CAIT and NJDOT it was determined that this bridge would be a modeling only study. An additional bridge, the East Anderson St Bridge, was identified by NJDOT for load rating and testing. This bridge consisted of 12 spans of adjacent prestressed concrete box beams which exhibited deterioration and the load rating was of concern to NJDOT. This bridge was modeled, instrumented, and load rated as part of this scope and was submitted previously in a separate report.

In addition to the eight bridges mentioned above, 2 skewed superstructures were also scheduled for monitoring under this task, bringing the total number of bridges to eleven. Since the skew bridges were reliant on such structures being constructed during the year, only one skewed superstructure was identified by NJDOT for monitoring during the first year and the work performed on the selected structure is presented in a separate synthesis report. The selected bridges studied under this task are shown in the list below and they are distributed over the entire state of New Jersey. An overview of their geographic location is given in Figure 1.

1. Structure 0118150 – US206 over Cedar Branch
2. Structure 1703152 – US40 over Salem Creek
3. Structure 0324152 – US206 over Springers Brook
4. Structure 1512152 – NJ72 over Mill Creek
5. Structure 1516152 – NJ166 over Toms River
6. Structure 1103152 – US1 over D&R Canal
7. Structure 1237155 – NJ18 over Raritan River
8. Structure 1701151– US40 over W. Branch of Game Creek
9. Route 3 over NJ Transit – New construction
10. Structure 020023A – East Anderson St (CR60) over Hackensack River

Following the selection of bridges by NJDOT, a methodology to model, rate, and test the structures was implemented. The methodology used to obtain updated load ratings using modeling and testing is detailed in the following sections.

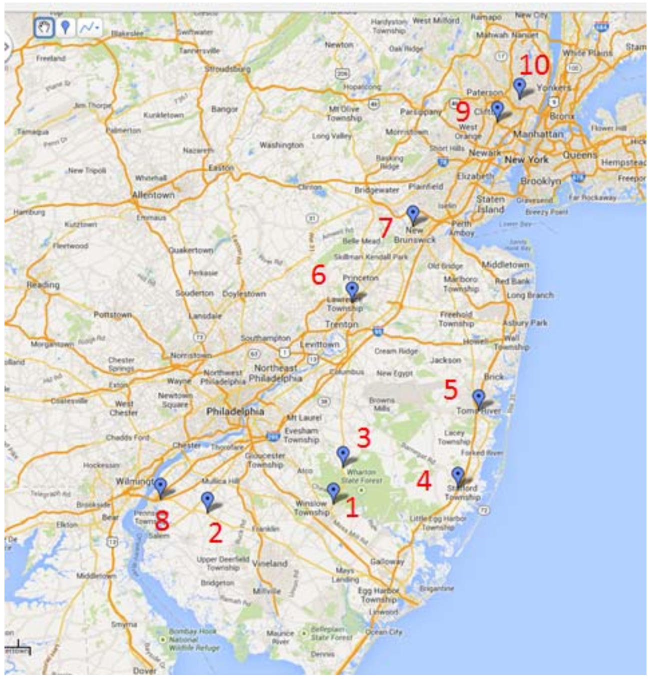


Figure 1: Geographic Location of Bridges

Load Rating Challenges

Several bridges rated in this report presented challenges during the development of finite element models and capacity estimation. For example, the Mill Creek and Toms River bridges are bridges constructed using encased or partially encased steel girders. The D&R canal bridge is a skewed portal frame structure whose width along the skew is nearly 400 feet. The RT18 bridge over the Raritan River is comprised of twin curved girder/floorbeam/stringer structures over 800' in length. These bridges provided unique challenges to developing load ratings due to the complexity of their structural systems and also their unique geometry.

The slab, T-beam, and portal frame structures rated in this report have low AASHTO rating factors due to the use of conservative distribution factors. Modeling and testing show improved distribution of load which provides increased rating factors. The encased girder structures are complex structural systems that are difficult to rate unless assumptions are made regarding the participation of the concrete encasement. The complexity of the system coupled with conservative distribution of load results in low rating factors. The RT18 Bridge over the Raritan River has low rating factors using traditional analysis due to the complexity of the structural system and conservative distribution factors. Table 1 provides a summary of the bridges rated, a comparison of the controlling ratings at the inventory level from both AASHTO Load Factor and refined rating, and a brief explanation of the reason for the low ratings calculated using traditional AASHTO methodology.

Table 1: Summary of AASHTO and Refined Load Ratings

	AASHTO Rating - Inventory - tons (RF)	Rating Methodology	Refined Rating - Inventory - tons (RF)	Rating Methodology	Reason for Low AASHTO Load Ratings
Cedar Branch	26 (0.72)	AASHTO Load Factor	55 (1.54)	FE Model and Load Test	Conservative load distribution
Salem Creek	25 (0.69)	AASHTO Load Factor	44 (1.25)	FE Model and Load Test	Conservative load distribution
Springers Brook	25 (0.69)	AASHTO Load Factor	159 (4.42)	FE Model	Conservative load distribution
Mill Creek	20 (0.56)	AASHTO Load Factor	44 (1.22)	FE Model	Complex structural system
Toms River	22 (0.61)	AASHTO Load Factor	58 (1.61)	FE Model	Complex structural system
D&R Canal	13 (0.36)	AASHTO Load Factor	49 (1.36)	FE Model	Conservative load distribution
Raritan River	22 (0.61)	AASHTO Load Factor	119 (3.31)	FE Model	Complex structural system
Game Creek	19 (0.53	AASHTO Load Factor	74 (2.06)	FE Model	Conservative load distribution

Approach to Finite Element Modeling

The finite element models developed for use in this task were constructed using the Strand7 finite element software package. Strand7 was chosen for its ability to interface with MATLAB computational software and the included API allowed for the facilitation of automated sensitivity studies, model calibration, and extraction of model responses. The following sections detail the steps taken to develop the a-priori models used for refined load rating.

Observation and Conceptualization

The first step in developing a finite element is to obtain all available documentation showing the geometry, section properties, and material properties for the structure. These documents could include as built drawings, design drawings, inspection reports, photos, etc. It also may be necessary to visit each structure to take additional photos, document dimensions shown on plans, and view any other details identified as uncertain during the document collection and review process. The site visit is also valuable for establishing site constraints and logistical hurdles for instrumenting and load testing the structure. A site visit was made to each of the structures selected for this task to view the structure, confirm overall geometry, and also to document the site conditions around each structure.

Selection of Model Form

The second step in developing a finite element model is to select the appropriate model form for representing the structure. For the purposes of the load ratings performed in this task, the discussion is limited to the use of physics based models (specifically FEM). A decision between a macro or element level model and a micro level geometric replica FE model is required. Element level models are useful for modeling structures with discrete members such as beam slab or truss structures. Geometric replica models can be used to model components of structures or model those structures that are monolithic in nature such as cast in place concrete structures. Both macro and micro level models were used in this study.

Geometry

Once the model form is selected, the overall geometry of the physical structure is developed and translated into the Strand7 modeling software via direct input or transformation from an external CAD program. For the geometric replica models developed in this project, AutoCAD was used to develop the geometry which was imported into the Strand7 FE software for further manipulation and meshing. For element level models, the geometry was developed directly within the Strand7 software. An example of the geometry of a geometric replica model is given in Figure 2.

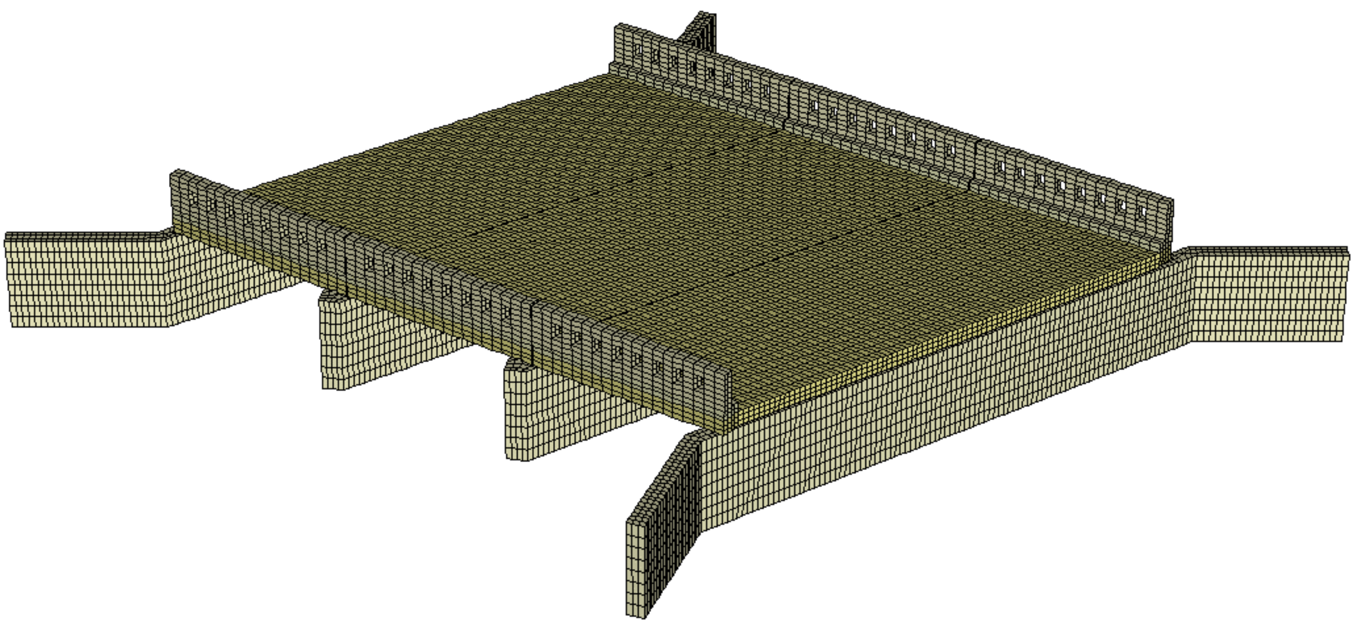


Figure 2: Example Geometric Replica Model

Material and Section Properties

Element level models utilizing beam elements, require the definition of a cross section for each element. The cross sections can be built in CAD and imported into the software or directly selected from a library of available cross sections. For element level models that include shell elements, the planar geometry is defined and the user selects a thickness dimension. For many element level models, shells are used to model the bridge deck and the user would supply the deck thickness as the third dimension. In geometric replica models, the 3D geometry is already explicitly defined which negates the need for developing cross sections.

Once the member sections are defined, each element is assigned a material property. The relevant material properties can be obtained from design drawings, design/analysis code recommendations, or direct results from material sampling. In the case of the bridges presented in this report, the material properties were either obtained for the design drawings or from the AASHTO Manual for Bridge Evaluation (2011). However, these are only estimates of the material properties used in the construction of bridges during this time period. Changes in the material properties will influence the capacity used in the ratings and should be considered as part of future refined load ratings.

Boundary Conditions

Once the material and section properties are assigned, the boundary conditions supporting the model are identified and implemented in the models. For most superstructure rating analyses, the model is truncated at the interface between the superstructure and the substructure and an assumed support condition is modeled. Models are supported by restraining translation and rotation based on the boundary conditions identified in the as built drawings. For calibration purposes, support conditions can be modeled as springs whose value can be tuned to influence the model response.

Miscellaneous Assignments

This stage of model construction can include the assignment of non-structural mass and link elements. Nonstructural mass is included in a model to add dead load to the model without adding any additional stiffness. Nonstructural mass could be used to model an asphalt overlay or add the dead load due to attached utilities or other ancillary structures that do not add stiffness to the structure. Links are used to connect different elements together that are at different points in space and are defined to relate the translations and rotations at one node to those at a separate node. Link types include master/slave, coupling, pinned, rigid, shrink, attachment, and multi-point. Such an example would be using a rigid link to connect a beam element to a shell element so the beam and shell act compositely as would be the case with a concrete deck on steel girder structure.

Load Cases

The final step is to define the loads to be applied to the model. The loads can range from the load due to self-weight, snow loads, live loads from vehicular traffic and/or wind, and loads due to temperature. In the case of rating analysis for live loads, the dead load and vehicular live load are the critical load cases needed for the analysis. For load rating, a moving load analysis is conducted where the static loads are moved across the structure and the critical load positions corresponding to the maximum load effect at a particular location or member are identified. The rating analysis then uses the maximum load effects extracted from the model.

FE Model QA/QC

Before the model can be used for load rating, the model must be error screened by an independent party to identify if the model has been built without any blatant errors or bias. The following details the general approach used by IIS for the error screening of finite element models.

Overall Geometry

The most fundamental error screening begins by ensuring the overall geometry is correct, which is limited to the primary load carrying elements. The checker must become familiarized with the bridge documentation gathered for the model construction process. The checker then proceeds to verify overall dimensions such as length, width and depth of the structural elements. In addition to overall dimensions, the checker also verifies the number of elements in comparison to the plans (for example, quantity of stringers).

Common errors - At this early stage, common errors include unit errors in the conversion from plan units to model units, and simple misinterpretation of the plans. By reviewing such global information first, the checker does not waste much time in the case of an error, and can turn the model back to the person who built it for revision.

Secondary Geometry

The second stage of error screening is aimed at confirming the geometry of all remaining model items, and is limited to secondary load carrying elements. This would consist of transverse bracing, barriers, and decks. The checker must ensure that all intersections between pairs of beam elements or shell elements have nodes at the intersection, and that all intersecting elements share the same node. The overall quantity of elements must also be confirmed against as-built plans for the secondary members.

At this phase, errors commonly include duplicate nodes at element intersections, missing secondary elements, or incorrect geometry. Most modeling software platforms provide built-in error screening functions which search the model for duplicate nodes and elements, streamlining the checker's responsibilities. If at this stage, minor geometric errors or omissions are discovered, the checker is encouraged to complete the remainder of the thorough review since the correction of these errors are easily incorporated.

Property Assignments

The third stage focuses on the assignments of section and material properties to all members. The checker must ensure that the geometric assignments to each element are appropriate compared to the as-built drawings. For elements, all cross-sectional properties must be checked for accuracy in terms of shape and assignment locations within the model. Additionally, elements must be checked for proper local axis orientation. This is facilitated by plotting the local axes on the model GUI in addition to plotting the extruded shape of the member elements, if possible. The material properties of all members are then checked against those specified in the as-built drawings.

Errors include simple mistakes when selecting members for assignment, so it is critical that the checker meticulously ensure the member assignments are correct. It is also important to ensure the units of the member assignments are consistent with the global units of the model.

Constitutive Assignments

The model at this stage is checked for proper constitutive assignments. The checker must ensure that analytical representations of physical phenomena are simulated in a reliable manner, and that the assumed values are justified. This requires the checking of rigid offsets or links, boundary conditions, joint constraints, master/slave interfaces, among many other types of constitutive assignments.

Common errors: Errors at this stage are commonly associated with the user selecting members by accident when assigning definitions, or poor judgment in simulating physical phenomena.

Analysis Settings

The checker must review the analyses set up by the analyst in building the model. This also includes ensuring that the assigned global degrees of freedom are consistent with the analysis being carried out and the geometry selected. Similarly, the checker must ensure that gravity is oriented correctly in relation to reality, and only for load cases considering self-weight. The checker is also responsible for reviewing the assignment of all live load cases for properly simulated loading scenarios as well as superimposed dead load cases for properly assigned extraneous mass sources.

Errors at this stage are commonly related to mistakes on the behalf of the user related to properly setting up the analysis files and settings. The checker must run all analyses at this stage of the review process, which must include: (1) dead load, (2) unit load in each direction, and (3) modal analysis. The methodology of using these analyses for model error screening is described below.

Using Model Analyses as an Error Screening Tool

Following the completion of all model error screening through the execution of analyses, the checker must carefully review the analysis results for each load case discussed above for specific errors.

Dead load - In reviewing the dead load analysis, the checker will be reviewing the model for missed element connections, overall symmetry in the displaced shape, overall symmetry in boundary conditions, unintended breaks or discontinuities. The checker shall provide a rough estimate of total dead load, if it is not stated within the as-built drawings, for comparison to the total dead load reactions tabulated from the model.

Unit load in each direction - This specific analysis tends to amplify the presence of unintended breaks and discontinuities. The checker shall also ensure that the sum of applied loads at the nodes is equal to the sum of reaction forces, confirming that no 'floating' nodes are present and that equilibrium is satisfied.

Modal analysis - Modal analysis, specified to run with at least 15 modes, will also tend to amplify the presence of unintended breaks or discontinuities for the checker's review. Additionally, the mode shapes provide insight towards the distribution of mass around the structure. Finally, the presence of rigid body modes in a dynamic analysis would also indicate issues related to improperly assigned boundary conditions.

Comparison to Simple Hand Calculations

The final step of the error screening process is to compute simple hand calculations of a structural response for comparison with the model. For example, it may be that the checker could provide a quick estimate of midspan deflection due to a unit point load by assuming it only acts on one stringer and using simple beam theory. Taking such steps adds another layer of confidence to the model before moving on to further system design.

Mesh Sensitivity Analysis

Once the model has been screened for the errors mentioned previously, a mesh sensitivity study is performed to determine the appropriateness of the mesh size used in the model. A model response, such as midspan displacement is chosen and compared with models of increasing refinement. The initial mesh is halved and the displacement predicted again. Once the rate of change of the predicted response between subsequent models is minimized, the mesh has been refined to an appropriate level. After this final step, the model is ready to be used for further analysis such as the load ratings performed in this study.

Rating Methodology

Initially the structures selected for refined load rating were rated using current AASHTO Load and Resistance Factor methodology. However, after presenting these ratings to NJDOT it was discussed that they should be revised using the Load Factor (LFR) methodology. The LFR methodology is applied as specified by the AASHTO Manual for Bridge Evaluation (2011) and modified by the specifications shown in Section 43 of the New Jersey Department of Transportation Design Manual for Bridges and Structures (2009). The nominal capacities of member sections are developed in accordance with the provisions specified in AASHTO Standard Specifications for Load Factor Design and modified based on deterioration observed in the field and documented in available inspection reports. The bridges rated in this report included, reinforced concrete slabs, reinforced concrete T-beams, concrete encased steel girder bridges, girder/floorbeam/stinger bridges, and prestressed concrete girder bridges. Depending on the structural system, several decisions and assumptions are required for developing the capacity used in load rating.

Flexural Capacity

For slab type bridges, a one foot width of the slab is rated. The flexural capacity is developed based on assuming the one foot section is a reinforced concrete beam, and the demand from the model is distributed to the one foot wide section. The flexural capacity of the T-beam bridges were based on developing the capacity of the flanged T-beam assuming the effective flange width is determined using LFD provisions. Once the effective flange width is known, the depth of the equivalent stress block is computed and then the nominal moment capacity is calculated by multiplying the force in the reinforcing steel at yield by the moment arm between the compressive force and the tensile force. The portal frame structure top slab was rated using the AASHTO strip width method for rating reinforced concrete slabs.

The flexural capacity of the concrete encased sections was assumed to equal the capacity of the non-composite steel section. The non-composite flexural capacity is equal to the plastic moment of the section according to LFD provisions. This is a conservative assumption and the capacity would increase if the contribution of the concrete encasement and slab were taken into account and the section was treated as composite. The positive and negative flexural capacity of the non-encased steel sections rated in this report is equal to the plastic moment of the section either based on whether the section

Shear Capacity

The shear capacity of each bridge was calculated using LFD methodology. Concrete sections without shear reinforcement, such as reinforced concrete slabs, have their shear capacity calculated solely on the shear capacity of the concrete in a one foot wide section. Concrete sections with shear reinforcement, such as T-beams, have the shear capacity calculated as a combination of the shear capacity of the concrete plus the additional shear capacity afforded by the shear reinforcement. The shear capacity of steel I sections was calculated using 58% of the yield stress of steel multiplied by the geometry of the web as specified in the LFD provisions.

Dead and Live Load Demand

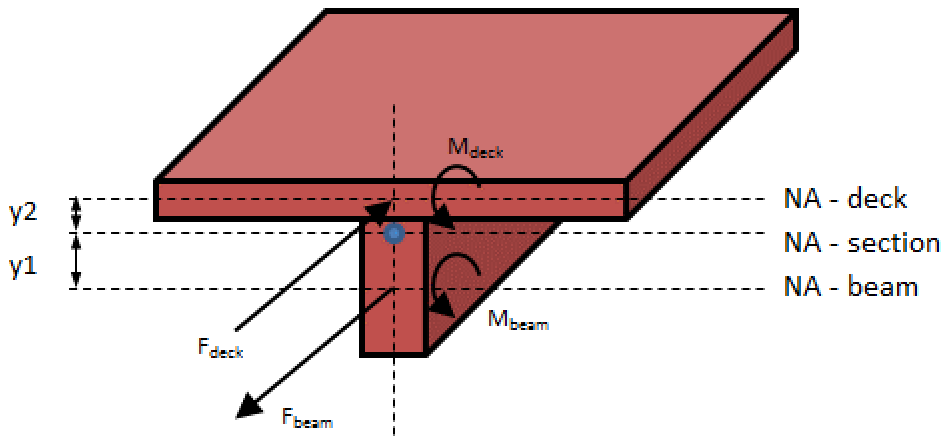
The dead load and live load demands are developed directly from the FE models for each structure. To develop live load demands, the finite element models were loaded using the following loads:

- HS20
- Type 3
- NJDOT Type 3S2
- Type 3-3
- Single Unit Specialized Hauling Vehicles (SU4-SU7)

After establishing the fixed geometry of each truck, a single truck was placed in each lane and were moved across the model to determine the maximum load effect for a particular member or area of the bridge. Once the truck position(s) that produced the maximum load effect were located, the loads were automatically placed at this location and the maximum load effect was extracted from the model.

Extraction of Maximum Load Effect

The LFR strength method compares section capacity to the maximum member actions (forces and moments) from the dead and live loads applied to the bridge. The selected model form dictates how the modeler extracts the maximum member action for different vehicular loadings. Element level models can directly output forces and moments for each element, while a geometric replica model outputs stresses that require further manipulation to obtain forces and moments. If an element level model is selected that uses beam elements linked to a shell element deck for modeling a composite section, the total moment acting on the composite section is calculated using a summation of the moment in the deck, the moment in the beam, the axial force in the beam multiplied by the distance between the neutral axis of the beam and the neutral axis of the composite section, and the axial force in the deck multiplied by the distance between the neutral axis of the deck and the neutral axis of the composite section. This process is shown graphically in Figure 3. To calculate the shear force in the composite section, the shear force in the vertical plane of the beam and the vertical plane of deck are summed.



$$M_{NA-section} = M_{deck} + M_{beam} + F_{deck} * y2 + F_{beam} * y1$$

Figure 3: Extraction of Bending Moment from Composite Element Level Model

If a solid element model is used, obtaining bending moments requires further manipulation of the model responses. Solid elements do not directly output moments or shear forces but they do output stresses. Therefore, a conversion from stress to moment/shear force is required. To obtain bending moments from a solid element, the longitudinal stresses are contoured on the model, a cut at the section of interest is taken, and the bending moment is calculated by integrating the normal stress distribution on the section. This process is shown using the simple benchmark model shown in Figure 4. The benchmark compared the midspan moment of a simply supported beam using two modeling approaches, one model was constructed using beam elements and the second model was constructed using solid elements. The section properties, material properties, and boundary conditions were equivalent in both models. The midspan bending moment due to a point load applied at the center was predicted using both models and compared. The solid element and beam element representations produced equivalent moments, verifying the approach used to extract bending moment from solid elements. To calculate the shear force on the section, the shear stress is contoured on the model, a cut is taken at the critical section for shear, and the direct sum of the shear stress is tabulated to obtain the total shear force on the section.

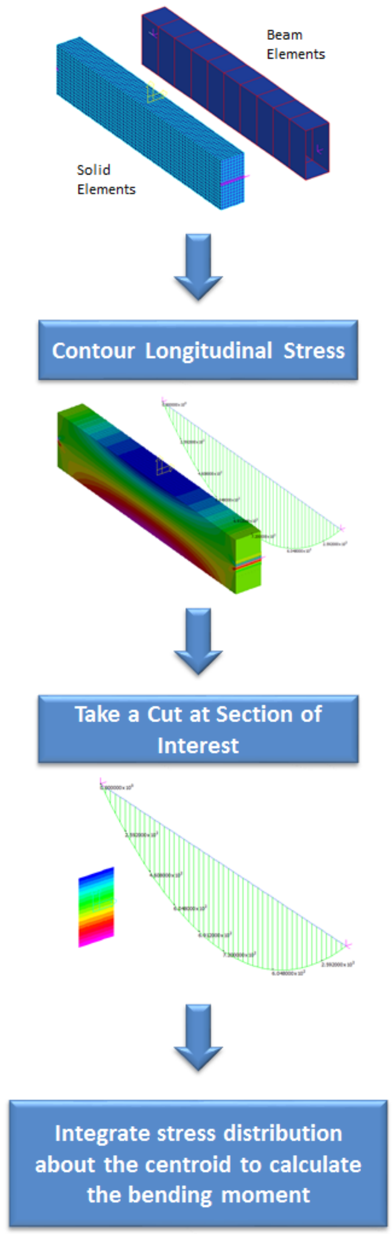


Figure 4: Extraction of Bending Moment from Solid Elements

Load Factor Rating

Following the extraction and summarization of the maximum member actions from the model, the maximum member actions are used as inputs into the bridge rating equations. To obtain the Load Factor ratings presented in this report, the flexural strength approach was used for both shear and moment. The general load factor rating equations for flexural and shear strength at the inventory and operating levels are shown the equation below.

$$RF = \frac{\phi R_n - (A_1 D + S)}{A_2 L(1 + I)}$$

Where:

RF = Rating factor
ϕR_n = Nominal strength of a section (moment and shear should both be evaluated)
D = Dead load demand
S = Unfactored secondary effects
L = Live load demand
I = Live load impact factor
A_1 = Dead load demand factor – equal to 1.3 for both inventory and operating levels
A_2 = Live load demand factor – equal to 2.17 for inventory level and 1.3 for operating level

Summary of Load Testing Approach

In order to provide confirmation of the as-is load ratings using calibrated finite element models, structural testing was implemented on several structures. Using the calibrated models, the initial load ratings were updated to reflect the as-is behavior of the structure. Load testing is divided into two categories, diagnostic and proof level load testing. Diagnostic load testing was chosen to evaluate the structures load carrying capacity. According to AASHTO (2011), diagnostic load tests are employed to improve the Engineer's understanding of the behavior of a bridge and to reduce uncertainties related to material properties, boundary conditions, cross-section contributions, effectiveness of repair, influence of damage and deterioration, and other similar variables. Load tests measure the response of critical bridge members to develop an input versus output relationship in order to compare with and calibrate an analytical model.

During a diagnostic load test, loaded trucks are positioned at various locations on a bridge span to maximize the effects at a particular instrumented location. The loaded trucks are held at each load position until all readings have stabilized and to provide sufficient data for averaging in the data reduction and interpretation stage. The instrumentation during a load test generally consists of strain, displacement, and rotation sensors. For the load tests described in this report, strain gages aimed at developing field measured input output relationships, and the distribution of strain within the structure were the primary sensors installed. A single truck was provided for testing and was positioned at several points along each travel lane. The truck was also crawled along each lane to develop an influence line describing the effect of the position of the loaded truck on each instrumented location.

Data Reduction and Interpretation

Following the implementation of load testing, there are a large number of measurements to reduce and synthesize into metrics describing the response of the structure. However before the data can be reduced and interpreted, the raw data requires error screening. In this phase, the data is screened for anomalies and/or blatant errors which could be attributed to dead sensors, erroneous zeroing, or significant noise that were not recognized during the test. If these errors are identified in the collected data, steps to correct errors may include omitting data from nonfunctioning or incorrectly zeroed sensors and filtering the data to remove noise. Following error screening, data is plotted versus time and reduced into tables by identifying the beginning and end of each load stage manually and averaging each of the sensors over that time period to minimize the effects of sensor or electrical noise.

Once summary tables of responses at each instrumented location and load case are generated, the responses can then be interpreted. Data interpretation can be segregated into two phases: direct interpretation, meaning information could be obtained with minimal further analysis or computation, and interpretation through model calibration, which requires significant additional analyses. In the load testes presented in this report, the data was directly interpreted to document nominal strain measurements in the instrumented members and distribution of strains across the structure. The summarized and interpreted strains were then used to calibrate the a-priori models and update the load ratings.

Model Calibration

Sensitivity

Sensitivity analyses were conducted using the a-priori model to identify the model parameters that directly influence the model responses of interest. These parameters included material properties, boundary, continuity, and compatibility conditions. During each sensitivity analysis these parameters were varied between their identified feasible bounds to quantify the effect on the predicted responses. Since the bounds for boundary, continuity, and compatibility conditions were assumed to be varied between qualitative bounds (i.e. for boundary conditions the bounds were varied between free and fixed), a translation from qualitative values to quantitative values was required. The sensitivity study helps to identify if the initial assumed bounds of the parameter are valid

Calibration

Upon completion of the data reduction, direct interpretation and analysis, the FE model was prepared for calibration efforts. IIS utilized MATLAB’s Optimization Toolbox to employ optimization algorithms to update the unknown parameters identified in the sensitivity studies in an effort to minimize the difference between observed and analytical results. For each parameter, the algorithm was allowed to explore all possible options over a range between the identified feasible parameter bounds. In the case of the load tests presented within this study, a least squares nonlinear optimization algorithm was chosen to minimize the discrepancies between the measured and predicted results.

Load Ratings

The following tables provide a summary of the controlling inventory and operating load ratings for each structure for flexure and shear.

Table 2: Summary of Flexural Load Ratings - Inventory

	Summary of Flexural Load Ratings - Inventory (tons)							
Truck	**Cedar Branch**	**Salem Creek**	**Springers Brook**	**Mill Creek**	**Toms River**	**D&R Canal**	**Raritan River**	**Game Creek**
HS20 (36T)	382 (156)	93 (123)	159	80	75	49	139	82
Type 3 (25T)	320 (131)	82 (119)	147	77	149	35	136	58
Type 3-3 (40T)	644 (264)	160 (282)	247	173	183	91	166	113
Type 3S2 (40T)	513 (210)	132 (174)	198	247	239	76	157	96
SU4 (27T)	424 (173)	78 (11)	152	28	86	52	133	55
SU5 (31T)	508 (208)	87 (124)	147	29	117	53	135	62
SU6 (35T)	517 (211)	95 (141)	172	28	99	58	136	65
SU7 (39T)	569 (233)	106 (159)	192	29	109	61	137	72
NRL (40T)	579 (237)	106 (175)	198	28	111	60	125	74

Table 3: Summary of Flexural Load Ratings - Operating

Summary of Flexural Load Ratings - Operating (tons)								
Truck	**Cedar Branch**	**Salem Creek**	**Springers Brook**	**Mill Creek**	**Toms River**	**D&R Canal**	**Raritan River**	**Game Creek**
HS20 (36T)	638 (261)	156 (206)	265	134	125	82	232	136
Type 3 (25T)	535 (219)	138 (200)	246	130	249	58	228	97
Type 3-3 (40T)	1083 (444)	267 (470)	412	290	305	152	277	189
Type 3S2 (40T)	856 (350)	221 (292)	331	412	399	127	262	161
SU4 (27T)	708 (290)	131 (190)	253	47	144	87	222	93
SU5 (31T)	813 (333)	150 (215)	291	54	166	89	255	104
SU6 (35T)	863 (353)	159 (235)	287	48	166	96	227	109
SU7 (39T)	950 (389)	177 (266)	320	49	183	101	229	120
NRL (40T)	968 (396)	178 (294)	331	46	186	101	208	123

Table 4: Summary of Shear Load Ratings - Inventory

Summary of Shear Load Ratings - Inventory (tons)								
Truck	**Cedar Branch**	**Salem Creek**	**Springers Brook**	**Mill Creek**	**Toms River**	**D&R Canal**	**Raritan River**	**Game Creek**
HS20 (36T)	26 (55)	34 (45)	253	44	58	127	119	74
Type 3 (25T)	27 (57)	28 (41)	152	52	56	127	112	60
Type 3-3 (40T)	46 (97)	55 (97)	331	83	102	266	221	119
Type 3S2 (40T)	67 (142)	45 (59)	281	98	78	180	184	92
SU4 (27T)	27 (57)	30 (44)	145	26	36	132	93	57
SU5 (31T)	29 (61)	35 (50)	166	29	47	150	110	65
SU6 (35T)	30 (63)	36 (53)	187	32	40	147	113	67
SU7 (39T)	36 (76)	40 (60)	208	32	38	187	127	70
NRL (40T)	40 (85)	38 (63)	215	34	36	180	130	69

Table 5: Summary of Shear Load Ratings – Operating

Summary of Shear Load Ratings - Operating (tons)								
Truck	**Cedar Branch**	**Salem Creek**	**Springers Brook**	**Mill Creek**	**Toms River**	**D&R Canal**	**Raritan River**	**Game Creek**
HS20 (36T)	44 (93)	58 (77)	423	81	97	212	199	124
Type 3 (25T)	46 (97)	47 (68)	254	94	94	213	187	100
Type 3-3 (40T)	77 (164)	93 (164)	553	150	170	445	369	199
Type 3S2 (40T)	112 (238)	76 (100)	470	175	131	300	307	153
SU4 (27T)	46 (97)	51 (74)	242	48	61	221	156	96
SU5 (31T)	49 (104)	59 (84)	278	48	79	251	184	108
SU6 (35T)	51 (108)	60 (89)	312	53	67	245	188	111
SU7 (39T)	60 (127)	67 (101)	348	53	63	313	212	118
NRL (40T)	68 (144)	64 (106)	359	57	61	300	218	116

* Calibrated refined rating presented in parentheses

Summary of Findings

- The finite element models are used to generate the demand used in the load ratings presented in this report. Where AASHTO standard specifications supply empirical distribution factors to distribute moment and shear to the load carrying members, the refined load ratings use the geometry and stiffness of the structure to distribute moment and shear. The model is creating its own distribution factor by taking advantage of the multidimensional nature of the models.
- HS20 ratings at the inventory level are greater than one for all structures in flexure and shear except

for Salem Creek (shear) and Cedar Branch (shear). These structures were load tested following the a-priori ratings to provide updated ratings.

- Shear ratings govern for seven of the eight rated structures
- A-priori refined load ratings indicate reserve flexural capacity for the structures. A-priori refined load ratings also indicate reserve shear capacity for most structures except for Cedar Branch and Salem Creek. Additional deterioration or damage will decrease the load ratings and the effects of additional damage and deterioration should be incorporated in future load ratings.
- The measured strains for Salem Creek indicate lower strains than predicted by the a-priori models
- The measured strains for Salem Creek also indicate a more favorable distribution of load than predicted by the a-priori model
- Measured strains for Cedar Branch show the a-priori model under predicts the magnitude of the measured strains within the slab. The magnitude of strain results in decrease in the flexural rating of the slab. However, the rating factors are still in excess of 1 for flexure for all rating vehicles. Shear rating remains the controlling load effect for this bridge.
- For Cedar Branch, the distribution of strains within the slab is generally limited to one third of the width of the structure. The reinforced concrete slab is comprised of three sections which use shear keys to lock the slab sections together. The load test data indicates the transfer of load between the sections is limited.

Recommendations

- The rating factors presented in this report use a conservative 25-30% impact factor applied to the live load effects. Reduction of this factor will increase the reported rating factors and inclusion of lower impact factors should be considered based on site specific conditions, such as the approach roadway condition, average daily truck traffic, and truck type.
- The capacity calculations could be updated using measured material properties. In cases where load ratings are low, sampling of material properties maybe performed and the actual material properties used in the capacity calculations. Improvement of the ratings is possible with the use of measured material properties especially in older reinforced concrete structures. Material sampling should be considered as a first step after conducting a finite element model rating.

Structure 0118150 – US206 over Cedar Branch

Description of Structure

Structure 0118150 - US206 over Cedar Branch is a single span reinforced concrete slab structure located in Hammonton, NJ. The structure spans Cedar Brook, a small stream that is non-navigable to marine traffic with a maximum depth of approximately three feet. The deck slab is reinforced concrete with bituminous overlay and concrete sidewalks and railings. Three expansion joints run in the direction of the span of the superstructure. The substructure consists of reinforced concrete breast walls and wing walls. It was built in 1953 and has not since undergone any major widening or rehabilitation. It carries two lanes of traffic, with one lane in each direction. The bridge has an NBI rating of satisfactory (6) and is currently has a load factor rating at the HS20 inventory level rating of 20 tons. A snapshot of the structure is shown in Figure 5.

Figure 5: Overview of Structure 0118150 – US206 over Cedar Branch

Figure 6: Structure 0118150 – Aerial View

Description of FE Model

Due to the monolithic construction of the bridge, a solid element model was selected to represent the geometry of the structure. The model geometry was first created in AutoCAD, then imported into strand7 as an .iges geometry file. The model was discretized by first surface auto meshing into quad4 plate elements with a max edge length of 6”, then solid auto meshing into tetrahedral elements also with a maximum edge length of 6”. Several views of the model are shown in Figure 7 through Figure 8.

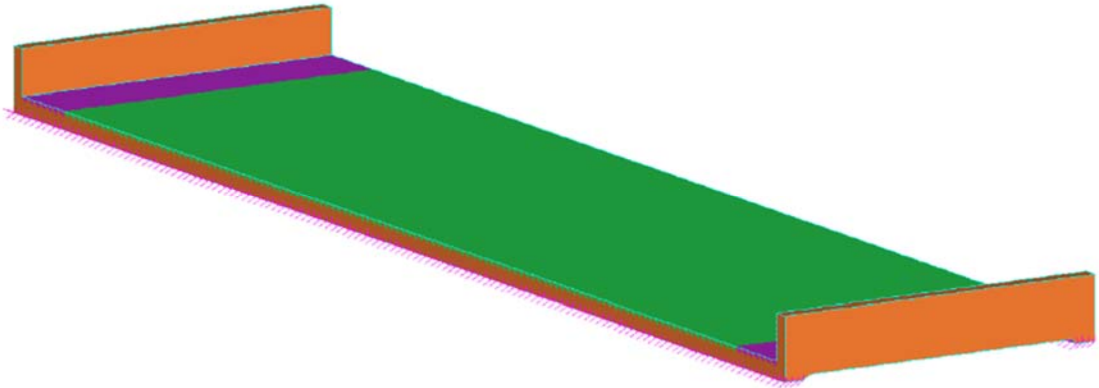


Figure 7: Structure 0118150 – Dimetric Model View

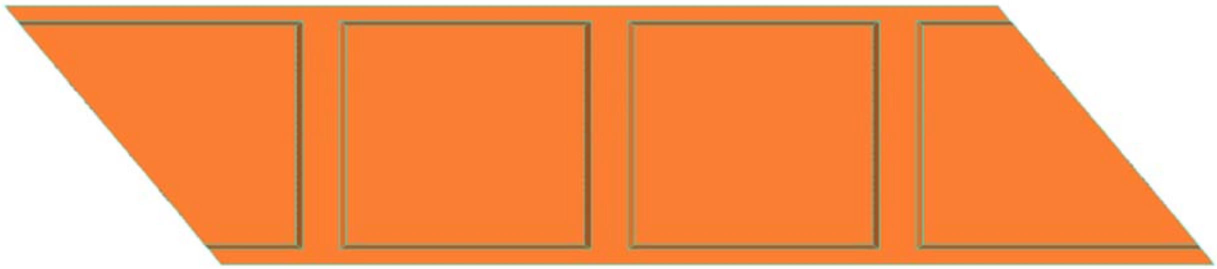


Figure 8: Structure 0118150 – Underside

Overall Geometry

Dimensions

Length – 17’ along stiffening ribs

Width– 65’4”

Skew - 50^0

Slab Depth – 1’-1”

Expansion Joint Ribs –Three total spaced at 16’4” at center of deck.

Expansion Joints

The structure has four sections of reinforced concrete slab that are separated by expansion joints. These expansion joints run normal the bridge and have trapezoidal concrete ribs on the underside of the deck. These ribs have been included in the model, but the actual expansion joint gaps have not been included in the model geometry and the deck has been modeled as continuous. The original drawings show a slight crown to the deck which has not been included in the model geometry. See Figure 9 for dimensions of the expansion joint ribs.

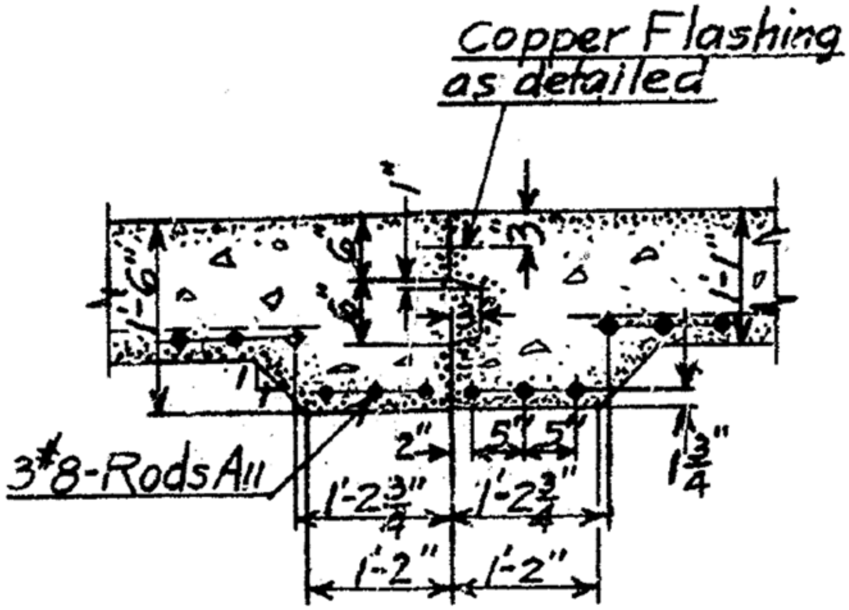


Figure 9: Structure 0118150 – Expansion Joint and Stiffening Ribs

Railing

The structure has a solid reinforced concrete railing with a rounded top. In the model, this has been simplified into a rectangle of similar dimensions. From the contract drawings, the railing reinforcement is embedded into the deck to a depth of approximately 2', so compatibility was assumed between the two elements. The ends of the railings consist of a portion that is 3" wider for a length of 2-6". For model simplicity, this change in geometry was not modeled and instead the 9" intermediate width was used for the entire length of the railing. See Figure 10 for railing details.

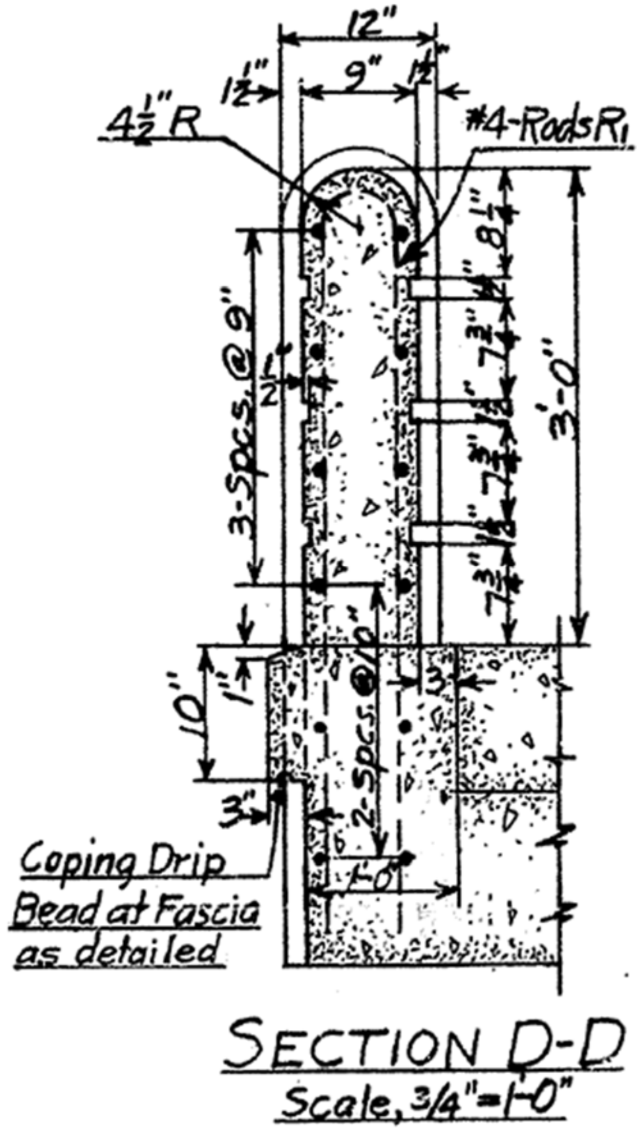


Figure 10: Structure 0118150 – Reinforced Concrete Railing

Sidewalk:

A concrete sidewalk is located on both sides of the bridge. The drawings do not indicate steel reinforcement in the sidewalk. The sidewalk was poured separately from the deck and railing, with no connection details between the two besides roughing the surface of the deck below the sidewalk to create a bond. Therefore, the connection between the sidewalk and the deck were not modeled compositely. The sidewalk was modeled as a non-structural mass contributing dead load to the model but no additional stiffness. For more details see Figure 11.

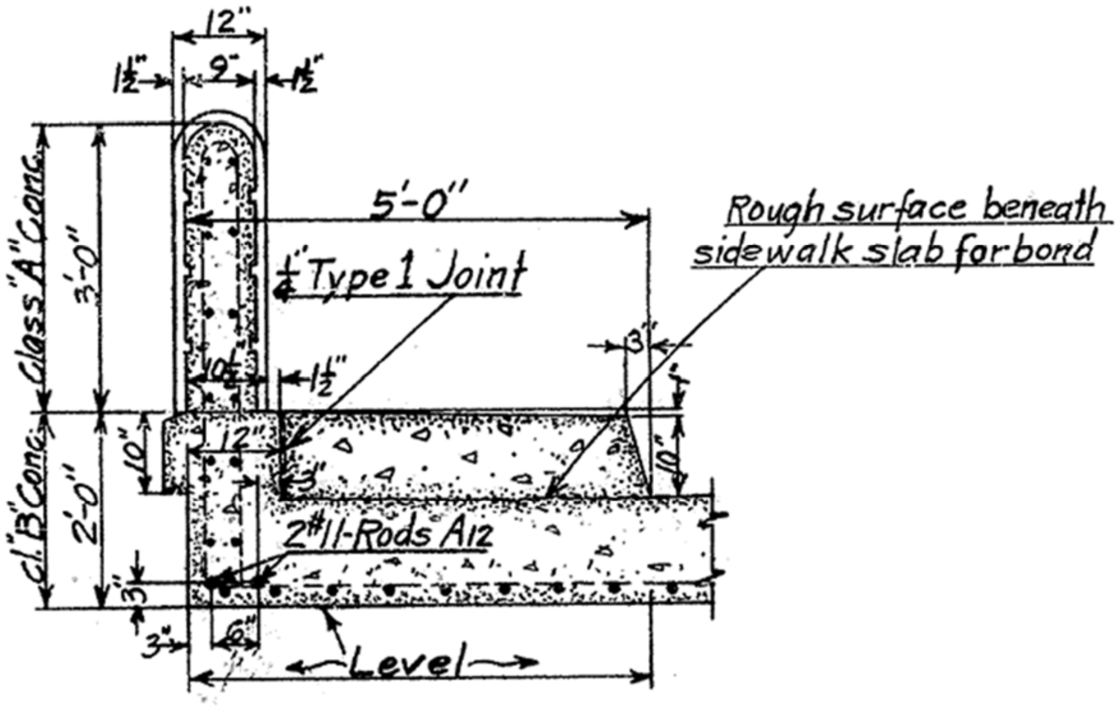


Figure 11: Structure 0118150 – Deck and Sidewalk

<u>Asphalt Overlay:</u>

The asphalt overlay is not indicated in the drawings, but the inspection report notes the curb heights in the deck cross section on page 16-26. From the drawings and inspection report, it is known that the distance from the top of the deck to the top of curb is 10”, so the thickness of the asphalt overlay can be determined. The curb height is 6” on one side of the bridge and 4” on the other. These values were averaged to get 5”, making the thickness of the asphalt 5”. This was then applied as a nonstructural mass with a unit weight of 145 pounds per cubic foot.

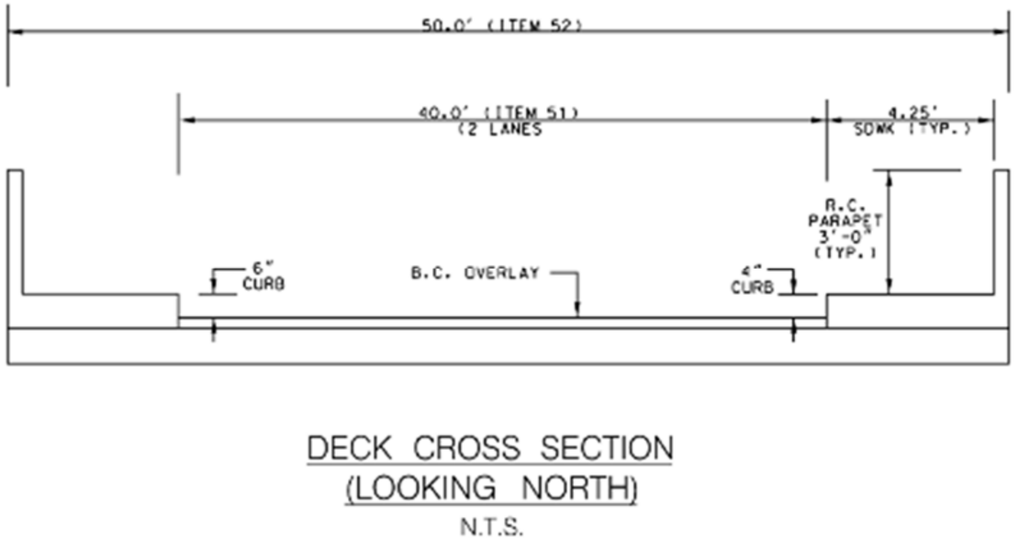


Figure 12: Structure 0118150 – Cross Section Sketch from Inspection Report

Material Properties

Given the age of this bridge, it was difficult to find properties of concrete that are specified on the drawings. The concrete properties for this model were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi as shown in Figure 13. The specified reinforcing steel yield strength was not given in the plans so an assumed value of 33 ksi was used based on the data of construction. The yield strength was selected from Figure 14.

Table 6A.5.2.1-1—Minimum Compressive Strength of Concrete by Year of Construction

Year of Construction	Compressive Strength, f'_c, ksi
Prior to 1959	2.5
1959 and Later	3.0

Figure 13: AASHTO Specified Concrete Compressive Strength for Bridges with Unknown Details

Table 6A.5.2.2-1—Yield Strength of Reinforcing Steel

Type of Reinforcing Steel	Yield Strength, f_y, ksi
Unknown steel constructed prior to 1954	33.0
Structural grade	36.0
Billet or intermediate grade, Grade 40, and unknown steel constructed during or after 1954	40.0
Rail or hard grade, Grade 50	50.0
Grade 60	60.0

Figure 14: AASHTO Steel Reinforcement Yield Strength for Bridges with Unknown Details

Miscellaneous Calculations

Unit Weight of Fill / Asphalt Topping:

$$Deck\ Pressure\ due\ to\ Sidewalk = 149\ pcf \times \left(\frac{10"}{12"}\right) ft = 124.17\ psf$$

$$Deck\ Pressure\ due\ to\ Asphalt\ Overlay = 145\ pcf \times \left(\frac{8"}{12"}\right) ft = 96\ psf$$

Concrete Modulus

$$f'_c = 57{,}000\sqrt{2500} = 2850000\ \text{psi} = 2850\ \text{ksi}$$

Assumptions, Limitations

- The crown of the deck has been modeled as a flat surface.
- The expansion joints have not been modeled.
- The balustrades and curb have been simplified to basic rectangular shapes.
- All boundary conditions have been modeled as pinned. This is because the drawings indicate that the superstructure is dowelled into the substructure.
- The sidewalk and bituminous overlay have been modeled as non-structural masses and therefore contribute no stiffness.
- The thickness of the asphalt overlay changes across the length of the deck according to the inspection report deck cross section – it was assumed this change was linear across the deck profile.
- The thinnest portion of the slab was used in the capacity calculations

- The impact factor was calculated at 30% and this is conservative given a visual inspection of the bridge approach conditions shown in Figure 5.

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles (HS20, Type 3, 3S2, 3-3, and SU4-SU7 and NRL) and the critical moment and shear demands extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 6 and the a-priori model based ratings versus those provided in the inspection report are given in Table 7.

Table 6: Structure 0118150 – A-Priori Model Based Load Ratings

US206 Over Cedar Branch - A-Priori Load Ratings				
	Flexure		Shear	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	382	638	26	44
Type 3 (25T)	320	535	27	46
Type 3-3 (40T)	644	1083	46	77
Type 3S2 (40T)	513	856	67	112
SU4 (27T)	424	708	27	46
SU5 (31T)	508	813	29	49
SU6 (35T)	517	863	30	51
SU7 (39T)	569	950	36	60
NRL (40T)	579	968	40	68

Table 7: Structure 0118150 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model - Shear		Inspection Report - Flexure	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	26	44	20	34
Type 3 (25T)	27	46	17	29
Type 3-3 (40T)	46	77	34	57
Type 3S2 (40T)	67	112	28	47

The a-priori model based load ratings indicate the structure has adequate flexural capacity but the shear capacity is uncertain. The bridge has a very short span from abutment to abutment of approximately 17 feet which precludes the bridge from being loaded fully by full weight of longer wheelbase vehicles. Shorter wheelbase vehicles with heavy axle weights will control the ratings for this structure. The model based load ratings are governed by shear while the load ratings reported in the inspection report are governed by the flexural response of the bridge. Given the uncertainty with the a-priori load ratings especially in shear, instrumentation and testing were necessary to provide data for model calibration and calibrated model based load ratings.

Sensitivity Analysis

A sensitivity analysis was conducted to identify the model parameters that directly influence the model responses in the T-Beam structure. The parameters chosen were the longitudinal support stiffness, transverse support stiffness, vertical support stiffness, rotational support stiffness about the transverse axis, and concrete modulus of elasticity. The concrete modulus was based on varying the compressive strength of concrete from 1000 psi to 10000 psi. The bounds for each of these parameters is shown in Table 8. Example sensitivity plots are given in Figure 15 and Figure 16.

Table 8: Structure 0118150 – Model Calibration Parameter Bounds

Parameter	Lower Bound	Upper Bound
Longitudinal Support Stiffness - East	Free	Fixed
Longitudinal Support Stiffness - West	Free	Fixed
Vertical Support Stiffness	Free	Fixed
Rotational Support Stiffness	Free	Fixed
Modulus of Concrete	1802	5700

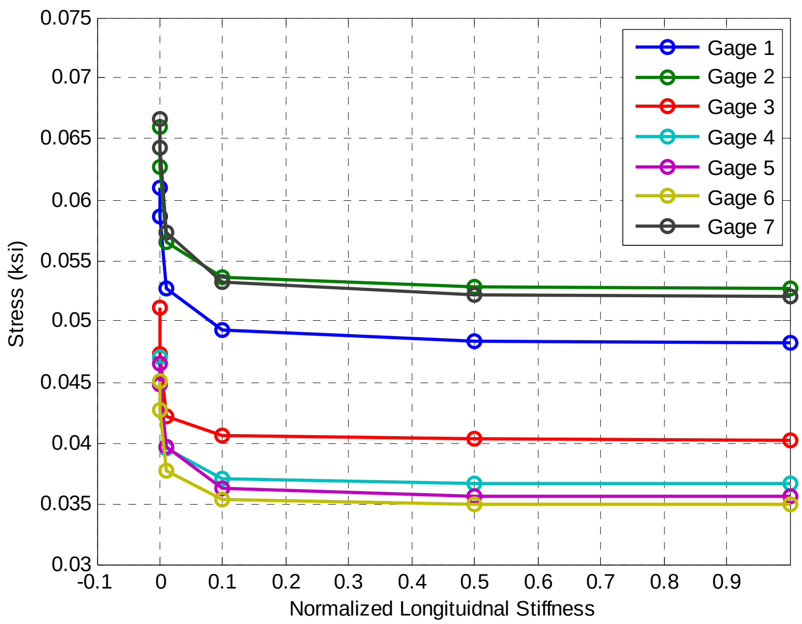


Figure 15: Structure 0118150 – Example Sensitivity Plot – Longitudinal Support Stiffness

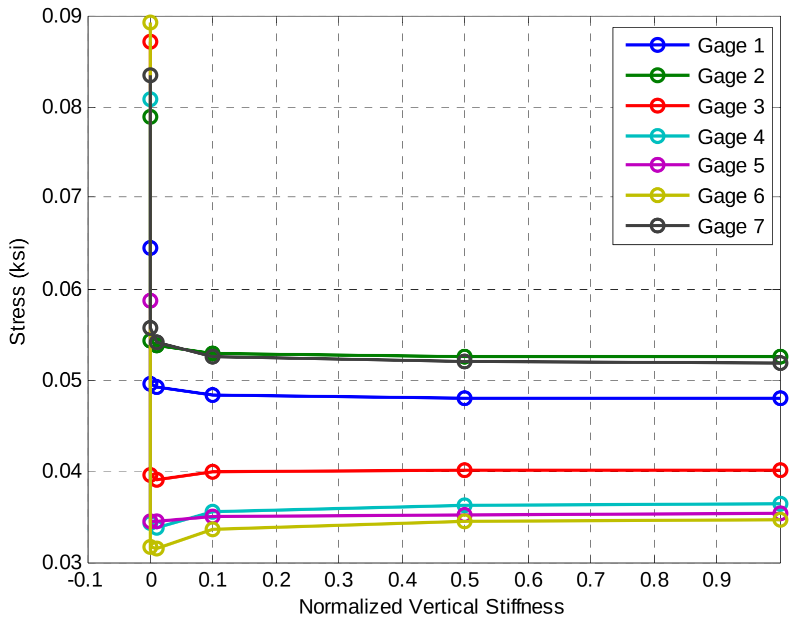


Figure 16: Structure 0118150 – Example Sensitivity Plot – Vertical Support Stiffness

Instrumentation and Load Testing

Due to the low shear ratings from the a-priori finite element model, a load test was scheduled for this structure. The instrumentation was designed to capture the distribution of strain in the slab under a known load and also to identify whether possible shear cracks on the edge of the slab were active under the same known loads. A photo of the sensor installation to measure the crack on the side of the slab is shown in Figure 17 while the test truck is shown in Figure 18.

A loaded truck of known weight (64 kips) was provided for the test and served as the vehicle used for all testing. Eight load cases were used to test the structure. The first load case involved positioning the truck in the upstream lane of traffic at quarter, mid, and three-quarter span. This pattern was repeated in the downstream lane, along the downstream curb line, along the upstream curb line, and also straddling both traffic lanes for a total of 18 loading configurations. The three other load cases were crawl speed tests in the upstream lane, downstream lane, and straddling configurations. The truck was positioned off of the bridge and subsequently proceeded to crawl over the bridge at approximately 5mph.

Figure 17: Sensor Installation on Structure 0118150

Figure 18: Truckload Test on Structure 0118150

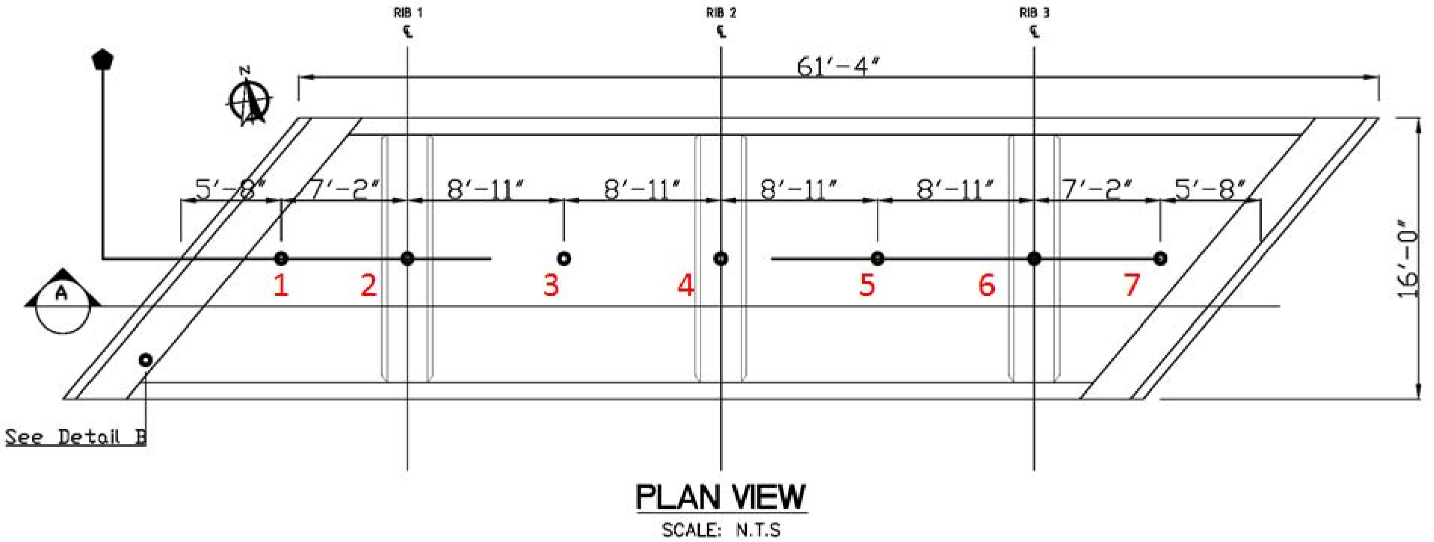


Figure 19: Structure 0118150 – Instrumentation Plan View

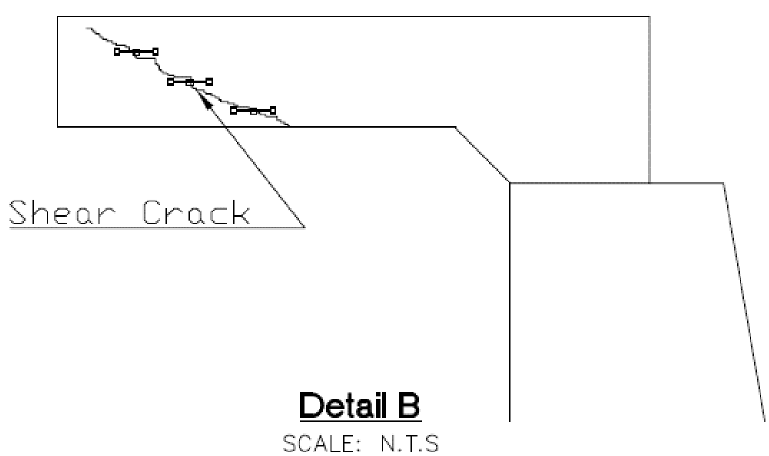


Figure 20: Structure 0118150 – Instrumentation Detail at Possible Shear Crack

An example of the data collected from the load test is given in Figure 21. The time histories show three plateaus which correspond to the three locations the truck was positioned during each load case (downstream lane, upstream lane, lane straddle). To obtain the nominal strain at each position the average strain once the readings had stabilized were taken at each plateau and from each gage. The values were used further in the model calibration procedure.

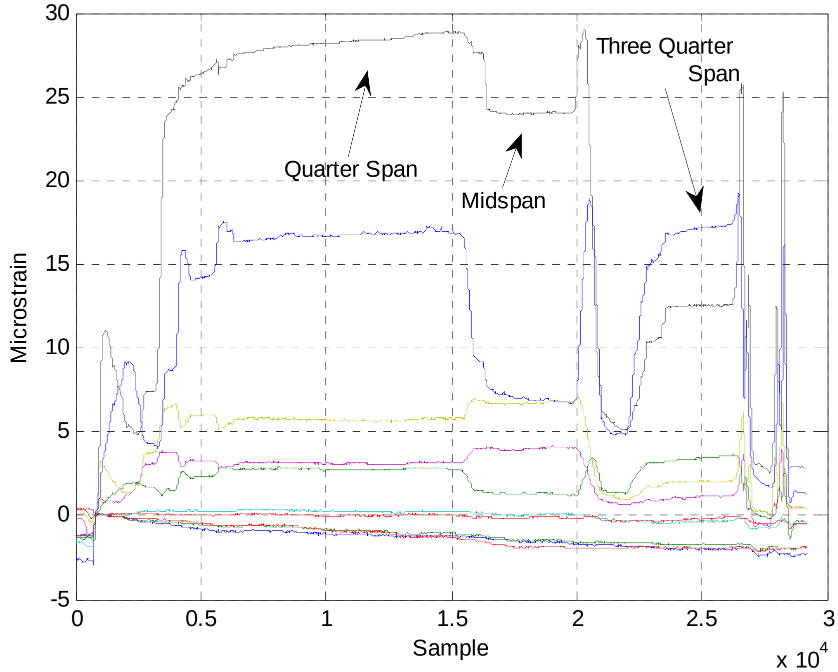


Figure 21: Example Strain Data from Load Test on Structure 0118150

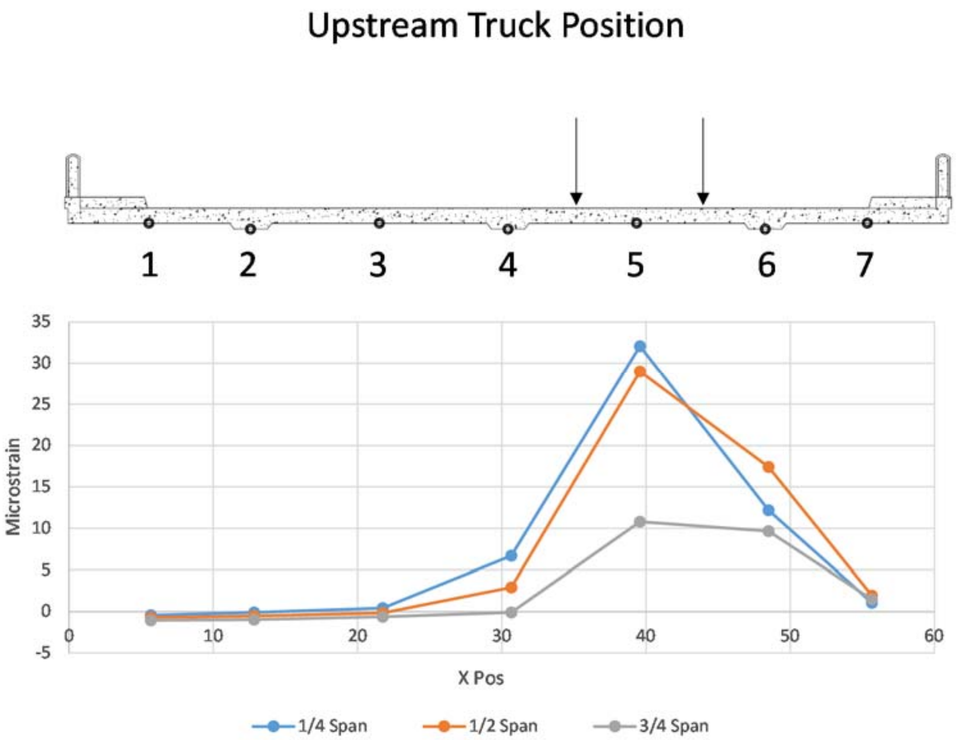


Figure 22: Distribution of Strain versus Upstream Lane Truck Positions - Structure 0118150

When the truck is in the upstream lane the strain is resisted by the slab directly under the loads. The drop off in measured strain at gage 4 is due to the longitudinal joint along the centerline of the stiffening rib where gage 4 was installed. The gage was installed to the left of this joint away from the loads shown in Figure 22. This can also be shown when the load is in the downstream lane as is shown in Figure 23. Gage 4 measures more strain showing that minimal load is transferred across the longitudinal joint at each rib.

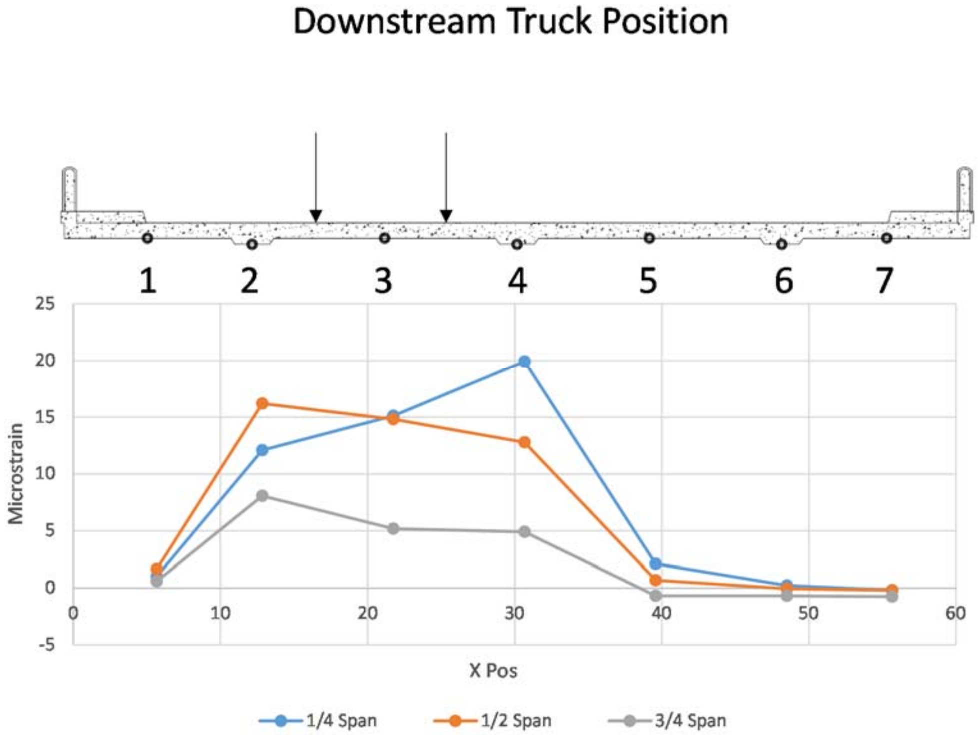


Figure 23: Distribution of Strain versus Downstream Lane Truck Positions - Structure 0118150

The three gages installed over the crack on side of the slab indicated very small strain values even when the truck was positioned against the curb line. The maximum strain observed in these gages was -4 microstrain and there did not appear to be any opening of the crack during the 8 load cases. The strain readings were typically negative indicating the gages were being compressed as the crack closed. As a comparison of the a-priori models' ability to predict the measured response, a comparison table is given for the load case where the test truck was positioned at the midspan of the upstream lane.

Table 9: Structure 1703152 – Comparison of A-Priori Model Predicted Stress versus Measured Stress – Upstream Load Case

	Model (ksi)	Measured (ksi)	Error (%)
Gage 4	0.0024	0.0047	48.93%
Gage 5	0.0163	0.0308	47.05%
Gage 6	0.009	0.0275	67.27%
Gage 7	0.001	0.004	75.92%

Model Calibration

The data obtained from the load test was used as a baseline with which to modify the finite element model. The following details the modifications made to the model to accurately reflect the measured data. Four parameters were identified as being influential to the responses of interest that were measured during the load test. These parameters included the longitudinal support stiffness at the east end of the beams, longitudinal support stiffness at the west end of the beams, rotational support stiffness, and modulus of concrete. The parameters and their bounds are shown in Table 8. During the sensitivity analysis these parameters were varied between their bounds to quantify the effect on the response at proposed instrumentation locations. The final model parameters are given in Table 10. After calibration, the errors between the measured and predicted stresses have been reduced to under 20%.

Table 10: Structure 0118150 – Model Calibration - Final Parameter Values

Parameter	Value
Longitudinal Support Stiffness - East	0.873
Longitudinal Support Stiffness - West	0.721
Rotational Support Stiffness	0.005
Modulus of Concrete	3256

Table 11: Structure 1703152 – Comparison of Calibrated Model Predicted Stress versus Measured Stress– Upstream Load Case

	Model (ksi)	Measured (ksi)	Error (%)
Gage 4	0.0039	0.0047	17.02%
Gage 5	0.0281	0.0308	8.72%
Gage 6	0.0234	0.0275	14.91%
Gage 7	0.0034	0.004	18.14%

Calibrated Model Load Rating

The calibrated model based load ratings following the load testing and model experiment are given in Table 19.

Table 12: Structure 0118150 – Calibrated Load Ratings

US206 Over Cedar Branch - Calibrated Model Load Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	157	262	55	94
Type 3 (25T)	131	219	58	98
Type 3-3 (40T)	264	444	98	164
Type 3S2 (40T)	210	351	143	239
SU4 (27T)	174	290	58	98
SU5 (31T)	208	333	62	104
SU6 (35T)	212	354	64	109
SU7 (39T)	233	390	77	128
NRL (40T)	237	397	85	145

Since the magnitude of strain within the slab is increased and the distribution of flexural strain within in the slab has been increased, the load ratings are updated to reflect these changes. The flexural ratings have decreased from the initial a-priori model but still are in excess of the respective vehicle weights at the inventory level of evaluation. The shear ratings have increased above one due to better distribution of the strains within the slab.

Structure 1703152 – US40 over Salem Creek

Description of Structure

Route 40 over Salem Creek is a single span structure in Pilesgrove, Salem County, NJ that crosses Salem Creek, a small non-navigable stream with a depth of less than 1' according to the inspection report. The bridge is a combination of an original reinforced concrete T-Beam structure and a reinforced concrete slab that was added as part of a widening project. The original bridge was constructed in 1919 and the widening project occurred in 1929. The widening project left the majority of the structure in place and added an additional 6'-10" to each side of the bridge. The drawings indicate that the original substructure's wing walls and superstructure parapet were demolished during the widening project. The bridge has a concrete parapet and handrail and no sidewalk. An overview of the structure is shown in Figure 24 and an aerial view of the bridge is given in Figure 25.

Figure 24: Overview of Structure 1703152 – US40 over Salem Creek

Figure 25: Structure 1703152 – Aerial View

Description of FE Model

A solid model was chosen for this bridge. The slab portion of the bridge was most accurately represented using brick elements in Strand7. A plate line model could have been used for the T-beam construction of the original bridge, but for consistency's sake brick elements were used for the entire structure. The model is separated into three sections, each assigned to a group. These were first modeled in AutoCAD, exported to an .iges files, and then imported separately. Each section was discretized by first surface auto meshing into shell elements with a max edge length of 4", then solid auto meshing into brick tetrahedral elements also with a maximum edge length of 4". This resulted in the model shown in Figure 26 with the mesh shown in Figure 27.

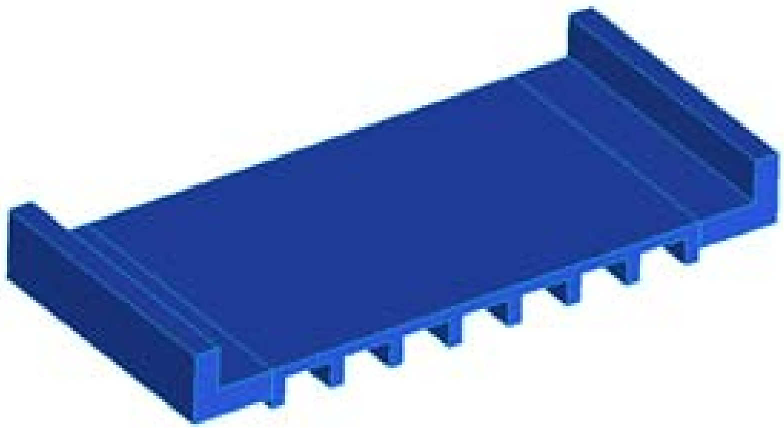


Figure 26: Structure 1703152 – Dimetric Model View

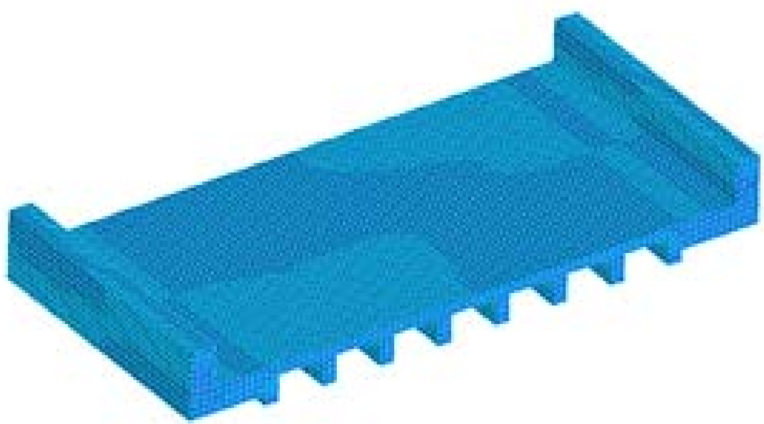


Figure 27: Structure 1703152 – Mesh

Overall Geometry

Dimensions

Original T-Beam Structure

Length - 20’ clear span, 24’ out to out. Modeled to center of bearing surface, so length in model is 21’ 6”.

Width – 24’-10” x 2*(2-8 ½” + 1’) = 32’-3”, rounded to 32’

Skew – The Bridge is not skewed

Beams and Deck

The beam and deck dimensions and reinforcement details are shown in Figure 28.

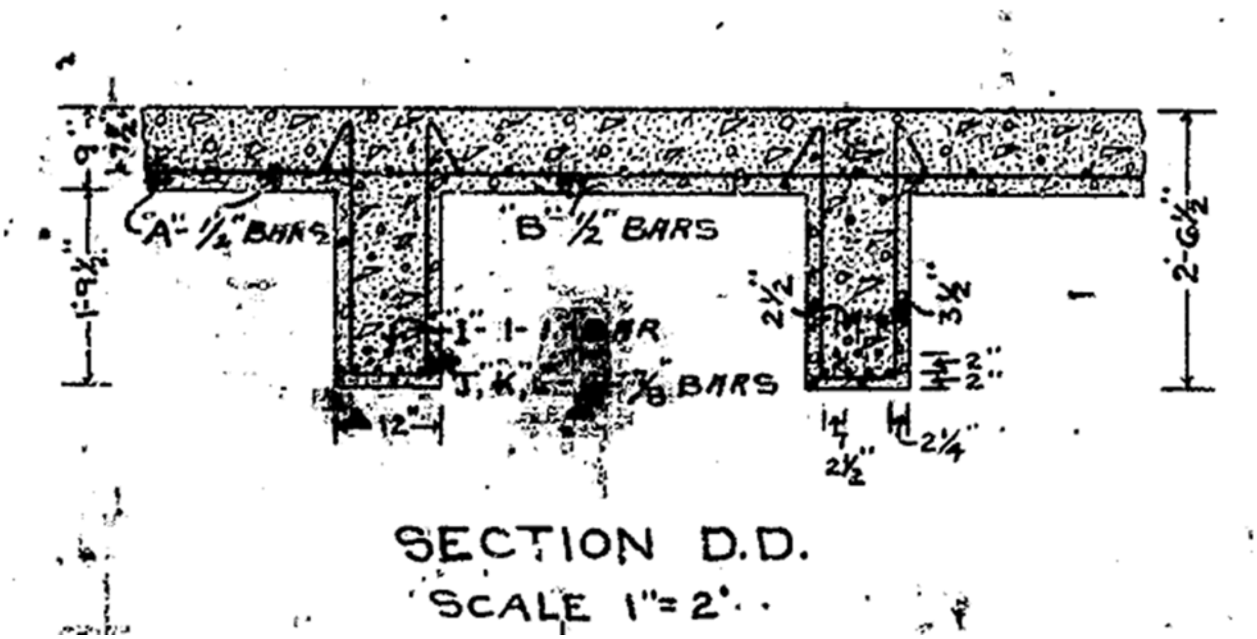


Figure 28: Structure 1703152 – Beam and Deck Dimensions

Widened Structure

Width – 5’-6” + 2” + 2” = 5’-10”

Length – 20’-0” clear span, 22’8” out to out. Modeled to center of bearing surface, so length in model is 21’6”.

Slab Thickness – 1’-9 ½”

Parapet Dimensions -

- Width - 1’-8” + 4” – 3” = 21”
- Height – 3’-10 7/8” + 7” = 53.875”

Depth of Fill –. The inspection report listed the curb height to be 8”. The depth of fill was determined by subtracting the height of curb and the thickness of the slab from the overall height of the parapet:

- 53.875”-21.5”-8” = 24.375”

Material Properties

Given the age of this bridge, it was difficult to find properties of concrete that are specified on the drawings. The concrete properties for this model were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi as shown in Figure 13.

Miscellaneous Calculations

Unit Weight of Fill / Asphalt Topping:

$$Deck\ Pressure\ due\ to\ fill\ \&\ asphalt\ topping = 145\ pcf\ \times \left(\frac{24.375}{12}\right) ft^3 = 294.5\ psf$$

Rotational Stiffness Calculation:

$$\frac{152.45\ kip - ft}{0.00093\ radian} = 163{,}874\ \frac{kip - ft}{radian}$$

Concrete modulus:

$$f'_c = 57{,}000\sqrt{2500} = 2850000\ \text{psi} = 2850\ \text{ksi}$$

Assumptions, Limitations

- The change in elevation when the original structure meets the new structure is not indicated anywhere on the drawings. It was assumed that during construction of the widening project, they poured the top of the deck for new structure to be flush with the top of the deck of the old structure.
- The interface between the old and new structure has been modeled as a continuous structure.
- The asphalt and fill contribute no stiffness (modeled as structural mass).
- The unit weight of the fill and asphalt has been modeled as a constant 145 pcf – this is accurate for the asphalt topping but the composition of the fill is an unknown. It’s likely that this value is too high for the fill.
- The additional mass from the crown of the roadway has not been omitted
- The railing that sits atop the parapet has not been modeled. This is justified simply by assuming that the stiffness of the handrail will be negligible compared to that of the slab and parapet.
- Bearing fixity of the original structure has been modeled using rotational spring constant from a representative slice of the bridge
- The substructure is not included in the solid model

- The impact factor was selected as 30% since the riding surface and its influence on the impact of trucks on the bridge superstructure is unknown.

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described before and the critical moment and shear demands extracted from the model and input into the load factor rating equations. An example of truckload case that would produce a maximum moment effect is shown in Figure 29 The Load Factor ratings developed from the a-priori finite element model are given in Table 13 while a comparison of the controlling rating from the model and those reported in the inspection report are given in Table 14.

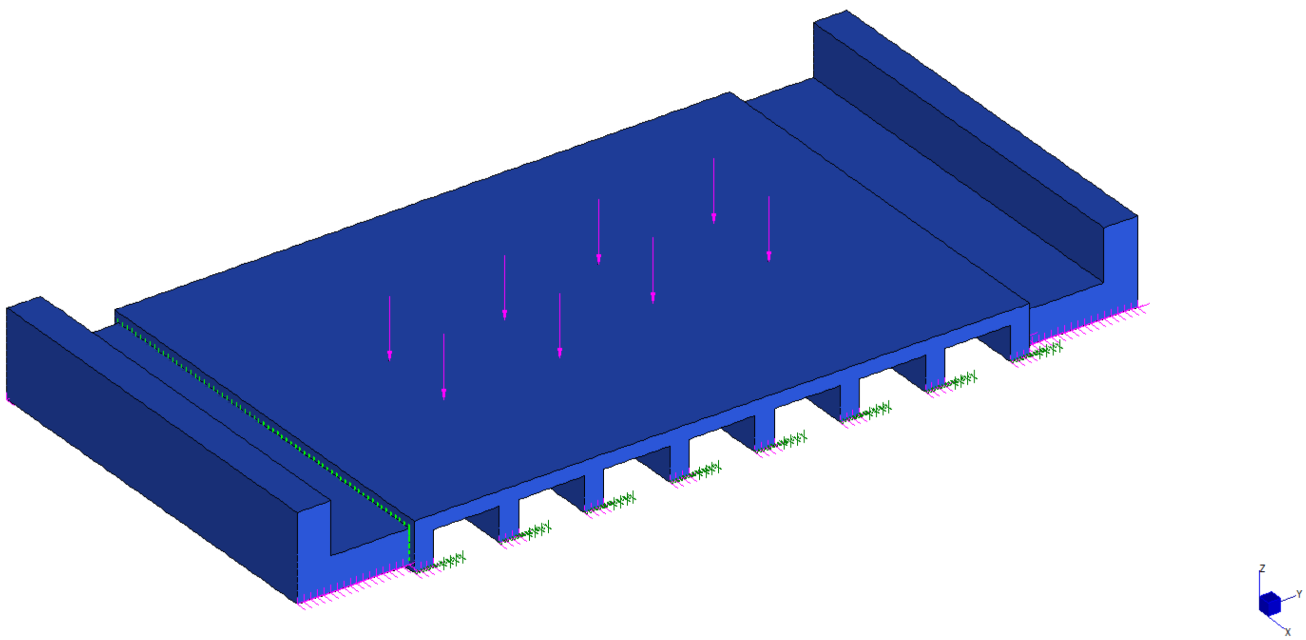


Figure 29: Structure 1703152 – Example Truckload Applied to Model

Table 13: Structure 1703152 – A-Priori Model Load Factor Ratings

Salem Creek - A-Priori Model Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	93	156	34	58
Type 3 (25T)	82	138	28	47
Type 3-3 (40T)	160	267	55	93
Type 3S2 (40T)	132	221	45	76
SU4 (27T)	78	131	30	51
SU5 (31T)	87	150	35	59
SU6 (35T)	95	159	36	60
SU7 (39T)	106	177	40	67
NRL (40T)	106	178	38	64

Table 14: Structure 1703152 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	34	58	25	42
Type 3 (25T)	28	47	20	34
Type 3-3 (40T)	55	93	32	54
Type 3S2 (40T)	45	76	39	66

The comparison of ratings shows the rating for the FE model are controlled by the shear capacity while the inspection report does not specify whether shear or moment controls the rating.

Sensitivity Analysis

A sensitivity analysis was conducted to identify the model parameters that directly influence the model responses in the T-Beam structure. The parameters chosen were the longitudinal support stiffness, transverse support stiffness, vertical support stiffness, rotational support stiffness about the transverse axis, and concrete modulus of elasticity. The concrete modulus was based on varying the compressive strength of concrete from 1000 psi to 10000 psi. The bounds for each of these parameters is shown in Table 15. An example sensitivity plot is given in Figure 30.

Table 15: Structure 1703152 – Model Calibration Parameter Bounds

Parameter	Lower Bound	Upper Bound
Longitudinal Support Stiffness - East	Free	Fixed
Longitudinal Support Stiffness - West	Free	Fixed
Rotational Support Stiffness - East	Free	Fixed
Rotational Support Stiffness - East	Free	Fixed
Modulus of Concrete -Beams	1802	5700
Modulus of Concrete -Deck	1802	5700

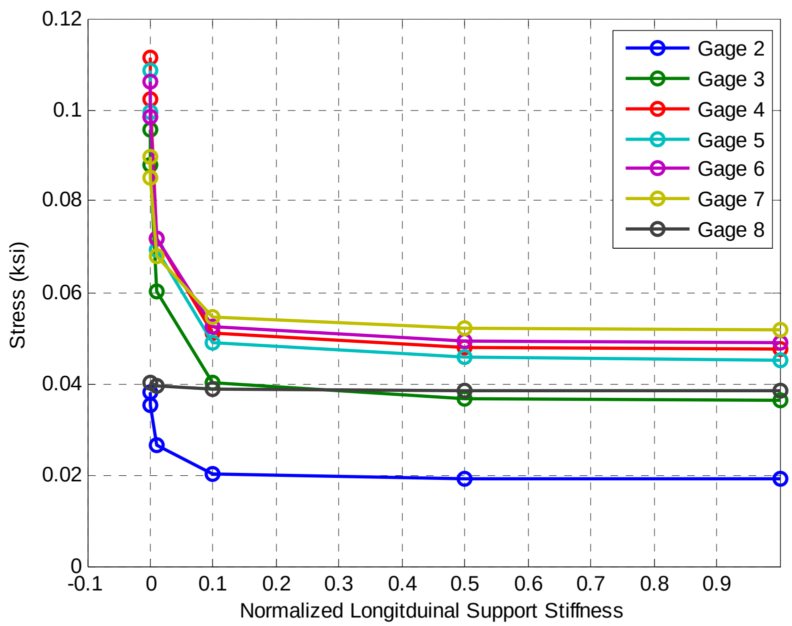


Figure 30: Structure 1703152 – Example Sensitivity Plot – Longitudinal Support Stiffness

Instrumentation and Load Testing

Due to the lower ratings developed from the a-priori finite element model, a load test was scheduled for this structure. The instrumentation was designed to capture the strain in the T-beams under a known load. To capture the strain responses due to the loaded truck and also ambient traffic, an instrumentation plan was developed to measure the bottom fiber strains at midspan of the T-beams. Vibrating wire train gages were installed on the eight different T-Beams at various heights on the section. The variable installation height on the section was due to the condition of the concrete. In order to measure reliable strain values, unsound concrete and cracks are to be avoided. Therefore, the instrumentation was installed on sound concrete which necessitated the variability in installation location and height.

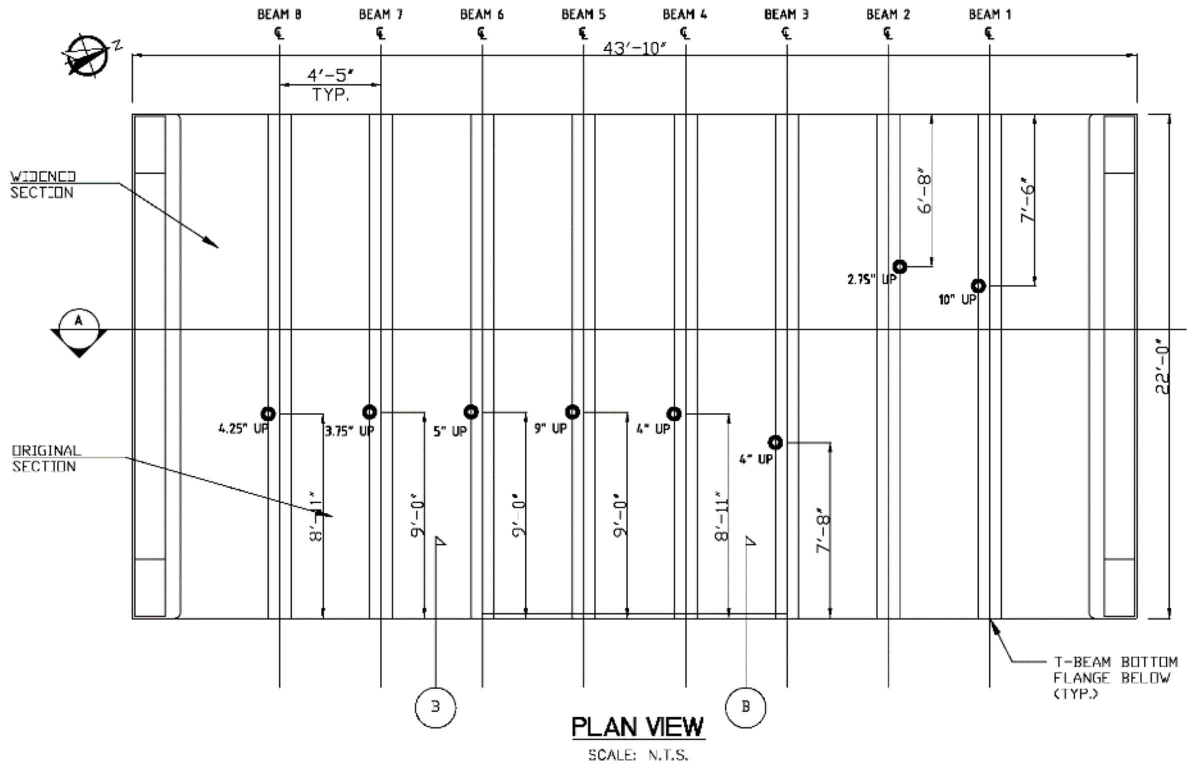


Figure 31: Sensor Instrumentation Plan for Structure 1703152

A loaded truck of known weight (64 kips) was provided for the test and served as the vehicle used for all testing. Since one truck was available, 6 load cases were implemented during the test. The first load case involved positioning the truck in the upstream lane of traffic at quarter, mid, and three-quarter span. This pattern was repeated in the downstream lane and also straddling both traffic lanes. The three other load cases were crawl speed passes in each traffic lane. The truck crawled across the bridge in the upstream, downstream, and straddling configurations.

Figure 32: Truckload Test on Structure 1703152

The strains shown in Figure 33 are for when the truck was positioned in the upstream lane and are shown as an example of the readings taken under the loaded truck. A comparison of the a-priori predicted stresses versus those measured during the test are given in Table 16. The comparison shows the model over predicts the stresses in the beams directly under the load while it under predicts the stress in the beams away from the load. The measured data indicates more load distribution than the a-priori model predicts.

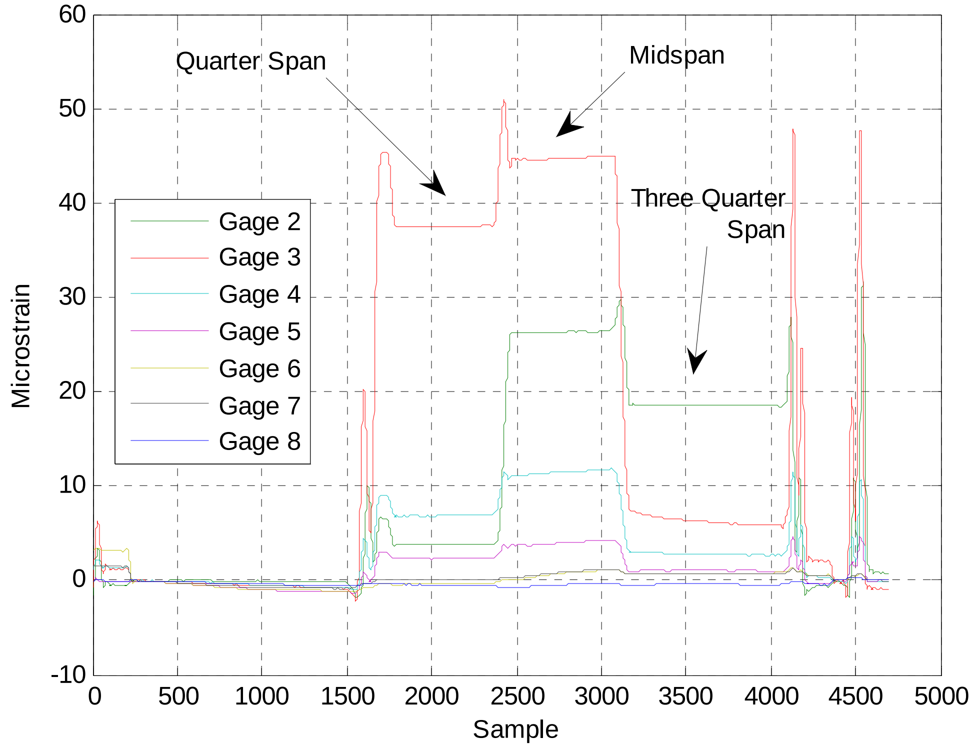


Figure 33: Example Strain Data from Load Test on Structure 1703152

Table 16: Structure 1703152 – Comparison of A-Priori Model Predicted Stress versus Measured Stress

	Model (ksi)	Measured (ksi)	Error (%)
Gage 2	0.054	0.025	-114.75%
Gage 3	0.118	0.091	-29.24%
Gage 4	0.079	0.050	-56.36%
Gage 5	0.017	0.047	63.29%
Gage 6	0.003	0.015	79.12%
Gage 7	0.001	0.010	94.06%
Gage 8	0.001	0.004	71.93%

Model Calibration

The data obtained from the load test was used as a baseline with which to modify the finite element model. The following details the modifications made to the model to accurately reflect the measured data. Four model parameters were identified as being uncertain including the longitudinal support stiffness at the east end of the beams, longitudinal support stiffness at the west end of the beams, rotational support stiffness and lateral support stiffness. . During the sensitivity analysis these parameters were varied between their identified bounds to quantify the effect on the stresses at the measured locations. The parameters and their final values are shown in Table 17. A comparison of the calibrated model predicted stresses versus the measured stresses is given in Table 18. The calibrated model predicts the stresses in the T-beams directly under the load when the truck is placed in the upstream lane within 15% error. The calibrated model indicates the stresses in the T-beams away from the load small which agrees with the distribution of strain obtained from the truckload test indicating that the load is applied to the T-beams directly in the vicinity of the load and.

Table 17: Structure 1703152 – Final Model Parameter Values - Normalized

Parameter	Value
Longitudinal Support Stiffness	0.011
Longitudinal Support Stiffness – West	0.905
Rotational Support Stiffness – East	0.015
Rotational Support Stiffness – West	0.003
Modulus of Concrete –Beams	0.567
Modulus of Concrete –Deck	0.747

Table 18: Structure 1703152 – Comparison of Calibrated Model Predicted Stress versus Measured Stress

	Model (ksi)	Measured (ksi)	Error (%)
Gage 2	0.028	0.025	-12.88%
Gage 3	0.097	0.091	-5.99%
Gage 4	0.054	0.050	-7.56%
Gage 5	0.046	0.047	1.27%
Gage 6	0.018	0.015	-21.48%
Gage 7	0.005	0.010	49.44%
Gage 8	-0.001	0.004	118.87%

Calibrated Model Load Rating

The calibrated model based load ratings are shown in Table 19.

Table 19: Structure 1703152 – Calibrated Model Based Load Ratings

Salem Creek - Calibrated Model Ratings				
	Flexure		Shear	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	123	206	45	77
Type 3 (25T)	119	200	41	68
Type 3-3 (40T)	282	470	97	164
Type 3S2 (40T)	174	292	59	100
SU4 (27T)	113	190	44	74
SU5 (31T)	137	237	50	84
SU6 (35T)	141	235	53	89
SU7 (39T)	159	266	60	101
NRL (40T)	175	294	63	106

The calibrated model load ratings reflect the improved distribution of strain between T-beams that was revealed when the distribution of strain from the initial model and experiment were compared. The improved distribution has provided additional flexural capacity along with improved shear capacity so that all ratings are greater than the respective vehicle weight.

Structure 0324152 – US206 over Springers Brook

Description of Structure

Bridge 0324152 is a three span cast in place simply supported reinforced concrete slab bridge along US Route 206 crossing Springers Brook near Shamong, NJ. The structure was built in 1929 and consists of a 16 inch deep reinforced concrete slab supporting an asphalt overlay. The bridge carries two lanes of US206 over Springers Brook. An overview of the structure is shown in Figure 34 and an overhead aerial view is shown in Figure 35.

Figure 34: Overview of Structure 0324152 – US206 over Springers Brook

Figure 35: Structure 0324152 – Aerial View

Description of FE Model

A solid model was chosen for this bridge due to its monolithic construction. The slab portion of the bridge was most accurately represented using brick elements in Strand7. The model is separated into three spans, each assigned to a group. These were first modeled in AutoCAD, exported to an IGES geometry file, and then imported separately. Each section was discretized by first surface meshing the cross section into shell elements with a max edge length of 6 inches and then extruding the shell elements into brick elements also with a maximum edge length of 6 inches. The substructure elements were omitted and the model was truncated at interface between the slab and the substructure. This resulted in the model shown in Figure 26.

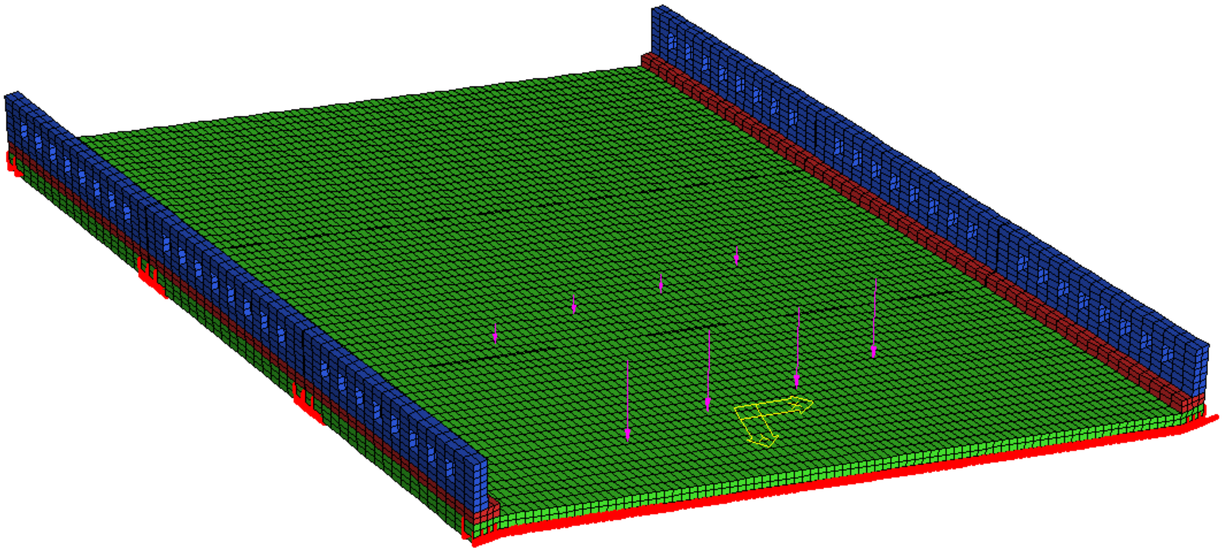


Figure 36: Structure 0324152 – Dimetric Model View

Overall Geometry

<u>Dimensions</u>

Length – Span 1 and Span 3 – 16' 1.5" from centerline of bearing areas, Span 2 – 16' 3" from centerline of bearing areas

Width – 40.3' curb to curb, 44' out to out

Skew – 20 degrees

<u>Material Properties</u>

Given the age of this bridge, it was difficult to find properties of concrete specified on the drawings. The concrete properties for this model were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi as shown in Figure 13. The specified reinforcing steel yield strength was not given in the plans so an assumed value of 33 ksi was used based on the data of construction. The yield strength was selected from Figure 14.

<u>Miscellaneous Calculations</u>

In order to calculate the dead load due to the asphalt overlay on the structure, the depth of the asphalt overlay was estimated from the most recent inspection report and field measurements. The following calculation was used to calculate a non-structural mass to be applied to the structure to accurately model the dead load effects from the asphalt overlay. The concrete modulus for an assumed concrete compressive strength is also show.

Asphalt Overlay

$$Deck\ Pressure\ due\ to\ asphalt\ overlay = 145\,pcf\ \times \left(\frac{6.5}{12}\right) ft = 78.5\,psf$$

Concrete Modulus

$$f'_c = 57{,}000\sqrt{2500} = 2850000\text{ psi} = 2850\text{ ksi}$$

Assumptions, Limitations

- All concrete within model is assumed to be 2.5 ksi concrete (E = 2850 ksi)
- Reinforcing steel was not included in the model
- The substructures were not included in the model
- The asphalt overlay was included in the model
- The parapet is an approximation of the actual parapet due to the complex geometry and changes in cross section along the length
- The boundary conditions were assumed to be pinned at the piers and allow longitudinal expansion at the abutments
- A ten percent reduction in the cross section of the reinforcing steel was included to account for exposed and deteriorated reinforcement
- The impact factor was selected as a maximum of 30%.

A-Priori Load Ratings

The a-priori model was loaded using the specified rating vehicles and the critical moment and shear demands extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model generated demands are given in Table 20 while the controlling ratings from the model versus those presented in the inspection report are given in Table 21. The model and inspection report both show bending moment as the controlling load effect for structure 0324152.

Table 20: Structure 0324152 – A-Priori Model Load Factor Ratings

Springers Brook - A- Priori Model Load Ratings				
	Flexure		Shear	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	159	265	253	423
Type 3 (25T)	147	246	152	254
Type 3-3 (40T)	247	412	331	553
Type 3S2 (40T)	198	331	281	470
SU4 (27T)	152	253	145	242
SU5 (31T)	147	291	166	278
SU6 (35T)	172	287	187	312
SU7 (39T)	192	320	208	348
NRL (40T)	198	331	215	359

Table 21: Structure 0324152 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	159	265	25	42
Type 3 (25T)	147	246	21	36
Type 3-3 (40T)	247	412	41	69
Type 3S2 (40T)	198	331	34	57

Structure 1512152 – NJ72 over Mill Creek

Description of Structure

Route 72 over Mill Creek is a combination of structural types resulting from a widening. The original structure was built in 1930 and consists of concrete-encased steel beams. The widened portion is comprised of prestressed adjacent box beams and was constructed in 1968. The bridge sees an ADT of 45,000 with 4% truck traffic. There is a median with a hollowed curb section for utilities as well as a utility bridge over Mill Creek immediately adjacent to the structure. The bridge carries RT72 over the non-navigable Mill Creek in Manahawkin, NJ. An overview of the structure is shown in Figure 37 and an overhead aerial view is shown in Figure 38

Figure 37: Overview of Structure 1512152 – NJ72 over Mill Creek

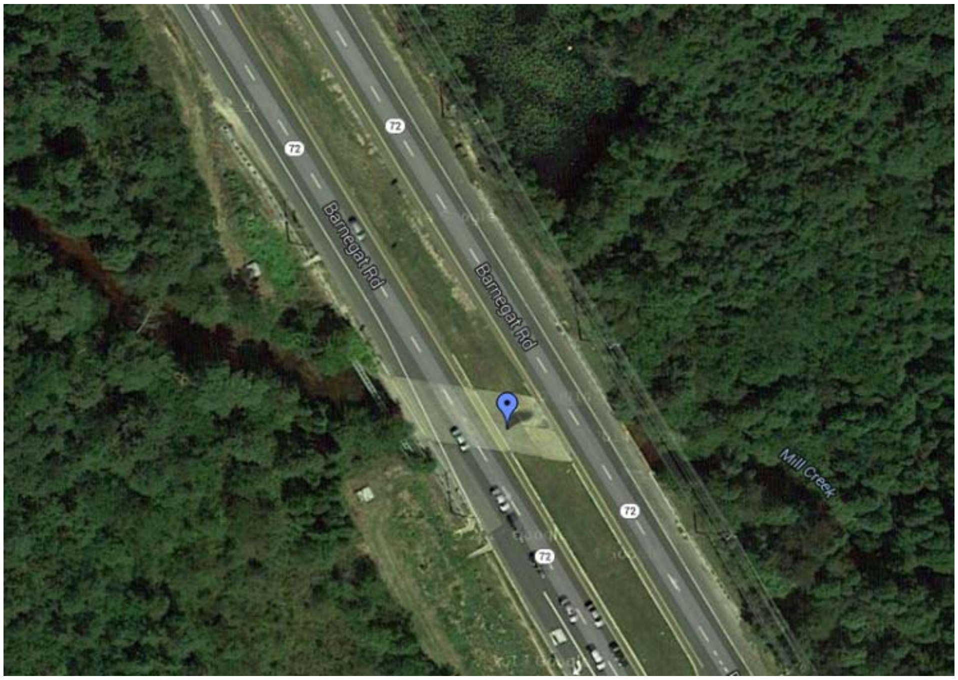


Figure 38: Structure 1512152 – Aerial View

Description of FE Model

An element level model of the structure was created to represent the beams and deck of the structure. For the encased I sections two beam elements were defined, one for the encasement and one for the steel section as shown in Figure 39. Full compatibility between the two sections was assumed. The concrete deck was modeled as shell elements. The newer section was constructed using adjacent composite prestressed concrete box beams. Combining the aforementioned elements resulted in the model shown in Figure 40.

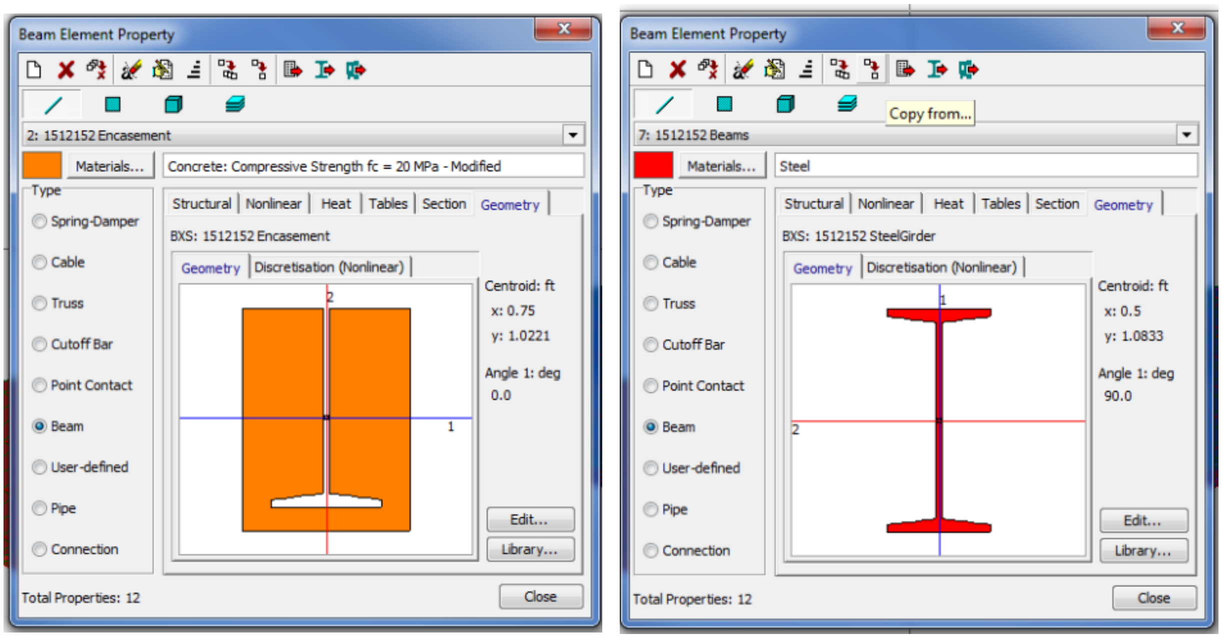


Figure 39: Structure 1512152 – Concrete Encased Beam Cross Sections

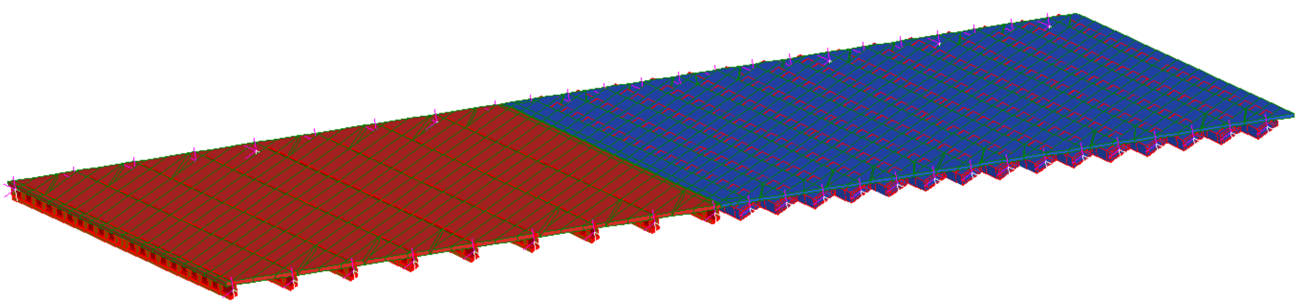


Figure 40: Structure 1512152 – Dimetric Model View

The boundary conditions of the structure were assumed to be pinned at one end and expansion at the opposite abutment. At the pinned side translation was restrained in the X, Y, And Z directions while rotation was restrained about the Y and Z directions. At the expansion end, translations were restrained in the Y and Z directions and rotations were restrained about the Y and Z directions.

Overall Geometry

Dimensions

Length – 42 feet

Width – 107 feet curb to curb with, 115.7 ft. out to out

Skew – 40 degree skew

Beam and Deck

The original encased I beam span cross sectional dimensions are shown in Figure 41 and the span is characterized by steel I beams fully encased in concrete. The concrete deck is composite with the encasement and provides the appearance of a reinforced concrete T-Beam section. The minimum deck thickness above the steel beams is approximately 3.5 inches above which a 4" asphalt overlay has been placed. The widened section consists of the adjacent prestressed box beams shown in Figure 42. The span is comprised of 15 adjacent beams topped with a variable composite concrete deck that changes thickness from a minimum of 5 inches to a maximum of 7 inches.

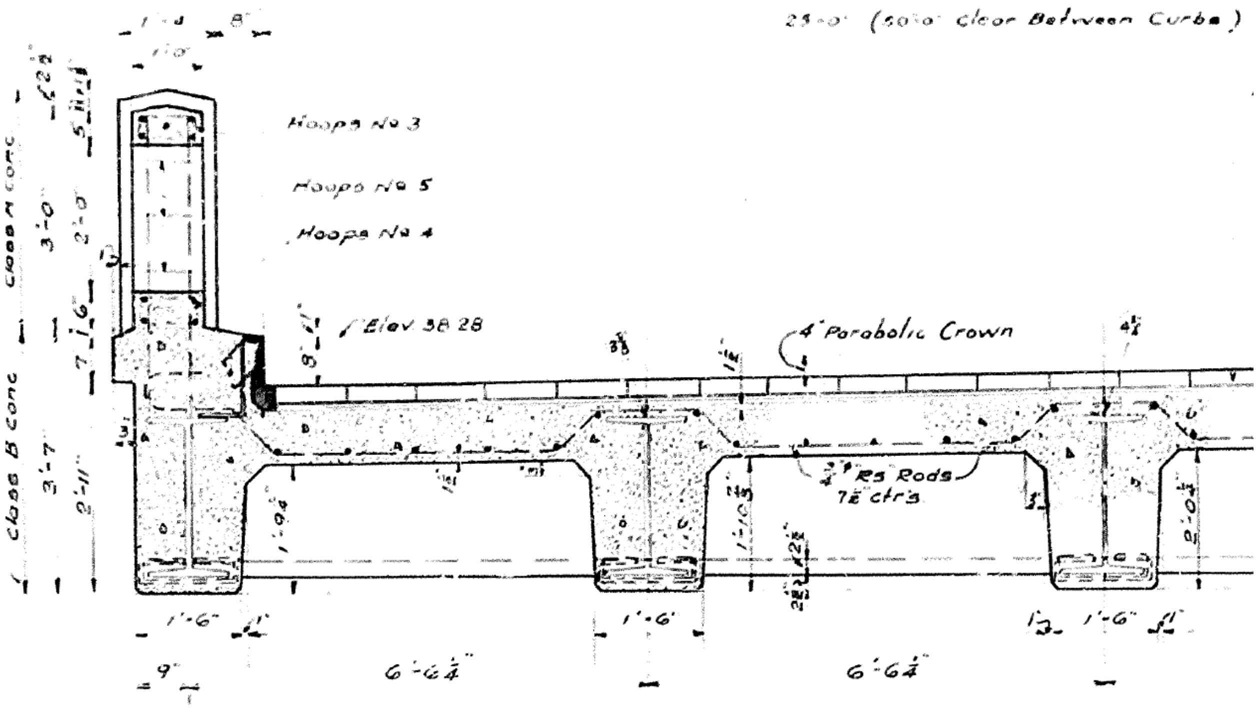


Figure 41: Structure 1512152 – Encased I-Sections

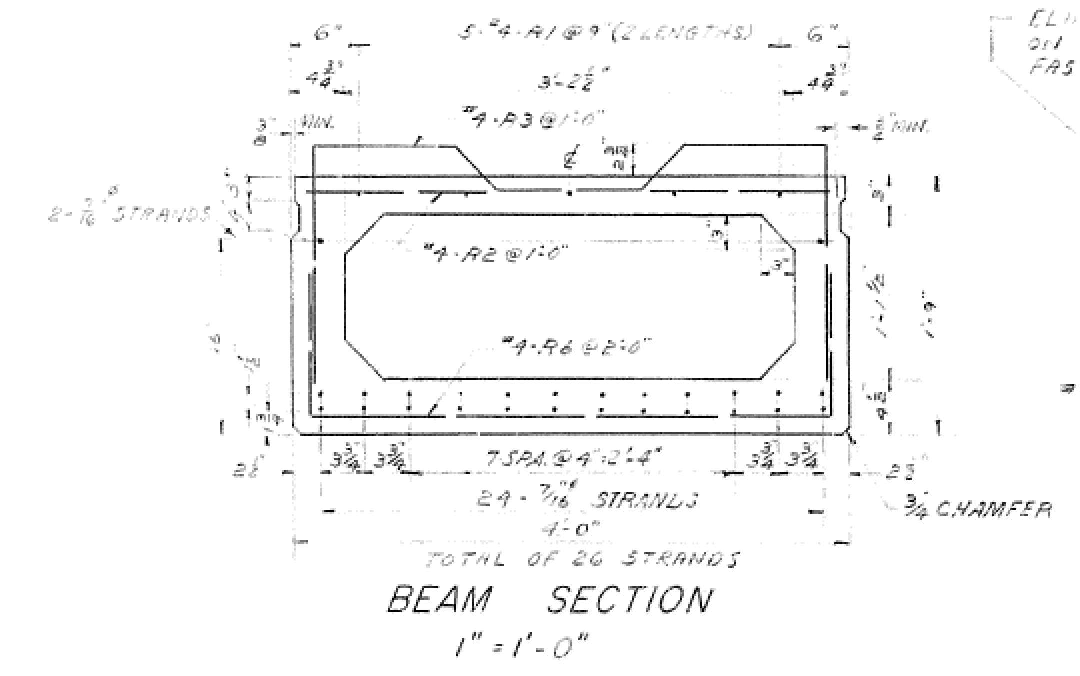


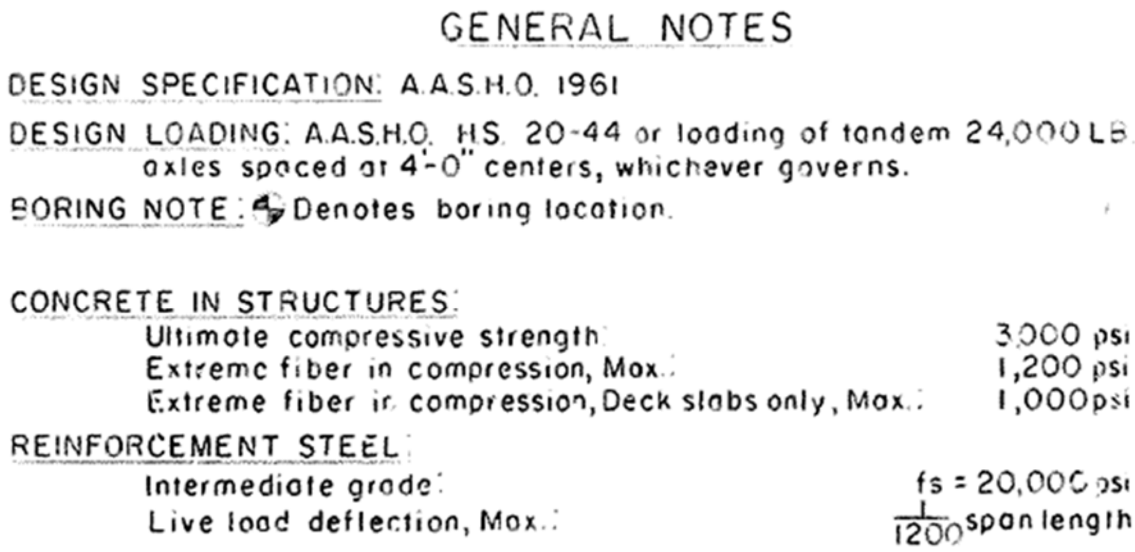
Figure 42: Structure 1512152 – Prestressed Adjacent Box Beam

Sidewalk and Parapet

The bridge has a sidewalk and parapet on each side. The original span has a variable cross section parapet as shown in Figure 37 and the widened section has a solid concrete parapet shown in Figure 37.

<u>Material Properties</u>

Given the age of the original bridge span, it was difficult to find properties of concrete that are specified on the drawings. The concrete properties for the concrete encasement and deck were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi as shown in Figure 13. For the prestressed adjacent box beam span the material properties shown in Figure 43 were used.



GENERAL NOTES

DESIGN SPECIFICATION: A.A.S.H.O. 1961

DESIGN LOADING: A.A.S.H.O. H.S. 20-44 or loading of tandem 24,000 LB. axles spaced at 4'-0" centers, whichever governs.

BORING NOTE: Denotes boring location.

CONCRETE IN STRUCTURES:

Ultimate compressive strength:	3,000 psi
Extreme fiber in compression, Max.:	1,200 psi
Extreme fiber in compression, Deck slabs only, Max.:	1,000 psi

REINFORCEMENT STEEL:

Intermediate grade:	fs = 20,000 psi
Live load deflection, Max.:	$\frac{1}{1200}$ span length

Figure 43: Structure 1512152 – Material Properties used In Prestressed Adjacent Box Beam Span

<u>Miscellaneous Calculations</u>

No additional calculations were performed in the construction of the model.

Assumptions, Limitations

- The stiffness contribution of parapets was ignored, dead load was included as a nonstructural mass. The shape of the parapet cross section was assumed to be rectangular.
- Capacity calculations were based on steel section only. Concrete encasement was ignored resulting in a conservative capacity estimate.
- Prestressed sections rated for flexure only

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described before and the critical moment and shear demands extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 22. A comparison of the controlling load rating from the model versus those presented in the inspection report are given in Table 24. The ratings from the model are controlled by shear while the controlling load effect for the ratings presented in the inspection report is unknown.

Table 22: Structure 1512152 – Encased Steel Beams – A-Priori Model Load Factor Ratings

NJ72 Over Mill Creek - A-Priori Load Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	80	134	44	74
Type 3 (25T)	77	130	52	87
Type 3-3 (40T)	173	290	83	163
Type 3S2 (40T)	247	412	98	139
SU4 (27T)	28	47	66	110
SU5 (31T)	29	54	67	112
SU6 (35T)	28	48	72	121
SU7 (39T)	29	49	77	130
NRL (40T)	28	46	80	134

Table 23: Structure 1512152 – Prestressed Concrete Beams – A-Priori Model Load Factor Ratings

NJ72 Over Mill Creek - A-Priori Load Ratings		
	Flexure	
Truck (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	115	193
Type 3 (25T)	152	254
Type 3-3 (40T)	321	536
Type 3S2 (40T)	197	329
SU4 (27T)	49	82
SU5 (31T)	55	95
SU6 (35T)	53	89
SU7 (39T)	57	95
NRL (40T)	100	167

Table 24: Structure 1512152 – Encased Steel Beams – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	44	81	20	34
Type 3 (25T)	52	94	18	31
Type 3-3 (40T)	83	150	36	60
Type 3S2 (40T)	98	175	28	48

Structure 1516152 – NJ166 over Toms River

Description of Structure

Structure 1516152 carries Rt. 166 over Toms River in Dover Township, Ocean County. The bridge was built in 1928 without any major widening or rehabs. The length is 50' and the width is 60"-1". The structure is a single simply supported span comprised of partially encased steel girders. These girders frame into a large end plate girder which is supported in several locations along its length. The outside stringers are fully encased, with a fairly large span of deck between them. The structure is considered scour critical according to the inspection report. Overall condition is considered fair, primarily due to underside deterioration of the deck. The bridge has an ADT of 25000 with 4% truck traffic. An overview of the structure is shown in Figure 44 and an overhead aerial view is given in Figure 45.

Figure 44: Overview of Structure 1516152 – NJ166 over Toms River

Figure 45: Structure 1516152 – Aerial View

Description of FE Model

An element level model of the structure was created to represent the beams and deck of the structure. For the partially encased I sections two beam elements were defined, one for the encasement and one for the steel section as shown in Figure 46. For the fully encased section two beam elements were used to represent the steel beam and the concrete encasement as shown in Figure 47. Full compatibility between the sections was assumed. For the end floor beams into which the stringers frame, beam elements were selected to model these sections. The concrete deck between the encased girders was modeled as shell elements which were linked to the encasement sections using rigid links. Combining the beam elements and shell elements resulted in the model shown in Figure 48.

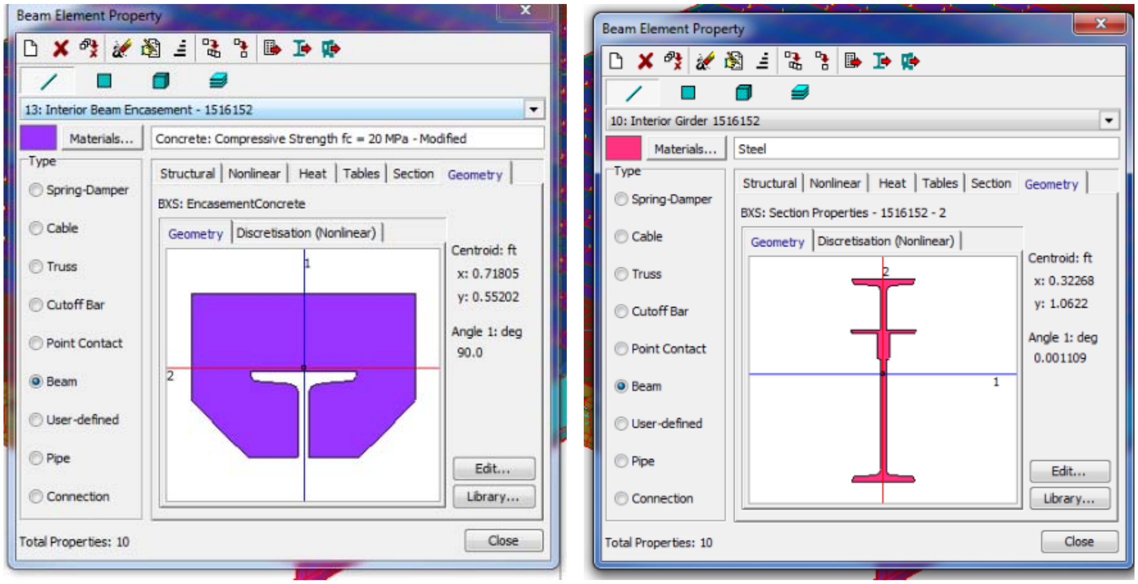


Figure 46: Structure 1516152 – Partially Encased Beam Cross Sections

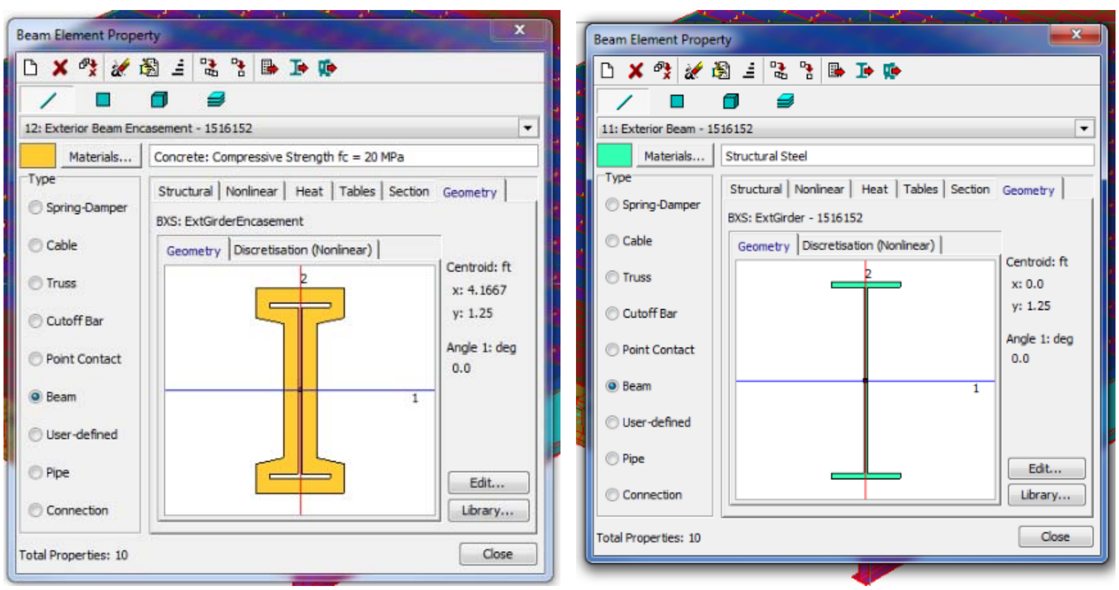


Figure 47: Structure 1516152 – Fully Encased Beam Cross Sections

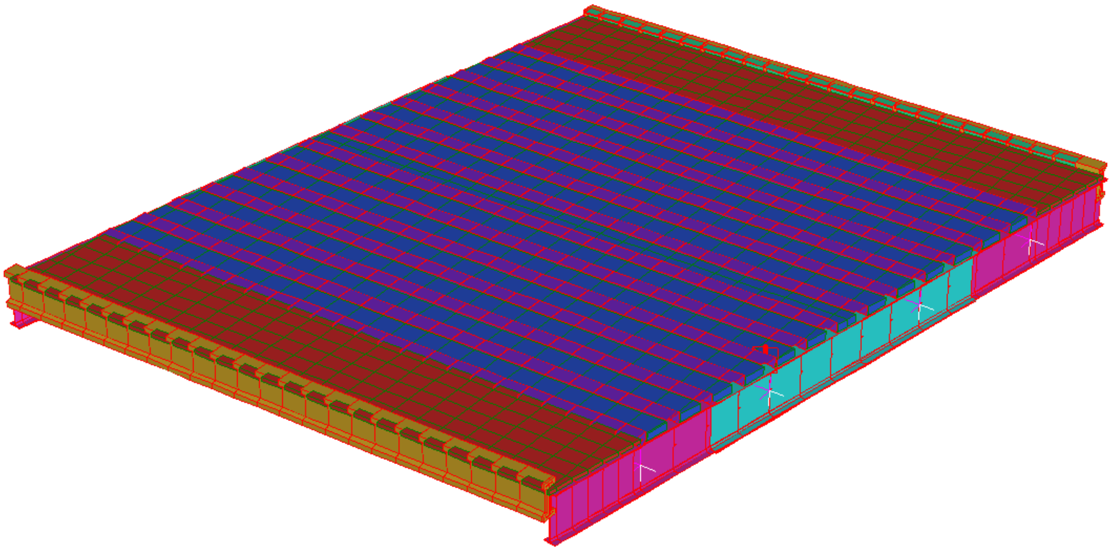


Figure 48: Structure 1516152 – Dimetric Model View

Overall Geometry

<u>Dimensions</u>

Length – 47 feet

Width – 40.1 feet curb to curb width, 60.1 ft. out to out

Skew – 10 degree skew

Beam and Deck

The interior stringers are partially encased in concrete as shown in Figure 49. The concrete encasement is supported by angles running the length of the beam. The composite concrete deck is approximately 4 inches above the top flange of the partially encased section. The fully encased exterior beams support a portion of the sidewalk but will see minimal loading from vehicles since the exterior girder does not directly support the concrete deck.

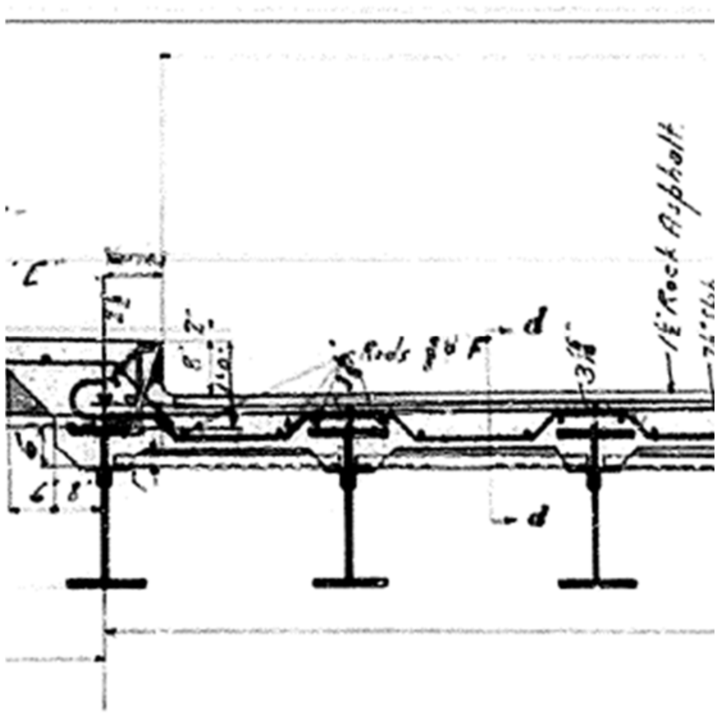


Figure 49: Structure 1516152 – Partially Encased Interior I Sections

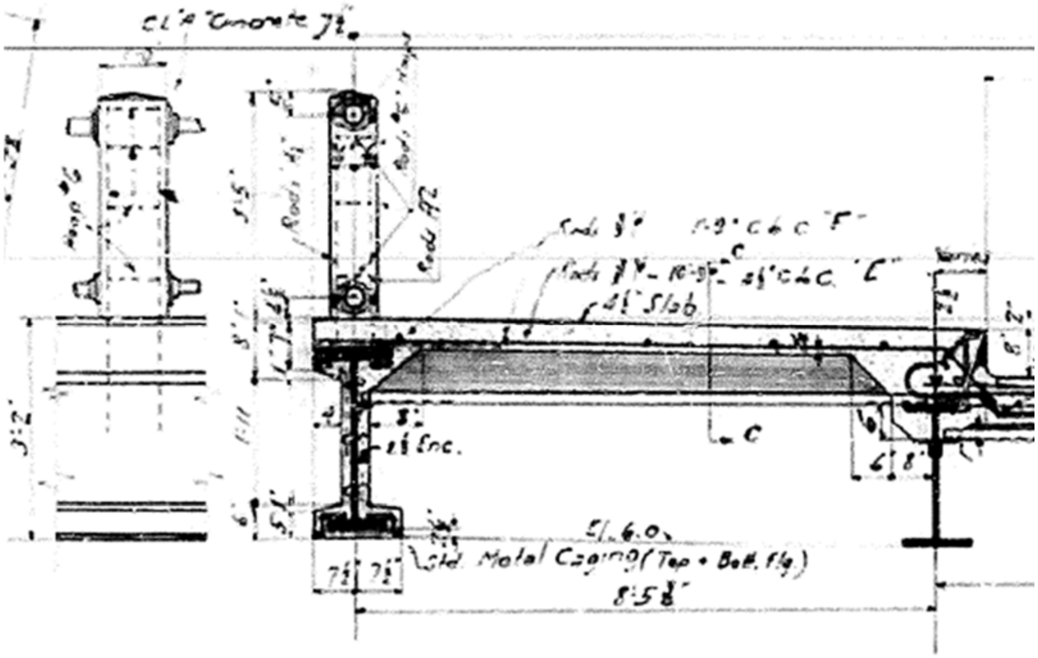


Figure 50: Structure 1516152 – Encased Exterior I Sections

Sidewalk and Parapet

The bridge has a sidewalk and parapet on each side. The sidewalk is supported by the exterior stringer and the first interior stringer. The parapet is a steel railing and was not included in the model since it does not provide any stiffness to the system and contributes a small amount of dead load.

Material Properties

Given the age of the bridge, it was difficult to find properties of concrete that are specified on the drawings. The concrete properties for the concrete encasement and deck were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi as shown in Figure 13.

Miscellaneous Calculations

A benchmark model was developed to determine the efficacy of modeling the partially encased girders using a combination of two beam elements and shell elements. A slice of the model was taken (as shown in Figure 51) and load applied to this model to determine if the modeling method modeled the composite beam/deck interface correctly.

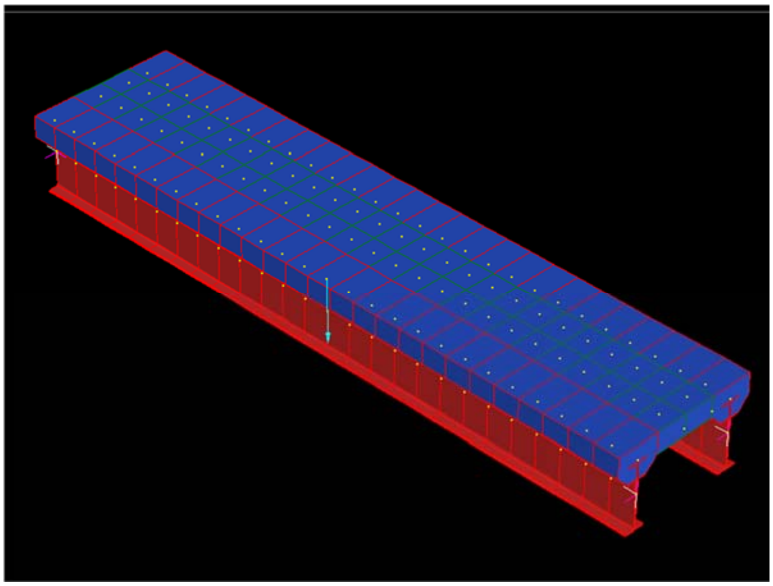


Figure 51: Structure 1516152 – Benchmark Model

The model results shown in Figure 52 show the chosen modeling method for modeling composite action between the beams and the deck is in fact providing realistic results. The benchmark shows that the beams and deck are deflecting under load in a composite manner.

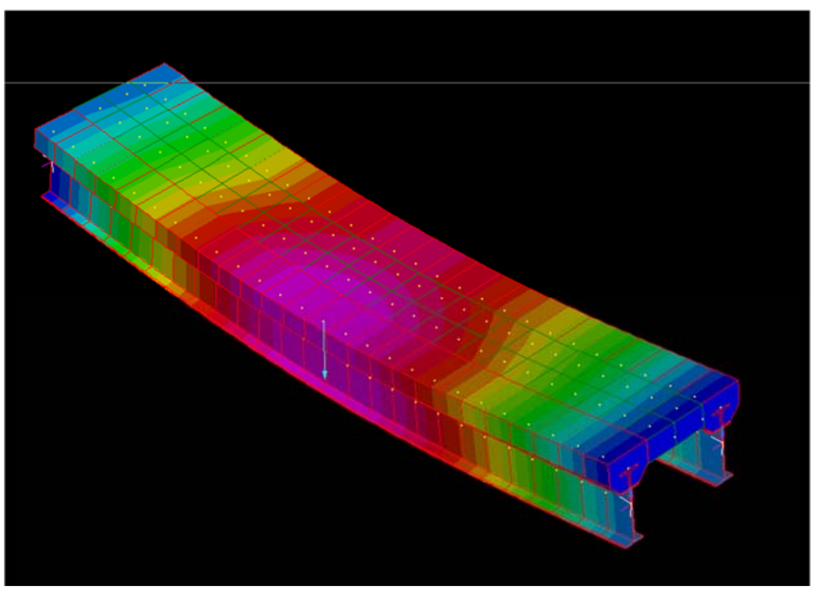


Figure 52: Structure 1516152 – Benchmark Model Results

Assumptions, Limitations

- The rotation about the Y axis was assumed to be unrestrained
- Both ends of the bridge were assumed to be pinned boundaries
- Diaphragm at midspan ignored in model (conservative assumption)
- Concrete encasement omitted in capacity calculations resulting in conservative estimation of the stringer capacity

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described previously and the critical moment and shear demands extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 25. A comparison of the controlling ratings from the model and those presented in the inspection report is give in Table 26. The model based ratings are controlled by shear while the controlling load effect for the ratings presented in the inspection report is unknown.

Table 25: Structure 1516152 – A-Priori Model Load Factor Ratings

NJ166 over Toms River - A-Priori Load Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	75	125	58	97
Type 3 (25T)	149	249	56	94
Type 3-3 (40T)	183	305	102	170
Type 3S2 (40T)	239	399	78	131
SU4 (27T)	86	144	36	61
SU5 (31T)	117	166	47	79
SU6 (35T)	99	166	40	67
SU7 (39T)	109	183	38	63
NRL (40T)	111	186	36	61

Table 26: Structure 1516152 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	58	97	22	37
Type 3 (25T)	56	94	20	34
Type 3-3 (40T)	102	170	39	65
Type 3S2 (40T)	78	131	32	53

Structure 1103152 – US1 over D&R Canal

Description of Structure

Structure 1103152 carries US Route 1 over the D&R canal near Lawrence, NJ. The bridge is a concrete rigid frame structure that has been widened to accommodate the expansion of US Route 1. The bridge carries 4 lanes of traffic and accommodates a ramp from I295. The first structure was built in 1959 and the widened structure was completed in 1974. An overview of the structure is shown in Figure 53 and an overhead aerial view is shown in Figure 54.

Figure 53: Overview of Structure 1103152 – US1 over D&R Canal

Figure 54: Structure 1103152 – Aerial View

Description of FE Model

Due to the monolithic construction of the RC frames, solid elements would be a good model form. However, due to the size of the model and number of elements needed it was decided to use shell elements drawn at the centerline of the frame walls, deck, and foundations. For the purposes of rating this structure, only the top slab will be rated. An overview of the model is shown in Figure 55.

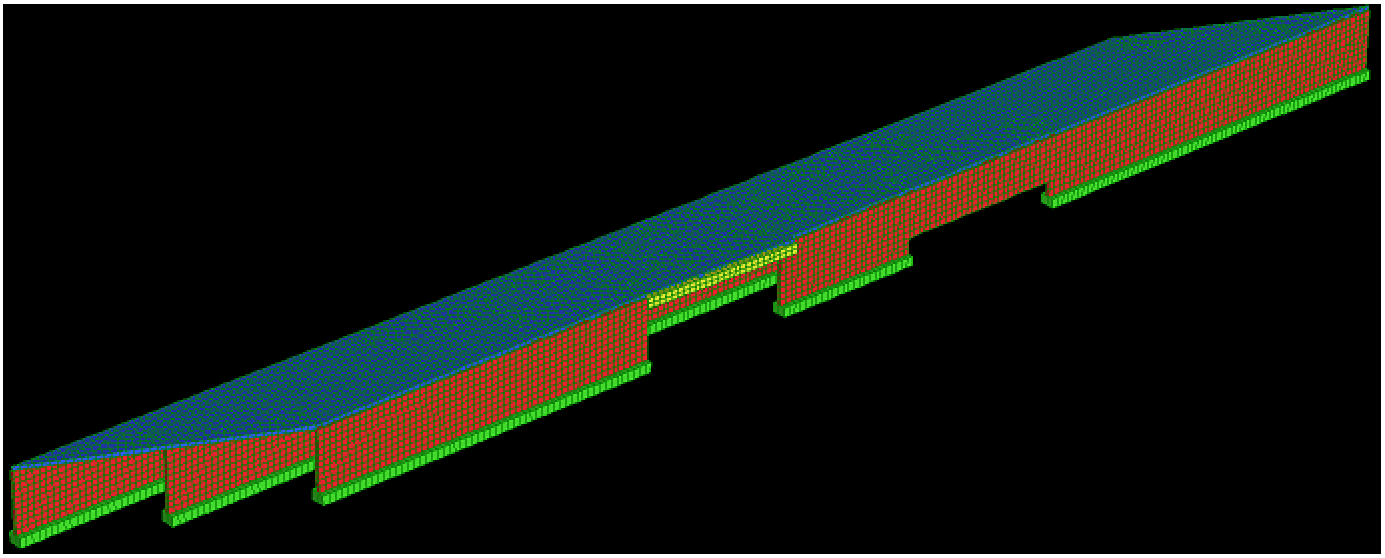


Figure 55: Structure 1103152 – Dimetric Model View

Overall Geometry

<u>Dimensions</u>

Length – 80 feet

Width – 159 feet out to out, 390 feet along the skew angle

Skew – 66 degree skew

The elevation of the RC frame is shown in Figure 56, and indicates a deck slab thickness of 13 inches. The rebar spacing is also specified. The elevation shown is the nominal elevation. There are other elevations where the frame walls are shorter in areas where the original bridge foundations were reused. An elevation of the sections where the original foundation was used is shown in Figure 57.

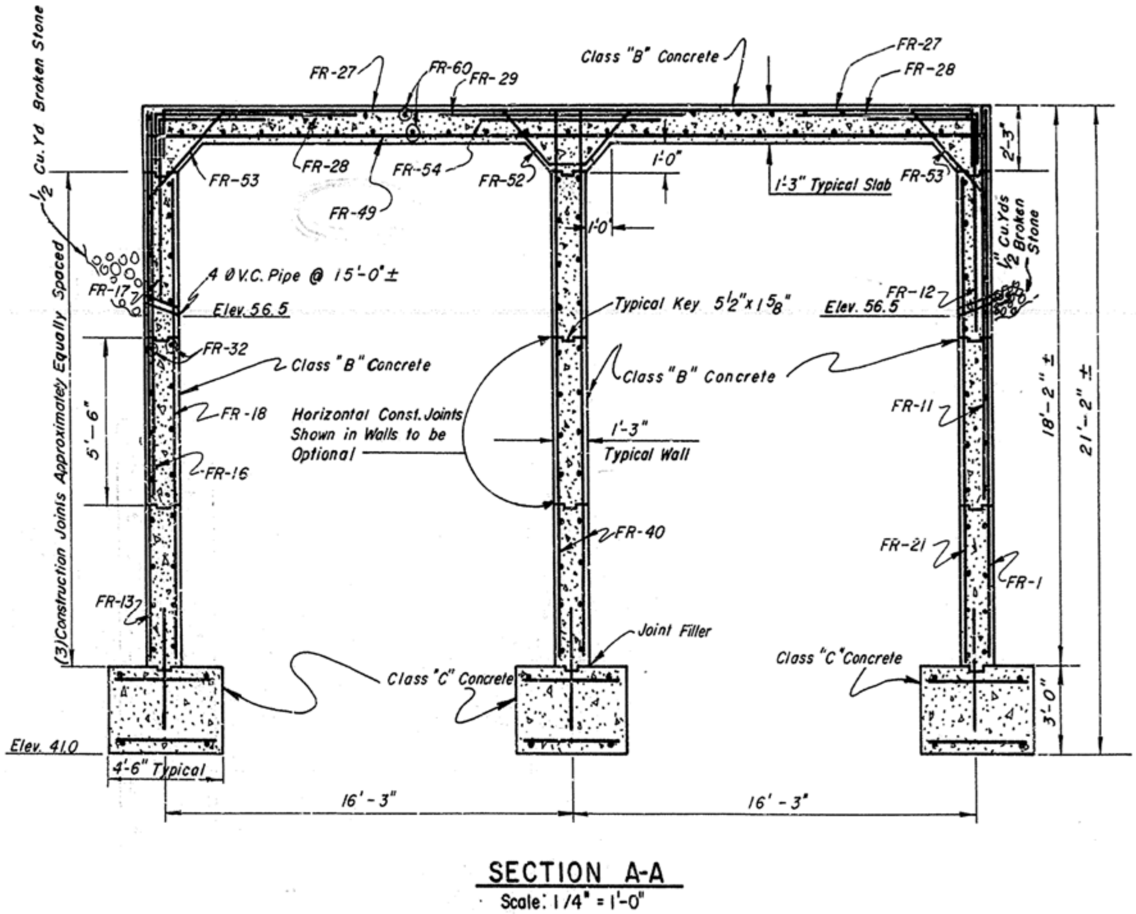


Figure 56: Structure 1103152 – RC Frame Elevation

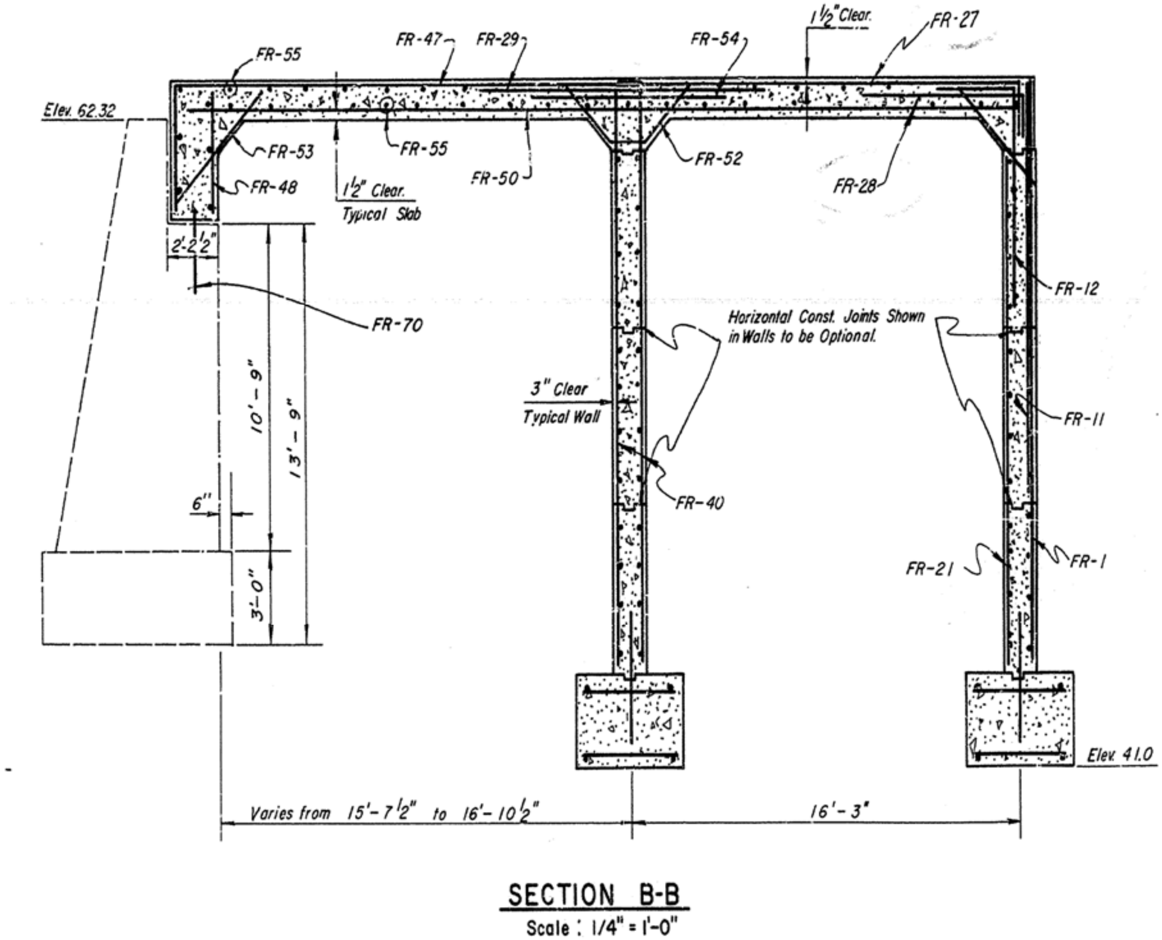


Figure 57: Structure 1103152 – RC Frame Elevation 2

<u>Material Properties</u>

The material properties for the portion of the frame constructed in 1959 are not given so an assumption regarding the material properties is necessary. The recommended material properties from AASHTO MBE are taken as 3 ksi for compressive strength of concrete and 40 ksi for the reinforcing steel bars.

<u>Miscellaneous Calculations</u>

$$Deck\ Pressure\ due\ to\ asphalt\ overlay = 145\, pcf \times \left(\frac{6.5}{12}\right) ft = 78.5\, psf$$

$$Nonstructural\ Mass\ from\ Barriers = 145\, pcf \times (3.16333\ X\ 1)\ ft\text{^}2 = 458\, plf$$

Assumptions, Limitations

- The foundations were assumed to be consistent along the width of the structure and areas where the previous bridges foundation was used were omitted

- Soil pressure was not considered on the outside walls of the frame
- All concrete was assumed to have a compressive strength of 3 ksi
- Reinforcing steel was not included in the model
- Parapets, barriers, and sidewalks were not included explicitly in the model but rather with added mass at the appropriate locations
- The model was auto meshed with a maximum edge size of 2 ft.
- The deck elements were offset upwards half of its thickness (7.5") to remove the overlap with the frame walls
- The foundations were offset half of their thickness (18") downward to remove the overlap with the frame walls

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described before and the critical moment and shear demands extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 27. Table 28 presents a comparison of the controlling load ratings developed from the finite element model versus those reported in the inspection report. For both ratings, the positive moment in the top slab of the reinforced concrete frame is the controlling load effect.

Table 27: Structure 1103152 – A-Priori Model Load Factor Ratings

US1 over D&R Canal - A Priori Load Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	49	82	127	212
Type 3 (25T)	35	58	127	213
Type 3-3 (40T)	91	152	266	445
Type 3S2 (40T)	76	127	180	300
SU4 (27T)	52	87	132	221
SU5 (31T)	53	89	150	251
SU6 (35T)	58	96	147	245
SU7 (39T)	61	101	187	313
NRL (40T)	60	101	180	300

Intelligent Infrastructure Systems

Table 28: Structure 1103152 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	49	82	13	22
Type 3 (25T)	35	58	16	26
Type 3-3 (40T)	91	152	31	52
Type 3S2 (40T)	76	127	25	42

Structure 1237155 – NJ18 over Raritan River

Description of Structure

Structure 1237155 carries Route 18 over the Raritan River in New Brunswick, NJ. The bridge is two separate superstructures (1 northbound and 1 southbound). The bridge is curved in plan and is comprised of a girder/floor beam/stringer superstructure. The girders are continuous for 4 spans and there is a 5th simply supported span. The girders range. The structure was completed in 1980. Several pictures and an aerial view of the bridge are shown in Figure 58 Figure 59 respectively.

Figure 58: Structure 1237155 – NJ18 over Raritan River

Figure 59: Structure 1237155 – Aerial View

Description of FE Model

The continuous spans and simply supported spans making up the Rt18 Bridge were modeled separately to reduce the computational resources required to analyze such large models. The models were built using a hybrid approach where the main girders were modeled using shell elements while the floorbeams, stringers, and other members were modeled with beam elements. The model construction is described in the following sections.

Overall Geometry

Dimensions

Length along centerline – Continuous Spans – 870’, Simple Span – 168’

Width – Southbound – 51 ft, Northbound – 42.5 ft

Skew –

Simple Span **-** Abutment A - 16°**,** Pier A - 16°

Continuous Span – Pier A - 41°, Pier 1 -40°, Pier 2 - 36°, Pier 3 - 32°, Abutment B - 27°

An elevation view of the structure from the as built plans is given in Figure 60.

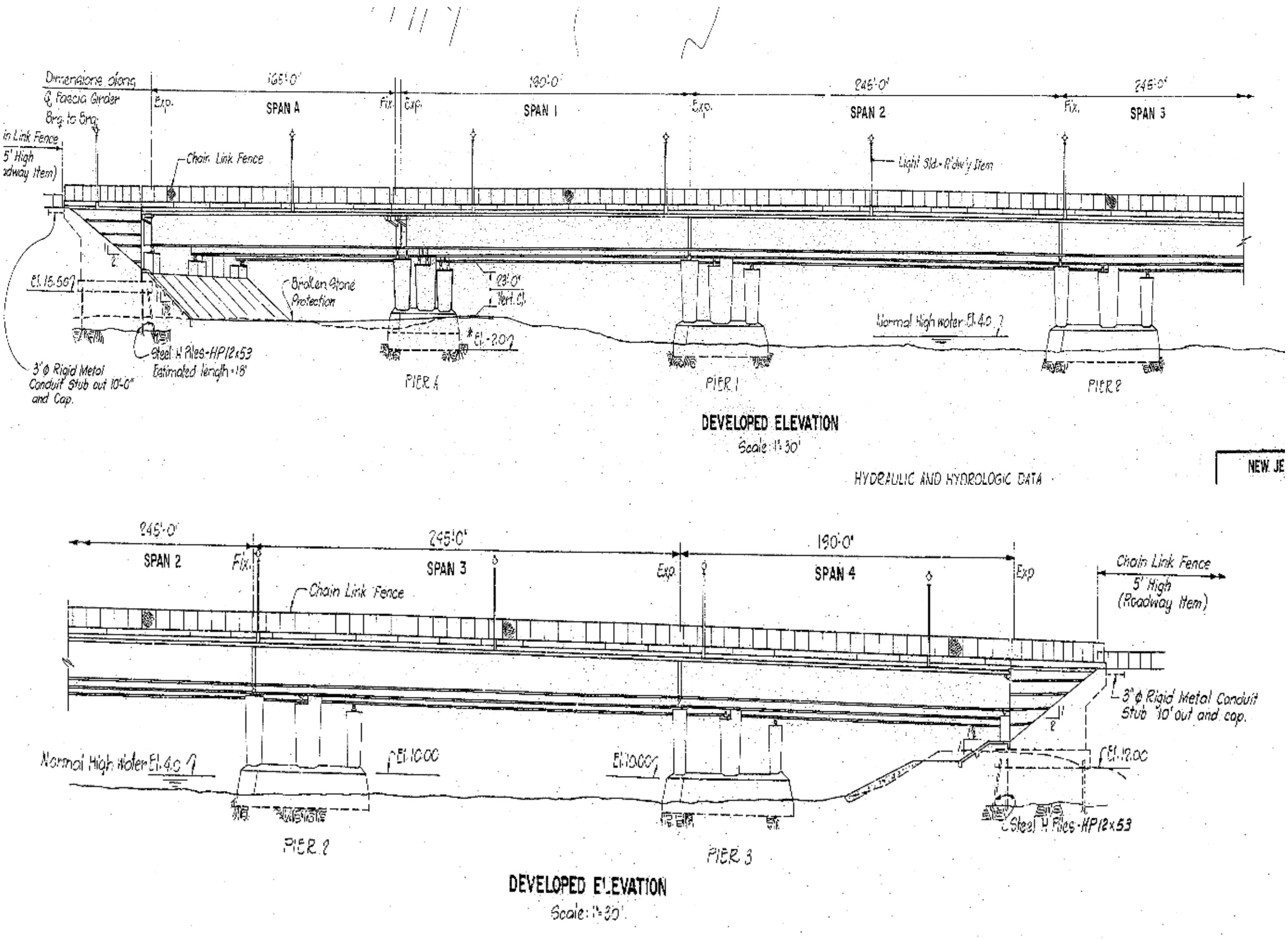


Figure 60: Structure 1237155 – Elevation

Girders

The girders of the structure are built up from and vary from 10' (Westbound structure), to 12' (Eastbound structure). The flanges range 1.75" to 3" thick and the webs range from ¾" to 5/8" thick. Since the bridge is curved in plan it was decided that the girders would be modeled using shell elements. The girders were modeled as shell elements since beam elements would not allow the inclusion of warping effects due to torsion which is more prevalent in curved bridges than straight bridges due to the eccentricity of the girders from a line drawn between the substructures. An example of the girder and its mesh density is shown in Figure 61.

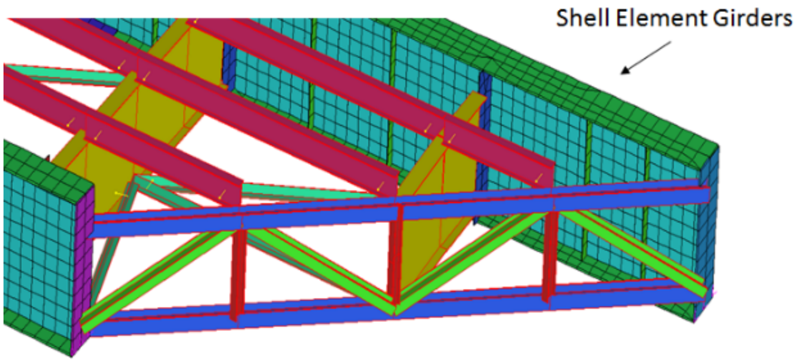


Figure 61: Structure 1237155 – Shell Element Girder

Floorbeams

The floorbeams that support the stringers are varied depending on the location in the structure. Typically, there are two types of floorbeams per span, an end floorbeam and a midspan floorbeam section. An example of a floorbeam section is shown in Figure 63. The floorbeams typically have a web depth of either 84" or 105" depending on their location (eastbound or westbound structure). The floorbeam members were modeled using beam elements and the floorbeams were drawn at their member neutral axis. The floorbeams were then connected to the curved girders through the floorbeam connection plate. Since the floorbeams have a full height connection at each end, the end nodes of the floorbeam were rigidly linked to the nodes along the height of the floorbeam connection plate. The floorbeams and their connection to the girders are shown in Figure 62.

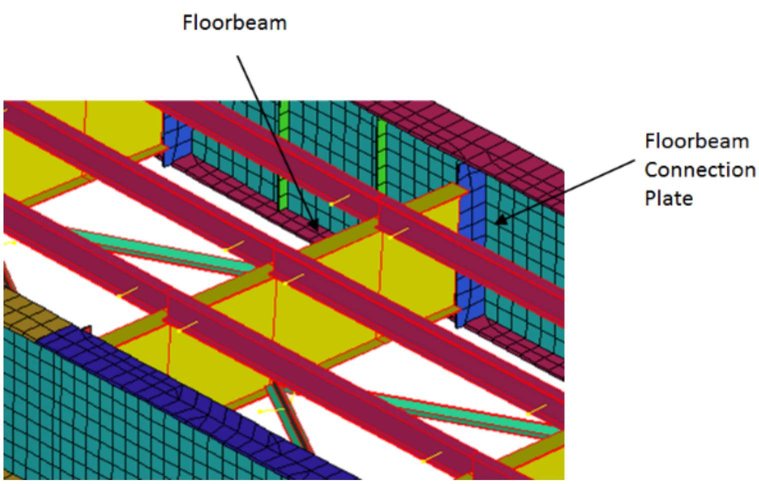


Figure 62: Structure 1237155 – Floorbeam

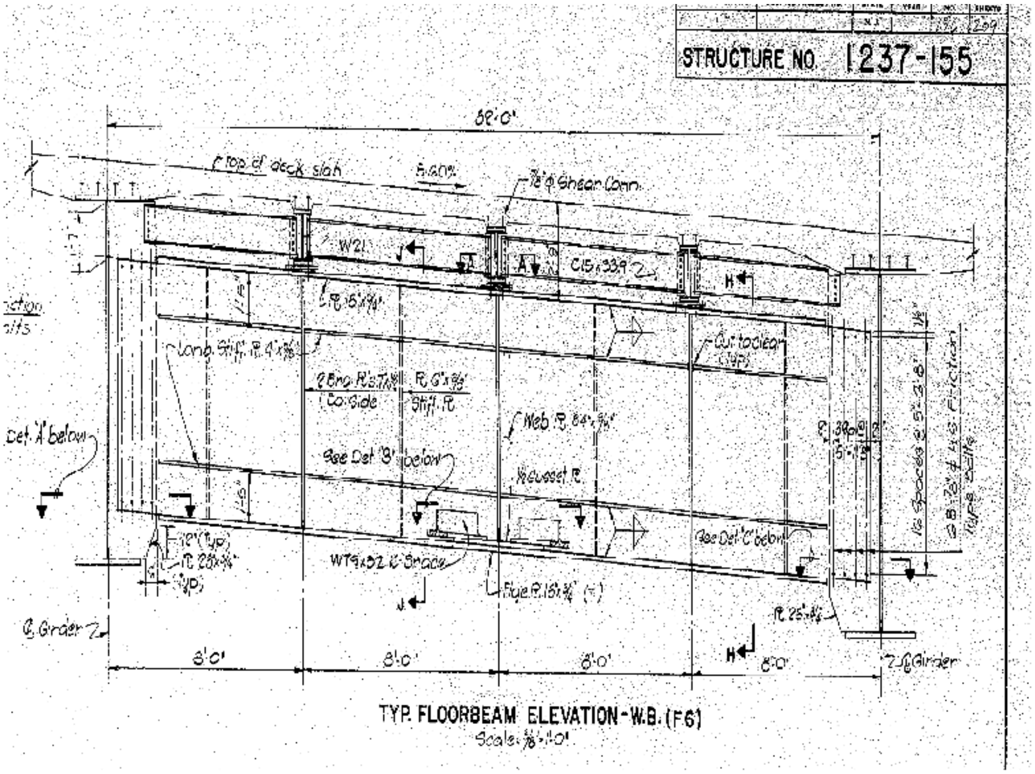


Figure 63: Structure 1237155 – Example Floorbeam Section

Stringers, Diaphragms, and Secondary Members

The stringers supporting the roadway are the same between all structures and are modeled using W21 x 53 rolled sections. The diaphragms between the stringers are modeled using channel sections of type C15 x 33.9. The stringers were drawn at the centerline of their location in the XYZ coordinate system and then the section was offset so the insertion point for the section was at the bottom flange. The stringers at each floorbeam intersection were rigidly linked to the centerline of the floorbeam members. The stringers are shown in Figure 64. The end diaphragm trusses and wind bracing were modeled using beam elements with the proper section and material properties assigned to each member based on the as built drawings. The end diaphragm and wind bracing members are shown in Figure 65.

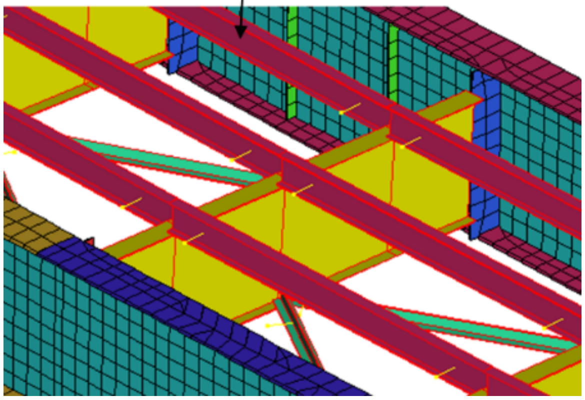


Figure 64: Structure 1237155 – Stringers

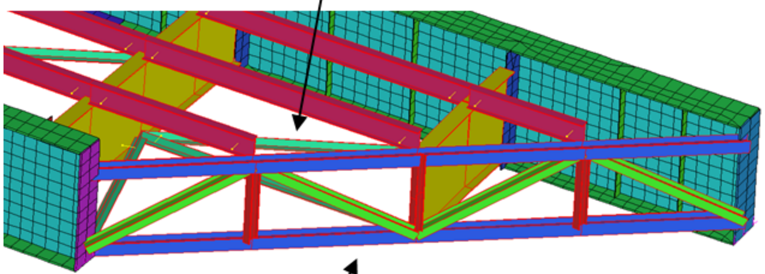


Figure 65: Structure 1237155 – Secondary Members

Deck

The deck of RT 18 continuous span was modeled using shell elements. The edges of shell elements were defined from the same nodes that define the stringers and the middle of the top flange of the curved girders (or the uppermost row of shells in the flanges of the curved girders). Because the curved girders are approximated using a series of straight line segments, the rectangular regions defined by these straight segments and the stringers were used to create an initial set of shells, which were manually subdivided into approximately 1'x1' segments. This meant that the stringers also had to be subdivided to the same resolution as the deck in order for the nodes to align and enforce compatibility. The deck was then offset to the necessary height, in this case 6".

The deck overhangs were modeled by creating a UCS for each straight of the curved girders, then offsetting the end nodes of them by the appropriate x and z coordinate to account for its length and change in elevation due to slope. These four nodes were then used to create quad4 shell elements which were then further discretized. The barrier on the north overhang was simplified into a beam element with a rectangular cross section and modeled using shared nodes, which were then offset to the necessary height. A screenshot of the deck and barrier is shown in Figure 66 and Figure 67.

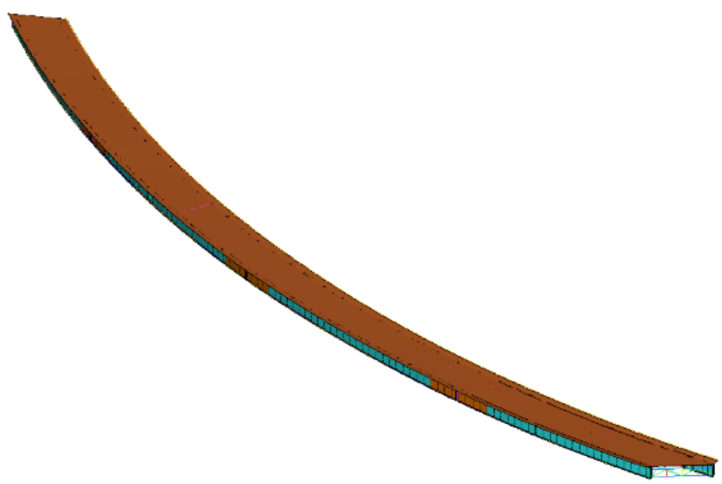


Figure 66: Structure 1237155 – Continuous Span - Finite Element Model

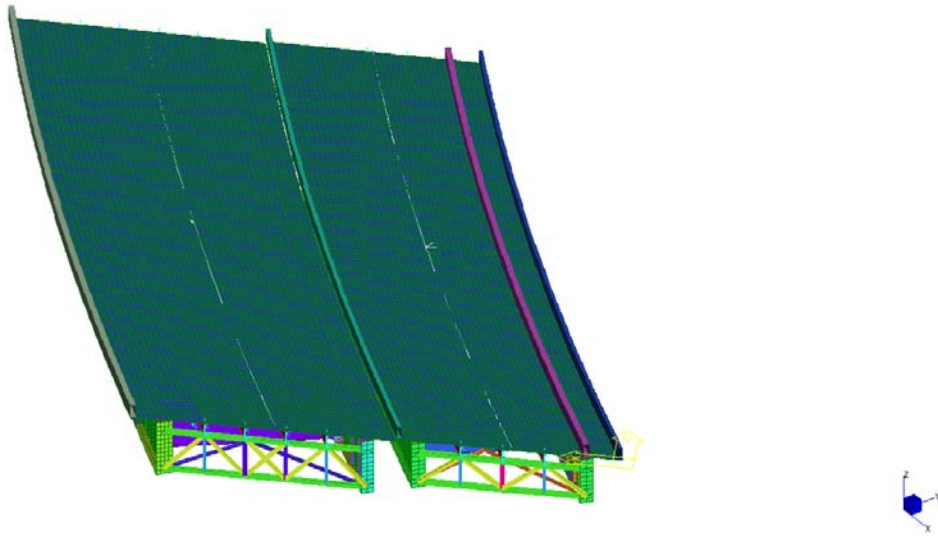


Figure 67: Structure 1237155 – Simply Supported Span - Finite Element Model

<u>Material Properties</u>

Steel – A36 – f_y = 36 ksi

A441 - f_y = 50 ksi

A588 – f_y = 50 ksi

Concrete f'_c = 3 ksi

<u>Miscellaneous Calculations</u>

Assumptions, Limitations

- The substructure elements were not explicitly included in the model. The boundary conditions were applied to the bottom flange of the girders at the pier and abutment locations.
- The deck was modeled as an 8" thick section and does not account for the haunches and other areas where the deck thickness changes
- The barriers were included in the model
- Girder camber was ignored
- Differential elevation of the girders was included
- All flange thickness transitions were included
- Girder web stiffeners, floorbeam connection gusset plates, and bearing stiffeners were included and

modeled using shell elements

- Floorbeams were connected to shell element girders via rigid links along the height of the floorbeam.
- Stringers were connected to the floorbeams using rigid links
- Stiffening plates on the webs of the floorbeams were omitted from the model

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described previously and the critical moment and shear demands were extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 29. The controlling ratings and member type are given for the ratings. From the presented ratings it can be seen that flexure of the stringers control the ratings of the RT18 bridge. However, all ratings are above 1 for the a-priori model based approach. A comparison of the controlling ratings obtained from the model based load rating versus those presented in the inspection report is given in Table 30. The controlling load effect and member for model based load rating are shear in the stringers of the simply supported end span. The inspection report lists an interior stringer of the simply supported end span as the controlling member for ratings. However, the controlling load effect is not reported in the inspection report.

Table 29: Structure 1237155 – A-Priori Model Load Factor Ratings

RT18 Over Raritan River - A-Priori Load Ratings						
	Flexure			Shear		
Truck (tons)	Inventory (tons)	Operating (tons)	Member	Inventory (tons)	Operating (tons)	Member
HS20 (36T)	139	232	Girder	119	199	Stringer
Type 3 (25T)	136	228	Girder	112	187	Stringer
Type 3-3 (40T)	166	277	Girder	221	369	Stringer
Type 3S2 (40T)	157	262	Girder	184	307	Stringer
SU4 (27T)	133	222	Girder	93	156	Stringer
SU5 (31T)	135	255	Girder	110	184	Stringer
SU6 (35T)	136	227	Girder	113	188	Stringer
SU7 (39T)	137	229	Girder	127	212	Stringer
NRL (40T)	125	208	Girder	130	218	Stringer

Table 30: Structure 1237155 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

	FE Model		Inspection Report	
Truck (tons)	**Inventory (tons)**	**Operating (tons)**	**Inventory (tons)**	**Operating (tons)**
HS20 (36T)	119	199	22	37
Type 3 (25T)	112	187	16	28
Type 3-3 (40T)	221	369	32	53
Type 3S2 (40T)	184	307	23	40

Structure 1701151– US40 over W. Branch of Game Creek

Description of Structure

Bridge 1701151 is a two span cast in place reinforced concrete T-beam structure bridge that carries US40 across W Branch of Game Creek in Carneys Point Township, NJ. This structure consists of 15 T-beam elements supporting a reinforced concrete deck that carries two lanes of traffic westbound on US40. The T-beam span was constructed in 1941. From the available information and a site visit a finite element model was constructed of the bridge. It should be noted that the rebar layout and spacing is unknown and an assumed layout was used to compute the capacity of the beam. An overview of the structure is shown in Figure 68 and an aerial view is shown in Figure 69

Figure 68: Structure 1701151– US40 over W. Branch of Game Creek

Figure 69: Structure 1701151– Aerial View

Description of FE Model

Due to the monolithic construction of the reinforced concrete T-beams and deck, solid elements were chosen to model the entire structure. The geometry of the model was constructed from information taken from the inspection report and also field measurements made during a site visit to the structure. An overview of the FE model is shown in Figure 70.

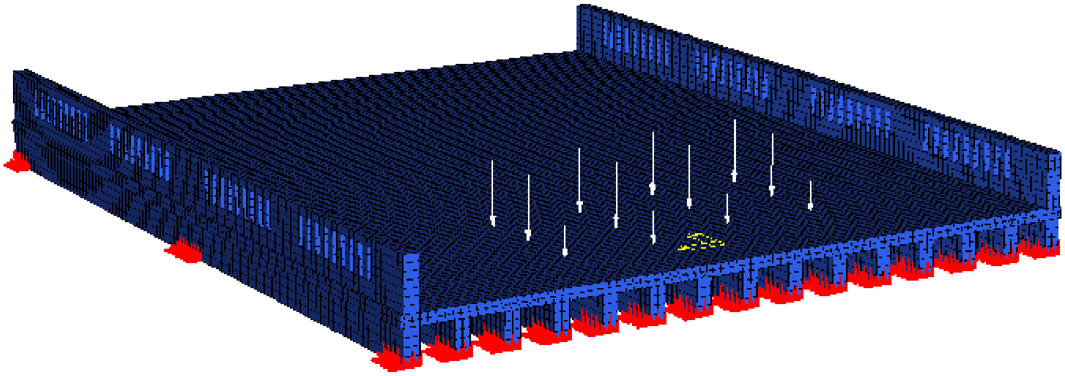


Figure 70: Structure 1701151– Dimetric Model View

Overall Geometry

Dimensions

Length – 25 ft (maximum span length) – 53 ft total length

Width – 38.4 ft curb to curb, 45 ft out to out

Skew – no skew

Material Properties

Given the age of the bridge, the concrete properties for the bridge were taken from the AASHTO Manual for Bridge Evaluation 2nd Edition (2011). The recommended minimum compressive strength for concrete in a bridge constructed prior to 1959 is 2.5 ksi and the minimum yield strength of reinforcing steel is 33ksi as shown in Figure 13 and Figure 14.

Miscellaneous Calculations

The thickness of the asphalt overlay on the deck was estimated from the inspection report and field measurements and determined to be approximately 2.5 inches. These values were used to calculate a nonstructural mass that was added to model to represent the dead load from the overlay.

$$mass\ due\ to\ asphalt\ overlay = 145pcf * \left(\frac{2.5in}{12in}\right) * 1ft = 30.2\ psf$$

Assumptions, Limitations

- Steel reinforcement size and spacing is unknown so a pattern and bar size were assumed. A single layer of reinforcement was also assumed. The size and spacing of reinforcement was based on the values seen on other similar structures built during this era.
- Stirrup size and spacing was unknown and was assumed based on the values seen in other bridges built during this era

A-Priori Load Ratings

The a-priori model was loaded using the rating vehicles described previously and the critical moment and shear demands were extracted from the model and input into the load factor rating equations. The Load Factor ratings developed from the a-priori finite element model are given in Table 31. A comparison of the controlling ratings developed from the finite element model versus those presented in the inspection report is given in Table 32. The controlling load effect for the model based ratings is shear while the controlling load effect for the ratings presented in the inspection report is unknown.

Table 31: Structure 1701151 – A-Priori Model Load Factor Ratings

US40 Over W. Branch of Game Creek - A-Priori Load Ratings				
	Flexure		Shear	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	82	136	74	124
Type 3 (25T)	58	97	60	100
Type 3-3 (40T)	113	189	119	199
Type 3S2 (40T)	96	161	92	153
SU4 (27T)	55	93	57	96
SU5 (31T)	62	104	65	108
SU6 (35T)	65	109	67	111
SU7 (39T)	72	120	70	118
NRL (40T)	74	123	69	116

Table 32: Structure 1701151 – Comparison of Model Based Load Ratings versus Inspection Report Ratings

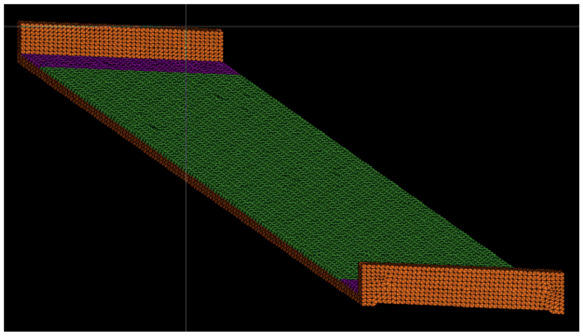
	FE Model		Inspection Report	
Truck (tons)	Inventory (tons)	Operating (tons)	Inventory (tons)	Operating (tons)
HS20 (36T)	74	124	19	32
Type 3 (25T)	60	100	15	26
Type 3-3 (40T)	119	199	30	50
Type 3S2 (40T)	92	153	24	40

Appendix – Load Rating Calculations

Structure 0118150 – US206 over Cedar Branch

Bridge 0118150 - US206 Over Cedar Branch

Slab Rating



General Bridge Information

1) Built in 1953
2) Length - 24.4' - NBIS length - 19.58' - Actual length - 16' 1"
3) Width - 50'
4) ADT - 8663 ADTT - 173 one direction
5) Structure in satisfactory condition (Condition rating = 6)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the reinforced concrete slab for the following rating vehicles: HS20, Type 3, Type 3-3,Type 3S2, and Lane loading

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 0118150 - US206 over Cedar Branch\Drawings

- As-built drawings, NJDOT, **Bridge 0118150 - Drawings with markups.pdf**
- Inspection report - Cycle 16 - **0118150_20110624cy16.pdf**

2) AASHTO, LFD Specifications 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum moments in slab for each rating vehicle
6) Calculate the load ratings.

Assumptions and Limitations:

1) A unit width of the thinnest portion of the slab was used as the governing section for capacity
2) Superimposed dead load was distributed over the entire width of the structure

Demand:

Constants

$\gamma_c := 150 \cdot pcf$	Unit weight on concrete
$\gamma_a := 144 \cdot pcf$	Unit weight of asphalt
$UW := 1 \cdot ft$	Unit width
$L1 := (1 \cdot ft + 1 \cdot in)$	Span length - centerline of bearing to centerline of bearing
$Skew := 50 \cdot deg$	Skew angle
$W_{road} := 52 \cdot ft$	Roadway width aligned with skew
$W_{edge} := 65 \cdot ft$	Bridge out to out width aligned with skew
$h := 13 \cdot in$	Slab depth
$lane := 20 \cdot ft$	Lane width

Dead Load

Dead Load Moment Due to Concrete Elements

$$M_{DL\_tot} := \frac{(114 \cdot kip \cdot ft \cdot ft)}{W_{edge}} = 1.75\ kip \cdot ft$$

Total dead load moment per foot of width - taken from FE model

$$V_{DL\_tot} := \frac{(131 \cdot kip \cdot ft)}{W_{edge}} = 2.02\ kip$$

Total dead load shear per foot of width - taken from FE model

Live Load Analysis

Live Load Moments

$$I_{LFR} := min\left(\frac{50\ ft}{L1 + 150 \cdot ft}, 0.30\right) = 0.30$$

$$M_{HS20} := 1.05 \cdot kip \cdot ft = 1.05\ kip \cdot ft$$

$$M_{type3S2} := 0.87 \cdot kip \cdot ft = 0.87\ ft \cdot kip$$

$$M_{type3} := 0.87 \cdot kip \cdot ft = 0.87\ ft \cdot kip$$

$$M_{type33} := 0.77 \cdot kip \cdot ft = 0.77\ ft \cdot kip$$

Live load moments derived from solid element stress distribution obtained from FE influence analysis

$$M_{Lane\_LFR} := 0.58 \cdot kip \cdot ft = 0.58\ ft \cdot kip$$

$$M_{SU4} := (0.71 \cdot kip \cdot ft) = 0.71\ ft \cdot kip$$

$$M_{SU5} := (0.68 \cdot kip \cdot ft) = 0.68\ ft \cdot kip$$

$$M_{SU6} := (0.75 \cdot kip \cdot ft) = 0.75\ ft \cdot kip$$

$$M_{SU7} := (0.76 \cdot kip \cdot ft) = 0.76\ ft \cdot kip$$

$$M_{NRL} := (0.77 \cdot kip \cdot ft) = 0.77\ ft \cdot kip$$

Live Load Shears

To calculate shear force from solid elements take the direct sum of shear stress on the section

$$V_{HS20} := 4.52 \cdot kip = 4.52\ kip$$

$$V_{type3} := 3.05 \cdot kip = 3.05\ kip$$

$$V_{type33} := 2.9 \cdot kip = 2.90\ kip$$

$V_{type3S2} := 2 \cdot kip = 2.00\ kip$

$V_{Lane\_LFR} := 4.5 \cdot kip = 4.50\ kip$

Live load shears (per foot) derived from solid element stress distribution from FE influence analysis.

$V_{SU4} := (3.3 \cdot kip) = 3.30\ kip$

$V_{SU5} := 3.5 \cdot kip = 3.50\ kip$

$V_{SU6} := (3.8 \cdot kip) = 3.80\ kip$

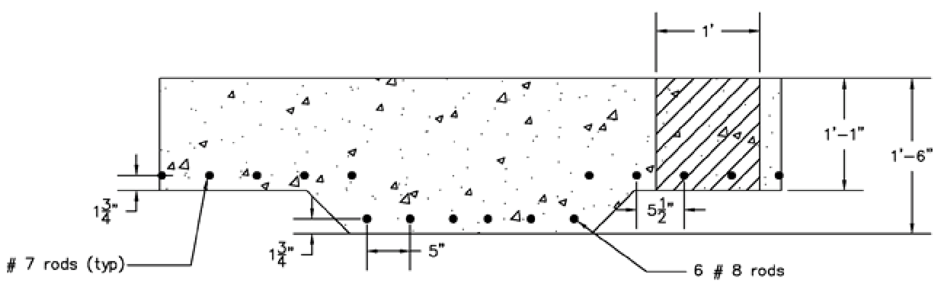
$V_{SU7} := (3.6 \cdot kip) = 3.60\ kip$

$V_{NRL} := (3.3 \cdot kip) = 3.30\ kip$

Capacity

Compute the nominal flexural resistance of a unit width section of the slab - Use thinnest slab section

Cross Section from as built plans



Slab – Unit Width

Constants

$f_c := 2500 \cdot psi$ Compressive strength of concrete for a bridge constructed before 1959 MBE Table 6A.5.2.1-1

$f_y := 33000 \cdot psi$ Yield stress of reinforcing steel for a bridge constructed before 1959 MBE Table 6A.5.2.2-1

$d_{bar} := \frac{7}{8} \cdot in$ — Reinforcement bar diameter

$bar_{space} := 5.5 \cdot in$ — Bar spacing

$n_{bars} := \frac{(12 \cdot in)}{bar_{space}} = 2.18$ — Number of bars in a 1' section

$A_s := n_{bars} \cdot \frac{\left(\pi \cdot (d_{bar})^2\right)}{4} = 1.31\ in^2$ — Area of reinforcing steel - 7/8" diameter rods @ 5.5" O.C

$\beta_1 := 0.85$

$b := UW = 12.00\ in$ — Width of slab section

$a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b} = 1.70\ in$ — Distance from the extreme compression fiber to the neutral axis of the section

$d_s := h - 1.75 \cdot in = 11.25\ in$ — Distance to C.G of steel from extreme compression fiber

Nominal Flexural Capacity

$$M_n := A_s \cdot f_y \cdot \left(d_s - \left(\frac{a}{2}\right)\right) = 37.53\ (kip \cdot ft)$$

Compute Nominal Shear Capacity

Slab does not have any prestressing or shear reinforcement so shear capacity is solely based on the shear capacity of concrete

$b := 12 \cdot in$

$$V_n := 2 \cdot psi \cdot \left(\sqrt{2500}\right) \cdot b \cdot d_s = 13.50\ kip$$

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.3$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.9$

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 10.63$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 17.75$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot \left(M_{Lane\_LFR} \cdot \left(1 + I_{LFR}\right)\right)} = 19.25$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 32.13$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 12.83$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 21.42$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\left(M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 16.23$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 27.09$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 12.83$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 21.42$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 15.72$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 26.25$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU5} \cdot \left(1 + I_{LFR}\right)} = 16.42$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 26.25$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 14.89$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 24.85$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 14.69$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 24.52$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 14.50$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 24.20$$

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{\left(\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(V_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 0.75$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.25$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{\left(\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(V_{Lane\_LFR} \cdot \left(1 + I_{LFR}\right)\right)} = 0.75$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.25$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{\left(\left(\phi_s \cdot V_n\right) - A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{type3} \cdot \left(1 + I_{LFR}\right)} = 1.11$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.85$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{\left(\phi_s \cdot V_n - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 1.16$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.94$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 1.69$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.82$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 1.02$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.71$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 0.97$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.61$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 0.89$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.48$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 0.94$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.57$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_n) - A_1 \cdot (V_{DL\_tot})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 1.02$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.71$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $Type3S2 := 40 \cdot ton$ $SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $SU4 := 27 \cdot ton$ $SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $SU5 := 31 \cdot ton$ $NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 382.77 \\ 320.80 \\ 649.05 \\ 513.29 \\ 424.55 \\ 508.95 \\ 517.27 \\ 569.22 \\ 579.95 \end{bmatrix} \textit{ton} \qquad FlexureOperating = \begin{bmatrix} 638.92 \\ 535.50 \\ 1083.42 \\ 856.80 \\ 708.67 \\ 813.65 \\ 863.44 \\ 950.16 \\ 968.07 \end{bmatrix} \textit{ton}$$

$$ShearInventory = \begin{bmatrix} 26.91 \\ 27.69 \\ 46.60 \\ 67.56 \\ 27.64 \\ 29.92 \\ 30.89 \\ 36.36 \\ 40.95 \end{bmatrix} \textit{ton} \qquad ShearOperating = \begin{bmatrix} 44.91 \\ 46.22 \\ 77.78 \\ 112.78 \\ 46.14 \\ 49.95 \\ 51.57 \\ 60.70 \\ 68.35 \end{bmatrix} \textit{ton}$$

Structure 1703152 – US40 over Salem Creek

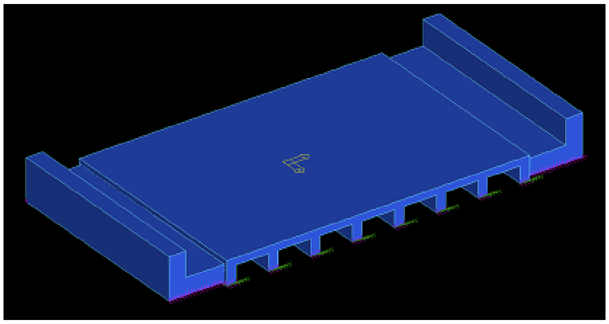
Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 1703152

By: CTY, 8-28-13
Chk'd: JBP 10-11-13

Bridge 1703152 - RT40 Over Salem Creek

T-Beam Rating - Positive Flexure and Shear



General Bridge Information

1) Built in 1919 - widened - 1929
2) Length - 24' - clear span 20'
3) Width - 44.2'
4) ADT - 15300 ADTT - 1071 one direction
5) Structure in fair condition (Condition rating = 5)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the reinforced concrete T-Beam for the following rating vehicles: HS20, Type 3, Type 3-3, Type 3S2, SU4-SU7 and NRL

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 1703152 - Route 40 over Salem Creek-EM\Drawings

- As-built drawings, NJDOT, **Bridge 1703152 - Drawings_Markup.pdf**
- Inspection report - Cycle 16 - **1703152_20110421cy16_Report.pdf**

2) AASHTO, LFD Bridge Design Specifications 1th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum force effects for each rating vehicle
6) Calculate the load ratings.

Assumptions and Limitations:

1) Superimposed dead load was distributed over 8 beams
2) Live load effects were distributed over 8 beams

Demand:

Constants

$\gamma_c := 150 \cdot pcf$	Unit weight on concrete
$\gamma_a := 144 \cdot pcf$	Unit weight of asphalt
$L := 20 \cdot ft = 240.00\ (in)$	Span length
$W_{road} := 40 \cdot ft + 3 \cdot in$	Roadway width
$W_{edge} := 44.2 \cdot ft$	Bridge out to out width
$S := 4.464 \cdot ft$	Beam spacing
$lane := 12 \cdot ft$	Lane width
$beams := 8$	

Dead Load

Dead Load Moment & Shear Due to Concrete Elements

$M_{DL\_tot} := ((18 \cdot kip \cdot ft)) = 18.00\ kip \cdot ft$ Total dead load moment on T-beam section

$V_{DL\_tot} := ((9 \cdot kip)) = 9.00\ kip$

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Project: NJDOT Bridge Resource Program
Subject: Bridge 1703152

By: CTY, 8-28-13
Chk'd: JBP 10-11-13

Live Load Analysis

Live Load Moments

$I_{LFR} := 0.30$ Impact factor - Unknown riding surface - MBE Table C6A.4.4.3-1

$M_{HS20} := 37 \cdot (\boldsymbol{kip} \cdot \boldsymbol{ft}) = 37.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{type3} := (29 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 29.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{tandem} := (42 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 42.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{type33} := (24 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 24.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{type3S2} := (29 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 29.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{Lane} := 6 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 6.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

Live load moments taken from FE model

$M_{SU4} := (33 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 33.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{SU7} := (35 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 35.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{SU5} := (34 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 34.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{NRL} := (36 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 36.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_{SU6} := (35 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft}) = 35.00\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

Live Load Shears

$V_{HS20} := 13 \cdot \boldsymbol{kip} = 13.00\ \boldsymbol{kip}$

$V_{type3} := 11 \cdot \boldsymbol{kip} = 11.00\ \boldsymbol{kip}$

$V_{tandem} := 15 \cdot \boldsymbol{kip} = 15.00\ \boldsymbol{kip}$

$V_{type33} := 9 \cdot \boldsymbol{kip} = 9.00\ \boldsymbol{kip}$

Live load shears taken from FE model

$V_{type3S2} := 11 \cdot \boldsymbol{kip} = 11.00\ \boldsymbol{kip}$

$V_{Lane} := 2 \cdot \boldsymbol{kip} = 2.00\ \boldsymbol{kip}$

$V_{SU4} := (11 \cdot \boldsymbol{kip}) = 11.00\ \boldsymbol{kip}$

$V_{SU7} := (12 \cdot \boldsymbol{kip}) = 12.00\ \boldsymbol{kip}$

$V_{SU5} := (11 \cdot \boldsymbol{kip}) = 11.00\ \boldsymbol{kip}$

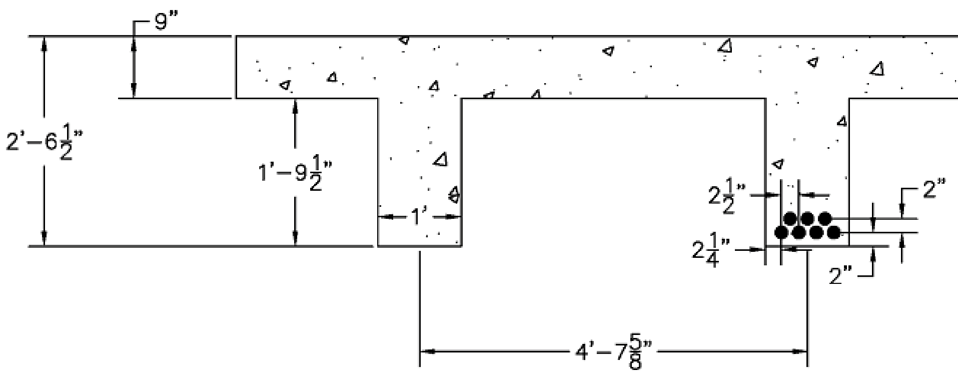
$V_{NRL} := (14 \cdot \boldsymbol{kip}) = 14.00\ \boldsymbol{kip}$

$V_{SU6} := (12 \cdot \boldsymbol{kip}) = 12.00\ \boldsymbol{kip}$

Capacity

Compute the nominal flexural capacity of an interior T-beam

Cross Section from as built plans



Constants

$f_c := 2500 \cdot psi$ Compressive strength of concrete for a bridge constructed before 1959 MBE Table 6A.5.2.1-1

$f_y := 33000 \cdot psi$ Yield stress of reinforcing steel for a bridge constructed before 1959 MBE Table 6A.5.2.2-1

$h := 2 \cdot ft + 6.5 \cdot in = 30.50\ in$ depth of T-beam section

$t_w := 12 \cdot in$ thickness of T-beam web

$d_{slab} := 9 \cdot in$ thickness of concrete slab

$b_{ftop} := 12 \cdot in$

$cover1 := 2 \cdot in$ distance from bottom of section to first rebar layer

$cover2 := 2 \cdot in + 2 \cdot in = 4.00\ in$ distance from bottom of section to second rebar layer

$d_{bar\_1} := \frac{7}{8} \cdot in$ Outside reinforcement bar diameter

$d_{bar\_2} := 1 \cdot in$ Center reinforcement bar diameter

$n_{bars} := 7$ Number of bars in T-beam

$n_{bars\_1} := 4$ Number of bars in first layer of rebar

$n_{bars\_2} := 3$ Number of bars in second layer of rebar

Reinforcement is comprised of 2 layers. The first layer has (4) 7/8" bars while the second layer has (2) 7/8" bars surrounding a 1" bar

$$A_s := n_{bars\_1} \cdot \frac{\left(\pi \cdot \left(d_{bar\_1}\right)^2\right)}{4} + \left(n_{bars\_2} - 1\right) \cdot \frac{\left(\pi \cdot \left(d_{bar\_1}\right)^2\right)}{4} + \frac{\left(\pi \cdot \left(d_{bar\_2}\right)^2\right)}{4} = 4.39\ in^2$$

$$y := \frac{\left(\left(cover1 \cdot n_{bars\_1}\right) + \left(cover2 \cdot n_{bars\_2}\right)\right)}{n_{bars}} = 2.86\ in$$ Distance to C.G of steel from bottom of section

$$d_s := h - y = 27.64\ in$$ depth from extreme compression fiber to reinforcing steel C.G

Flexure

Calculate effective slab width - LFD 8.10.1.1

Flange width is taken as the lesser of:

*1/4 * L*

12 ts + greater of tw or 1/2 bf top*

S (beam spacing)

$$b_{eff} := min\left(0.25 \cdot L, 12 \cdot d_{slab} + \left(\max\left(t_w, 0.5 \cdot b_{ftop}\right)\right), S\right) = 53.57\ in$$

$$\rho_{act} := \frac{A_s}{b_{eff} \cdot d_s} = 0.0030$$

Mu (ultimate moment on section) is found in accordance with AASHTO LFD Article 8.16

Consider a rectangular section with compression limited to top slab. Check MBE 6B.5.3.2 requirements for 75 percent of balanced condition

$$\beta_1 := 0.85$$

$$\rho_{bal} := \frac{\left(0.85 \cdot \beta_1 \cdot f_c\right)}{f_y} + \frac{87000 \cdot \mathbf{psi}}{87000 \cdot \mathbf{psi} + f_y} = 0.78$$ **LFD Eq. 8-18**

$$\rho_{max} := 0.75 \cdot \rho_{bal} = 0.58$$

$$\textbf{if}\left(\rho_{act} < \rho_{max}, \text{“OK”}, \text{“Not Ok”}\right) = \text{“OK”}$$

Calculate depth of equivalent stress block

$$a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b_{eff}} = 1.27\ in$$ Depth of equivalent stress block - LFD Eq 8-17

$$\textbf{if}\ (a < d_{slab}, \text{"OK – Within slab"}, \text{"Not within slab"}) = \text{"OK – Within slab"}$$

Nominal Flexural Capacity

$$M_n := A_s \cdot f_y \cdot \left(d_s - \left(\frac{a}{2}\right)\right) = 326.28\ \boldsymbol{kip \cdot ft}$$ **LFD Eq. 8-16**

Shear

Compute Nominal Shear Capacity

Stirrups: # 3 bars at 9.25" OC

$$d_{stirrup} := \frac{3}{8} \cdot \boldsymbol{in}$$ Stirrup diameter

$$s_{stirrup} := 9.25 \cdot \boldsymbol{in} = 9.25\ in$$ Stirrup spacing

$$A_v := 2 \cdot \left(\frac{\pi}{4}\right) \cdot (d_{stirrup})^2 = 0.22\ \boldsymbol{in}^2$$ Stirrup steel area

Shear Capacity

Shear resistance of a concrete section is taken as the sum of the shear strength provided by the concrete and the shear strength provided by the shear reinforcement

$$b_w := t_w = 12.00\ \boldsymbol{in}$$ Minimum beam web width

$$V_c := \left(2 \cdot \boldsymbol{psi} \cdot \sqrt{2500} \cdot b_w \cdot d_s\right) = 33.17\ kip$$ LFD Design Eq. 8-49

$$V_s := \frac{(A_v \cdot f_y \cdot d_s)}{s_{stirrup}} = 21.78\ \boldsymbol{kip}$$ LFD Design Eq. 8-53

$$V_n := V_c + V_s = 54.96\ \boldsymbol{kip}$$

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.85$

Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 2.60$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.34$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 3.31$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.53$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 4.00$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.69$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 3.31$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.53$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU4} \cdot (1 + I_{LFR})} = 2.91$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.86$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 2.83$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.86$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 2.75$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.58$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 2.75$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.58$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 2.67$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.46$$

Shear Ratings

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 0.97$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.61$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 1.14$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.91$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot V_{type33} \cdot (1 + I_{LFR})} = 1.40$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.33$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 1.14$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.91$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 1.14$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.91$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 1.14$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.91$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 1.05$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.75$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 1.05$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.75$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_f \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 0.97$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.61$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

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Project: NJDOT Bridge Resource Program
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By: CTY, 8-28-13
Chk'd: JBP 10-11-13

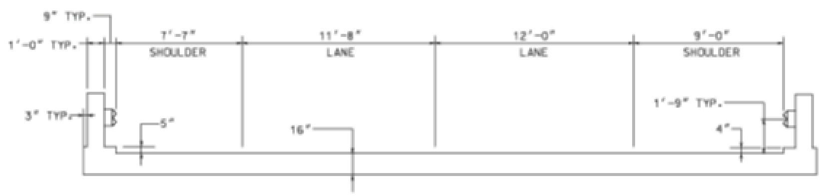
$$FlexureInventory=\begin{bmatrix}93.52\\82.86\\160.20\\132.58\\78.64\\87.64\\95.43\\106.42\\106.80\end{bmatrix}\ \mathbf{ton} \qquad FlexureOperating=\begin{bmatrix}156.11\\138.31\\267.41\\221.30\\131.27\\150.72\\159.30\\177.63\\178.27\end{bmatrix}\ \mathbf{ton}$$

$$ShearInventory=\begin{bmatrix}34.81\\28.57\\55.87\\45.71\\30.86\\35.43\\36.40\\40.59\\38.70\end{bmatrix}\ \mathbf{ton} \qquad ShearOperating=\begin{bmatrix}58.11\\47.69\\93.26\\76.30\\51.50\\59.14\\60.76\\67.76\\64.60\end{bmatrix}\ \mathbf{ton}$$

Structure 0324152 – US206 over Springers Brook

Bridge 0324152 - US206 Over Springers Brook

Slab Rating - Span 1



General Bridge Information

1) Built in 1929
2) Length - 55' - Span lengths 16'-1.5"', 16'-3"', 16'-1.5"
3) Width - 44'-3"
4) ADT - 8190 ADTT - 290 one direction
5) Superstructure in poor condition

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the reinforced concrete slab for the following rating vehicles: HL93, HS20, Type 3, Type 3-3, and Type 3S2

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring \Refined Load Ratings\Bridge 0324152 - US206 over Springers Brook\Drawings And Documents

- As-built drawings, NJDOT, **Bridge 0324152 - Drawings.pdf**
- Inspection report - Cycle 17 - **0324152_20110811cy17.PDF.pdf**

2) AASHTO, LFD Bridge Design Specifications 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum moments in slab for each rating vehicle
6) Calculate the load ratings.

Assumptions and Limitations:

1) 10% section loss has occurred to the reinforcment as specified in inspection report

Demand:

Constants

$\gamma_c := 150 \cdot pcf$ — Unit weight on concrete

$\gamma_a := 144 \cdot pcf$ — Unit weight of asphalt

$UW := 1 \cdot ft$ — Unit width

$L1 := (16 \cdot ft + 1.5 \cdot in)$ — Span length - centerline of bearing to centerline of bearing

$Skew := 20 \cdot deg$ — Skew angle

$W_{road} := 40 \cdot ft + 3 \cdot in$ — Roadway width

$W_{edge} := 44.25 \cdot ft$ — Bridge out to out width

$h := 16 \cdot in$ — Slab depth

$lane := 12 \cdot ft$ — Lane width

Dead Load

Dead Load Moment Due to Concrete Elements

The dead load moment for a unit width section is claculated by integrating the maximum bending stress distribution under dead load over the area of the unit width slab section

$M_{DL\_tot} := (8.2) \cdot kip \cdot ft = 8.20\ kip \cdot ft$

Total dead load moment per foot of width - taken from FE model

$V_{DL\_tot} := 1.5 \cdot kip = 1.50\ kip$

Live Load Analysis

Live Load Moments

$$I_{LFR} := min\left(\frac{50\ ft}{L1 + 150 \cdot ft}, 0.30\right) = 0.30$$

To calculate moments from solid elements take the area integral of the stress distribution on the section of interest

$$M_{HS20} := 3\ kip \cdot ft = 3.00\ kip \cdot ft$$

$$M_{type3} := 2.3\ kip \cdot ft = 2.30\ kip \cdot ft$$

$$M_{type33} := 2.2\ kip \cdot ft = 2.20\ kip \cdot ft$$

$$M_{type3S2} := 2.7 \cdot kip \cdot ft = 2.70\ kip \cdot ft$$

$$M_{Lane\_LFR} := 2 \cdot kip \cdot ft = 2.00\ ft \cdot kip$$

Live load moments derived from solid element stress distribution from FE influence analysis

$$M_{SU4} := 2.4 \cdot kip \cdot ft = 2.40\ ft \cdot kip$$

$$M_{SU5} := 2.8\ kip \cdot ft = 2.80\ ft \cdot kip$$

$$M_{SU6} := 2.7 \cdot kip \cdot ft = 2.70\ ft \cdot kip$$

$$M_{SU7} := 2.7 \cdot kip \cdot ft = 2.70\ ft \cdot kip$$

$$M_{NRL} := 2.7 \cdot kip \cdot ft = 2.70\ ft \cdot kip$$

Live Load Shears

To calculate shear force from solid elements take the direct sum of shear stress on the section

$V_{HS20} := 0.7 \cdot kip = 0.70\ kip$

$V_{type3} := 0.8 \cdot kip = 0.80\ kip$

$V_{type33} := 0.6 \cdot kip = 0.60\ kip$

$V_{type3S2} := 0.7 \cdot kip = 0.70\ kip$

$V_{Lane\_LFR} := 0.4 \cdot kip = 0.40\ kip$

$V_{SU4} := (0.9 \cdot kip) = 0.90\ kip$

$V_{SU5} := (0.9 \cdot kip) = 0.90\ kip$

Live load shears (per foot) derived from solid element stress distribution from FE influence analysis

$V_{SU6} := (0.9 \cdot kip) = 0.90\ kip$

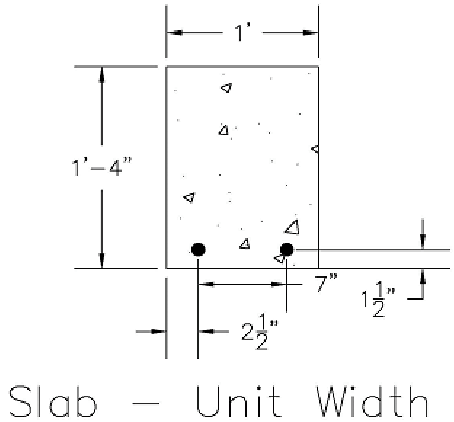
$V_{SU7} := (0.9 \cdot kip) = 0.90\ kip$

$V_{NRL} := (0.9 \cdot kip) = 0.90\ kip$

Capacity

Compute the nominal flexural capacity of a unit width section of the slab

Cross Section from as built plans



Constants

$f_c := 2.5 \cdot ksi$ Compressive strength of concrete for a bridge constructed before 1959 MBE Table 6A.5.2.1-1

$f_y := 33 \cdot ksi$ Yield stress of reinforcing steel for a bridge constructed before 1959 MBE Table 6A.5.2.2-1

$d_{bar} := 1 \cdot in$ Reinforcement bar diameter

$bar_{space} := 7 \cdot in$ Bar spacing

$n_{bars} := \frac{(12 \cdot in)}{bar_{space}} = 1.71$ Number of bars in a 1' section

$A_s := 0.9 \cdot n_{bars} \cdot \frac{\left(\pi \cdot (d_{bar})^2\right)}{4} = 1.21\ in^2$ Area of reinforcing steel - 1" diameter rods @ 7" - Includes 10% section loss specified in inspection report

$\beta_1 := 0.85$

$b := UW = 12.00\ in$ — Width of slab section

$$a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b} = 1.57\ in$$
Depth of equivalent stress block - LFD Eq 8-17

$d_s := h - 1.5 \cdot in = 14.50\ in$ — Distance to C.G of steel from extreme compression fiber

Nominal Flexural Capacity

$$M_n := A_s \cdot f_y \cdot \left(d_s - \left(\frac{a}{2}\right)\right) = 45.71\ kip \cdot ft$$
LFD Eq 8-16

Compute Nominal Shear Capacity

Slab does not have any prestressing or shear reinforcement so shear capacity is solely based on the shear capacity of concrete

$b_w := 12 \cdot in$

$V_c := 2 \cdot psi \cdot \sqrt{2500} \cdot b_w \cdot d_s = 17.40\ kip$ — LFD Eq. 8-49

$V_n := V_c = 17.40\ kip$

Load Factor Rating

Flexure

Load Factors:

$A_1 := 1.25$ — dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ — live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ — live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$

$\phi_s := 0.85$

LFD strength reduction factors

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{HS20}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 4.42$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.37$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - A_1 \cdot M_{Lane\_LFR}\right)}{A_{2\_inv} \cdot \left(M_{Lane\_LFR} \cdot \left(1 + I_{LFR}\right)\right)} = 6.85$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.43$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{type3}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 5.90$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.84$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{type33}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 6.19$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.32$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{type3S2})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 4.96$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.28$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU4})}{A_{2\_inv} \cdot M_{SU4} \cdot (1 + I_{LFR})} = 5.63$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.40$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU5})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 4.76$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.40$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU6})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 4.96$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.28$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU7})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 4.96$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.28$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{NRL})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 4.96$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.28$$

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{HS20}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 7.05$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.76$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{Lane\_LFR}))}{A_{2\_inv} \cdot (V_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 12.66$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 21.14$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{type3}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 6.11$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.20$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{\left(\phi_s \cdot V_n - \left(A_1 \cdot V_{type33}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 8.29$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 13.85$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{type3S2}\right)}{A_{2\_inv} \cdot V_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 7.05$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.76$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{SU4}\right)}{A_{2\_inv} \cdot V_{SU4} \cdot \left(1 + I_{LFR}\right)} = 5.38$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.98$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{SU5}\right)}{A_{2\_inv} \cdot V_{SU5} \cdot \left(1 + I_{LFR}\right)} = 5.38$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.98$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{SU6}\right)}{A_{2\_inv} \cdot V_{SU6} \cdot \left(1 + I_{LFR}\right)} = 5.38$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.98$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU7})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 5.38$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.98$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_n) - A_1 \cdot (V_{NRL})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 5.38$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.98$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 159.03 \\ 147.42 \\ 247.40 \\ 198.30 \\ 152.08 \\ 147.71 \\ 172.28 \\ 192.11 \\ 198.30 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 265.46 \\ 246.08 \\ 412.97 \\ 331.01 \\ 253.86 \\ 291.47 \\ 287.57 \\ 320.67 \\ 331.01 \end{bmatrix} ton$$

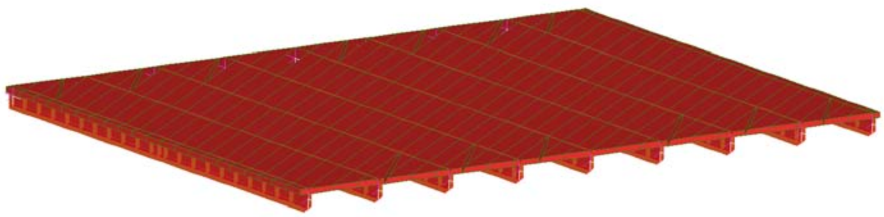
$$ShearInventory = \begin{bmatrix} 253.68 \\ 152.76 \\ 331.80 \\ 281.87 \\ 145.32 \\ 166.85 \\ 187.03 \\ 208.56 \\ 215.29 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 423.45 \\ 254.99 \\ 553.85 \\ 470.50 \\ 242.57 \\ 278.51 \\ 312.20 \\ 348.14 \\ 359.37 \end{bmatrix} ton$$

Structure 1512152 – NJ72 over Mill Creek

Intelligent Infrastructure Systems	Project: NJDOT Bridge Resource Program	By: CTY, 5-23-13
A Pennoni Company	Subject: Concrete Encased Steel I Section	Chk'd: JBP, 1-7-14

Bridge 1512152 - NJ 72 Over Mill Creek

Concrete Encased Steel Stringers



General Bridge Information

1) Built in 1930 - rehabbed in 1968
2) Length - 44'
3) Width - 115' 8"
4) ADT - 8190 ADTT - 290 one direction
5) Good condition

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for simply supported, concrete encased steel stringer bridge

References:

1) As-built drawings, NJDOT, **Bridge 1512152 - Drawings.pdf**
2) AASHTO, LFD Specifications, 17th Edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) Inspection report - Cycle 17 - **1512152_20101109cy17.PDF.pdf**

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum moments in members
6) Calculate the load ratings.

Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Concrete Encased Steel I Section

By: CTY, 5-23-13
Chk'd: JBP, 1-7-14

Assumptions and Limitations:

1) Vehicle combinations were limited to the rating vehicle placed in each lane. Combinations of different vehicles was not evaluated
2) Concrete encasement is neglected in capacity calculations

Material Properties:

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.5 \cdot ksi$ Compressive of concrete for a bridge constructed before 1959 - MBE Table 6A.5.2.1-1

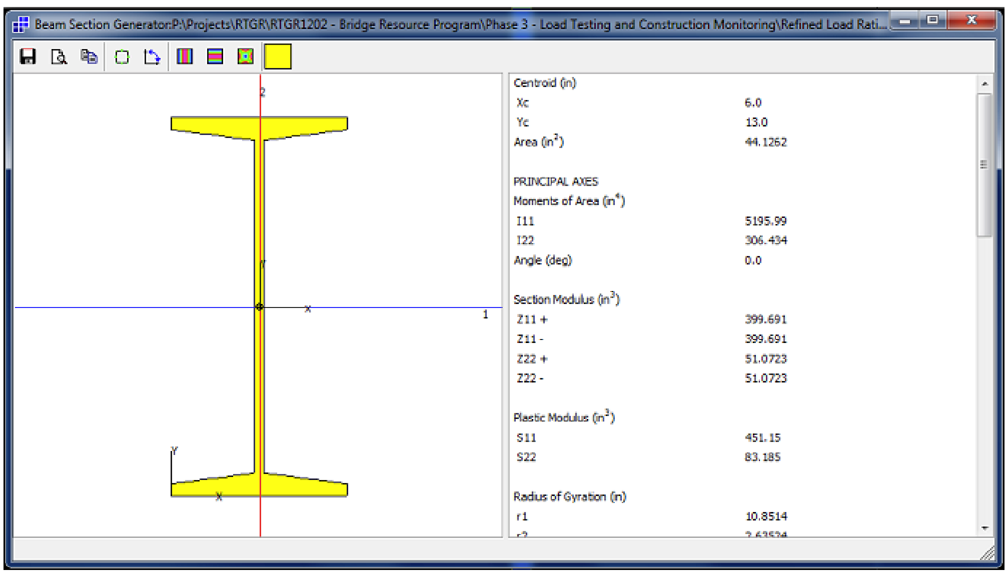
$f_{y\_s} := 30 \cdot ksi$ Yield strength of steel for a bridge constructed between 1905 and 1936 - MBE Table 6A.6.2.1-1

$f_{y\_rs} := 33 \cdot ksi$ Yield stess of reinforcing steel for a bridge constructed before 1959 - MBE Table 6A.5.2.2-1

$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ Concrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

Section Properties:



Noncomposite Section Properties Calculated From Strand 7

$d := 26 \cdot in$ depth of section

$t_f := 0.875 \cdot in$ flange thickness

$b_f := 12 \cdot in$ flange width

$t_w := 0.62 \cdot in$ web thickness

$A_{nc} := 32.7996 \cdot in^2$ area

$y_{bnc} := 12.7469 \cdot in$ distance to neutral axis from bottom of stringer

$I_{nc} := 5195.99 \cdot in^4$ moment of intertia

$Z_{nc} := 451.15 \cdot in^3$ plastic section modulus

$S_{t\_nc} := \frac{I_{nc}}{d - y_{bnc}} = 392.06\ in^3$ section modulus at top of steel

$S_{b\_nc} := \frac{I_{nc}}{y_{bnc}} = 407.63\ in^3$ section modulus at bottom of steel

Capacity:

Compute the nominal moment resistance of the the stringer at midspan assuming the capacity is dervied from the plastic moment of the non-composite section

Check compactness of the section

Compression Flange

$$\textbf{if}\left(\frac{b_f}{t_f} \le 0.75 \cdot \frac{4110}{\sqrt{30}}, \text{“Meets Criteria”}, \text{“Does not meet criteria”}\right) = \text{“Meets Criteria”}$$ LFD 10-93

Web Thickness

$$\text{if}\left(\frac{d-(2\cdot t_f)}{t_w} \le 0.75\cdot\frac{19230}{\sqrt{30}}, \text{“Meets Criteria”}, \text{“Does not meet criteria”}\right) = \text{“Meets Criteria”}$$

LFD 10-94

Section passes the 75% limit requirement on the above equalities so the interaction equation in LFD 10-95 is not required. The section is also continuously braced along the top flange by the composite encasement so the lateral bracing check is not necessary

$M_{ps} := f_{y\_s} \cdot Z_{nc} = 1127.88\ \boldsymbol{ft \cdot kip}$ Plastic moment of the noncomposite section - LFD 10-92

$M_n := M_{ps} = 1127.88\ \boldsymbol{ft \cdot kip}$ Nominal moment resistance

Compute the nominal shear resistance of the stringer - **LFD 10.48.7**

$k := 5$ assumed unstiffened web

$D := d - 2 \cdot t_f = 24.25\ \boldsymbol{in}$ Clear distance between flanges

$$C := \text{if}\ \frac{D}{t_w} < \frac{(6000\cdot\sqrt{k})}{\sqrt{30}} \;\; \Big\| \;\; 1.0 \;\; \text{if}\ \frac{(6000\cdot\sqrt{k})}{\sqrt{30}} \le \frac{D}{t_w} \le \frac{(7500\cdot\sqrt{k})}{\sqrt{30}} \;\; \Big\| \;\; \frac{(6000\cdot\sqrt{k})}{\left(\frac{D}{t_w}\right)\cdot\sqrt{30}} \;\; \text{if}\ \frac{D}{t_w} > \frac{(7500\cdot\sqrt{k})}{\sqrt{30}} \;\; \Big\| \;\; \frac{((4.5\cdot 10^7)\cdot k)}{\left(\frac{D}{t_w}\right)^2\cdot 30} \quad = 1.00$$

LFD 10-116/117

Calculate nominal shear capacity

$V_p := 0.58 \cdot f_{y\_s} \cdot D \cdot t_w = 261.61\ \boldsymbol{kip}$ LFD 10-115

$V_u := C \cdot V_p = 261.61\ \boldsymbol{kip}$ LFD 10-119

$V_n := V_u$

Demand:

$$I_{LFR} := min\left(\frac{50\ \boldsymbol{ft}}{44 \cdot \boldsymbol{ft} + 150 \cdot \boldsymbol{ft}}, 0.30\right) = 0.26$$

Moment Demand from FE Model

$M_{DL\_tot} := 187\ \boldsymbol{ft} \cdot \boldsymbol{kip} = 187.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{HS20} := 138 \cdot \boldsymbol{ft} \cdot \boldsymbol{kip} = 138.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{type3} := 104 \cdot \boldsymbol{ft} \cdot \boldsymbol{kip} = 104.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{type33} := 86 \cdot \boldsymbol{ft} \cdot \boldsymbol{kip} = 86.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{type3S2} := 56 \cdot \boldsymbol{ft} \cdot \boldsymbol{kip} = 56.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{Lane\_LFR} := 103 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 103.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{SU4} := 122.5 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 122.50\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{SU5} := 131 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 131.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{SU6} := 144 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 144.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{SU7} := 153 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 153.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

$M_{NRL} := 160 \cdot \boldsymbol{kip} \cdot \boldsymbol{ft} = 160.00\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

Shear Demand from FE Model

$V_{DL\_tot} := 47 \cdot \boldsymbol{kip} = 47.00\ \boldsymbol{kip}$

$V_{HS20} := 48 \cdot \boldsymbol{kip} = 48.00\ \boldsymbol{kip}$

$V_{type3} := 32 \cdot \boldsymbol{kip} = 32.00\ \boldsymbol{kip}$

$V_{type33} := 28 \cdot \boldsymbol{kip} = 28.00\ \boldsymbol{kip}$

$V_{type3S2} := 32 \cdot \boldsymbol{kip} = 32.00\ \boldsymbol{kip}$

$V_{Lane\_LFR} := 27 \cdot \boldsymbol{kip} = 27.00\ \boldsymbol{kip}$

$V_{SU4} := 28\ \boldsymbol{kip} = 28.00\ \boldsymbol{kip}$

$V_{SU5} := 31\ \boldsymbol{kip} = 31.00\ \boldsymbol{kip}$

$V_{SU6} := (32 \cdot \boldsymbol{kip}) = 32.00\ \boldsymbol{kip}$

$V_{SU7} := (33 \cdot \boldsymbol{kip}) = 33.00\ \boldsymbol{kip}$

$V_{NRL} := (33 \cdot \boldsymbol{kip}) = 33.00\ \boldsymbol{kip}$

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.85$

Flexure Rating

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{HS20}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 2.24$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.73$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - A_1 \cdot M_{Lane\_LFR}\right)}{A_{2\_inv} \cdot \left(M_{Lane\_LFR} \cdot \left(1 + I_{LFR}\right)\right)} = 3.15$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.26$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{type3}))}{A_{2\_inv} \cdot M_{type3} \cdot (1 + I_{LFR})} = 3.12$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.21$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{((M_n) - (A_1 \cdot M_{type33}))}{A_{2\_inv} \cdot M_{type33} \cdot (1 + I_{LFR})} = 4.35$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.26$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{type3S2})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 6.18$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.32$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU4})}{A_{2\_inv} \cdot M_{SU4} \cdot (1 + I_{LFR})} = 2.58$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.30$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU5})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 2.38$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.30$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU6})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 2.12$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.55$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU7})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 1.97$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.29$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{NRL})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 1.87$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.12$$

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{HS20}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 1.24$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.07$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{Lane\_LFR}))}{A_{2\_inv} \cdot (V_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 2.56$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.27$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{\left(\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{type3}\right)\right)}{A_{2\_inv} \cdot V_{type3} \cdot \left(1 + I_{LFR}\right)} = 2.09$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.49$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{\left(\phi_s \cdot V_n - \left(A_1 \cdot V_{type33}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 2.45$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.09$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{type3S2}\right)}{A_{2\_inv} \cdot V_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 2.09$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.49$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{\left(\phi_s \cdot V_n\right) - \left(A_1 \cdot V_{SU4}\right)}{A_{2\_inv} \cdot V_{SU4} \cdot \left(1 + I_{LFR}\right)} = 2.45$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.09$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU5})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 2.17$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.62$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU6})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 2.09$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.49$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU7})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 2.01$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.36$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_n) - A_1 \cdot (V_{NRL})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 2.01$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.36$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

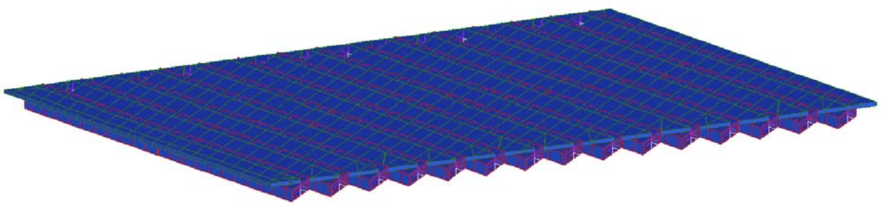
$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 80.54 \\ 77.96 \\ 173.89 \\ 247.34 \\ 69.61 \\ 73.81 \\ 73.84 \\ 76.45 \\ 74.66 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 134.43 \\ 130.13 \\ 290.26 \\ 412.87 \\ 116.19 \\ 133.41 \\ 123.25 \\ 127.61 \\ 124.63 \end{bmatrix} ton$$

$$ShearInventory = \begin{bmatrix} 44.62 \\ 52.20 \\ 98.07 \\ 83.52 \\ 66.20 \\ 67.28 \\ 72.56 \\ 77.92 \\ 80.44 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 74.48 \\ 87.14 \\ 163.71 \\ 139.42 \\ 110.50 \\ 112.30 \\ 121.12 \\ 130.07 \\ 134.27 \end{bmatrix} ton$$

Bridge 1512152 - NJ72 Over Mill Creek

Prestressed Concrete Adjacent Box Beam Rating



General Bridge Information

1) Built in 1930 - widened - 1968
2) Length - 44'
3) Width - 115' 8"
4) ADT - 45,380 ADTT - 1815 one direction
5) Structure in fair condition (Condition rating = 7)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the prestressed concrete adjacent box beam for the following rating vehicles: HS20, Type 3, Type 3-3,Type 3S2, SU4-SU7 and NRL

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 1512152 - NJ Rt72 over Mill Creek\Drawings

- As-built drawings, NJDOT, **Bridge 1512152 - Drawings_Markup.pdf**
- Inspection report - Cycle 16 - **1512152_20101109cy17.pdf**

2) AASHTO, LFD Specifications, 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

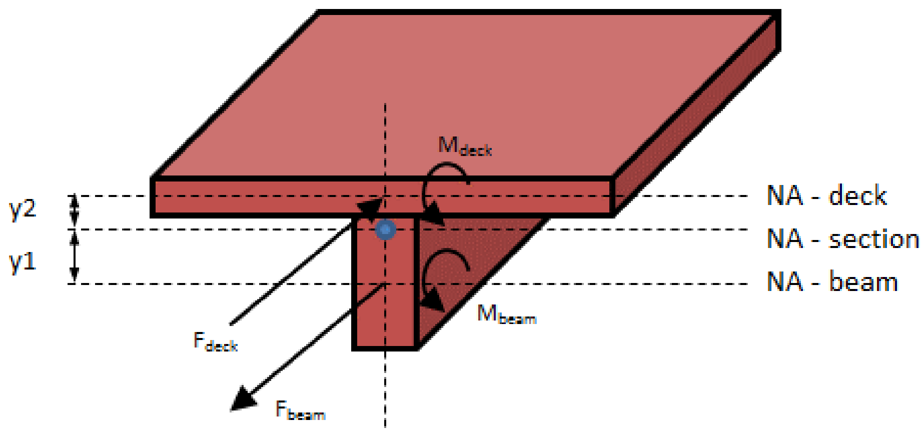
1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum force effects for each rating vehicle
6) Calculate the load ratings.

Demand:

Constants

$\gamma_c := 150 \cdot pcf$	Unit weight of normal weight concrete
$\gamma_a := 144 \cdot pcf$	Unit weight of asphalt
$L := 44 \cdot ft = 528.00\ (in)$	Span length
$W_{edge} := 115 \cdot ft + 8 \cdot in$	Bridge out to out width
$lane := 12 \cdot ft$	Lane width

The moment demand was derived from the beam/shell FE model using the following methodology and equation:



$$M_{NA-section} = M_{deck} + M_{beam} + F_{deck} * y2 + F_{beam} * y1$$

Dead Load

Dead Load Moment Due to Concrete Elements

$M_{DL\_tot} := 167 \cdot kip \cdot ft$ Total dead load moment on PS Box

Live Load Analysis

Live Load Moments

$$I_{LFR} := min\left(\frac{50\ ft}{44 \cdot ft + 150 \cdot ft}, 0.30\right) = 0.26$$

$$M_{HS20} := 75 \cdot kip \cdot ft = 75.00\ ft \cdot kip$$

$$M_{type3S2} := 51 \cdot kip \cdot ft = 51.00\ ft \cdot kip$$

$$M_{type3} := 42 \cdot kip \cdot ft = 42.00\ ft \cdot kip$$

$$M_{type33} := 36 \cdot kip \cdot ft = 36.00\ ft \cdot kip$$

$$M_{Lane\_LFR} := 55 \cdot kip \cdot ft = 55.00\ ft \cdot kip$$

$$M_{SU4} := (120 \cdot kip \cdot ft) = 120.00\ ft \cdot kip$$

$$M_{SU5} := (122 \cdot kip \cdot ft) = 122.00\ ft \cdot kip$$

$$M_{SU6} := (138 \cdot kip \cdot ft) = 138.00\ ft \cdot kip$$

$$M_{SU7} := (142 \cdot kip \cdot ft) = 142.00\ ft \cdot kip$$

$$M_{NRL} := (93 \cdot kip \cdot ft) = 93.00\ ft \cdot kip$$

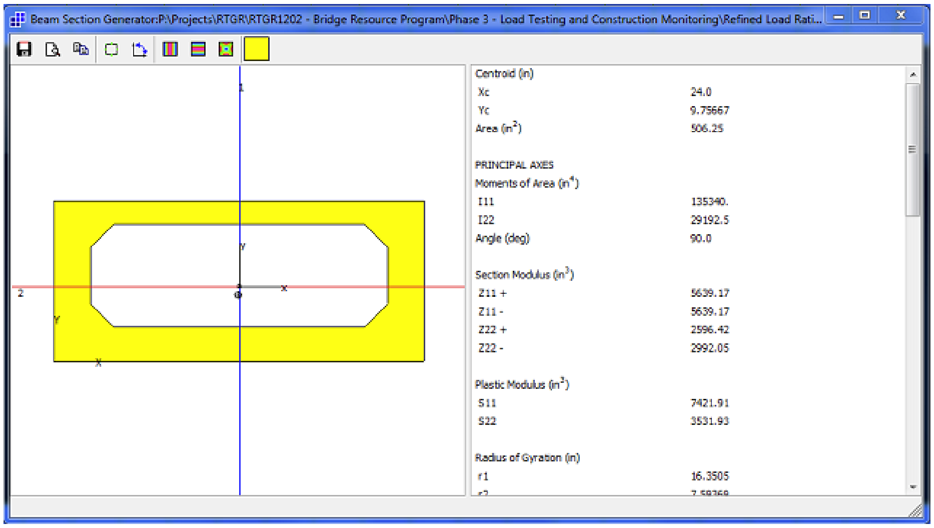
Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 1512152

By: JBP, 3-7-13
Chk'd: MTY, 3-22-13

Capacity

Box Beam Section Properties



$h := 21 \cdot in$	Height of section	$S_{bot} := 2992.05 \cdot in^3$	Section modulus (bottom)
$b := 48 \cdot in$	Width of section	$S_{top} := 2596.42 \cdot in^3$	Section modulus (top)
$A_c := 506.25 \cdot in^2$	Cross section area		
$I_x := 29152.5 \cdot in^4$	Moment of inertia		

Constants

$f_c := 3750 \cdot psi$ Compressive strength of prestressed concrete for a bridge constructed after 1959 — MBE Table 6A.5.2.1-1

$f_s := 36000 \cdot psi$ Yield strength of reinforcing steel for a bridge constructed after 1959 — MBE Table 6A.5.2.2-1

$f_{ci} := \dfrac{7}{4 + 0.85 \cdot 7} \cdot f_c = 2638.19\ psi$ Compressive strength of concrete at initial prestress assuming typical moist cured type-I cement (Nawy 2003)

$f_y := 36000 \cdot psi$ Yield strength of reinforcing steel for a bridge constructed after 1959 - MBE Table 6A.5.2.2-1

$f_{pu} := 250000 \cdot psi$ Tensile strength of prestressing strand constructed after 1963 - MBE Table 6A.5.2.3-1

$f_s := f_{pu} \cdot 0.8 = 200000.00\ psi$

$\gamma_1 := 0.28$ Assumes low relaxation strands

$\phi_{flex} := 1.0$ Flexural capacity reduction factor

$\phi_{shear} := 0.9$ Shear capacity reduction factor

Compute Nominal Flexural Capacity

$cg := 2.5 \cdot in$ location of prestressing strand center of gravity from bottom of section

$d_p := h - cg = 18.50\ in$ depth of prestressing center of gravity from extreme compression fiber

$n_{strands} := 17$ number of prestressing strands

$d_{strand} := \left(\frac{1}{2}\right) \cdot in = 0.50\ in$ diameter of prestressing strand

$A_{ps} := n_{strands} \cdot \frac{\left(\pi \cdot d_{strand}^{\ 2}\right)}{4} = 3.34\ in^2$ Area of prestressing strands

$\beta_1 := 0.8$

$p := \frac{A_{ps}}{A_c} = 0.01$ **Area ratio of steel to concrete**

$f_{su} := f_s \cdot \left(1 - \left(\left(\frac{\gamma_1}{\beta_1}\right) \cdot \left(p \cdot \frac{f_s}{f_c}\right)\right)\right) = 175384.39\ psi$ Average stress in steel at ultimate load

$M_n := \left(A_{ps} \cdot f_{su} \cdot h \cdot \left(1 - 0.6 \cdot \frac{\left(p \cdot f_{su}\right)}{f_c}\right)\right) = 834.94\ kip \cdot ft$ Flexural Strength - LFD Eq. 9-13

Compute Nominal Shear Capacity

Shear resistance of a prestressed beam is the sum of the shear resistance of concrete and the shear resistance provided by web reinforcement

Shear resistance of concrete should be taken as the lesser of Vci and Vcw

Vci need not be taken less than:

$V_{ci} := 1.7 \cdot psi \cdot \sqrt{5000} \cdot b \cdot 0.8 \cdot h = 96935.85\ lbf$ LFD Eq 9-27

$f_{pc} := 0 \cdot psi$

$V_p := 0 \cdot lbf$

$V_{cw} := \left(3.5 \cdot psi \cdot \sqrt{5000} + 0.3 \cdot f_{pc}\right) \cdot b \cdot 0.8 \cdot h + V_p = 199.57\ kip$ LFD Eq 9-29

$V_c := min\left(V_{ci}, V_{cw}\right) = 96.94\ kip$

Calculate shear resistance from web reinforcement

$s_d := \frac{4}{8} \cdot in$ Stirrup diameter

$A_v := 2 \cdot \pi \cdot \left(\frac{s_d^{\ 2}}{4}\right) = 0.39\ in^2$ Area of web reinforcement

$s := 6 \cdot in$ Stirrup spacing

$V_s := min\left(\frac{\left(A_v \cdot f_y \cdot 0.8 \cdot h\right)}{s}, 8 \cdot psi \cdot \sqrt{5000} \cdot b \cdot 0.8 \cdot h\right) = 39.58\ kip$ Shear resistance due to web reinforcement

$V_n := \left(V_c + V_s\right) = 136.52\ kip$ Unfactored nominal shear resistance

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.85$ strength reduction factors

Flexure Ratings

HS20 Rating

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{HS20}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 3.21$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.36$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - A_1 \cdot M_{Lane\_LFR}\right)}{A_{2\_inv} \cdot \left(M_{Lane\_LFR} \cdot \left(1 + I_{LFR}\right)\right)} = 4.55$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.59$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{type3}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 6.10$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.18$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\left(M_n\right) - \left(A_1 \cdot M_{type33}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 8.04$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 13.42$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{type3S2})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 4.94$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.25$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU4})}{A_{2\_inv} \cdot M_{SU4} \cdot (1 + I_{LFR})} = 1.84$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.07$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU5})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 1.80$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.07$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU6})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 1.54$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.57$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{SU7})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 1.48$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.47$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{NRL})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 2.50$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.18$$

Summary Rating Table

$HS20 := 36 \cdot ton$	$Type3S2 := 40 \cdot ton$	$SU6 := 34.75 \cdot ton$
$Type3 := 25 \cdot ton$	$SU4 := 27 \cdot ton$	$SU7 := 38.75 \cdot ton$
$Type33 := 40 \cdot ton$	$SU5 := 31 \cdot ton$	$NRL := 40 \cdot ton$

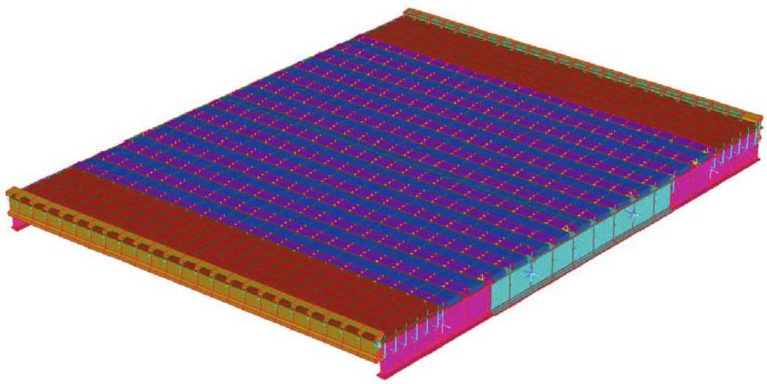
$Truck$	$FlexureInventory$	$FlexureOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$

$$FlexureInventory = \begin{bmatrix} 115.67 \\ 152.43 \\ 321.59 \\ 197.62 \\ 49.58 \\ 55.76 \\ 53.42 \\ 57.39 \\ 100.10 \end{bmatrix} \, ton \qquad FlexureOperating = \begin{bmatrix} 193.08 \\ 254.45 \\ 536.81 \\ 329.88 \\ 82.76 \\ 95.03 \\ 89.16 \\ 95.79 \\ 167.09 \end{bmatrix} \, ton$$

Structure 1516152 – NJ166 over Toms River

Bridge 1516152 - NJ Route 166 over Toms River

Interior Stringer Rating



General Bridge Information

1) Built in 1928
2) Length - 50' - NBIS length
3) Width - 60'
4) ADT - 24,690 ADTT - 988 one direction
5) Superstructure in fair condition (Condition rating = 5)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for an interior partially encased steel stringer. The rating vehicles include HS20, Type 3, Type 3-3, Type 3S2, SU4-SU7 and NRL

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 1516152 - NJ166 over Toms River\Drawings and Inspection Reports

- As-built drawings, NJDOT, **Bridge 1516152 - Drawings.pdf**
- Inspection report - Cycle 16 - **1516152_20101005cy16.pdf**

2) AASHTO, LFD Specifications, 17th Edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43
5) NCHRP Research Results Digest Number 234 - Manual for Bridge Rating through Load Testing
6) AISC Manual of Steel Construction - LRFD - 3rd edition

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum moments in members
6) Calculate the load ratings.

Assumptions and Limitations:

1) Assuming partial concrete encasement provides composite action between beams and deck for both capacity and demand - in model the modulus of the concrete in the encasement was reduced to 0 in order to eliminate all stiffness.
2) Neglecting the contribution of the partial encasement to the capacity of the beam section
3) Neglecting negative end moments of stringers (I-Beams C & D in model)

Material Properties:

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.5 \cdot ksi$ Compressive of concrete for a bridge constructed before 1959 - MBE Table 6A.5.2.1-1

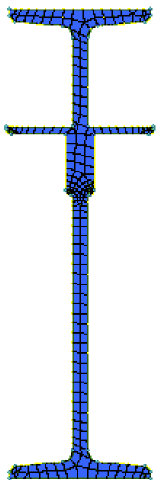
$f_{y\_s} := 30 \cdot ksi$ Yield strength of steel for a bridge constructed between 1905 and 1936 - MBE Table 6A.6.2.1-1

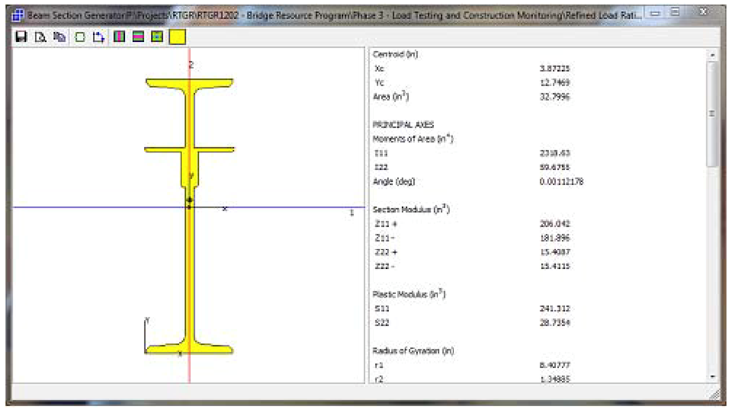
$f_{y\_rs} := 33 \cdot ksi$ Yield stess of reinforcing steel for a bridge constructed before 1959 - MBE Table 6A.5.2.2-1

$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ Concrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

Section Properties:





Noncomposite Section Properties Calculated From Strand 7

$d := 24 \cdot in$	depth of section
$t_f := 0.870 \cdot in$	flange thickness
$b_f := 7.25 \cdot in$	flange width
$t_w := 0.745 \cdot in$	web thickness
$t_h := 0\ in$	haunch thickness
$d_w := d - (2 \cdot t_f) = 22.26\ in$	web depth
$t_{ang} := \frac{3}{8} \cdot in$	angle leg thickness
$b_{ang} := 3.5 \cdot in$	angle leg length
$A_{nc} := 32.7996 \cdot in^2$	area
$y_{bnc} := 12.7469 \cdot in$	distance to neutral axis from bottom of stringer
$I_{nc} := 2318.63 \cdot in^4$	moment of intertia
$Z_{nc} := 240.197 \cdot in^3$	plastic section modulus

$S_{t\_nc} := \dfrac{I_{nc}}{d - y_{bnc}} = 206.04\ in^3$ section modulus at top of steel

$S_{b\_nc} := \dfrac{I_{nc}}{y_{bnc}} = 181.90\ in^3$ section modulus at bottom of steel

Composite Section Properties

$t_s := 5.5 \cdot in$ thickness of deck slab from top of beam (ignores encasement depth)

$S := 38.5 \cdot in$ interior stringer spacing

$L := 50 \cdot ft = 600.00\ in$ stringer length

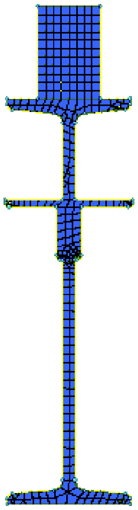
Effective Flange Width, beff

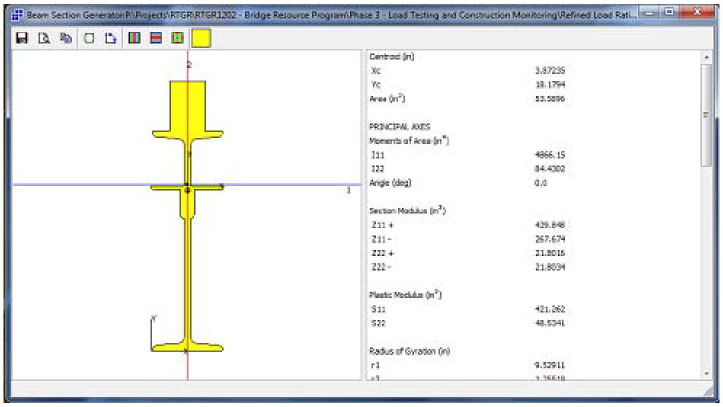
$b_{eff} := min\left(0.25 \cdot L, 12 \cdot t_s + \max\left(t_w, 0.5 \cdot b_f\right), S\right) = 38.50\ in$ Minimum flange width

Modular ratio, n

$n := \dfrac{E_s}{E_c} = 10.18$

Short term composite (n)





Short Term Composite Section Properties from Strand 7

$y_{b\_stc} := 18.1794 \cdot in$ — distance to neutral axis form bottom of beam

$I_{stc} := 4866.15 \cdot in^4$ — short tem composite moment of inertia

$$S_{t\_stc} := \frac{I_{stc}}{d - y_{b\_stc}} = 836.02\ in^3$$

section modulus at top of steel

$$S_{b\_stc} := \frac{I_{stc}}{y_{b\_stc}} = 267.67\ in^3$$

section modulus at bottom of steel

Compute Nominal Flexural Capacity of Section

For braced, compact, composite section Mr = Mu - LFD 10.50.1.1

where Mu is found in accordance with applicable load factor provisions of AASHTO

Check assumptions:

1. Section is fully braced along top flange by deck for live load and secondary dead load

2. To check if section is compact, need to apply provisions of AASHTO LFD 10.50.1.1.1 as follows

The compressive force in the slab C is equal to the smallest value given by the following equations (LFD 10.50.1.1.1a):

$C_{conc} := 0.85 \cdot f_c \cdot b_{eff} \cdot t_s = 449.97\ kip$ — LFD Eq. 10-123

$C_{STL} := A_{nc} \cdot f_{y\_s} = 983.99\ kip$ — LFD Eq. 10-124

$$\textbf{if}\left(C_{conc} < C_{STL}, \text{“Concrete controls”}, \text{“Steel controls”}\right) = \text{“Concrete controls”}$$

$$C := min\left(C_{conc}, C_{STL}\right) = 449.97\ kip$$

Capacity

$$C' := \frac{\left(C_{STL} - C_{conc}\right)}{2} = 267.01\ kip$$

LFD Eq. 10-126

Compare C' and AFy - top flange - AASHTO LFD 10.50.1.1.1(d)

$$\textbf{if}\left(t_f \cdot b_f \cdot f_{y\_s} > C', \text{“NA is in the top flange”}, \text{“NA is in the web”}\right) = \text{“NA is in the web”}$$

$D := d - 2 \cdot t_f = 22.26 \; \boldsymbol{in}$ Unsupported distance between flange components

$$y1 := t_f + \frac{\left(C' - \left(t_f \cdot b_f \cdot f_{y\_s}\right)\right)}{t_w \cdot \left(d - 2 \cdot t_f\right) \cdot f_{y\_s}} \cdot D = 4.35 \; \boldsymbol{in}$$ **LFD Eq. 10-128**

Since the compressive force in the steel is below the top flange at the plastic moment, web slenderness and ducitility should be checked

$D_{cp} := y1 - t_f = 3.48 \; \boldsymbol{in}$

$$\textbf{if}\left(2 \cdot \frac{D_{cp}}{t_w} < \frac{19230 \cdot \boldsymbol{psi}}{\sqrt{36000 \cdot \boldsymbol{psi}}}, \text{“Compact”}, \text{“Noncompact”}\right) = \text{“Compact”}$$ **LFD Eq. 10-129**

$D_p := t_s + y1 = 9.85 \; \boldsymbol{in}$ Distance from top of slab to PNA

$\beta := 0.9$

$$D' := \beta \cdot \frac{\left(d + t_s + t_h\right)}{7.5} = 3.54 \; \boldsymbol{in}$$

$$\textbf{if}\left(\frac{D_p}{D'} \le 5, \text{“Ductility satisfied”}, \text{“Ductility not satisfied”}\right) = \text{“Ductility satisfied”}$$

Dp exceeds D' LFD eq 10-129c is required to calculate Mu. Therefore calculation of the yield moment and the plastic moment is needed

$M_y := f_{y\_s} \cdot S_{b\_stc} = 669.18 \; \boldsymbol{kip \cdot ft}$

Calculate plastic moment - Take moments about the PNA

$P_{conc} := 0.85 \cdot f_c \cdot b_{eff} \cdot t_s = 449.97 \; \boldsymbol{kip}$

$$y_{conc} := D_p - \left(\frac{t_s}{2}\right) = 7.10 \; \boldsymbol{in}$$

$P_t := \left(\left(t_f \cdot b_f\right) + \left(t_w \cdot \left(D_p - t_s - t_f\right)\right)\right) \cdot f_{y\_s} = 267.01 \; \boldsymbol{kip}$

$$y_t := \frac{\left(\left(t_f \cdot b_f \cdot \left(D_p - t_s - \frac{t_f}{2}\right)\right) + \left(t_w \cdot \left(D_p - t_s - t_f\right) \cdot \frac{\left(D_p - t_s - t_f\right)}{2}\right)\right)}{\left(t_f \cdot b_f\right) + t_w \cdot \left(D_p - t_s - t_f\right)} = 3.28 \; \boldsymbol{in}$$

$$P_w := t_w \cdot (d - D_p - t_f) \cdot f_{y\_s} = 296.80\ \textit{kip}$$

$$y_w := \frac{(d - D_p - t_f)}{2} = 6.64\ \textit{in}$$

$$P_b := t_f \cdot b_f \cdot f_{y\_s} = 189.23\ \textit{kip}$$

$$y_b := d - \left(\frac{t_f}{2}\right) = 23.57\ \textit{in}$$

$$M_p := P_{conc} \cdot y_{conc} + P_t \cdot y_t + P_w \cdot y_w + P_b \cdot y_b = 875.08\ \textit{kip} \cdot \textit{ft}$$

$$M_R := \frac{(5 \cdot M_p - 0.85 \cdot M_y)}{4} + \frac{(0.85 \cdot M_y - M_p)}{4} \cdot \left(\frac{D_p}{D'}\right) = 738.59\ \textit{kip} \cdot \textit{ft}$$

LFD Eq. 10-129c

$$M_n := M_R$$

Compute Nominal Shear Capacity of Section

$$V_p := 0.58 \cdot f_{y\_s} \cdot D \cdot t_w = 288.56\ \textit{kip}$$

Calculate the buckling coefficient of the girder web

$k := 5$ Unstiffened web

$$F := \left\| \begin{array}{l} \text{if } \left(\dfrac{D}{t_w} < \dfrac{(6000 \cdot \sqrt{k})}{\sqrt{36000}}\right) \\ \quad \| \, C \leftarrow 1.0 \\ \text{else if } \dfrac{(6000 \cdot \sqrt{k})}{\sqrt{36000}} \le \dfrac{D}{t_w} \le \dfrac{(7500 \cdot \sqrt{k})}{\sqrt{36000}} \\ \quad \left\| \, C \leftarrow \dfrac{(6000 \cdot \sqrt{k})}{\dfrac{D}{t_w} \sqrt{36000}} \right. \\ \text{else} \\ \quad \left\| \, C \leftarrow \dfrac{4.5 \cdot 10^7\ k}{\left(\dfrac{D}{t_w}\right)^2 \sqrt{36000}} \right. \end{array} \right\| = 1.00$$

Calculate the buckling coefficient of the girder web. This is equal to the buckling shear stress divided by the shear yield stress. ***LFD Eq. 10-116***

$C_{buck} := F$

$V_u := C_{buck} \cdot V_p = 288.56\ \textbf{kip}$

Nominal shear capacity. ***LFD Eq. 10-113***

$V_n := V_u$

Demand:

$I_{LFR} := 0.30$ Impact factor - Unknown riding surface - MBE Table C6A.4.4.3-1

Moment Demand from FE Model

$M_{DL\_tot} := 46 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{HS20} := 103 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{Lane\_LFR} := 33 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{type3} := 36 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{type33} := 47 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{type3S2} := 36 \cdot \textbf{ft} \cdot \textbf{kip}$

$M_{SU4} := 67 \cdot \textbf{kip} \cdot \textbf{ft}$

$M_{SU5} := 57 \cdot \textbf{kip} \cdot \textbf{ft}$

$M_{SU6} := 75 \cdot \textbf{kip} \cdot \textbf{ft}$

$M_{SU7} := 76 \cdot \textbf{kip} \cdot \textbf{ft}$

$M_{NRL} := 77 \cdot \textbf{kip} \cdot \textbf{ft}$

Live load moments taken from FE model. HL93 loading is a notional loading taken as the greater of the HS20 or tandem force effect

Live Load Shears

To calculate shear force from solid elements take the direct sum of shear stress on the section

$V_{HS20} := 42 \cdot \textbf{kip} = 42.00\ \textbf{kip}$

$V_{DL\_total} := 35 \cdot \textbf{kip} = 35.00\ \textbf{kip}$

$V_{type3} := 32 \cdot \textbf{kip} = 32.00\ \textbf{kip}$

$V_{type33} := 29 \cdot \textbf{kip} = 29.00\ \textbf{kip}$

$V_{type3S2} := 36 \cdot \textbf{kip} = 36.00\ \textbf{kip}$

$V_{SU5} := (44 \cdot \textbf{kip}) = 44.00\ \textbf{kip}$

$V_{SU6} := (54 \cdot \textbf{kip}) = 54.00\ \textbf{kip}$

$V_{SU7} := (61 \cdot \textbf{kip}) = 61.00\ \textbf{kip}$

$V_{NRL} := (64 \cdot \textbf{kip}) = 64.00\ \textbf{kip}$

$V_{Lane\_LFR} := 34 \cdot kip = 34.00\ kip$

Live load shears (per foot) derived from solid element stress distribution from FE influence analysis

$V_{SU4} := (48 \cdot kip) = 48.00\ kip$

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$

Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$

Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.85$ strength reduction factors

Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{HS20} \cdot (1 + I_{LFR}))} = 2.09$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.49$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type3} \cdot (1 + I_{LFR})} = 5.98$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.98$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 4.58$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.64$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 5.98$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.98$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 3.21$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.36$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU5} \cdot \left(1 + I_{LFR}\right)} = 3.78$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.36$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU6} \cdot \left(1 + I_{LFR}\right)} = 2.87$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.79$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 2.83$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.73$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 2.80$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.67$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR} := \frac{((\phi_f \cdot M_n) - A_1 \cdot M_{Lane\_LFR})}{A_{2\_inv} \cdot (M_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 6.70$$

$$OR_{Lane\_LFR} := IR_{Lane\_LFR} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.18$$

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{HS20}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 1.63$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.72$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{Lane\_LFR}))}{A_{2\_inv} \cdot (V_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 2.11$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.53$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{type3}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 2.27$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.80$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{(\phi_s \cdot V_n - (A_1 \cdot V_{type33}))}{A_{2\_inv} \cdot V_{type33} \cdot (1 + I_{LFR})} = 2.56$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.26$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{type3S2})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 1.97$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.29$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU4})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 1.37$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.28$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU5})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 1.53$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.56$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU6})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 1.17$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.95$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{SU7})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 0.98$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.64$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_n) - A_1 \cdot (V_{NRL})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 0.92$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 1.53$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 75.23 \\ 149.48 \\ 183.20 \\ 239.17 \\ 86.74 \\ 117.07 \\ 99.73 \\ 109.75 \\ 111.82 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 125.58 \\ 249.52 \\ 305.79 \\ 399.23 \\ 144.80 \\ 166.25 \\ 166.48 \\ 183.20 \\ 186.65 \end{bmatrix} ton$$

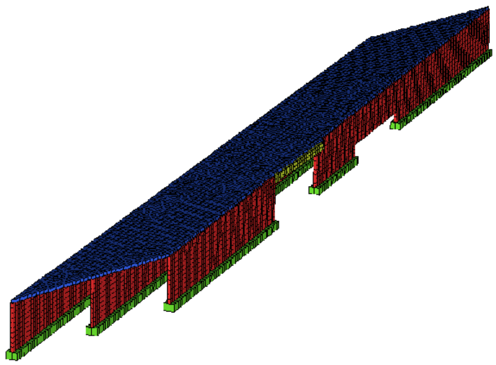
$$ShearInventory = \begin{bmatrix} 58.57 \\ 56.85 \\ 102.20 \\ 78.88 \\ 36.94 \\ 47.52 \\ 40.55 \\ 38.06 \\ 36.62 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 97.77 \\ 94.89 \\ 170.60 \\ 131.67 \\ 61.67 \\ 79.32 \\ 67.69 \\ 63.53 \\ 61.12 \end{bmatrix} ton$$

Structure 1103152 – US1 over D&R Canal

Intelligent Infrastructure Systems	Project: NJDOT Bridge Resource Program	By: JBP, 3-4-13
A Pennoni Company	Subject: Bridge 1103152	Chk'd:MTY 3-5-13 Rev JBP 8-1-13

Bridge 1103152 - Route US1 Over D&R Canal

Two-Span Continuous Slab Rating - Positive Flexure and Shear



General Bridge Information

1) Built in 1959, widened 1974
2) NBIS length - 77' - Actual length (from drawings, in model) - 80'
3) Width (out-to-out) - 159'
4) ADT - 2080 ADTT
5) Structure in satisfactory condition (Condition rating = 6)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the two span continuous reinforced concrete slab for the following rating vehicles: HS20, Type 3, Type 3-3, Type 3S2, SU4, SU5, SU6, SU7, and NRL

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 1103152 - Route US1 over D&R Canal\Drawings and Documents

- As-built drawings, NJDOT, **Bridge 1103152 - Drawings.pdf**
- Inspection report - Cycle 16 - **1103152_20101019cy16.pdf**

2) AASHTO, LFD Bridge Design Specifications 17th Edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum moments in slab for each rating vehicle
6) Calculate the load ratings.

Demand:

Constants

$\gamma_c := 150 \cdot pcf$ Unit weight on concrete

$\gamma_a := 144 \cdot pcf$ Unit weight of asphalt

$UW := 1 \cdot ft$ Unit width

$L1 := \left(\frac{57}{2} \cdot ft\right)$ Span length - centerline of bearing to centerline of bearing

$Skew := 50 \cdot deg$ Skew angle

$W_{road} := 113 \cdot ft$ Roadway width - 1 merge lane + 2 NB lanes + 2 SB lanes

$W_{edge} := 159 \cdot ft$ Bridge out to out width

$h := 15 \cdot in$ Slab depth

$lane := 18 \cdot ft$ Lane width

Dead Load

Dead Load Moment Due to Concrete Elements

$M_{DL\_tot} := 3.25 \cdot kip \cdot ft$ Total dead load moment per foot width - taken from FE model

$V_{DL\_tot} := 0.8 \cdot kip$ Total dead load shear per foot width - taken from FE model

Live Load Analysis

Live Load Moments

$I_{LFR} := min\left(\frac{50\ ft}{L1 + 150 \cdot ft}, 0.30\right) = 0.28$ Impact factor

Live load moments derived from plate elements (per foot)

$M_{HS20} := 5.76 \cdot kip \cdot ft$ $M_{SU4} := 4.05 \cdot kip \cdot ft$

$M_{type3S2} := 4.13 \cdot kip \cdot ft$ $M_{SU5} := 4.58 \cdot kip \cdot ft$

$M_{type3} := 5.59 \cdot kip \cdot ft$ $M_{SU6} := 4.72 \cdot kip \cdot ft$

$M_{type33} := 3.46 \cdot kip \cdot ft$ $M_{SU7} := 5.01 \cdot kip \cdot ft$

$M_{Lane} := 5.13 \cdot kip \cdot ft$ $M_{NRL} := 5.19 \cdot kip \cdot ft$

Live Load Shears

Live load shears derived from plate elements (per foot)

$V_{HS20} := 1.3 \cdot kip$ $V_{SU4} := 0.936 \cdot kip$

$V_{type3S2} := 1.02 \cdot kip$ $V_{SU5} := 0.945 \cdot kip$

$V_{type3} := 0.9 \cdot kip$ $V_{SU6} := 1.086 \cdot kip$

$V_{type33} := 0.689 \cdot kip$ $V_{SU7} := 0.949 \cdot kip$

$V_{Lane} := 1.55 \cdot kip$ $V_{NRL} := 1.02 \cdot kip$

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A Pennoni Company
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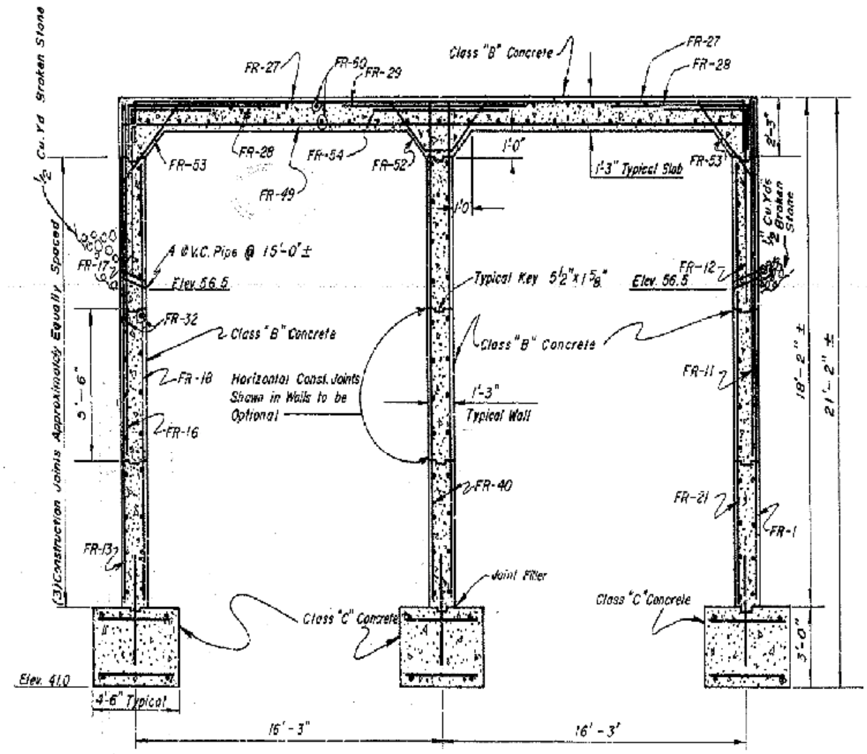
Chk'd:MTY 3-5-13 Rev JBP 8-1-13

Capacity

Flexural Capacity

Compute the nominal flexural capacity of a unit width section of the slab - Use thinnest slab section

Cross Section from as built plans



SECTION A-A
Scale: 1/4" = 1'-0"

Constants

$f_c := 3 \cdot ksi$ Compressive strength of concrete for a bridge constructed 1959 or later MBE Table 6A.5.2.1-1

$f_y := 40 \cdot ksi$ Yield stress of reinforcing steel specified in bridge plans

$d_{bar} := \frac{6}{8} \cdot in$ Reinforcement bar diameter

$bar_{space} := 8 \cdot in$ Bar spacing

$n_{bars} := \frac{(12 \cdot in)}{bar_{space}} = 1.50$ Number of bars in a 1' section

$A_s := n_{bars} \cdot \frac{\left(\pi \cdot (d_{bar})^2\right)}{4} = 0.66\ in^2$ Area of reinforcing steel - 6/8" diameter bars @ 8" O.C

$b := UW = 12.00\ in$ Width of slab section

$a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b} = 0.87\ in$ Depth of equivalent stress block, LFD 8-17

$d_s := h - 1.5 \cdot in = 13.50\ in$ Distance to C.G of steel from extreme compression fiber

Nominal Flexural Capacity

$M_n := A_s \cdot f_y \cdot \left(d_s - \left(\frac{a}{2}\right)\right) = 28.86\ kip \cdot ft$ LFD Eq 8-16

Shear

Compute Nominal Shear Capacity

Slab does not have any prestressing or shear reinforcement so shear capacity is solely based on the shear capacity of concrete

$b_w := 12 \cdot in$

$V_c := 2 \cdot psi \cdot \sqrt{2500} \cdot b_w \cdot d_s = 16.20\ kip$ LFD Eq. 8-49

$V_n := V_c = 16.20\ kip$

Load Factor Rating

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$

Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$

Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.85$ Strength reduction factors

Flexure Ratings

HS20 Rating

$$IR_{HS20str} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{HS20} \cdot (1 + I_{LFR}))} = 1.37$$

$$OR_{HS20str} := IR_{HS20str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.29$$

Lane Load Rating

$$IR_{Lanestr} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{Lane} \cdot (1 + I_{LFR}))} = 1.54$$

$$OR_{Lanestr} := IR_{Lanestr} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.57$$

AASHTO Type 3 Rating

$$IR_{type3str} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type3} \cdot (1 + I_{LFR})} = 1.41$$

$$OR_{type3str} := IR_{type3str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.36$$

AASHTO Type 3-3 Rating

$$IR_{type33str} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type33} \cdot (1 + I_{LFR})} = 2.28$$

$$OR_{type33str} := IR_{type33str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.81$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 1.91$$

$$OR_{type3S2str} := IR_{type3S2str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.19$$

AASHTO SU4 Rating

$$IR_{SU4str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU4} \cdot (1 + I_{LFR})} = 1.95$$

$$OR_{SU4str} := IR_{SU4str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.25$$

AASHTO SU5 Rating

$$IR_{SU5str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 1.72$$

$$OR_{SU5str} := IR_{SU5str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.88$$

AASHTO SU6 Rating

$$IR_{SU6str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 1.67$$

$$OR_{SU6str} := IR_{SU6str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.79$$

AASHTO SU7 Rating

$$IR_{SU7str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 1.57$$

$$OR_{SU7str} := IR_{SU7str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.63$$

AASHTO NRL Rating

$$IR_{NRLstr} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 1.52$$

$$OR_{NRLstr} := IR_{NRLstr} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.54$$

Shear Ratings

HS20 Rating

$$IR_{HS20sh} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 3.54$$

$$OR_{HS20sh} := IR_{HS20sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.90$$

Lane Load Rating

$$IR_{Lanesh} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{Lane} \cdot (1 + I_{LFR}))} = 2.97$$

$$OR_{Lanesh} := IR_{Lanesh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.95$$

AASHTO Type 3 Rating

$$IR_{type3sh} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 5.11$$

$$OR_{type3sh} := IR_{type3sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.53$$

AASHTO Type 3-3 Rating

$$IR_{type33sh} := \frac{((\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot V_{type33} \cdot (1 + I_{LFR})} = 6.67$$

$$OR_{type33sh} := IR_{type33sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.14$$

AASHTO Type 3S2 Rating

$$IR_{type3S2sh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 4.51$$

$$OR_{type3S2sh} := IR_{type3S2sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.52$$

AASHTO SU4 Rating

$$IR_{SU4sh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 4.91$$

$$OR_{SU4sh} := IR_{SU4sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.20$$

AASHTO SU5 Rating

$$IR_{SU5sh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 4.86$$

$$OR_{SU5sh} := IR_{SU5sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.12$$

AASHTO SU6 Rating

$$IR_{SU6sh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 4.23$$

$$OR_{SU6sh} := IR_{SU6sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.07$$

AASHTO SU7 Rating

$$IR_{SU7sh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 4.84$$

$$OR_{SU7sh} := IR_{SU7sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.09$$

AASHTO NRL Rating

$$IR_{NRLsh} := \frac{(\phi_s \cdot V_n) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 4.51$$

$$OR_{NRLsh} := IR_{NRLsh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.52$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $Type3S2 := 40 \cdot ton$ $SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $SU4 := 27 \cdot ton$ $SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $SU5 := 31 \cdot ton$ $NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str}$	$HS20 \cdot OR_{HS20str}$	$HS20 \cdot IR_{HS20sh}$	$HS20 \cdot OR_{HS20sh}$
$Type3$	$Type3 \cdot IR_{type3str}$	$Type3 \cdot OR_{type3str}$	$Type3 \cdot IR_{type3sh}$	$Type3 \cdot OR_{type3sh}$
$Type33$	$Type33 \cdot IR_{type33str}$	$Type33 \cdot OR_{type33str}$	$Type33 \cdot IR_{type33sh}$	$Type33 \cdot OR_{type33sh}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str}$	$Type3S2 \cdot OR_{type3S2str}$	$Type3S2 \cdot IR_{type3S2sh}$	$Type3S2 \cdot OR_{type3S2sh}$
$SU4$	$SU4 \cdot IR_{SU4str}$	$SU4 \cdot OR_{SU4str}$	$SU4 \cdot IR_{SU4sh}$	$SU4 \cdot OR_{SU4sh}$
$SU5$	$SU5 \cdot IR_{SU5str}$	$SU5 \cdot OR_{SU5str}$	$SU5 \cdot IR_{SU5sh}$	$SU5 \cdot OR_{SU5sh}$
$SU6$	$SU6 \cdot IR_{SU6str}$	$SU6 \cdot OR_{SU6str}$	$SU6 \cdot IR_{SU6sh}$	$SU6 \cdot OR_{SU6sh}$
$SU7$	$SU7 \cdot IR_{SU7str}$	$SU7 \cdot OR_{SU7str}$	$SU7 \cdot IR_{SU7sh}$	$SU7 \cdot OR_{SU7sh}$
NRL	$NRL \cdot IR_{NRLstr}$	$NRL \cdot OR_{NRLstr}$	$NRL \cdot IR_{NRLsh}$	$NRL \cdot OR_{NRLsh}$

$$FlexureInventory=\begin{bmatrix}49.31\\35.28\\91.20\\76.41\\52.59\\53.40\\58.08\\61.02\\60.80\end{bmatrix}\ ton \qquad FlexureOperating=\begin{bmatrix}82.31\\58.89\\152.24\\127.54\\87.79\\89.13\\96.95\\101.86\\101.49\end{bmatrix}\ ton$$

$$ShearInventory=\begin{bmatrix}127.30\\127.70\\266.88\\180.28\\132.61\\150.80\\147.10\\187.71\\180.28\end{bmatrix}\ ton \qquad ShearOperating=\begin{bmatrix}212.50\\213.16\\445.49\\300.93\\221.35\\251.73\\245.54\\313.33\\300.93\end{bmatrix}\ ton$$

Structure 1237155 – NJ18 over Raritan River – Example Calculations

Bridge 1237155 Span 1-4 - Curved Girder Rating - Negative Moment - Girder 2

A. General Bridge Information

1) Built in 1980
2) Length - 870'
3) Width - 32'
4) ADT - 36,070 ADTT - 988 one direction
5) Superstructure in satisfactory condition (Condition rating = 6)

B. Description & Purpose

The purpose of the following calculations are to determine the inventory and operating load for the curved girders of the structure. The rating vehicles include HS20, Type 3, Type 3-3, Type 3S2 SU4, SU5, SU6, SU7, Formula B NRL, and lane load cases

C. References

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction MonitoringRefined Load Ratings\Bridge 1237155 - Rt18 over Raritan River\Documents and Drawings

- As-built drawings, NJDOT, **Bridge 1237155 - Drawings.pdf**
- Inspection report - Cycle 16 - **1237155_20100719cy13.PDF**

1) AASHTO. Standard Specifications for Highway Bridges, 17th Edition 2002
2) AASHTO, LFD Specifications 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43
5) NCHRP Research Results Digest Number 234 - Manual for Bridge Rating through Load Testing
6) AISC Manual of Steel Construction - LRFD - 3rd edition

D. Assumptions and Limitations

E. Legend

Highlighted values indicate inputs

Boxed values indicate critical checks or results

F. Material Properties

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.2 \cdot ksi$ Minimum 30 daycompressive strength of Class B concrete constructed per 1973 AASHTO

$f_{y\_s} := 32 \cdot ksi$ Minimum yield strength of A36 steel per AISC

$f_{y\_rs} := 36 \cdot ksi$ Yield stess of structural grade reinforcing steel

$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ conncrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

G. Section Properties

G.1. Steel Section Properties

$d_{web} := 120 \cdot in$	depth of web
$t_{tf} := 3.5 \cdot in$	top flange thickness
$t_{bf} := 3.5 \cdot in$	bottom flange thickness
$d := d_{web} + t_{tf} + t_{bf} = 127.00\ in$	total depth of section
$b_f := 32 \cdot in$	flange width
$t_w := \frac{7}{8} \cdot in$	web thickness
$A_{st} := d_{web} \cdot t_w + b_f \cdot (t_{tf} + t_{bf}) = 329.00\ in^2$	area of steel
$a_{bf} := t_{bf} \cdot b_f = 112.00\ in^2$	area of bottom flange
$a_{tf} := t_{tf} \cdot b_f = 112.00\ in^2$	area of top flange
$a_{web} := d_{web} \cdot t_w = 105.00\ in^2$	area of web

G.2. Slab Section Properties

$t_s := 8 \cdot in$	thickness of deck slab from top of beam (ignores haunch)
$t_h := 2 \cdot in = 2.00\ in$	thickness of haunch
$S := 6.5\ ft + \frac{7.75}{2}\ ft = 124.50\ in$	physical flange width
$L := 870 \cdot ft = 10440.00\ in$	curved girder length
$B := min\left(0.25 \cdot L, 12 \cdot t_s + max\left(t_w, 0.5 \cdot b_f\right), S\right) = 112.00\ in$	Effective Flange Width, be

G.3. Composite Section Properties

Composite Section Properties - LRFD Design 4.6.2.6.1

$t_s := 8 \cdot in$	thickness of deck slab from top of beam (ignores haunch)
$t_h := 2 \cdot in = 2.00\ in$	thickness of haunch
$S := 6.5\ ft + \frac{7.75}{2}\ ft = 124.50\ in$	physical flange width
$L := 870 \cdot ft = 10440.00\ in$	curved girder length

Effective Flange Width, be

$$B := min\left(0.25 \cdot L, 12 \cdot t_s + max\left(t_w, 0.5 \cdot b_f\right), S\right) = 112.00\ in$$ Minimum flange width

Modular ratio, n

$$n := \frac{E_s}{E_c} = 10.18$$

Short term composite (n)

$$b_e := \frac{B}{n} = 11.01\ in$$ Effective flange width

$$a_{ts} := b_e \cdot t_s = 88.06\ in^2$$ Area of transformed slab

Short term composite moment of intertia

$$y_{stc} := \frac{\left(\left(\frac{d_{web}}{2} + t_{bf}\right) \cdot a_{web} + \frac{t_{bf}}{2} \cdot a_{bf} + \left(\frac{t_{tf}}{2} + d_{web} + t_{bf}\right) \cdot \left(a_{tf}\right) + a_{ts} \cdot \left(t_{tf} + d_{web} + t_{bf} + t_h + \frac{t_s}{2}\right)\right)}{a_{web} + a_{tf} + a_{bf} + a_{ts}} = 78.17\ in$$

$$I_{bf} := \frac{\left(t_{bf}^{\ 3} \cdot b_f\right)}{12} = 114.33\ in^4$$

$$I_{tf} := \frac{\left(t_{tf}^{\ 3} \cdot b_f\right)}{12} = 114.33\ in^4$$

$$I_{web} := \frac{\left(t_w \cdot d_{web}^{\ 3}\right)}{12} = 126000.00\ in^4$$

$$I_{ts\_c} := \frac{\left(b_e \cdot t_s^{\ 3}\right)}{12} = 469.63\ in^4$$

$$d_{cbf\_stc} := y_{stc} - \frac{t_{bf}}{2} = 76.42\ in$$

$$d_{cw\_stc} := t_{bf} + \frac{d_{web}}{2} - y_{stc} = -14.67\ in$$

$$d_{ctf\_stc} := d_{cw\_stc} + \frac{d_{web}}{2} + \frac{t_{tf}}{2} = 47.08\ in$$

$$d_{cts\_stc} := d_{ctf\_stc} + \frac{t_{tf}}{2} + t_h + \frac{t_s}{2} = 54.83\ in$$

$$_{stc} := \left(I_{bf} + a_{bf} \cdot d_{cbf\_stc}{}^{2}\right) + \left(I_{tf} + a_{tf} \cdot d_{ctf\_stc}{}^{2}\right) + \left(I_{web} + a_{web} \cdot d_{cw\_stc}{}^{2}\right) + \left(I_{ts\_c} + a_{ts} \cdot d_{cts\_stc}{}^{2}\right) = 1316350.83\ in^{4}$$

H. Flexural Capacity

H.1. Plastic Moment Calculation

Reference: **AASHTO LFD 10.50.1.1**

$C_{con} := 0.85 \cdot f_c \cdot B \cdot t_s = 1675.52\ kip$ Check Compression in Slab

$C_{st} := f_{y\_s} \cdot \left(a_{bf} + a_{web} + a_{tf}\right) = 10528.00\ kip$ Check Compression in Steel

$C := min\left(C_{st}, C_{con}\right) = 1675.52\ kip$ Compressive force in the slab is taken as the minimum of C in concrete and C in steel

$C' := \dfrac{\left(C_{st} - C_{con}\right)}{2} = 4426.24\ kip$ Top Portion of steel section subjected to compression when Cst >Ccon

$$\begin{aligned} NeutralAxis(C') := \text{if } & C_{st} > C_{con} \\ & \text{if } C' < a_{tf} \cdot f_{y\_s} \\ & \quad y_{tos} \leftarrow \frac{C'}{a_{tf} \cdot f_{y\_s}} \cdot t_{tf} \\ & \quad \text{return } y_{tos} \\ & \text{else} \\ & \quad y_{tos} \leftarrow t_{tf} + \frac{C' - a_{tf} \cdot f_{y\_s}}{a_{web} \cdot f_{y\_s}} \cdot d_{web} \\ & \quad \text{return } y_{tos} \end{aligned}$$

Determine distance from Top of Steel to NA

$y_{tos} := NeutralAxis(C') = 33.58\ in$ Distance to Centroid from top of steel

Check for compactness of Section

$$\begin{aligned} Compactcheck\left(y_{tos}\right) := \text{if } & \frac{2 \cdot y_{tos}}{t_w} \le \frac{19230 \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \\ & \quad \text{return "Section is Compact"} \\ & \text{else} \\ & \quad \text{return "Section is not Compact"} \end{aligned}$$

$Compact := Compactcheck\left(y_{tos}\right) = \text{"Section is Compact"}$

Check Ducility Requirements

$a := \dfrac{C}{0.85 \cdot f_c \cdot B} = 8.00\ in$ Depth of stress block

$D_{cb} := y_{tos} - t_{tf} = 30.08\ in$ Depth of web in compression from top of web

$D_p := y_{tos} + t_h + t_s = 43.58\ in$ Distance from top of slab to the neutral axis at the palstic moment

$\beta := 0.9$

$D' := \beta \cdot \dfrac{\left(d + t_s + t_h\right)}{7.5} = 16.44\ in$

$$Ductility\left(D_{cb}\right) := \begin{array}{l} \text{if } \left(\dfrac{D_p}{D'}\right) \le 5 \\ \quad \text{return "OK"} \\ \text{else} \\ \quad \text{return "Not OK"} \end{array}$$

Check Ductility Requirements

$Ductility := Ductility\left(D_{cb}\right) = \text{"OK"}$

Calculate Plastic Moment

Centroid steel above PNA

$a_{web\_a} := D_{cb} \cdot t_w = 26.32\ in^2$ Area of web above NA

$a_{tf} = 112.00\ in^2$ Area of top flange

$y_{web\_a} := \dfrac{D_{cb}}{2} = 15.04\ in$ distance from PNA to center of web above NA

$y_{tf\_a} := D_{cb} + \dfrac{t_{tf}}{2} = 31.83\ in$ distance from PNA to center of flange above NA

$y_a := \dfrac{a_{web\_a} \cdot y_{web\_a} + a_{tf} \cdot y_{tf\_a}}{a_{web\_a} + a_{tf}} = 28.64\ in$ Centroid of steel above PNA

Centroid of steel below PNA

$a_{web\_b} := \left(d - D_{cb}\right) \cdot t_w = 84.81\ in^2$ Area of web below NA

Area of bottom flange

$a_{bf} = 112.00\ in^2$ Area of bottom flange

$y_{web\_b} := \dfrac{d - D_{cb}}{2} = 48.46\ in$ distance from PNA to center of web below NA

$y_{bf\_b} := d - D_{cb} + \dfrac{t_{bf}}{2} = 98.67\ in$ distance from PNA to center of flange below NA

$y_b := \dfrac{a_{web\_b} \cdot y_{web\_b} + a_{bf} \cdot y_{bf\_b}}{a_{web\_b} + a_{bf}} = 77.03\ in$ Centroid of steel below PNA

Calculate Plastic Moment

$C = 1675.52\ kip$

$C' = 4426.24\ kip$

$T := C + C' = 6101.76\ kip$

$M_p := C \cdot \left(D_p - \dfrac{a}{2}\right) + C' \cdot y_a + T \cdot y_b = 55258.88\ kip \cdot ft$

H.2. Capacity of Section

Reference: **AASHTO Standard Specs 10.50.2.1**

If the section meets the requirements of compactness and ductility in the negative moment region, the capacity of the section is taken as the full plastic moment

$M_r := M_p$

I. Demand

I.1. Impact Factor

$I_{LFR} := 0.3$

I.2. Moment

Stresses taken from FE Model

$\sigma_{xx\_DL} := 16 \cdot ksi$

$\sigma_{xx\_HS20} := 1.29 \cdot ksi$

$\sigma_{xx\_type3} := 0.91 \cdot ksi$

$\sigma_{xx\_type33} := 1.41 \cdot ksi$

$\sigma_{xx\_type3S2} := 1.43 \cdot ksi$

$\sigma_{xx\_SU4} := 0.98 \cdot ksi$

$\sigma_{xx\_SU5} := 1.12 \cdot ksi$

$\sigma_{xx\_SU6} := 1.25 \cdot ksi$

$\sigma_{xx\_SU7} := 1.4 \cdot ksi$

$\sigma_{xx\_NRL} := 1.59 \cdot ksi$

$\sigma_{xx\_Lane\_Moment} := 0.75 \cdot 2.91 \cdot ksi$

Moment Demand from FE Model

Moment calculated from plate stresses and transformed section properties

$$M_{DL\_tot} := \frac{\sigma_{xx\_DL} \cdot I_{stc}}{y_{stc}} = 22451.66\ kip \cdot ft$$

$$M_{HS20} := \frac{\sigma_{xx\_HS20} \cdot I_{stc}}{y_{stc}} = 1810.17\ kip \cdot ft$$

$$M_{type3} := \frac{\sigma_{xx\_type3} \cdot I_{stc}}{y_{stc}} = 1276.94\ kip \cdot ft$$

$$M_{type33} := \frac{\sigma_{xx\_type33} \cdot I_{stc}}{y_{stc}} = 1978.55\ kip \cdot ft$$

$$M_{type3S2} := \frac{\sigma_{xx\_type3S2} \cdot I_{stc}}{y_{stc}} = 2006.62\ kip \cdot ft$$

$$M_{SU4} := \frac{\sigma_{xx\_SU4} \cdot I_{stc}}{y_{stc}} = 1375.16\ kip \cdot ft$$

$$M_{SU5} := \frac{\sigma_{xx\_SU5} \cdot I_{stc}}{y_{stc}} = 1571.62\ kip \cdot ft$$

$$M_{SU6} := \frac{\sigma_{xx\_SU6} \cdot I_{stc}}{y_{stc}} = 1754.04\ kip \cdot ft$$

$$M_{SU7} := \frac{\sigma_{xx\_SU7} \cdot I_{stc}}{y_{stc}} = 1964.52\ kip \cdot ft$$

$$M_{NRL} := \frac{\sigma_{xx\_NRL} \cdot I_{stc}}{y_{stc}} = 2231.13\ kip \cdot ft$$

$$M_{Lane\_LFR} := \frac{\sigma_{xx\_Lane\_Moment} \cdot I_{stc}}{y_{stc}} = 3062.55\ kip \cdot ft$$

J. Load Factor Rating

J1. Factors & Equations

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.9$ strength reduction factors

J2. Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 4.24$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.08$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 6.02$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.04$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 3.88$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.48$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{\left(\phi_f \cdot M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 3.83$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.39$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\left(\phi_f \cdot M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 5.59$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.32$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 4.89$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.32$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 4.38$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.31$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 3.91$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.53$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 3.44$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.75$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{(\phi_f \cdot (M_r) - A_1 \cdot M_{Lane\_LFR})}{A_{2\_inv} \cdot (M_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 5.31$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.87$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $Type3S2 := 40 \cdot ton$ $SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $SU4 := 27 \cdot ton$ $SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $SU5 := 31 \cdot ton$ $NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$

$$FlexureInventory = \begin{bmatrix} 152.76 \\ 150.38 \\ 155.29 \\ 153.12 \\ 150.81 \\ 151.51 \\ 152.17 \\ 151.51 \\ 137.71 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 254.99 \\ 251.02 \\ 259.21 \\ 255.59 \\ 251.74 \\ 289.03 \\ 254.01 \\ 252.90 \\ 229.87 \end{bmatrix} ton$$

Bridge 1237155 Span 1-4 - Curved Girder Rating - Positive Moment - Girder 2

A. General Bridge Information

1) Built in 1980
2) Length - 870'
3) Width -
4) ADT - 36,070 ADTT - 988 one direction
5) Superstructure in satisfactory condition (Condition rating = 6)

B. Description & Purpose

The purpose of the following calculations are to determine the inventory and operating load for the curved girders of the structure. The rating vehicles include HS20, Type 3, Type 3-3, Type 3S2 SU4, SU5, SU6, SU7, Formula B NRL, and lane load cases

C. References

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction MonitoringRefined Load Ratings\Bridge 1237155 - Rt18 over Raritan River\Documents and Drawings

- As-built drawings, NJDOT, **Bridge 1237155 - Drawings.pdf**
- Inspection report - Cycle 16 - **1237155_20100719cy13.PDF**

1) AASHTO. Standard Specifications for Highway Bridges, 17th Edition 2002
2) AASHTO, LRFD Bridge Design Spec. 2007, with 2008 interim revisions.
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43
5) NCHRP Research Results Digest Number 234 - Manual for Bridge Rating through Load Testing
6) AISC Manual of Steel Construction - LRFD - 3rd edition

D. Assumptions and Limitations

E. Legend

Highlighted values indicate inputs

Boxed values indicate critical checks or results

F. Material Properties

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.2 \cdot ksi$ Minimum 30 daycompressive strength of Class B concrete constructed per 1973 AASHTO

$f_{y\_s} := 32 \cdot ksi$ Minimum yield strength of A36 steel per AISC

$f_{y\_rs} := 36 \cdot ksi$ Yield stess of structural grade reinforcing steel

$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ conncrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

G. Section Properties

G.1. Steel Section Properties

$d_{web} := 120 \cdot in$	depth of web
$t_{tf} := 2.75 \cdot in$	top flange thickness
$t_{bf} := 2.75 \cdot in$	bottom flange thickness
$d := d_{web} + t_{tf} + t_{bf} = 125.50\ in$	total depth of section
$b_f := 32 \cdot in$	flange width
$t_w := 0.75 \cdot in$	web thickness
$A_{st} := d_{web} \cdot t_w + b_f \cdot (t_{tf} + t_{bf}) = 266.00\ in^2$	area of steel
$a_{bf} := t_{bf} \cdot b_f = 88.00\ in^2$	area of bottom flange
$a_{tf} := t_{tf} \cdot b_f = 88.00\ in^2$	area of top flange
$a_{web} := d_{web} \cdot t_w = 90.00\ in^2$	area of web

G.2. Slab Section Properties

$t_s := 8 \cdot in$	thickness of deck slab from top of beam (ignores haunch)
$t_h := 2 \cdot in = 2.00\ in$	thickness of haunch
$S := 6.5\ ft + \frac{7.75}{2}\ ft = 124.50\ in$	physical flange width
$L := 870 \cdot ft = 10440.00\ in$	curved girder length
$B := min\left(0.25 \cdot L, 12 \cdot t_s + max\left(t_w, 0.5 \cdot b_f\right), S\right) = 112.00\ in$	Effective Flange Width, be

G.3. Composite Section Properties

Composite Section Properties - LRFD Design 4.6.2.6.1

$t_s := 8 \cdot in$	thickness of deck slab from top of beam (ignores haunch)
$t_h := 2 \cdot in = 2.00\ in$	thickness of haunch
$S := 6.5\ ft + \frac{7.75}{2}\ ft = 124.50\ in$	physical flange width

$L := 870 \cdot ft = 10440.00\ in$ curved girder length

Effective Flange Width, be

$B := min\left(0.25 \cdot L, 12 \cdot t_s + max\left(t_w, 0.5 \cdot b_f\right), S\right) = 112.00\ in$ Minimum flange width

Modular ratio, n

$$n := \frac{E_s}{E_c} = 10.18$$

Short term composite (n)

$b_e := \frac{B}{n} = 11.01\ in$ Effective flange width

$a_{ts} := b_e \cdot t_s = 88.06\ in^2$ Area of transformed slab

Short term composite moment of intertia

$$y_{stc} := \frac{\left(\left(\frac{d_{web}}{2} + t_{bf}\right) \cdot a_{web} + \frac{t_{bf}}{2} \cdot a_{bf} + \left(\frac{t_{tf}}{2} + d_{web} + t_{bf}\right) \cdot \left(a_{tf}\right) + a_{ts} \cdot \left(t_{tf} + d_{web} + t_{bf} + t_h + \frac{t_s}{2}\right)\right)}{a_{web} + a_{tf} + a_{bf} + a_{ts}} = 79.85\ in$$

$$I_{bf} := \frac{\left(t_{bf}^{\ 3} \cdot b_f\right)}{12} = 55.46\ in^4$$

$$I_{tf} := \frac{\left(t_{tf}^{\ 3} \cdot b_f\right)}{12} = 55.46\ in^4$$

$$I_{web} := \frac{\left(t_w \cdot d_{web}^{\ 3}\right)}{12} = 108000.00\ in^4$$

$$I_{ts\_c} := \frac{\left(b_e \cdot t_s^{\ 3}\right)}{12} = 469.63\ in^4$$

$$d_{cbf\_stc} := y_{stc} - \frac{t_{bf}}{2} = 78.47\ in$$

$$d_{cw\_stc} := t_{bf} + \frac{d_{web}}{2} - y_{stc} = -17.10\ in$$

$$l_{ctf\_stc} := d_{cw\_stc} + \frac{d_{web}}{2} + \frac{t_{tf}}{2} = 44.28\ in$$

$$l_{cts\_stc} := d_{ctf\_stc} + \frac{t_{tf}}{2} + t_h + \frac{t_s}{2} = 51.65\ in$$

$${}_{stc} := \left(I_{bf} + a_{bf} \cdot d_{cbf\_stc}{}^{2}\right) + \left(I_{tf} + a_{tf} \cdot d_{ctf\_stc}{}^{2}\right) + \left(I_{web} + a_{web} \cdot d_{cw\_stc}{}^{2}\right) + \left(I_{ts\_c} + a_{ts} \cdot d_{cts\_stc}{}^{2}\right) = 1084241.12\ in^{4}$$

H. Flexural Capacity

H.1. Plastic Moment Calculation

Reference: **AASHTO Standard Specs 10.50.1.1**

$C_{con} := 0.85 \cdot f_c \cdot B \cdot t_s = 1675.52\ kip$ — Check Compression in Slab

$C_{st} := f_{y\_s} \cdot \left(a_{bf} + a_{web} + a_{tf}\right) = 8512.00\ kip$ — Check Compression in Steel

$C := min\left(C_{st}, C_{con}\right) = 1675.52\ kip$ — Compressive force in the slab is taken as the minimum of C in concrete and C in steel

$C' := \frac{\left(C_{st} - C_{con}\right)}{2} = 3418.24\ kip$ — Top Portion of steel section subjected to compression when Cst >Ccon

$$NeutralAxis(C') := \text{if } C_{st} > C_{con} \quad \left\| \begin{array}{l} \text{if } C' < a_{tf} \cdot f_{y\_s} \\ \quad \left\| \begin{array}{l} y_{tos} \leftarrow \dfrac{C'}{a_{tf} \cdot f_{y\_s}} \cdot t_{tf} \\ \text{return } y_{tos} \end{array} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} y_{tos} \leftarrow t_{tf} + \dfrac{C' - a_{tf} \cdot f_{y\_s}}{a_{web} \cdot f_{y\_s}} \cdot d_{web} \\ \text{return } y_{tos} \end{array} \right. \end{array} \right.$$

Determine distance from Top of Steel to NA

$y_{tos} := NeutralAxis(C') = 27.84\ in$ — Distance to Centroid from top of steel

Check for compactness of Section

$$Compactcheck\left(y_{tos}\right) := \text{if } \frac{2 \cdot y_{tos}}{t_w} \le \frac{19230 \cdot ksi^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \text{return } \text{"Section is Compact"}$$
$$\text{else}$$
$$\quad \text{return } \text{"Section is not Compact"}$$

$$Compact := Compactcheck\left(y_{tos}\right) = \text{"Section is Compact"}$$

Check Ducility Requirements

$$a := \frac{C}{0.85 \cdot f_c \cdot B} = 8.00\ in$$ Depth of stress block

$$D_{cb} := y_{tos} - t_{tf} = 25.09\ in$$ Depth of web in compression from top of web

$$D_p := y_{tos} + t_h + t_s = 37.84\ in$$ Distance from top of slab to the neutral axis at the palstic moment

$$\beta := 0.9$$

$$D' := \beta \cdot \frac{\left(d + t_s + t_h\right)}{7.5} = 16.26\ in$$

$$Ductility\left(D_{cb}\right) := \text{if } \left(\frac{D_p}{D'}\right) \le 5$$
$$\quad \text{return } \text{"OK"}$$
$$\text{else}$$
$$\quad \text{return } \text{"Not OK"}$$

Check Ductility Requirements

$$Ductility := Ductility\left(D_{cb}\right) = \text{"OK"}$$

Calculate Plastic Moment

Centroid steel above PNA

$$a_{web\_a} := D_{cb} \cdot t_w = 18.82\ in^2$$ Area of web above NA

$$a_{tf} = 88.00\ in^2$$ Area of top flange

$$y_{web\_a} := \frac{D_{cb}}{2} = 12.55\ in$$ distance from PNA to center of web above NA

$y_{tf\_a} := D_{cb} + \frac{t_{tf}}{2} = 26.47\ in$ distance from PNA to center of flange above NA

$y_a := \frac{a_{web\_a} \cdot y_{web\_a} + a_{tf} \cdot y_{tf\_a}}{a_{web\_a} + a_{tf}} = 24.02\ in$ Centroid of steel above PNA

Centroid of steel below PNA

$a_{web\_b} := (d - D_{cb}) \cdot t_w = 75.31\ in^2$ Area of web below NA

$a_{bf} = 88.00\ in^2$ Area of bottom flange

$y_{web\_b} := \frac{d - D_{cb}}{2} = 50.20\ in$ distance from PNA to center of web below NA

$y_{bf\_b} := d - D_{cb} + \frac{t_{bf}}{2} = 101.78\ in$ distance from PNA to center of flange below NA

$y_b := \frac{a_{web\_b} \cdot y_{web\_b} + a_{bf} \cdot y_{bf\_b}}{a_{web\_b} + a_{bf}} = 78.00\ in$ Centroid of steel below PNA

Calculate Plastic Moment

$C = 1675.52\ kip$

$C' = 3418.24\ kip$

$T := C + C' = 5093.76\ kip$

$M_p := C \cdot \left(D_p - \frac{a}{2}\right) + C' \cdot y_a + T \cdot y_b = 44674.64\ kip \cdot ft$

H.2. Capacity of Section

$S_t := \frac{I_{stc}}{d - y_{stc}} = 23750.37\ in^3$ Section modulus at top of steel

$M_y := f_{y\_s} \cdot S_t = 63334.31\ kip \cdot ft$ Moment Capacity at first yield of the compact positive moment section

$$Capacity(D') := \text{if } D_p \le D' \quad \| M_u \leftarrow M_p,\ \text{return } M_u$$
$$\text{else if } D_p \le D_p \le 5 \cdot D' \quad \| M_u \leftarrow \frac{5 \cdot M_p - 0.85 \cdot M_y}{4} + \frac{0.85 \cdot M_y - M_p}{4} \cdot \left(\frac{D_p}{D'}\right),\ \text{return } M_u$$
$$\text{else if } D_p > 5 \cdot D' \quad \| \text{return "Not OK"}$$

$M_u := Capacity(D') = 47714.20\ kip \cdot ft$

$M_r := M_u$

I. Shear Capacity

I.1. Stiffnener Spacing Requirements

$d_o := 4.667 \cdot ft = 56.00\ in$ Stiffener Spacing

$$Spacing\_Check(d_o) := \| \text{if } d_o \le 3 \cdot d_{web} \ \| \text{return "Spacing OK"};\ \text{else} \ \| \text{return "Spacing not OK"}$$

Check Spacing Requirements
AASHTO Standard Specs 10.48.8.3

$$Depth\_Check(d_{web}) := \text{if } \frac{d_{web}}{t_w} > 150 \ \| x \leftarrow \frac{d_{web}}{t_w},\ \text{return "Stiffeners Required"};\ \text{else} \ \| \text{return "Stiffeners not Required"}$$

Check web depth to thickness ratio
AASHTO Standard Specs 10.48.8.3

$$Handling\_Requirements(d_{web}) := \text{if } d_o < d_{web} \cdot \left(\frac{260}{\left(\frac{d_{web}}{t_w}\right)}\right)^2 \ \| \text{return "Handling OK"};\ \text{else} \ \| \text{return "Handling not OK"}$$

Check Handling requirements
AASHTO Standard Specs 10.48.8.3

$Spacing\_Check := Spacing\_Check\left(d_o\right) = \text{“Spacing OK”}$

$Depth\_Check := Depth\_Check\left(d_{web}\right) = \text{“Stiffeners Required”}$

Output check results

$Handling\_Requirements := Handling\_Requirements\left(d_{web}\right) = \text{“Handling OK”}$

I.1. Shear Capacity Calculation

$$k := 5 + \frac{5}{\left(\dfrac{d_o}{d_{web}}\right)^2} = 27.96$$

$$C\left(d_{web}\right) := \begin{array}{l} \text{if } \dfrac{d_{web}}{t_w} < \dfrac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \\ \quad \left\| C \leftarrow 1 \right. \\ \text{else if } \dfrac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \le \dfrac{d_{web}}{t_w} \le \dfrac{7500 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \\ \quad \left\| C \leftarrow \dfrac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\left(\dfrac{d_{web}}{t_w}\right) \cdot \sqrt{f_{y\_s}}} \right. \\ \text{else if } \dfrac{d_{web}}{t_w} > \dfrac{7500 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \\ \quad \left\| C \leftarrow \dfrac{4.5 \cdot 10^7 \cdot k \cdot ksi}{\left(\dfrac{d_{web}}{t_w}\right)^2 \cdot f_{y\_s}} \right. \end{array}$$

Determine C, defined as the buckling shear stress divided by the shear yield stress
AASHTO Standard Specs 10.48.8.1

$C := C\left(d_{web}\right) = 1.00$

Return C

$V_p := 0.58 \cdot f_{y\_s} \cdot d_{web} \cdot t_w = 1670.40\ kip$

Determine plastic shear force
AASHTO Standard Specs 10.48.8.1 eq. 10-115

$$V_u(c) := \left\| \begin{array}{l} \text{if } Spacing\_Check = \text{"Spacing OK"} \\ \quad \left\| \begin{array}{l} \text{if } Handling\_Requirements = \text{"Handling OK"} \\ \quad \left\| \begin{array}{l} V_u \leftarrow V_p \cdot \left(C + \dfrac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\dfrac{d_o}{d_{web}} \right)^2}} \right) \\ \text{return } V_u \end{array} \right. \end{array} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} V_u \leftarrow C \cdot V_p \\ \text{return } V_u \end{array} \right. \end{array} \right.$$

Determine Shear capacity by first determining if the provisions of article 10.48.8.3 are satisfied for the transverse stiffeners. If so, use 10-114 to calculate capacity. If not, use equation 10-113.
AASHTO Standard Specs 10.48.8.1

$V_u := V_u(C) = 1670.40\ kip$

J. Demand

J.1. Impact Factor

$I_{LFR} := 0.3$

J.2. Moment

Stresses taken from FE Model

$\sigma_{xx\_DL} := 7.5 \cdot ksi$

$\sigma_{xx\_HS20} := 2.62 \cdot ksi$

$\sigma_{xx\_type3} := 1.85 \cdot ksi$

$\sigma_{xx\_type33} := 2.44 \cdot ksi$

$\sigma_{xx\_type3S2} := 2.58 \cdot ksi$

$\sigma_{xx\_SU4} := 2.05 \cdot ksi$

$\sigma_{xx\_SU5} := 2.31 \cdot ksi$

$\sigma_{xx\_SU6} := 2.58 \cdot ksi$

$\sigma_{xx\_SU7} := 2.85 \cdot ksi$

$\sigma_{xx\_NRL} := 3.24 \cdot ksi$

$\sigma_{xx\_Lane\_Moment} := 2.6 \cdot ksi$

Moment Demand from FE Model

Moment calculated from plate stresses and transformed section properties

$$M_{DL\_tot} := \frac{\sigma_{xx\_DL} \cdot I_{stc}}{y_{stc}} = 8486.71\ kip \cdot ft$$

$$M_{HS20} := \frac{\sigma_{xx\_HS20} \cdot I_{stc}}{y_{stc}} = 2964.69\ kip \cdot ft$$

$$M_{type3} := \frac{\sigma_{xx\_type3} \cdot I_{stc}}{y_{stc}} = 2093.39\ kip \cdot ft$$

$$M_{type33} := \frac{\sigma_{xx\_type33} \cdot I_{stc}}{y_{stc}} = 2761.01\ kip \cdot ft$$

$$M_{type3S2} := \frac{\sigma_{xx\_type3S2} \cdot I_{stc}}{y_{stc}} = 2919.43\ kip \cdot ft$$

$$M_{SU4} := \frac{\sigma_{xx\_SU4} \cdot I_{stc}}{y_{stc}} = 2319.70\ kip \cdot ft$$

$$M_{SU5} := \frac{\sigma_{xx\_SU5} \cdot I_{stc}}{y_{stc}} = 2613.91\ kip \cdot ft$$

$$M_{SU6} := \frac{\sigma_{xx\_SU6} \cdot I_{stc}}{y_{stc}} = 2919.43\ kip \cdot ft$$

$$M_{SU7} := \frac{\sigma_{xx\_SU7} \cdot I_{stc}}{y_{stc}} = 3224.95\ kip \cdot ft$$

$$M_{NRL} := \frac{\sigma_{xx\_NRL} \cdot I_{stc}}{y_{stc}} = 3666.26\ kip \cdot ft$$

$$M_{Lane\_LFR} := \frac{\sigma_{xx\_Lane\_Moment} \cdot I_{stc}}{y_{stc}} = 2942.06\ kip \cdot ft$$

J.3. Shear

$V_{HS20} := 79 \cdot kip = 79.00\ kip$

$V_{DL\_total} := 278.5 \cdot kip = 278.50\ kip$

$V_{type3} := 57.5 \cdot kip = 57.50\ kip$

$V_{type33} := 79.2 \cdot kip = 79.20\ kip$

$V_{type3S2} := 85.2 \cdot kip = 85.20\ kip$

$V_{Lane\_LFR} := 102.2 \cdot kip = 102.20\ kip$

$V_{SU4} := 63.1 \cdot kip = 63.10\ kip$

$V_{SU5} := 70.9 \cdot kip = 70.90\ kip$

$V_{SU6} := 78.5 \cdot kip = 78.50\ kip$

$V_{SU7} := 86.6 \cdot kip = 86.60\ kip$

$V_{NRL} := 91.3 \cdot kip = 91.30\ kip$

K. Load Factor Rating

K1. Factors & Equations

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$ $\phi_s := 0.9$ strength reduction factors

K2. Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 3.87$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.45$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 5.48$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.14$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 4.15$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.93$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 3.93$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.55$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 4.94$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.25$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU5} \cdot \left(1 + I_{LFR}\right)} = 4.39$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.25$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 3.93$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.55$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 3.55$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.93$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 3.13$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.22$$

LFR Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{(\phi_f \cdot (M_r) - A_1 \cdot M_{Lane\_LFR})}{A_{2\_inv} \cdot (M_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 4.73$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.90$$

K3. Shear Ratings

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_u) - (A_1 \cdot V_{HS20}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 6.30$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.52$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{((\phi_s \cdot V_u) - (A_1 \cdot V_{Lane\_LFR}))}{A_{2\_inv} \cdot (V_{Lane\_LFR} \cdot (1 + I_{LFR}))} = 4.77$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.96$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{((\phi_s \cdot V_u) - (A_1 \cdot V_{type3}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 8.83$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 14.73$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{(\phi_s \cdot V_u - (A_1 \cdot V_{type33}))}{A_{2\_inv} \cdot V_{type33} \cdot (1 + I_{LFR})} = 6.29$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.49$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{type3S2})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 5.81$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}} \right) = 9.70$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{SU4})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 8.00$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}} \right) = 13.36$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{SU5})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 7.07$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}} \right) = 11.81$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{SU6})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 6.35$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}} \right) = 10.59$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{SU7})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 5.71$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}} \right) = 9.53$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_f \cdot V_u) - A_1 \cdot (V_{NRL})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 5.39$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.00$$

Summary Rating Table

$HS20 := 36 \cdot ton$	$Type3S2 := 40 \cdot ton$	$SU6 := 34.75 \cdot ton$
$Type3 := 25 \cdot ton$	$SU4 := 27 \cdot ton$	$SU7 := 38.75 \cdot ton$
$Type33 := 40 \cdot ton$	$SU5 := 31 \cdot ton$	$NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 139.18 \\ 136.88 \\ 166.06 \\ 157.04 \\ 133.41 \\ 135.94 \\ 136.43 \\ 137.72 \\ 125.05 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 232.33 \\ 228.49 \\ 277.19 \\ 262.14 \\ 222.69 \\ 255.69 \\ 227.74 \\ 229.89 \\ 208.74 \end{bmatrix} ton$$

$$ShearInventory = \begin{bmatrix} 226.90 \\ 220.63 \\ 251.43 \\ 232.47 \\ 216.07 \\ 219.27 \\ 220.51 \\ 221.29 \\ 215.76 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 378.74 \\ 368.28 \\ 419.69 \\ 388.05 \\ 360.67 \\ 366.02 \\ 368.08 \\ 369.38 \\ 360.15 \end{bmatrix} ton$$

Bridge 1237155 Span 1-4 - Floorbeam 2 Rating

A. General Bridge Information

1) Built in 1980
2) Length - 870' - NBIS length
3) Width - 93.5' NBIS Out to Out
4) ADT - 36,070 ADTT - 988 one direction
5) Superstructure in satisfactory condition (Condition rating = 6)

B. Description & Purpose

The purpose of the following calculations are to determine the inventory and operating load for the curved girders of the structure. The rating vehicles include HS20, Type 3, Type 3-3, Type 3S2 SU4, SU5, SU6, SU7, Formula B NRL, and lane load cases for moment and shear.

C. References

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction MonitoringRefined Load Ratings\Bridge 1516152 - NJ166 over Toms River\Drawings and Inspection Reports

- As-built drawings, NJDOT, **Bridge 1237155 - Drawings.pdf**
- Inspection report - Cycle 16 **- 1237155_20100719cy13.PDF**

1) AASHTO. Standard Specifications for Highway Bridges, 17th Edition 2002
2) AASHTO, LFD Specifications, 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43
5) NCHRP Research Results Digest Number 234 - Manual for Bridge Rating through Load Testing
6) AISC Manual of Steel Construction - LRFD - 3rd edition

D. Assumptions and Limitations

E. Legend

Highlighted values indicate inputs

Boxed values indicate critical checks or results

F. Material Properties

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.2 \cdot ksi$ Minimum 30 daycompressive strength of Class B concrete constructed per 1973 AASHTO

$f_{y\_s} := 32 \cdot ksi$ Minimum yield strength of A36 steel per AISC

$f_{y\_rs} := 36 \cdot ksi$ Yield stess of structural grade reinforcing steel

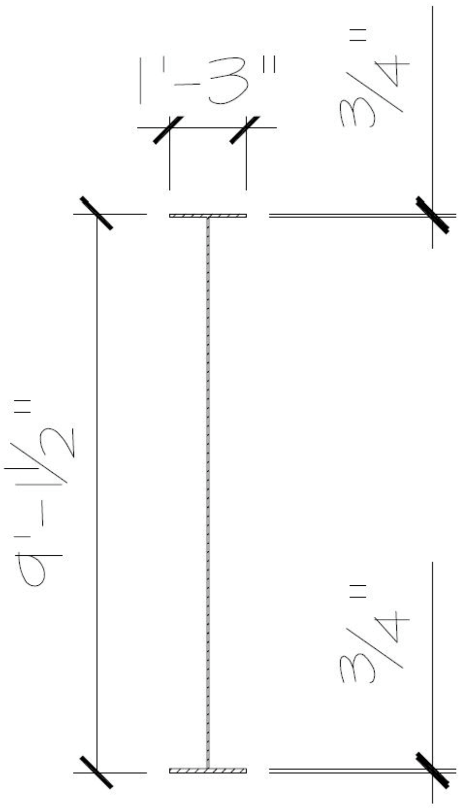
$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ conncrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

}

G. Section Properties

G.1. Global Section Properties



Note: Longitudinal Stiffeners not shown or included in capacity calculations

$t_f := 0.75 \cdot in$	Flange thickness
$d := 109.5 \cdot in$	Total depth of section
$b_f := 15 \cdot in$	Flange width
$t_w := 0.375 \cdot in$	Web thickness
$a_{web} := (d - 2 \cdot t_f) \cdot t_w = 40.50\ in^2$	Area of web
$a_f := t_f \cdot b_f = 11.25\ in^2$	Area of flange

}

$D := d - t_f \cdot 2 = 108.00\ in$ Depth of web

$A_{st} := a_{web} + 2 \cdot a_f = 63.00\ in^2$ Area of Section

G.2. Moment of Inertia & Section Modulus

$I_{web\_xx} := \dfrac{t_w \cdot D^3}{12} = 39366.00\ in^4$ Moment of Inertia of Web about x axis

$I_{flange\_xx} := \dfrac{b_f \cdot (t_f)^3}{12} = 0.53\ in^4$ Moment of Inertia of Flange about x axis

$d_{f\_xx} := \dfrac{D}{2} + \dfrac{t_f}{2} = 54.38\ in$ Distance from x centroid of section to centroid of flange

$y_{xx} := \dfrac{d}{2} = 54.75\ in$

$I_{xx} := I_{web\_xx} + I_{flange\_xx} + 2 \cdot \left(a_f \cdot d_{f\_xx}^{\ 2}\right) = 105890.94\ in^4$ Moment of Inertia about x axis

$S := \dfrac{I_{xx}}{y_{xx}} = 1934.08\ in^3$ Section Modulus about x axis

H. Demand

H.1. Impact Factor

$I_{LFR} := 0.3$

H.2. Moment

Moment Demand from FE Model

$M_{DL\_tot} := 192\ kip \cdot ft = 192.00\ kip \cdot ft$

$M_{HS20} := 234\ kip \cdot ft = 234.00\ kip \cdot ft$

$M_{type3} := 174\ kip \cdot ft = 174.00\ kip \cdot ft$

$M_{type33} := 141.3\ kip \cdot ft = 141.30\ kip \cdot ft$

$M_{type3S2} := 171.5\ kip \cdot ft = 171.50\ kip \cdot ft$

$M_{SU4} := 202.3\ kip \cdot ft = 202.30\ kip \cdot ft$

$M_{SU5} := 221.3\ kip \cdot ft = 221.30\ kip \cdot ft$

}

$M_{SU6} := 243.2 \cdot kip \cdot ft = 243.20\ kip \cdot ft$

$M_{SU7} := 264.5 \cdot kip \cdot ft = 264.50\ kip \cdot ft$

$M_{NRL} := 300\ kip \cdot ft = 300.00\ kip \cdot ft$

$M_{Lane} := 126 \cdot kip \cdot ft = 126.00\ kip \cdot ft$

H.3. Shear

$V_{DL\_tot} := 58 \cdot kip = 58.00\ kip$

$V_{HS20} := 46 \cdot kip = 46.00\ kip$

$V_{type3} := 33 \cdot kip = 33.00\ kip$

$V_{type33} := 28 \cdot kip = 28.00\ kip$

$V_{type3S2} := 29.6 \cdot kip = 29.60\ kip$

$V_{SU4} := 38.9 \cdot kip = 38.90\ kip$

$V_{SU5} := 42.5 \cdot kip = 42.50\ kip$

$V_{SU6} := 47 \cdot kip = 47.00\ kip$

$V_{SU7} := 51.1 \cdot kip = 51.10\ kip$

$V_{NRL} := 57.8 \cdot kip = 57.80\ kip$

$V_{Lane} := 25.8 \cdot kip = 25.80\ kip$

I. Flexural Capacity

I.1. Braced Noncompact Section Provisions

$L_b := 7.67 \cdot ft = 92.04\ in$

$$Flange\_Check := \text{if } \frac{b_f}{t_f} \le 24 \;\; \| \text{return "}Flange\ OK\text{"} \;\; \text{else} \;\; \| \text{return "}Flange\ not\ OK\text{"}$$

10.48.2.1 eq. 10-100

$$Web\_Check := \text{if } \frac{D}{t_w} \le \frac{73000 \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \;\; \| \text{return "}Web\ OK\text{"} \;\; \text{else} \;\; \| \text{return "}Web\ not\ OK\text{"}$$

10.48.6.1 eq 10-109

}

$$Brace\_Spacing := \text{if } L_b \cdot \frac{in^2}{kip} \le \frac{20000000 \cdot a_f}{f_{y\_s} \cdot d} \;\; \| \text{return "Spacing OK"} \;\; \text{else} \;\; \| \text{return "Spacing not OK"}$$

10.48.2.1 eq. 10-101

$Flange\_Check = \text{"Flange OK"}$

$Web\_Check = \text{"Web OK"}$

$Brace\_Spacing = \text{"Spacing OK"}$

$P_{max} := 0.15 \cdot A_{st} \cdot f_{y\_s} = 302.40\ kip$ 10.48.2.1 eq. 10-102

Pmax is greater than the maximum compressive forces in the floor beams from the model results, satisfying equation 10-102 in section 10.48.2.1.

I.2. Flexural Capacity

$$F_{cr} := min\left(\left(4400 \cdot \frac{t_f}{b_f}\right)^2 \cdot ksi, f_{y\_s}\right) = 32.00\ ksi$$

$$D_c := \frac{d}{2} = 54.75\ in$$

Parameters for 10.48.2 eq. 10-98

$$A_{fc} := \frac{A_{st}}{2} = 31.50\ in^2$$

$$\lambda := 15400$$

$$f_{b\_HS20} := \frac{M_{HS20} \cdot y_{xx}}{I_{xx}} = 1.45\ ksi$$

$$f_{b\_type3} := \frac{M_{type3} \cdot y_{xx}}{I_{xx}} = 1.08\ ksi$$

$$f_{b\_type33} := \frac{M_{type33} \cdot y_{xx}}{I_{xx}} = 0.88\ ksi$$

$$f_{b\_type3S2} := \frac{M_{type3S2} \cdot y_{xx}}{I_{xx}} = 1.06\ ksi$$

Calculation of fb for each load case for use in eq 10-103b

$$f_{b\_SU4} := \frac{M_{SU4} \cdot y_{xx}}{I_{xx}} = 1.26\ ksi$$

$$f_{b\_SU5} := \frac{M_{SU5} \cdot y_{xx}}{I_{xx}} = 1.37\ ksi$$

}

$$f_{b\_SU6} := \frac{M_{SU6} \cdot y_{xx}}{I_{xx}} = 1.51\ ksi$$

$$f_{b\_SU7} := \frac{M_{SU7} \cdot y_{xx}}{I_{xx}} = 1.64\ ksi$$

$$f_{b\_NRL} := \frac{M_{NRL} \cdot y_{xx}}{I_{xx}} = 1.86\ ksi$$

$$f_{b\_Lane} := \frac{M_{Lane} \cdot y_{xx}}{I_{xx}} = 0.78\ ksi$$

$$R_{b\_HS20} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_HS20}}}\right), 1\right) = 1.00$$

$$R_{b\_type33} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_type33}}}\right), 1\right) = 1.00$$

$$R_{b\_type3S2} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_type3S2}}}\right), 1\right) = 1.00$$

$$R_{b\_SU4} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_SU4}}}\right), 1\right) = 1.00$$

$$R_{b\_SU5} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_SU5}}}\right), 1\right) = 1.00$$

Calculation of Rb for each load case per 10.48.4.1 eq 10-103b

$$R_{b\_SU6} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_SU6}}}\right), 1\right) = 1.00$$

$$R_{b\_SU7} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_SU7}}}\right), 1\right) = 1.00$$

$$R_{b\_NRL} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_NRL}}}\right), 1\right) = 1.00$$

$$R_{b\_Lane} := min\left(1 - 0.002 \cdot \left(\frac{D_c \cdot t_w}{A_{fc}}\right) \cdot \left(\frac{D_c}{t_w} - \frac{\lambda \cdot \sqrt{ksi}}{\sqrt{f_{b\_Lane}}}\right), 1\right) = 1.00$$

}

$$M_{ult} := \begin{array}{l} x \leftarrow R_{b\_HS20} + R_{b\_type33} + R_{b\_type3S2} + R_{b\_SU4} + R_{b\_SU5} + R_{b\_SU6} + R_{b\_SU7} + R_{b\_NRL} + R_{b\_Lane} \\ \text{if } x = 9 \\ \quad M_u \leftarrow min\left(f_{y\_s} \cdot S, F_{cr} \cdot S\right) \\ \quad \text{return } M_u \\ \text{else} \\ \quad \text{return "Calculate Mu case by case"} \end{array}$$

$$M_u := M_{ult} = 5157.55\ ft \cdot kip$$

J. Shear Capacity

J.1. Stiffener Spacing Requirements

$$Depth\_Check := \begin{array}{l} \text{if } \dfrac{d}{t_w} > 150 \\ \quad x \leftarrow \dfrac{d}{t_w} \\ \quad \text{return "Stiffeners Required"} \\ \text{else} \\ \quad \text{return "Stiffeners not Required"} \end{array}$$

Check web depth to thickness ratio
AASHTO Standard Specs 10.48.8.3

$$Depth\_Check := Depth\_Check = \text{"Stiffeners Required"}$$

J.1. Capacity Calculation

$$d_o := 7.75 \cdot ft = 93.00\ in$$

Distance between transverse stiffeners 10.48.8.1

$$k := 5 + \frac{5 \cdot ksi}{\left(\dfrac{d_o}{D}\right)^2 \cdot f_{y\_s}} = 5.21$$

Buckling Coefficient 10.48.8.1

}

$$C(d) := \text{if } \frac{d}{t_w} < \frac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \| C \leftarrow 1$$
$$\text{else if } \frac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \le \frac{d}{t_w} \le \frac{7500 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \| C \leftarrow \frac{6000 \cdot \sqrt{k} \cdot ksi^{.5}}{\left(\frac{d}{t_w}\right) \cdot \sqrt{f_{y\_s}}}$$
$$\text{else if } \frac{d}{t_w} > \frac{7500 \cdot \sqrt{k} \cdot ksi^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \| C \leftarrow \frac{4.5 \cdot 10^{7} \cdot k \cdot ksi}{\left(\frac{d}{t_w}\right)^{2} \cdot f_{y\_s}}$$

Determine C, defined as the buckling shear stress divided by the shear yield stress
AASHTO Standard Specs 10.48.8.1

$$C := C(d) = 1.00$$

Return C

$$V_p := 0.58 \cdot f_{y\_s} \cdot d \cdot t_w = 762.12\ kip$$

Determine plastic shear force
AASHTO Standard Specs 10.48.8.1 eq. 10-115

$$V_u := V_p \cdot \left(C + \frac{0.87 \cdot (1 - C)}{1 + \left(\frac{d_o}{D}\right)^{2}} \right) = 762.12\ kip$$

Calculate Ultimate Shear Strength 10.48.8.1 eq. 10-114

K. Load Factor Rating

K1. Factors & Equations

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$

Moment Rating Equation

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$

Shear Rating Equation

}

Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 1237155

By: CTY 10-29-13
Chk'd: JBP 3-1-14

Load Factors:

$A_1 := 1.25$	dead load factor for concrete/asphalt (MBE - 6B.4.3)
$A_{2\_inv} := 2.17$	live load factor (inventory level) (MBE - 6B.4.3)
$A_{2\_op} := 1.3$	live load factor (operating level) (MBE - 6B.4.3)
$\phi_f := 0.9 \quad \phi_s := 0.85$	strength reduction factors

K2. Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{(\phi_f \cdot (M_u) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{HS20} \cdot (1 + I_{LFR}))} = 6.67$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.13$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{(\phi_f \cdot (M_u) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type3} \cdot (1 + I_{LFR})} = 8.97$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 14.97$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{(\phi_f \cdot (M_u) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type33} \cdot (1 + I_{LFR})} = 11.04$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 18.43$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{(\phi_f \cdot M_u) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{type3S2} \cdot (1 + I_{LFR})} = 9.10$$

}

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 15.19$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\left(\phi_f \cdot M_u\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 7.71$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.87$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{\left(\phi_f \cdot M_u\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU5} \cdot \left(1 + I_{LFR}\right)} = 7.05$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.87$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{\left(\phi_f \cdot M_u\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU6} \cdot \left(1 + I_{LFR}\right)} = 6.42$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.71$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{\left(\phi_f \cdot M_u\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU7} \cdot \left(1 + I_{LFR}\right)} = 5.90$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.85$$

}

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{(\phi_f \cdot M_u) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 5.20$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.68$$

LFR Lane Load - Moment

$$IR_{Lane\_m} := \frac{(\phi_f \cdot (M_u) - A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot (M_{Lane} \cdot (1 + I_{LFR}))} = 12.38$$

$$OR_{Lane\_}m := IR_{Lane\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 20.67$$

K3. Shear Ratings

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{((\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 4.43$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.40$$

LRFR Lane Load - Shear

$$IR_{Lane\_LRFR\_v} := \frac{((\phi_s \cdot V_u) - A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot (V_{Lane} \cdot (1 + I_{LFR}))} = 7.90$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LRFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 13.19$$

}

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{\left(\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot V_{type3} \cdot \left(1 + I_{LFR}\right)} = 6.18$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.32$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{\left(\phi_s \cdot V_u - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 7.28$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.16$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 6.89$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.50$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU4} \cdot \left(1 + I_{LFR}\right)} = 5.24$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.75$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU5} \cdot \left(1 + I_{LFR}\right)} = 4.80$$

}

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 8.01$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 4.34$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.24$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_tot})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 3.99$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.66$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_u) - A_1 \cdot (V_{DL\_tot})}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 3.53$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.89$$

Summary Rating Table

$HS20 := 36 \cdot ton$	$Type3S2 := 40 \cdot ton$	$SU6 := 34.75 \cdot ton$
$Type3 := 25 \cdot ton$	$SU4 := 27 \cdot ton$	$SU7 := 38.75 \cdot ton$
$Type33 := 40 \cdot ton$	$SU5 := 31 \cdot ton$	$NRL := 40 \cdot ton$

}

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A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 1237155

By: CTY 10-29-13
Chk'd: JBP 3-1-14

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

$$FlexureInventory = \begin{bmatrix} 240.06 \\ 224.19 \\ 441.72 \\ 363.93 \\ 208.25 \\ 218.58 \\ 222.96 \\ 228.60 \\ 208.05 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 400.71 \\ 374.23 \\ 737.33 \\ 607.49 \\ 347.62 \\ 399.12 \\ 372.16 \\ 381.58 \\ 347.28 \end{bmatrix} ton$$

$$ShearInventory = \begin{bmatrix} 159.60 \\ 154.50 \\ 291.34 \\ 275.59 \\ 141.55 \\ 148.75 \\ 150.78 \\ 154.65 \\ 141.13 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 266.41 \\ 257.89 \\ 486.31 \\ 460.02 \\ 236.28 \\ 248.30 \\ 251.69 \\ 258.14 \\ 235.58 \end{bmatrix} ton$$

}

Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 0324152

By: CTY, 10-29-13
Chk'd:JBP 3-3-14

Bridge 1237155 Span 1-4 - Interior Stringer Rating

A. General Bridge Information

1) Built in 1980
2) Length - 870' - 4 spans
3) Width - 93.5' NBIS Out to Out
4) ADT - 36,070 ADTT - 988 one direction
5) Superstructure in satisfactory condition (Condition rating = 6)

B. Description & Purpose

The purpose of the following calculations are to determine the inventory and operating load for the curved girders of the structure. The rating vehicles include HS20, Type 3, Type 3-3, Type 3S2 SU4, SU5, SU6, SU7, Formula B NRL, and lane load cases for moment and shear.

C. References

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction MonitoringRefined Load Ratings\Bridge 127155- Rt18 over Raritan River\Documents and Drawings

- As-built drawings, NJDOT, **Bridge 1237155 - Drawings.pdf**
- Inspection report - Cycle 16 - **1237155_20100719cy13.PDF**

1) AASHTO. Standard Specifications for Highway Bridges, 17th Edition 2002
2) AASHTO, LRFD Bridge Design Spec. 2007, with 2008 interim revisions.
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43
5) NCHRP Research Results Digest Number 234 - Manual for Bridge Rating through Load Testing
6) AISC Manual of Steel Construction - LRFD - 3rd edition

D. Assumptions and Limitations

E. Legend

Highlighted values indicate inputs

Boxed values indicate critical checks or results

F. Material Properties

The mechanical properties if the materials in this bridge are unknown so assumed values are taken from AASHTO

$f_c := 2.2 \cdot ksi$ Minimum 30 day compressive strength of Class B concrete constructed per 1973 AASHTO

$f_{y\_s} := 32 \cdot ksi$ Minimum yield strength of A36 steel per AISC

$f_{y\_rs} := 36 \cdot ksi$ Yield stess of structural grade reinforcing steel

$E_c := 57 \cdot ksi \cdot \sqrt{2500} = 2850.00\ ksi$ conncrete modulus of elasticity

$E_s := 29000 \cdot ksi$ Steel modulus of elasticity

G. Section Properties

G.1. Steel Section Properties

$t_f := 0.522 \cdot in$	flange thickness
$d := 20.8 \cdot in$	total depth of section
$b_f := 8.22 \cdot in$	flange width
$t_w := 0.375 \cdot in$	web thickness
$A_{st} := 16.2 \cdot in^2$	area of steel
$a_{web} := (d - 2 \cdot t_f) \cdot t_w = 7.41\ in^2$	area of web
$I_{xx} := 1140 \cdot in^4$	moment of interia about x axis AISC for W21x55
$a_f := t_f \cdot b_f = 4.29\ in^2$	area of flange
$d_{web} := d - t_f \cdot 2 = 19.76\ in$	depth of web

G.2. Slab Section Properties

$t_s := 8 \cdot in$	thickness of deck slab from top of beam (ignores haunch)
$t_h := 2 \cdot in = 2.00\ in$	thickness of haunch
$S := 7.667 \cdot ft$	physical flange width
$L := 22.5 \cdot ft = 270.00\ in$	unsupported stringer length
$B := min(0.25 \cdot L, 12 \cdot t_s + \max(t_w, 0.5 \cdot b_f), S) = 67.50\ in$	Effective Flange Width, be

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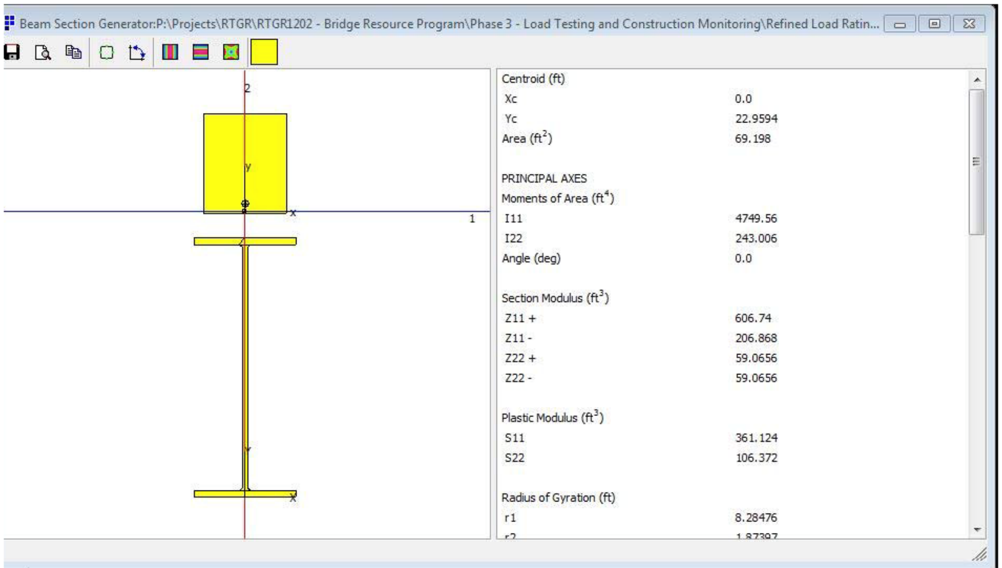
G.3. Composite Section Properties



Composite Section Properties - LRFD Design 4.6.2.6.1

Effective Flange Width, be

$$B := min\left(0.25 \cdot L, 12 \cdot t_s + \max\left(t_w, 0.5 \cdot b_f\right), S\right) = 67.50\ \boldsymbol{in}$$ Minimum flange width

Modular ratio, n

$$n := \frac{E_s}{E_c} = 10.18$$

Short term composite (n)

$$b_e := \frac{B}{n} = 6.63\ \boldsymbol{in}$$ Effective flange width

$$a_{ts} := b_e \cdot t_s = 53.07\ \boldsymbol{in}^2$$ Area of transformed slab

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Short term composite moment of intertia

$$a_{ts} := t_s \cdot b_e = 53.07\ \boldsymbol{in}^2$$
Area of transformed section

$$I_{ts} := \frac{b_e \cdot t_s^{\,3}}{12} = 283.03\ \boldsymbol{in}^4$$
Moment of Inertia of transformed section

$$y_{stc} := \frac{A_{st} \cdot \frac{d}{2} + a_{ts} \cdot \left(d + t_h + \frac{t_s}{2}\right)}{A_{st} + a_{ts}} = 22.96\ \boldsymbol{in}$$
Distance from bottom of steel to STC Centroid

$$d_{st} := y_{stc} - \frac{d}{2} = 12.56\ \boldsymbol{in}$$
Distance from STC centroid to Section Centroid

$$d_{ts} := \left(d - y_{stc}\right) + t_h + \frac{t_s}{2} = 3.84\ \boldsymbol{in}$$
Distance from STC Centroid to transformed section centroid

$$I_{stc} := I_{xx} + A_{st} \cdot d_{st}^{\,2} + I_{ts} + a_{ts} \cdot d_{ts}^{\,2} = 4761.18\ \boldsymbol{in}^4$$
STC Moment of Inertia

H. Flexural Capacity

H.1. Plastic Moment Calculation

ɘference: **AASHTO Standard Specs 10.50.1.1**

$$C_{con} := 0.85 \cdot f_c \cdot B \cdot t_s = 1009.80\ \boldsymbol{kip}$$
Check Compression in Slab

$$C_{st} := f_{y\_s} \cdot \left(2 \cdot a_f + a_{web}\right) = 511.69\ \boldsymbol{kip}$$
Check Compression in Steel

$$C := min\left(C_{st}, C_{con}\right) = 511.69\ \boldsymbol{kip}$$
Compressive force in the slab is taken as the minimum of C in concrete and C in steel

$$C' := \frac{\left(C_{st} - C_{con}\right)}{2} = -249.06\ \boldsymbol{kip}$$
Top Portion of steel section subjected to compression when Cst >Ccon

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$$NeutralAxis(C') := \text{if } C_{st} > C_{con} \; \left\| \begin{array}{l} \text{if } C' < a_f \cdot f_{y\_s} \\ \quad \left\| \begin{array}{l} y_{tos} \leftarrow \dfrac{C'}{a_f \cdot f_{y\_s}} \cdot t_f \\ \text{return } y_{tos} \end{array} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} y_{tos} \leftarrow t_f + \dfrac{C' - a_f \cdot f_{y\_s}}{a_{web} \cdot f_{y\_s}} \cdot d \\ \text{return } y_{tos} \end{array} \right. \end{array} \right. \; \text{else} \; \left\| \begin{array}{l} y_{tos} \leftarrow 0 \cdot in \\ \text{return } y_{tos} \end{array} \right.$$

Determine distance from Top of Steel to NA

$y_{tos} := NeutralAxis(C') = 0.00\ in$

Distance to Centroid from top of steel

Check for compactness of Section

$$Compactcheck(y_{tos}) := \text{if } \frac{2 \cdot y_{tos}}{t_w} \le \frac{19230 \cdot ksi^{.5}}{\sqrt{f_{y\_s}}} \; \left\| \text{return "Section is Compact"} \right. \; \text{else} \; \left\| \text{return "Section is not Compact"} \right.$$

$Compact := Compactcheck(y_{tos}) = \text{"Section is Compact"}$

Check Ducility Requirements

$a := \dfrac{C}{0.85 \cdot f_c \cdot B} = 4.05\ in$

Depth of stress block

$D_p := a = 4.05\ in$

Distance from top of slab to the neutral axis at the plastic moment

$\beta := 0.9$

$D' := \beta \cdot \dfrac{(d + t_s + t_h)}{7.5} = 3.70\ in$

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$$Ductility\left(D_p\right) := \begin{array}{l} \text{if } \left(\dfrac{D_p}{D'}\right) \le 5 \\ \quad \text{return "OK"} \\ \text{else} \\ \quad \text{return "Not OK"} \end{array}$$

Check Ductility Requirements

$Ductility := Ductility\left(D_p\right) = \text{"OK"}$

Calculate Plastic Moment

$C = 511.69\ \boldsymbol{kip}$

$S := C = 511.69\ \boldsymbol{kip}$

$arm := \dfrac{d}{2} + t_h + \left(t_s - a\right) = 16.35\ \boldsymbol{in}$

$M_p := C \cdot arm = 697.01\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

H.2. Capacity of Section

$S_t := \dfrac{I_{stc}}{d - y_{stc}} = -2199.65\ \boldsymbol{in}^3$

Section modulus at top of steel

$M_y := f_{y\_s} \cdot S_t = -5865.73\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

Moment Capacity at first yield of the compact positive moment section

$$Capacity(D') := \begin{array}{l} \text{if } D_p \le D' \\ \quad M_u \leftarrow M_p \\ \quad \text{return } M_u \\ \text{else if } D_p \le D_p \le 5 \cdot D' \\ \quad M_u \leftarrow \dfrac{5 \cdot M_p - 0.85 \cdot M_y}{4} + \dfrac{0.85 \cdot M_y - M_p}{4} \cdot \left(\dfrac{D_p}{D'}\right) \\ \quad \text{return } M_u \\ \text{else if } D_p > 5 \cdot D' \\ \quad \text{return "Not OK"} \end{array}$$

$M_u := Capacity(D') = 559.49\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M_r := M_u$

I. Shear Capacity

I.1. Stiffnener Spacing Requirements

$$Depth\_Check\left(d_{web}\right) := \text{if } \frac{d_{web}}{t_w} > 150$$
$$\quad \left\| \begin{array}{l} x \leftarrow \frac{d_{web}}{t_w} \\ \text{return "} Stiffeners\ Required\text{"} \end{array} \right.$$
$$\text{else}$$
$$\quad \left\| \text{return "} Stiffeners\ not\ Required\text{"} \right.$$

Check web depth to thickness ratio
AASHTO Standard Specs 10.48.8.3

$$Depth\_Check := Depth\_Check\left(d_{web}\right) = \text{"}Stiffeners\ not\ Required\text{"}$$

I.1. Capacity Calculation

$$k := 5 = 5.00$$

$$C\left(d_{web}\right) := \text{if } \frac{d_{web}}{t_w} < \frac{6000 \cdot \sqrt{k} \cdot \mathbf{ksi}^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \left\| C \leftarrow 1 \right.$$
$$\text{else if } \frac{6000 \cdot \sqrt{k} \cdot \mathbf{ksi}^{.5}}{\sqrt{f_{y\_s}}} \le \frac{d_{web}}{t_w} \le \frac{7500 \cdot \sqrt{k} \cdot \mathbf{ksi}^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \left\| C \leftarrow \frac{6000 \cdot \sqrt{k} \cdot \mathbf{ksi}^{.5}}{\left(\frac{d_{web}}{t_w}\right) \cdot \sqrt{f_{y\_s}}} \right.$$
$$\text{else if } \frac{d_{web}}{t_w} > \frac{7500 \cdot \sqrt{k} \cdot \mathbf{ksi}^{.5}}{\sqrt{f_{y\_s}}}$$
$$\quad \left\| C \leftarrow \frac{4.5 \cdot 10^7 \cdot k \cdot \mathbf{ksi}}{\left(\frac{d_{web}}{t_w}\right)^2 \cdot f_{y\_s}} \right.$$

Determine C, defined as the buckling shear stress divided by the shear yield stress
AASHTO Standard Specs 10.48.8.1

$$C := C\left(d_{web}\right) = 1.00$$

Return C

$$V_p := 0.58 \cdot f_{y\_s} \cdot d_{web} \cdot t_w = 137.50\ \mathbf{kip}$$

Determine plastic shear force
AASHTO Standard Specs 10.48.8.1 eq. 10-115

$$V_u := V_p \cdot C = 137.50\ \mathbf{kip}$$

J. Demand

J.1. Impact Factor

$I_{LFR} := 0.3 = 0.30$

J.2. Moment

Moment Demand from FE Model

$M_{DL\_tot} := 64.9\ \boldsymbol{kip \cdot ft} = 64.90\ \boldsymbol{kip \cdot ft}$

$M_{HS20} := 19.9\ \boldsymbol{kip \cdot ft} = 19.90\ \boldsymbol{kip \cdot ft}$

$M_{type3} := 17.8\ \boldsymbol{kip \cdot ft} = 17.80\ \boldsymbol{kip \cdot ft}$

$M_{type33} := 15.1\ \boldsymbol{kip \cdot ft} = 15.10\ \boldsymbol{kip \cdot ft}$

$M_{type3S2} := 17.74\ \boldsymbol{kip \cdot ft} = 17.74\ \boldsymbol{kip \cdot ft}$

$M_{SU4} := 20.5\ \boldsymbol{kip \cdot ft} = 20.50\ \boldsymbol{kip \cdot ft}$

$M_{SU5} := 20.6\ \boldsymbol{kip \cdot ft} = 20.60\ \boldsymbol{kip \cdot ft}$

$M_{SU6} := 22.7 \cdot \boldsymbol{kip \cdot ft} = 22.70\ \boldsymbol{kip \cdot ft}$

$M_{SU7} := 23.4\ \boldsymbol{kip \cdot ft} = 23.40\ \boldsymbol{kip \cdot ft}$

$M_{NRL} := 23.8\ \boldsymbol{kip \cdot ft} = 23.80\ \boldsymbol{kip \cdot ft}$

$M_{Lane} := 12.1\ \boldsymbol{kip \cdot ft} = 12.10\ \boldsymbol{kip \cdot ft}$

J.3. Shear

$V_{DL\_total} := 15.7 \cdot \boldsymbol{kip} = 15.70\ \boldsymbol{kip}$

$V_{HS20} := 11.1 \cdot \boldsymbol{kip} = 11.10\ \boldsymbol{kip}$

$V_{type3} := 8.2 \cdot \boldsymbol{kip} = 8.20\ \boldsymbol{kip}$

$V_{type33} :=$ [illegible] $\cdot$ **kip** $=$ [illegible] **kip**

$V_{type3S2} := 8.02 \cdot \mathbf{kip} = 8.02\ \mathbf{kip}$

$V_{SU4} := 10.64 \cdot \mathbf{kip} = 10.64\ \mathbf{kip}$

$V_{SU5} := 10.37 \cdot \mathbf{kip} = 10.37\ \mathbf{kip}$

$V_{SU6} := 11.33 \cdot \mathbf{kip} = 11.33\ \mathbf{kip}$

$V_{SU7} := 11.22 \cdot \mathbf{kip} = 11.22\ \mathbf{kip}$

$V_{NRL} := 11.29 \cdot \mathbf{kip} = 11.29\ \mathbf{kip}$

$V_{Lane} := 6.13 \cdot \mathbf{kip} = 6.13\ \mathbf{kip}$

K. Load Factor Rating

K1. Factors & Equations

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$	dead load factor for concrete/asphalt (MBE - 6B.4.3)
$A_{2\_inv} := 2.17$	live load factor (inventory level) (MBE - 6B.4.3)
$A_{2\_op} := 1.3$	live load factor (operating level) (MBE - 6B.4.3)
$\phi_f := 0.9$ $\quad \phi_s := 0.9$	strength reduction factors

K2. Flexure Ratings

HS20 Rating

$$IR_{HS20str\_m} := \frac{\left(\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})\right)}{A_{2\_inv} \cdot \left(M_{HS20} \cdot (1 + I_{LFR})\right)} = 7.52$$

$$OR_{HS20str\_m} := IR_{HS20str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.56$$

AASHTO Type 3 Rating

$$IR_{type3str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type3} \cdot \left(1 + I_{LFR}\right)} = 8.41$$

$$OR_{type3str\_m} := IR_{type3str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 14.04$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_m} := \frac{\left(\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 9.92$$

$$OR_{type33str\_m} := IR_{type33str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 16.55$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str\_m} := \frac{\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 8.44$$

$$OR_{type3S2str\_m} := IR_{type3S2str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 14.09$$

Formula B SU4

$$IR_{SU4str\_m} := \frac{\phi_f \cdot \left(M_r\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 7.30$$

$$OR_{SU4str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.19$$

Formula B SU5

$$IR_{SU5str\_m} := \frac{\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU5} \cdot (1 + I_{LFR})} = 7.27$$

$$OR_{SU5str\_m} := IR_{SU4str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 12.19$$

Formula B SU6

$$IR_{SU6str\_m} := \frac{\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU6} \cdot (1 + I_{LFR})} = 6.60$$

$$OR_{SU6str\_m} := IR_{SU6str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 11.01$$

Formula B SU7

$$IR_{SU7str\_m} := \frac{\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 6.40$$

$$OR_{SU7str\_m} := IR_{SU7str\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.68$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_m} := \frac{\phi_f \cdot (M_r) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 6.29$$

$$OR_{NRLstr\_m} := IR_{NRLstr\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.50$$

Lane Load - Moment

$$IR_{Lane\_LFR\_m} := \frac{(\phi_f \cdot (M_r) - A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot (M_{Lane} \cdot (1 + I_{LFR}))} = 12.38$$

$$OR_{Lane\_LFR\_m} := IR_{Lane\_LFR\_m} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 20.66$$

K3. Shear Ratings

Shear

HS20 Rating

$$IR_{HS20str\_v} := \frac{\left(\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_total}\right)\right)}{A_{2\_inv} \cdot \left(V_{HS20} \cdot \left(1 + I_{LFR}\right)\right)} = 3.33$$

$$OR_{HS20str\_v} := IR_{HS20str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.55$$

LFR Lane Load - Shear

$$IR_{Lane\_LFR\_v} := \frac{\left(\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_total}\right)\right)}{A_{2\_inv} \cdot \left(V_{Lane} \cdot \left(1 + I_{LFR}\right)\right)} = 6.02$$

$$OR_{Lane\_LFR\_v} := IR_{Lane\_LFR\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 10.05$$

AASHTO Type 3 Rating

$$IR_{type3str\_v} := \frac{\left(\left(\phi_s \cdot V_u\right) - \left(A_1 \cdot V_{DL\_total}\right)\right)}{A_{2\_inv} \cdot V_{type3} \cdot \left(1 + I_{LFR}\right)} = 4.50$$

$$OR_{type3str\_v} := IR_{type3str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.51$$

AASHTO Type 3-3 Rating

$$IR_{type33str\_v} := \frac{\left(\phi_s \cdot V_u - \left(A_1 \cdot V_{DL\_total}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 5.53$$

$$OR_{type33str\_v} := IR_{type33str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 9.24$$

AASHTO Type 3S-2 Rating

$$IR_{type3S2str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_total})}{A_{2\_inv} \cdot V_{type3S2} \cdot (1 + I_{LFR})} = 4.60$$

$$OR_{type3S2str\_v} := IR_{type3S2str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 7.68$$

Formula B SU4

$$IR_{SU4str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_total})}{A_{2\_inv} \cdot V_{SU4} \cdot (1 + I_{LFR})} = 3.47$$

$$OR_{SU4str\_v} := IR_{SU4str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.79$$

Formula B SU5

$$IR_{SU5str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_total})}{A_{2\_inv} \cdot V_{SU5} \cdot (1 + I_{LFR})} = 3.56$$

$$OR_{SU5str\_v} := IR_{SU5str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.94$$

Formula B SU6

$$IR_{SU6str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_total})}{A_{2\_inv} \cdot V_{SU6} \cdot (1 + I_{LFR})} = 3.26$$

$$OR_{SU6str\_v} := IR_{SU6str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.44$$

Formula B SU7

$$IR_{SU7str\_v} := \frac{(\phi_s \cdot V_u) - (A_1 \cdot V_{DL\_total})}{A_{2\_inv} \cdot V_{SU7} \cdot (1 + I_{LFR})} = 3.29$$

$$OR_{SU7str\_v} := IR_{SU7str\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.49$$

Formula B NRL (V'=10')

$$IR_{NRLstr\_v} := \frac{(\phi_s \cdot V_u) - A_1 \cdot V_{DL\_total}}{A_{2\_inv} \cdot V_{NRL} \cdot (1 + I_{LFR})} = 3.27$$

$$OR_{NRLstr\_v} := IR_{NRLstr\_v} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 5.46$$

Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 0324152

By: CTY, 10-29-13
Chk'd:JBP 3-3-14

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str\_m}$	$HS20 \cdot OR_{HS20str\_m}$	$HS20 \cdot IR_{HS20str\_v}$	$HS20 \cdot OR_{HS20str\_v}$
$Type3$	$Type3 \cdot IR_{type3str\_m}$	$Type3 \cdot OR_{type3str\_m}$	$Type3 \cdot IR_{type3str\_v}$	$Type3 \cdot OR_{type3str\_v}$
$Type33$	$Type33 \cdot IR_{type33str\_m}$	$Type33 \cdot OR_{type33str\_m}$	$Type33 \cdot IR_{type33str\_v}$	$Type33 \cdot OR_{type33str\_v}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str\_m}$	$Type3S2 \cdot OR_{type3S2str\_m}$	$Type3S2 \cdot IR_{type3S2str\_v}$	$Type3S2 \cdot OR_{type3S2str\_v}$
$SU4$	$SU4 \cdot IR_{SU4str\_m}$	$SU4 \cdot OR_{SU4str\_m}$	$SU4 \cdot IR_{SU4str\_v}$	$SU4 \cdot OR_{SU4str\_v}$
$SU5$	$SU5 \cdot IR_{SU5str\_m}$	$SU5 \cdot OR_{SU5str\_m}$	$SU5 \cdot IR_{SU5str\_v}$	$SU5 \cdot OR_{SU5str\_v}$
$SU6$	$SU6 \cdot IR_{SU6str\_m}$	$SU6 \cdot OR_{SU6str\_m}$	$SU6 \cdot IR_{SU6str\_v}$	$SU6 \cdot OR_{SU6str\_v}$
$SU7$	$SU7 \cdot IR_{SU7str\_m}$	$SU7 \cdot OR_{SU7str\_m}$	$SU7 \cdot IR_{SU7str\_v}$	$SU7 \cdot OR_{SU7str\_v}$
NRL	$NRL \cdot IR_{NRLstr\_m}$	$NRL \cdot OR_{NRLstr\_m}$	$NRL \cdot IR_{NRLstr\_v}$	$NRL \cdot OR_{NRLstr\_v}$

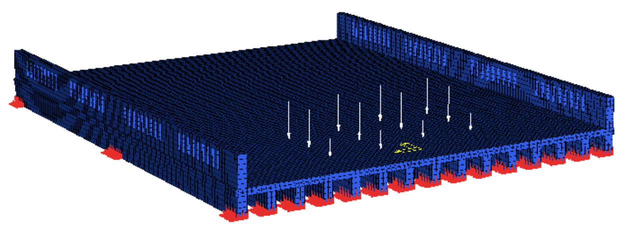
$$FlexureInventory = \begin{bmatrix} 270.89 \\ 210.31 \\ 396.66 \\ 337.63 \\ 197.22 \\ 225.34 \\ 229.23 \\ 247.97 \\ 251.66 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 452.17 \\ 351.05 \\ 662.12 \\ 563.59 \\ 329.20 \\ 377.97 \\ 382.63 \\ 413.91 \\ 420.09 \end{bmatrix} ton$$

$$ShearInventory = \begin{bmatrix} 119.71 \\ 112.53 \\ 221.36 \\ 184.10 \\ 93.67 \\ 110.34 \\ 113.21 \\ 127.48 \\ 130.77 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 199.83 \\ 187.85 \\ 369.50 \\ 307.30 \\ 156.35 \\ 184.19 \\ 188.97 \\ 212.79 \\ 218.29 \end{bmatrix} ton$$

Structure 1701151– US40 over W. Branch of Game Creek

Bridge 1701151 - RT40 Over W Branch Creek

T-Beam Rating - Positive Flexure and Shear



General Bridge Information

1) Built in 1919 - widened - 1929
2) Length - 24' - clear span 20'
3) Width - 44.2'
4) ADT - 15300 ADTT - 1071 one direction
5) Structure in fair condition (Condition rating = 5)

Description & Purpose:

The purpose of the following calculations are to determine the inventory and operating load rating for the reinforced concrete T-Beam for the following rating vehicles: HS20, Type 3, Type 3-3, Type 3S2, SU4, SU5, SU6, SU7, and NRL

References:

1) P:\Projects\RTGR\RTGR1202 - Bridge Resource Program\Phase 3 - Load Testing and Construction Monitoring\Refined Load Ratings\Bridge 1701151 - Route 40 WB over W Branch Creek

- Inspection report - Cycle 16 - **1701151_20100712cy16_Report.pdf**

2) AASHTO, LFD Specifications, 17th edition
3) AASHTO, Manual for Bridge Evaluation, 2011.
4) NJDOT Bridge Design Manual - Section 43

Approach:

1) Create finite element model of the structure from design drawings
2) Error screen model
3) Run load influence analysis and generate LL combinations
4) Perform linear analysis for vehicular LL combinations.
5) Identify maximum force effects for each rating vehicle
6) Calculate the load ratings.

Assumptions and Limitations:

1) Assumed rebar size and spacing
2) Assumed stirrup size and spacing
3) Assumed material properties according to AASHTO MBE

Demand:

Constants

$\gamma_c := 150 \cdot \frac{lbf}{ft^3}$	Unit weight on concrete
$\gamma_a := 144 \cdot \frac{lbf}{ft^3}$	Unit weight of asphalt
$L := 53 \cdot ft$	Span length
$W_{road} := 38 \cdot ft + 4 \cdot in$	Roadway width
$W_{edge} := 45 \cdot ft$	Bridge out to out width
$S := 3.143 \cdot ft$	Beam spacing
$lane := 12 \cdot ft$	Lane width
$beams := 15$	

Dead Load

Dead Load Moment Due to Concrete Elements

$M_{DL\_tot} := 41.7 \cdot kip \cdot ft$	Total dead load moment on interior T-beam section
$V_{DL\_tot} := 13.2 \cdot kip$	Total dead load shear on interior T-beam section

Live Load Analysis

Live Load Moments

$I_{LFR} := 0.30$ Impact factor - Unknown riding surface - MBE Table C6A.4.4.3-1

$M_{HS20} := 22 \cdot kip \cdot ft$ $M_{SU4} := 24.2 \cdot kip \cdot ft$

$M_{type3} := 21.5 \cdot kip \cdot ft$ $M_{SU5} := 24.9 \cdot kip \cdot ft$

$M_{type33} := 17.7 \cdot kip \cdot ft$ $M_{SU6} := 26.6 \cdot kip \cdot ft$ Live load moments taken from FE model

$M_{type3S2} := 20.7 \cdot kip \cdot ft$ $M_{SU7} := 26.9 \cdot kip \cdot ft$

$M_{Lane} := 12.7 \cdot kip \cdot ft$ $M_{NRL} := 27 \cdot kip \cdot ft$

Live Load Shears

$V_{HS20} := 6.36 \cdot kip$ $V_{SU4} := 6.17 \cdot kip$

$V_{type3} := 5.46 \cdot kip$ $V_{SU5} := 6.27 \cdot kip$

$V_{type33} := 4.42 \cdot kip$ $V_{SU6} := 6.83 \cdot kip$ Live load shears taken from FE model

$V_{type3S2} := 5.72 \cdot kip$ $V_{SU7} := 7.22 \cdot kip$

$V_{Lane} := 3.47 \cdot kip$ $V_{NRL} := 7.54 \cdot kip$

Capacity

Compute the nominal flexural resistance of an interior T-beam

Assumes dimensions taken from visual inspection

Assumes 1 layer of 4 #7 rebar

$h := 2 \cdot ft + 11 \cdot in = 35.00\ in$ depth of T-beam section

$d_{slab} := 11 \cdot in$ depth of slab

$t_w := 12 \cdot in$ thickness of T-beam web

$t_s := 9 \cdot in$ thickness of concrete slab

$b_{ftop} := 12 \cdot in$

$cover1 := 2 \cdot in$ distance from bottom of section to first rebar layer

$cover2 := 2 \cdot in + 2 \cdot in = 4.00\ in$ distance from bottom of section to second rebar layer

Constants

$f_c := 2500 \cdot psi$ Compressive strength of concrete for a bridge constructed before 1959 MBE Table 6A.5.2.1-1

$f_y := 33000 \cdot psi$ Yield stress of reinforcing steel for a bridge constructed before 1959 MBE Table 6A.5.2.2-1

$d_{bar} := \frac{7}{8} \cdot in$ Reinforcement bar diameter

$n_{bars} := 4$ Number of bars in T-beam

$n_{bars\_1} := 4$ Number of bars in first layer of rebar (Assume 1 layer)

$n_{bars\_2} := 0$ Number of bars in second layer of rebar

$A_s := n_{bars} \cdot \frac{\left(\pi \cdot (d_{bar})^2\right)}{4} = 2.41\ in^2$ Area of reinforcing steel - 7/8" diameter rods

$y := \frac{((cover1 \cdot n_{bars\_1}) + (cover2 \cdot n_{bars\_2}))}{n_{bars}} = 2.00\ in$ Distance to C.G of steel from bottom of section

$d_s := h - y = 33.00\ in$ Depth from extreme compression fiber to reinforcing steel C.G

Flexure

Calculate effective slab width - LFD 8.10.1.1

Flange width is taken as the lesser of:

*1/4 * L*

12 ts + greater of tw or 1/2 bf top*

S (beam spacing)

$$b_{eff} := min\left(0.25 \cdot L, 12 \cdot t_s + \left(\max\left(t_w, 0.5 \cdot b_{ftop}\right)\right), S\right) = 37.72\ in$$

$$\rho_{act} := \frac{A_s}{b_{eff} \cdot d_s} = 0.0019$$

Mu (ultimate moment on section) is found in accordance with AASHTO LFD Article 8.16

Consider a rectangular section with compression limited to top slab. Check MBE 6B.5.3.2 requirements for 75 percent of balanced condition

$$\beta_1 := 0.85$$

$$\rho_{bal} := \frac{\left(0.85 \cdot \beta_1 \cdot f_c\right)}{f_y} + \frac{87000 \cdot \boldsymbol{psi}}{87000 \cdot \boldsymbol{psi} + f_y} = 0.78$$ **LFD Eq. 8-18**

$$\rho_{max} := 0.75 \cdot \rho_{bal} = 0.58$$

$$\textbf{if}\left(\rho_{act} < \rho_{max}, \text{“}OK\text{”}, \text{“}Not\ Ok\text{”}\right) = \text{“}OK\text{”}$$

Calculate depth of equivalent stress block

$$a := \frac{\left(A_s \cdot f_y\right)}{0.85 \cdot f_c \cdot b_{eff}} = 0.99\ in$$ Depth of equivalent stress block - LFD Eq 8-17

$$\textbf{if}\left(a < d_{slab}, \text{“}OK - Within\ slab\text{”}, \text{“}Not\ within\ slab\text{”}\right) = \text{“}OK - Within\ slab\text{”}$$

Intelligent Infrastructure Systems
A Pennoni Company

Project: NJDOT Bridge Resource Program
Subject: Bridge 1701151

By: JBP 3-5-13
Chk'd: MTY 3-22-13 Rev. 8-28-13

Nominal Flexural Capacity

$$M_n := A_s \cdot f_y \cdot \left(d_s - \left(\frac{a}{2}\right)\right) = 215.00\ \boldsymbol{kip \cdot ft}$$ LFD Eq 8-16

Shear

Compute Nominal Shear Resistance

Stirrups: Stirrup size and spacing are unknown

$$d_{stirrup} := \frac{3}{8} \cdot \boldsymbol{in}$$ Assumed stirrup diameter

$$s_{stirrup} := 12 \cdot \boldsymbol{in}$$ Assumed stirrup spacing

$$A_v := 2 \cdot \left(\frac{\pi}{4}\right) \cdot \left(d_{stirrup}\right)^2 = 0.22\ \boldsymbol{in}^2$$ Stirrup steel area

Shear Capacity

Shear capacity of a concrete section is taken as the sum of the shear capacity of the concrete and the shear carried by shear reinforcement

$$b_w := t_w = 12.00\ \boldsymbol{in}$$ Minimum beam web width

$$V_c := \left(2 \cdot \boldsymbol{psi} \cdot \sqrt{2500} \cdot b_w \cdot d_s\right) = 39.60\ kip$$ LFD Design Eq. 8-49

$$V_s := \frac{\left(A_v \cdot f_y \cdot d_s\right)}{s_{stirrup}} = 20.05\ \boldsymbol{kip}$$ LRFD Design Eq. 5.8.3.3-4

$$V_n := V_c + V_s = 59.65\ \boldsymbol{kip}$$

Load Factor Rating

$$RF_{Moment} = \frac{C - A_1 \cdot M_{DC}}{A_2 \cdot M_{LL+IM}}$$ Moment Rating Equation

$$RF_{Shear} = \frac{C - A_1 \cdot V_{DC}}{A_2 \cdot V_{LL+IM}}$$ Shear Rating Equation

Load Factors:

$A_1 := 1.25$ dead load factor for concrete/asphalt (MBE - 6B.4.3)

$A_{2\_inv} := 2.17$ live load factor (inventory level) (MBE - 6B.4.3)

$A_{2\_op} := 1.3$ live load factor (operating level) (MBE - 6B.4.3)

$\phi_f := 0.9$

Flexure Ratings

HS20 Rating

$$IR_{HS20str} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{HS20} \cdot (1 + I_{LFR}))} = 2.28$$

$$OR_{HS20str} := IR_{HS20str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.80$$

Lane Load Rating

$$IR_{Lanestr} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot (M_{Lane} \cdot (1 + I_{LFR}))} = 3.95$$

$$OR_{Lanestr} := IR_{Lanestr} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.59$$

AASHTO Type 3 Rating

$$IR_{type3str} := \frac{((\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot}))}{A_{2\_inv} \cdot M_{type3} \cdot (1 + I_{LFR})} = 2.33$$

$$OR_{type3str} := IR_{type3str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.89$$

AASHTO Type 3-3 Rating

$$IR_{type33str} := \frac{\left(\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot M_{type33} \cdot \left(1 + I_{LFR}\right)} = 2.83$$

$$OR_{type33str} := IR_{type33str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.73$$

AASHTO Type 3S2 Rating

$$IR_{type3S2str} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 2.42$$

$$OR_{type3S2str} := IR_{type3S2str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.04$$

AASHTO SU4 Rating

$$IR_{SU4str} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU4} \cdot \left(1 + I_{LFR}\right)} = 2.07$$

$$OR_{SU4str} := IR_{SU4str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.46$$

AASHTO SU5 Rating

$$IR_{SU5str} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU5} \cdot \left(1 + I_{LFR}\right)} = 2.01$$

$$OR_{SU5str} := IR_{SU5str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.36$$

AASHTO SU6 Rating

$$IR_{SU6str} := \frac{\left(\phi_f \cdot M_n\right) - \left(A_1 \cdot M_{DL\_tot}\right)}{A_{2\_inv} \cdot M_{SU6} \cdot \left(1 + I_{LFR}\right)} = 1.88$$

$$OR_{SU6str} := IR_{SU6str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.14$$

AASHTO SU7 Rating

AASHTO SU7 Rating

$$IR_{SU7str} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{SU7} \cdot (1 + I_{LFR})} = 1.86$$

$$OR_{SU7str} := IR_{SU7str} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.11$$

AASHTO NRL Rating

$$IR_{NRLstr} := \frac{(\phi_f \cdot M_n) - (A_1 \cdot M_{DL\_tot})}{A_{2\_inv} \cdot M_{NRL} \cdot (1 + I_{LFR})} = 1.86$$

$$OR_{NRLstr} := IR_{NRLstr} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.10$$

Shear Ratings

HS20 Rating

$$IR_{HS20sh} := \frac{((\phi_f \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{HS20} \cdot (1 + I_{LFR}))} = 2.07$$

$$OR_{HS20sh} := IR_{HS20sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.46$$

Lane Load Rating

$$IR_{Lanesh} := \frac{((\phi_f \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot (V_{Lane} \cdot (1 + I_{LFR}))} = 3.80$$

$$OR_{Lanesh} := IR_{Lanesh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 6.34$$

AASHTO Type 3 Rating

$$IR_{type3sh} := \frac{((\phi_f \cdot V_n) - (A_1 \cdot V_{DL\_tot}))}{A_{2\_inv} \cdot V_{type3} \cdot (1 + I_{LFR})} = 2.41$$

$$OR_{type3sh} := IR_{type3sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.03$$

AASHTO Type 3-3 Rating

$$IR_{type33sh} := \frac{\left(\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)\right)}{A_{2\_inv} \cdot V_{type33} \cdot \left(1 + I_{LFR}\right)} = 2.98$$

$$OR_{type33sh} := IR_{type33sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 4.98$$

AASHTO Type 3S2 Rating

$$IR_{type3S2sh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{type3S2} \cdot \left(1 + I_{LFR}\right)} = 2.30$$

$$OR_{type3S2sh} := IR_{type3S2sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.85$$

AASHTO SU4 Rating

$$IR_{SU4sh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU4} \cdot \left(1 + I_{LFR}\right)} = 2.14$$

$$OR_{SU4sh} := IR_{SU4sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.57$$

AASHTO SU5 Rating

$$IR_{SU5sh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU5} \cdot \left(1 + I_{LFR}\right)} = 2.10$$

$$OR_{SU5sh} := IR_{SU5sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.51$$

AASHTO SU6 Rating

$$IR_{SU6sh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU6} \cdot \left(1 + I_{LFR}\right)} = 1.93$$

$$OR_{SU6sh} := IR_{SU6sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.22$$

AASHTO SU7 Rating

$$IR_{SU7sh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{SU7} \cdot \left(1 + I_{LFR}\right)} = 1.83$$

$$OR_{SU7sh} := IR_{SU7sh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 3.05$$

AASHTO NRL Rating

$$IR_{NRLsh} := \frac{\left(\phi_f \cdot V_n\right) - \left(A_1 \cdot V_{DL\_tot}\right)}{A_{2\_inv} \cdot V_{NRL} \cdot \left(1 + I_{LFR}\right)} = 1.75$$

$$OR_{NRLsh} := IR_{NRLsh} \cdot \left(\frac{A_{2\_inv}}{A_{2\_op}}\right) = 2.92$$

Summary Rating Table

$HS20 := 36 \cdot ton$ $\quad Type3S2 := 40 \cdot ton$ $\quad SU6 := 34.75 \cdot ton$

$Type3 := 25 \cdot ton$ $\quad SU4 := 27 \cdot ton$ $\quad SU7 := 38.75 \cdot ton$

$Type33 := 40 \cdot ton$ $\quad SU5 := 31 \cdot ton$ $\quad NRL := 40 \cdot ton$

$Truck$	$FlexureInventory$	$FlexureOperating$	$ShearInventory$	$ShearOperating$
$HS20$	$HS20 \cdot IR_{HS20str}$	$HS20 \cdot OR_{HS20str}$	$HS20 \cdot IR_{HS20sh}$	$HS20 \cdot OR_{HS20sh}$
$Type3$	$Type3 \cdot IR_{type3str}$	$Type3 \cdot OR_{type3str}$	$Type3 \cdot IR_{type3sh}$	$Type3 \cdot OR_{type3sh}$
$Type33$	$Type33 \cdot IR_{type33str}$	$Type33 \cdot OR_{type33str}$	$Type33 \cdot IR_{type33sh}$	$Type33 \cdot OR_{type33sh}$
$Type3S2$	$Type3S2 \cdot IR_{type3S2str}$	$Type3S2 \cdot OR_{type3S2str}$	$Type3S2 \cdot IR_{type3S2sh}$	$Type3S2 \cdot OR_{type3S2sh}$
$SU4$	$SU4 \cdot IR_{SU4str}$	$SU4 \cdot OR_{SU4str}$	$SU4 \cdot IR_{SU4sh}$	$SU4 \cdot OR_{SU4sh}$
$SU5$	$SU5 \cdot IR_{SU5str}$	$SU5 \cdot OR_{SU5str}$	$SU5 \cdot IR_{SU5sh}$	$SU5 \cdot OR_{SU5sh}$
$SU6$	$SU6 \cdot IR_{SU6str}$	$SU6 \cdot OR_{SU6str}$	$SU6 \cdot IR_{SU6sh}$	$SU6 \cdot OR_{SU6sh}$
$SU7$	$SU7 \cdot IR_{SU7str}$	$SU7 \cdot OR_{SU7str}$	$SU7 \cdot IR_{SU7sh}$	$SU7 \cdot OR_{SU7sh}$
NRL	$NRL \cdot IR_{NRLstr}$	$NRL \cdot OR_{NRLstr}$	$NRL \cdot IR_{NRLsh}$	$NRL \cdot OR_{NRLsh}$

$$FlexureInventory = \begin{bmatrix} 82.01 \\ 58.27 \\ 113.26 \\ 96.84 \\ 55.92 \\ 62.39 \\ 65.47 \\ 72.19 \\ 74.25 \end{bmatrix} ton \qquad FlexureOperating = \begin{bmatrix} 136.89 \\ 97.27 \\ 189.05 \\ 161.65 \\ 93.34 \\ 104.15 \\ 109.29 \\ 120.51 \\ 123.93 \end{bmatrix} ton$$

$$ShearInventory = \begin{bmatrix} 74.61 \\ 60.35 \\ 119.28 \\ 92.17 \\ 57.68 \\ 65.17 \\ 67.06 \\ 70.74 \\ 69.92 \end{bmatrix} ton \qquad ShearOperating = \begin{bmatrix} 124.53 \\ 100.74 \\ 199.10 \\ 153.85 \\ 96.28 \\ 108.78 \\ 111.94 \\ 118.08 \\ 116.72 \end{bmatrix} ton$$

APPENDIX 2E
SEVERELY SKEWED STEEL BRIDGES UNDER CONSTRUCTION

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March 22, 2013

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New Jersey Department of Transportation
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RE: Anderson Street Bridge (020023A)
Bergen County
Review of refined load rating, testing or analysis options

Mr. Kasbekar,

As requested by the Structural Evaluation office, the Bridge Resource Program Team has reviewed options to perform refined load rating, modeling and analyses of the Anderson Street Bridge (structure #020023A) in Bergen County. The bridge consists of six simply supported spans of non-composite adjacent prestressed box beams. The capacity of each box beam is highly dependent upon the condition of the prestressing strands, which is difficult to assess through routine visual inspection. The current inspection shows a loss of prestressing strands in various girders which significantly impacts the rated load carrying capacity of the structure. Additionally, load sharing between adjacent box beams in this bridge relies on the effectiveness of the transverse post tensioning and the condition of the shear keys. Since these elements are not easily inspected, similar to the prestressing strands, it is difficult to rely on calculated distribution factors that assume the shear keys and lateral post tensioning are fully effective.

Load testing is typically the most conclusive test method to identify reserve capacity in a bridge. However, adjacent prestressed box girder structures are inherently complex and very difficult to predict remaining life. Many elements are difficult to inspect including strands, post tensioning, and shear keys; which result in an increased need for conservative assumptions. Further, the unknown condition of these elements and overall poor bridge condition, add up to a potentially-unsafe load testing environment. In 2005, the Lakeview Drive Bridge over I-70 in western Pennsylvania collapsed under its own weight, serving as an example of how load carrying mechanisms, which are assumed to be effective, may in fact be significantly affected by deterioration. This also may be the case of the Bridge 020023A. Thus, we believe load testing is not appropriate for identifying the capacity of the structure due to the unknown condition of critical load carrying mechanisms. As an alternative to load testing, other methods exist that may offer insight into the current condition of the structure including dynamic testing and operational strain monitoring. The data from these methods could be used to calibrate a finite element model and produce a model based load rating. However, the outcome of this model/experiment correlation may not result in increased load ratings. The analytical load rating performed previously indicates loss of prestressing strands was taken into consideration, which may prove consistent with further modeling/testing. Our team believes that testing, in all likelihood, will not reveal additional capacity or redundant load paths that would improve the load ratings. Further, Even if our team did identify reserve capacity, it would be imprudent to suggest the findings inferred an increase in the structure’s life-expectancy.

It is our recommendation that the owner or agency in charge consider repair or replacement of the structure, in conformance with the latest inspection report or other engineering opinion previously submitted.

Sincerely,

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RU435056-2

Synthesis Report on Construction Deformations Severely Skewed Structural Steel Bridge Construction

Technical Memorandum
April, 2014

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Disclaimer Statement

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16. Abstract

More and more skewed bridges are being placed into service as available simple alignments vanish due to congestion in the built environment. Straight, skewed steel girder bridges can exhibit behavior far different from non-skewed steel girder bridges, approaching that of horizontally curved steel bridges. This behavior resulting in out-of-plane distortions can cause problems during construction. Skewed bridge behavior is characterized by a recently introduced skew index which delineates moderately and severely skewed bridge behavior. Based upon published and ongoing research activities, best practices and associated specifications for analysis, design, detailing, fabrication and erection are recommended for straight, skewed steel girder bridges. While approximate 1-D line-girder analysis can determine relatively accurate results for proportioning and detailing of skewed steel girders, any resulting out-of-plane distortions are not modeled. Out-of-plane distortions before or after concrete deck placement can only be anticipated with refined 2-D grid analysis. Moderately skewed bridges can be relatively easily erected when the crossframes are detailed to total dead-load fit (TDLF). Severely skewed bridges can be more easily erected when the crossframes are detailed to steel dead-load fit (SDLF), but unfortunately the webs will not be plumb in the final constructed position. As properly detailed crossframes provide geometry control for the erector, standard-size bolt holes must be used for crossframe-girder connections. In practice, out-of-plumb webs from acceptable fit within prescribed tolerances have exhibited no noted reductions in strength or serviceability. Finally, field studies of the recently constructed Route-3 Bridge at the Passaic River Crossing validate the recommendations.

17. Key Words Straight, skew, steel girder, skew index, structural analysis, fabrication, steel bridge erection, total dead load fit, steel dead load fit, 2-D grid analysis, crossframe, out-of-plane distortion	18. Distribution Statement

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1. Introduction

At the time of erection, steel girders in highly skewed bridges can deflect out-of-plumb due to differential deflections experienced at crossframe connections. Ideally, the webs would be theoretically plumb in the final constructed position under total dead load. Due to constructability issues, this is not always desirable. Two detailing options are available for straight, skewed girders: steel dead-load fit (SDLF) or total dead-load fit (TDLF). Erecting steel girders for severely skewed bridges require special consideration. AASHTO 6.7.4 requires for bridges with a skew greater than 20 degrees, that crossframes be installed normal to the main members. The National Steel Bridge Association (NSBA) Steel Bridge Design Handbook states that this practice results in large differential deflections between each end of the crossframes. It further suggests that special guidance should be provided to the fabricator and erector. NSBA indicates that crossframes and diaphragms tend to equalize deflections, further cautioning that designing the interior and exterior girders for different inertias and dead load deflections can result in significant differences in camber between girders. The effect of this differential camber during construction needs to be considered by the designer since the differences will likely complicate girder fabrication and erection.

Problem Statement and Research Objectives

NJDOT expressed interest in research focused on understanding the effects of highly skewed bridges, and developing recommendations for design, fabrication and erection of these structures. The following report documents the literature search performed on the construction of highly skewed bridges and distortions experienced during the erection and concrete deck casting process. The report includes contractors' means and methods for girder erection in these complex structures and fabricators diaphragm connection detailing to determine the impact that these variables may have on out-of-plane distortion. Lastly, the report also covers design considerations that should be outlined in the contract drawings to mitigate construction complexities and aid the fabricator during the fabrication of girders for these complex structures.

Organization

This synthesis report is subdivided into six main sections. Section 2 provides design comprehensive review of the design phase of a straight, severely skewed steel bridge. It covers design considerations including girder and crossframe layout, structure modeling and analysis, and differential deflections between crossframe connections. It also covers various recommended notes and guidance to be included in the construction plans.

Section 3 provides fabrication guidance in detailing, bracing, fit considerations, layover, theoretical plumbness and locked-in forces. This section will focus on guidance provided by steel girder fabricators that includes what they look for in the contract drawings, how they interpret the camber, girder and diaphragm positions, requested fit condition, and how they detail for the requested fit condition. The section will provide further detail on the various fit conditions and provide guidance in selecting fit condition based on skew indices.

Section 4 provides construction guidance during girder erection and concrete placement. It will cover key issues including tightening of crossframe-connection bolts, tying down girders, blocking bearings and uplift considerations.

Section 5 describes the modeling, instrumenting, data analysis and correlation of findings in the study of a straight, severely skewed steel girder bridge under construction (Structure #1601-162, Route 3 over NJ Transit, Passaic River crossings).

Finally, Section 6 provides a summary and conclusions from this research as well as directions for future work.

Proposed additions and revisions to NJDOT *Design Manual for Bridges and Structures* and the NJDOT *Standard Specifications for Road and Bridge Construction* (NJDOT, 2007) are compiled in Appendix A

Appendix B includes a proposed checklist for the review of the contractor's erection-plan-and-procedures submittal.

Appendix C provides a listing of references and documents reviewed as part of this study.

2. Design

Characterizing the Effects of Skew

Severely skewed, straight bridges can be classified as those with a skew index, I_s, greater than 0.30. The skew index introduced in NCHRP Report 725 (White, et al., 2012) characterizes the effects of skew not only by the skew angle but also the aspect ratio of the span, w_g/L. It is defined as:

$$I_s = \frac{w_g}{L} \tan q$$

Where:

w_g = width of steel framing between fascia girders (ft)

q = largest skew angle (radians)

L = average span length within a single span (ft)

As the aspect ratio and/or the skew angle increases, the effects of skew become more severe.

Web Plumbness

The theoretical erected web position (in other words, web plumbness) changes as skewed bridges are constructed. Unlike straight, non-skewed girders which deflect uniformly across the width of the bridge, skewed girder deflections vary across the bridge width as each girder has a different span length. When crossframes are installed, they force compatibility between the girders.

Detailing of the crossframes plays a significant role in the web plumbness and the constructability of straight, skewed girder bridges. Ideally, the webs would be theoretically plumb in the final constructed position under total dead load. Due to constructability issues, this is not always desirable. Two detailing options are available for straight, skewed girders: steel dead-load fit (SDLF) or total dead-load fit (TDLF).

Crossframes detailed for SDLF simply means detailing such that the crossframe drop between girders is based upon the differential girder deflection due to steel dead load only. Crossframe drop is illustrated by the heavy arrows in Figure 1, adapted from the contract drawings for Bridge No. 4 of Route 3 at the Passaic River Crossing. Crossframes detailed for SDLF should theoretically fit between girders with no applied force necessary to make the fit during erection. The webs are theoretically plumb before the crossframes are installed and after installation. When subsequent loads are applied to the bridge (such as the dead load of the deck), the girders again deflect differently, but the crossframes force compatibility at their connections to the girders and the girders also twist-rotate (commonly called layover). With the SDLF crossframes installed, the girders all still deflect differently but the twist-rotations of the all of girders are equal as the crossframes are much stiffer than the torsional stiffness of the girders. Crossframes detailed for SDLF will be theoretically stress-free prior to the deck placement. Girder webs of bridges with crossframes detailed for SDLF will be theoretically plumb prior to deck placement, but will layover and not be plumb in the final constructed position.

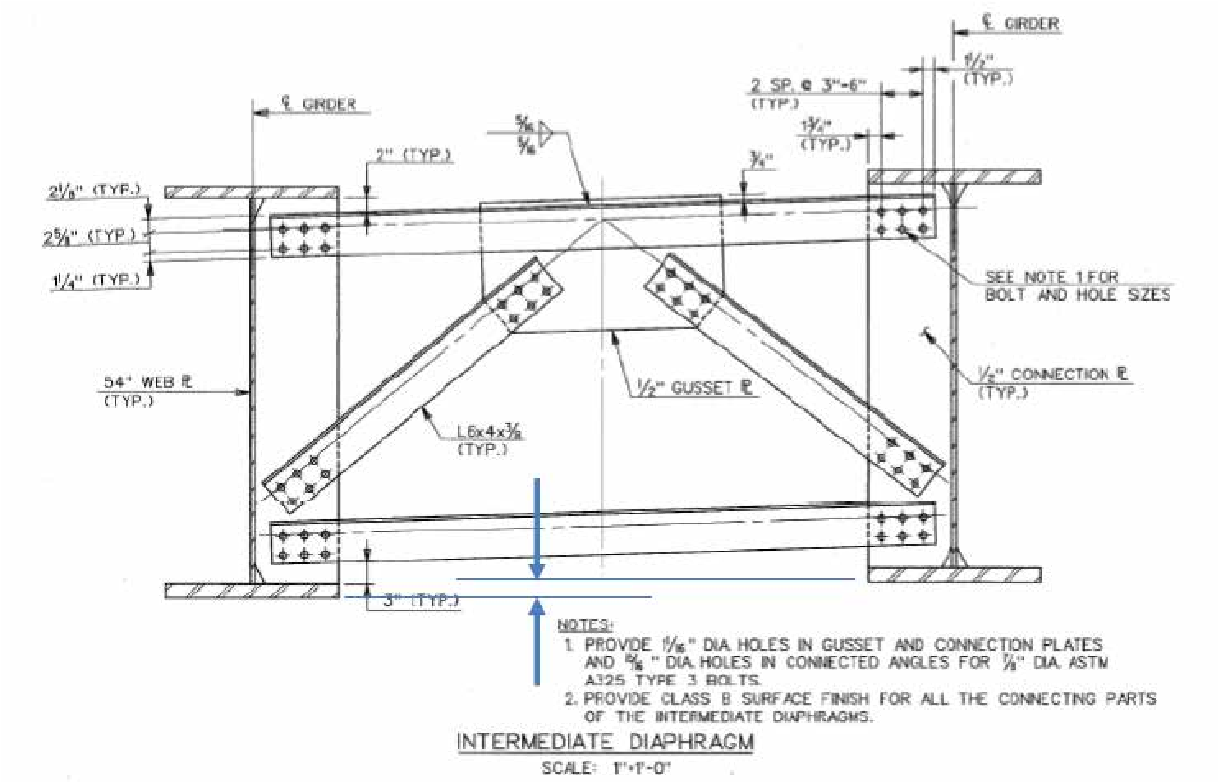


Figure 1 – Crossframe Drop on Bridge No. 4 of Route 3 at the Passaic River Crossing

Crossframes detailed for TDLF are fabricated to fit under total dead load; in other words, the crossframe drop is based upon the differential girder deflection at total dead load. Prior to installation of the crossframes, the webs will theoretically be plumb. However, since the girders are only subject to steel dead load at the time of crossframe installation, crossframes detailed for TDLF will not fit without applied force. The erector must twist the girders to allow the crossframes to fit and be installed. Since I-girders are weak in torsional resistance, the necessary applied force is usually relatively small and manageable. After application of this fitting force, the girders will no longer be plumb, nor will the crossframes be stress-free. However, with the further application of dead load, in the final constructed position at total dead load, the webs will theoretically be plumb and the crossframes theoretically stress-free.

Web plumbness in the final constructed position is contingent upon cambering the webs appropriately, detailing the crossframes for TDLF and applying a fitting force during erection. For straight girders of moderate skew (in other words, where $I_s \leq 0.30$), this force fitting is not typically difficult. For severely skewed girders (in other words, where $I_s > 0.30$), this force fitting may require greater force and be less than desirable as the skew index increases. Detailing the crossframes for steel dead-load fit (SDLF) allows less difficult erection, but the girder webs layover and are not plumb in the final constructed position.

No published literature or reported experience of bridge owners has been uncovered revealing a reduction in strength or serviceability due to expected out-of-plumbness from appropriate crossframe detailing for SDLF.

Standard-Size Bolt Holes

Article 6.13.1, General, of the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2014) specifies that:

> Unless otherwise permitted by the contract documents, standard-size bolt holes shall be used in connections in horizontally curved bridges.

The commentary to this article explains that:

> Standard-size bolt holes in connections in horizontally curved bridges ensure that the steel fits together in the field.

The same principle holds true for straight, skewed bridges, where differential deflection from girder to girder is analogous to horizontally curved girders. The use of oversize or slotted holes to minimize force fitting results in the loss of overall geometric control of skewed and/or horizontally curved steel bridges. The Route 3 Bridge was constructed using oversize holes in the diaphragm connection plates. The variability of the layovers measured at SDL and TDL at the bearing lines of the Route 3 Bridge (shown in Section 5) indicate the possible side effects of the use of oversize holes and finger tight bolts. The ability to provide an accurate estimation of layover at different conditions is difficult when oversize holes are used to minimize the forces associated with fitting the girders and crossframes together. Further, all bolts should be fully tightened prior to deck placement to maintain the specified geometry.

Specified Crossframe Fit

Article 6.7.2, Dead Load Camber, of the *LRFD Specifications* (AASHTO, 2014) specifies that:

> For straight skewed I-girder bridges and horizontally curved I-girder bridges with or without skewed supports, the contract documents should clearly state a theoretical erected web position of the girders and the crossframe fit condition under which that position is to be theoretically achieved.

The commentary to this article suggests two options for the theoretical erected web position: either plumb or out of plumb, and three options for the crossframe fit condition: either no load fit (NLF), steel dead load fit (SDLF) or total dead-load fit (TDLF). The *LRFD Specifications* provide no further guidance.

Table 1 summarizes the recommended crossframe fit conditions from the NSBA technical subcommittee report on Skewed and/or Curved Steel I-Girder Bridge Fit (NSBA, 2014). Crossframes detailed for NLF are not appropriate for skewed girders. Further, the report specifically recommended avoiding TDLF for bridges with a skew index, I_s, in excess of 0.30.

Table 1 – Recommended Crossframe Fit

Bridge Configuration	Crossframe Fit
straight, moderately skewed bridges ($I_s \leq 0.30$)	TDLF
straight, severely skewed bridges ($I_s > 0.30$)	SDLF

These recommendations suggest that for moderately skewed bridges, just as non-skewed bridges, webs can be theoretically plumb in the final constructed position. However, for severely skewed bridges, out-of-plumb webs consistent with SDLF should be tolerated for the sake of constructability.

According to the NSBA draft report on girder fit (NSBA 2014), the layover at the bearing lines can be predicted using the primary bending rotation of the girder due to dead load and bridge geometry. The estimated SDL and TDL cambers from analysis are typically similar and the crossframe drop at the bearing lines for Route 3 is small, the predicted layovers at the bearing lines for each condition should be similar. The SDL layover measurements indicate a variability of approximately 0.42 inches at the East bearing line and 0.3 at the west bearing line. At the TDL condition, the variability at the East bearing line is 0.17 inches and the West bearing line is 0.23 inches.

The Route 3 Bridge was detailed for the webs to be plumb at TDL. AASHTO/NSBA *Steel Bridge Erection Guide Specification* (AASHTO/NSBA, 2007) suggests a tolerance of ±⅛ inch x (web depth, in feet) When applying this tolerance to the layover measurements made at TDL, a number of the girders exceed the specified tolerance for web plumb. This may be due to the use of oversize holes, finger tight bolts until after the deck pour, the concrete pouring sequence, or other unidentified behaviors. For broad implementation of the web plumb tolerance, it is necessary for the geometry at SDL and TDL to be predictable. Predicting layover at different stages of construction assumes no additional deflection due to oversize holes and finger tight bolts. Unless these mechanisms can be included in the layover prediction, the use of a strict tolerance to judge the acceptability of out of plumb webs at the intended fit condition is not practical.

Design Analysis

The need for refined analysis for skewed and/or curved bridges was investigated in NCHRP Report 725 (White, et al., 2012). Table 2, adapted from this report and based upon comparison to 3-D finite-element analysis, suggests that only marginally more accurate design results can be determined from a 2-D grid model for force effects traditionally calculated for a straight girder. Crossframe forces and flange lateral bending stresses, not traditionally calculated for straight girders, are not calculated in a 1-D line-girder analysis. Thus, 1-D line-girder analysis is appropriate for the proportion and detailing of straight, skewed girders.

Table 2 – Rating of Analysis Methods

Force Effect	Geometry	Worst		Mode	
		2-D	1-D	2-D	1-D
major axis bending stresses	$I_s < 0.30$	B	B	A	A
	$0.30 \leq Is < 0.65$	B	C	B	B
	$I_s \geq 0.65$	D	D	C	C
vertical displacements	$I_s < 0.30$	B	A	A	A
	$0.30 \leq Is < 0.65$	B	B	A	B
	$I_s \geq 0.65$	D	D	C	C
girder layover at bearings	$I_s < 0.30$	B	A	A	A
	$0.30 \leq Is < 0.65$	B	B	A	B
	$I_s \geq 0.65$	D	D	C	C

Table 3 – Legend for Table 2

Grade	Normalized Mean Error
A	≤ 6%
B	7 – 12 %
C	13 - 20%
D	21 – 30%
F	> 30%

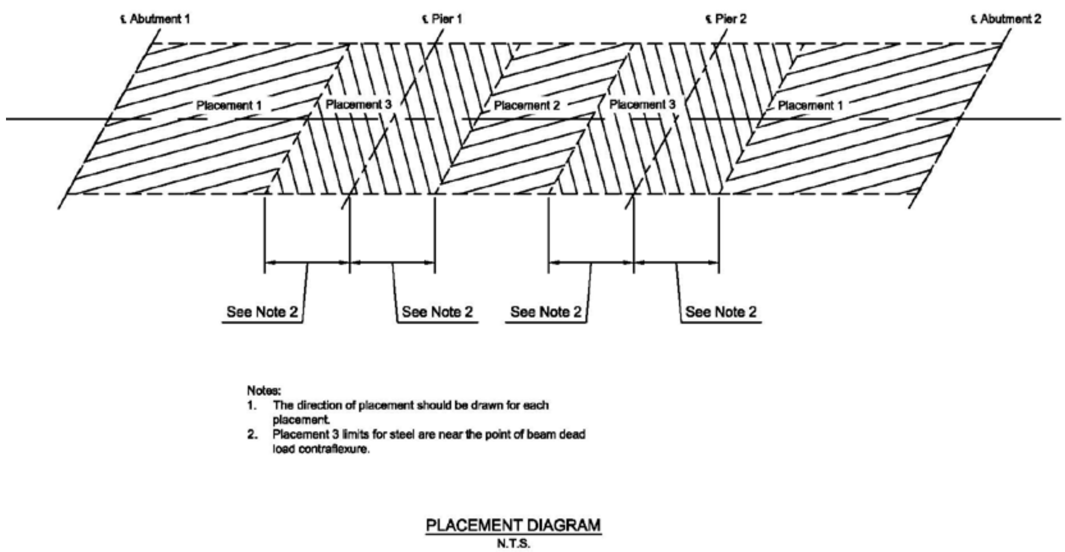
While a 1-D line-girder analysis is sufficient to design the girder, it cannot estimate the actual web position. If the out-of-plumbness at the final constructed position of girders with crossframes detailed for SLDF is desired, a 2-D grid analysis is necessary. Therefore, for straight, skewed girders with a skew index, I_s, greatly in excess of 0.30, a 2-D grid analysis is warranted to anticipate the final out-of-plumbness and determine whether it is acceptable.

The 2-D analysis of the effects of steel dead load only; in other words, the situation after erection but before the deck is placed; the designer can investigate the need for tie-downs or bearing blocking. Further, more appropriate loads and movements imposed upon the bearings can be determined for each construction condition and the final condition.

Modjeski and Masters is currently completing an FHWA project to develop a manual for the application of refined-analysis for bridges and the associated additions and revision to the *LRFD Specifications* to fully cover refined analysis. The products which should be useful for skewed bridges should be available within a year.

Proposed deck pouring sequence

The proposed or envisioned concrete deck pouring sequence directly affects the final position of the entire superstructure post-curing. Skewed steel girder bridges may require special deck placement sequences (see placement diagram). See recommendations and guidance Section 6 for more information related to deck pouring sequence.



3. Fabrication

Bolt Holes for Crossframe-to-Girder Connections

Items 2 and 3 of Article 24.9, Diaphragms and Crossframes, of the NJDOT *Design Manual for Bridges and Structures* (NJDOT, 2009) state:

2. The structural steel layout should be examined to determine if the location of relatively stiff intermediate diaphragms placed normal to the stringers introduce detrimental stresses in diaphragms and stringers due to twisting. If this condition exists, the spacing of the diaphragms should be staggered.

Also, the following note should be included on the plans:

> "Intermediate diaphragm connections to stringers shall be limited to finger tight bolts in oversized holes until the dead loads are in place. The bolts shall be tightened after the deck is in place."

3. Generally, the above note should be provided on final plans for most structural steel erection applications. Especially, final plans that are for those projects where stage construction is involved in the construction process.

The first note under the intermediate crossframe detail on the diaphragm details sheet of the contract drawings for Bridge No. 4 of Route 3 at the Passaic River Crossing, shown below, illustrates this provision in practice.

NOTES:
1. PROVIDE 1 1/16" DIA. HOLES IN GUSSET AND CONNECTION PLATES AND 15/16" DIA. HOLES IN CONNECTED ANGLES FOR 7/8" DIA. ASTM A325 TYPE 3 BOLTS.

Standard holes for 7/8 inch diameter bolts are 15/16 inches in diameter as indicated for the holes in connected angles. The 1-1/16 inch diameter holes for the connection plates and gusset are oversized.

When crossframe connections are fabricated with oversize holes, geometric control is lost during erection as discussed previously. Web plumbness and bearing locations cannot be assured. This was evident in the variability in web layover of the Route 3 Bridge at the SDL stage. This illustrates a possible effect of oversize holes.

Tightening bolts prior to pouring the concrete deck is critical to maintain geometric control. Article 11.6.5 of the AASHTO *LRFD Bridge Construction Specifications* requires that "splices and field connections shall have one-half of the holes filled with bolts and cylindrical erection pins (held bolts and half pins) before installing and tightening the balance of the high-strength bolts." All of this assumes standard holes.

Detailing and Fabricating Crossframes

The bridge geometry defined by the crossframe fit is specified on the contract drawings by the designer as per Article 6.7.2 of the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2014), as discussed previously.

Fabricators will detail the crossframes using the cambers calculated by the designer and shown on the contract drawings. The total dead-load camber is used for TDLF; the steel dead-load camber is used for SDLF. Under TDLF, the fabricator should care not to include future wearing surface as part of the cambers.

4. Erection

Prior to erection, the contractor should submit an erection plan and procedures prepared by a registered professional engineer to the owner for review. The AASHTO/NSBA *Steel Bridge Erection Guide Specification* (AASHTO/NSBA, 2007) suggests minimum requirements for steel girder erection procedures.

Girders with crossframes detailed for TDLF (in other words, moderately skewed girders) will be theoretically plumb in the final constructed position (under total dead load), but these girders will not be plumb after erection. Conversely, girders with crossframes detailed for SDLF (in other words, severely skewed girders) will be theoretically plumb after erection (under steel dead load only), but not in the final constructed position.

In our non-deterministic world, nothing is absolutely straight or absolutely plumb. As such a reasonable tolerance must be considered when defining plumbness. Article 9.2.2, Deviation from theoretical erected web position, of the AASHTO/NSBA *Steel Bridge Erection Guide Specification* (AASHTO/NSBA, 2007) suggests a tolerance of ±⅛ inch x (web depth, in feet). Specifying this tolerance at the theoretically plumb web conditions indicated above provides a reasonable field check on the design and erection.

Table 4, an extension of Table 1, indicates when web plumbness within the specified tolerance can be anticipated.

Table 4– Anticipated Web Position

Bridge Configuration	Crossframe Fit	Web Position
straight, moderately skewed bridges ($I_s \leq 0.30$)	TDLF	plumb in final constructed position
straight, severely skewed bridges ($I_s > 0.30$)	SDLF	plumb after erection

The skew index for Route 3 was 0.39 for stage 2 and was detailed for TDLF. Based on the NSBA report, this structure may have been better suited for SDLF. In addition a refined analysis may have been warranted regarding the deck pour and whether the sequence would allow the webs to rotate back to plumb.

5. Structural Response modeling, instrumentation, analysis and observations: Route 3 over NJ Transit

Introduction

The scope of this activity included modeling and instrumentation of a significantly skewed bridge to assist NJDOT in developing state specific methods and procedures for the construction of severely skewed bridge superstructures. Intelligent Infrastructure Systems (IIS) was able to perform this work on the ROUTE 3 Passaic River Crossing Bridge over NJ Transit with the assistance of both NJDOT, the bridge designer, and the contractor constructing the bridge.

Description of Route 3 Bridge

A skewed bridge was selected by NJDOT for modeling and instrumentation to provide quantitative information regarding the stresses and girder layover during different phases of construction. The RT3 Bridge is a 178 foot simple span bridge constructed to carry NJ Route 3 over NJ Transit rail lines near Rutherford, NJ. The bridge is characterized by a large camber and vertical curve due to the approach profiles and the required clearance over the rail lines. The deck is supported on 54 inch deep plate girders with two field splices. The girders were constructed using both 50 ksi and 70 ksi high performance unpainted weathering steel (ASTM A709 Grades HPS50W and HPS70W) since the required clearance did not allow for deeper girders that would make use of lower strength steel. The girders are supported on multi-rotational pot bearings (shown in Figure 3) at each end that allow different translations and rotations depending upon which girder they support. The skew index of the second construction stage of the RT3 Bridge is 0.39 which classifies it as a severely skewed bridge and makes it a candidate for refined analysis and monitoring.

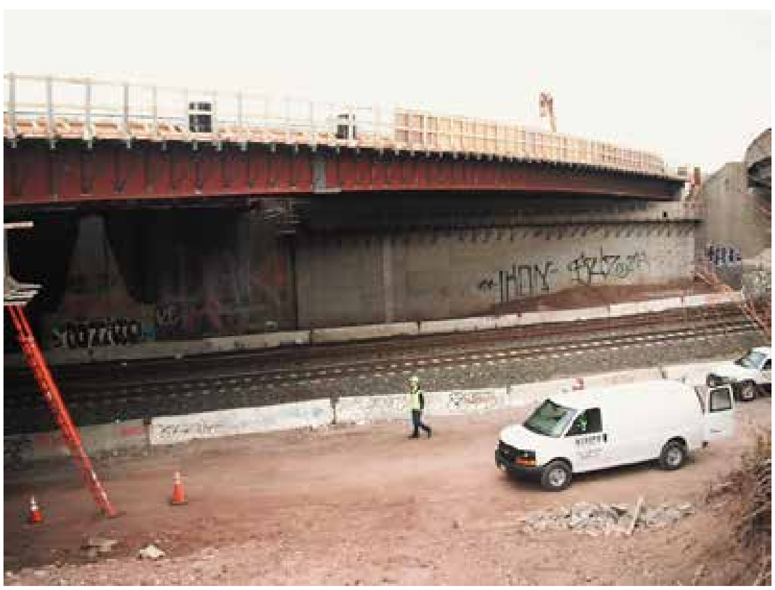


Figure 2: ROUTE 3 Passaic River Crossing

Figure 3: Multi-rotational Pot Bearing

Finite Element Modeling

IIS developed a 3D a priori finite element (FE) model based upon the information obtained from the designer and contractor. The FE model was developed using the Strand 7 simulation software due to its ability to communicate with the powerful computational software, MATLAB, which will be used to carry out the subsequent sensitivity and response prediction analyses. The a priori model was utilized to better understand the forces, rotations, and displacements developed during the construction of a skewed bridge superstructure. The scope of the model included modeling the superstructure of the bridge only. The model was terminated at the girder support bearings which omits the influence of the substructure elements on the response of the structure. Since the response of the superstructure due to dead loads was of importance, omitting the substructure elements was a determined to be a reasonable assumption.

Geometry

The 3D finite element model of the Route 3 Bridge was developed using the geometry detailed in the contract drawings supplied by the designer. The undeformed geometry was first developed in AutoCAD software and then imported into the Strand7 simulation software for further development of the geometry, element mesh, and material and section properties. Figure 4 shows the overall geometry of the 3D FE model developed for the Route 3 Bridge. The camber of each girder was modeled during development of the undeformed geometry in AutoCAD.

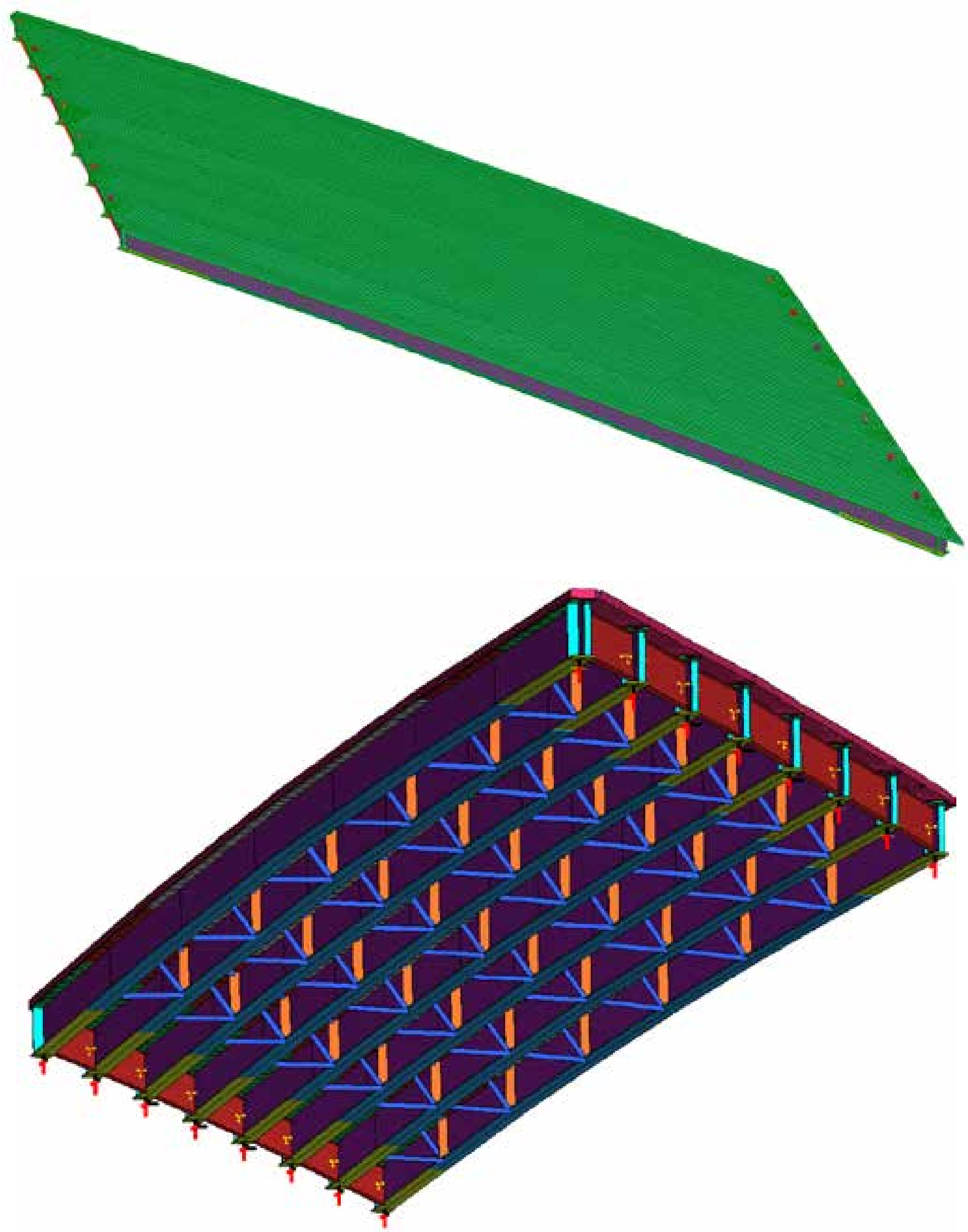


Figure 4: Overall Geometry of ROUTE 3 Passaic River Crossing FE Model

Element Types

The eight steel girders comprising the superstructure of the second stage of the Route 3 Bridge were modeled using shell elements (Quad4 Strand7 notation) for the flanges, webs, and stiffeners/cross frame connection plates. A view of the shell element girder construction is shown in Figure 5. Modeling the girders using shell elements allows for the prediction of lateral flange bending stresses and warping stresses from torsion of the girders. Standard 1D beam elements are not capable of modeling these behaviors, however a hybrid modeling of the girder is possible where the webs are modeled using shell elements while the flanges are modeled using beam elements. This modeling approach allows for prediction of lateral flange bending stresses while also accounting for warping stresses.

The FE model utilized six degree of freedom beam elements (Beam2 Strand7 notation) to represent the bearing line cross frames and cross frames (shown in Figure 5) between the girders. The beam elements were drawn at their neutral axis locations and rotated about their principal axes so the elements were displayed properly when viewed in extruded view. The deck was represented with shell elements (Quad4 Strand7 notation), which were drawn along curvature of the top flange of the beams and offset to 6.5 inches above the girder top flange. The deck was specified as 9 inches deep and therefore the 6.5 inch offset represents half the deck thickness plus a 2 inch haunch. The beam elements were discretized in a manner to both minimize computation time, and provide accurate representation of the physical structure. The maximum shell element size was roughly 1' x 1' while the beam elements ranged in overall length based upon their structural system. Beam elements generally had a discretization of one element per member.

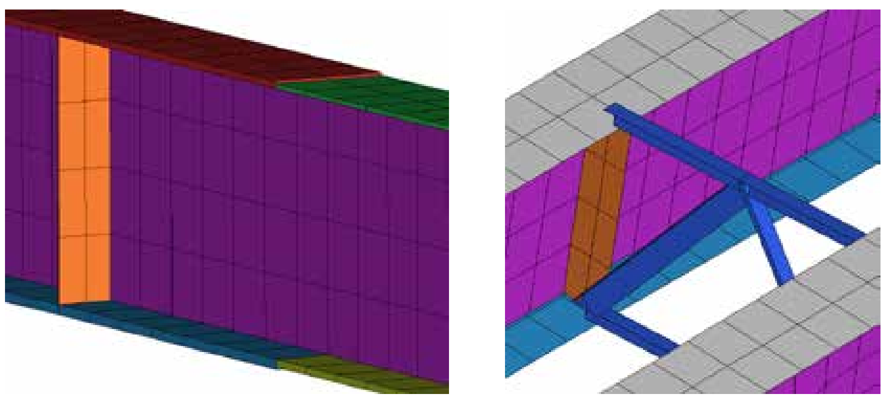


Figure 5: Elements used in ROUTE 3 FE Model

Boundary and Compatibility Conditions

The bounds of the model were located at the girder bearing locations at the east and west abutments. High load multi-rotational pot bearings were used to support the girders of the ROUTE 3 Bridge and each bearing was modeled according to the allowed translations/rotations specified in the contract drawings. The bearings were specified as follows and the type of bearing at each girder of ROUTE 3 stage two is shown in Figure 6 and Figure 7:

F – Fixed bearing (longitudinal and transverse translation restrained)

FVR – Fixed bearing (vertical restrained)

E – Expansion

***EGT* –** Expansion Guided Transverse (Longitudinal translation restrained)

***EGL* –** Expansion Guided Longitudinal (Transverse translation restrained)

***EGVTR* –** Expansion Guided Transverse Vertically Restrained (Longitudinal and Vertical Translation Restrained)

***EVR* –** Expansion Vertically Restrained

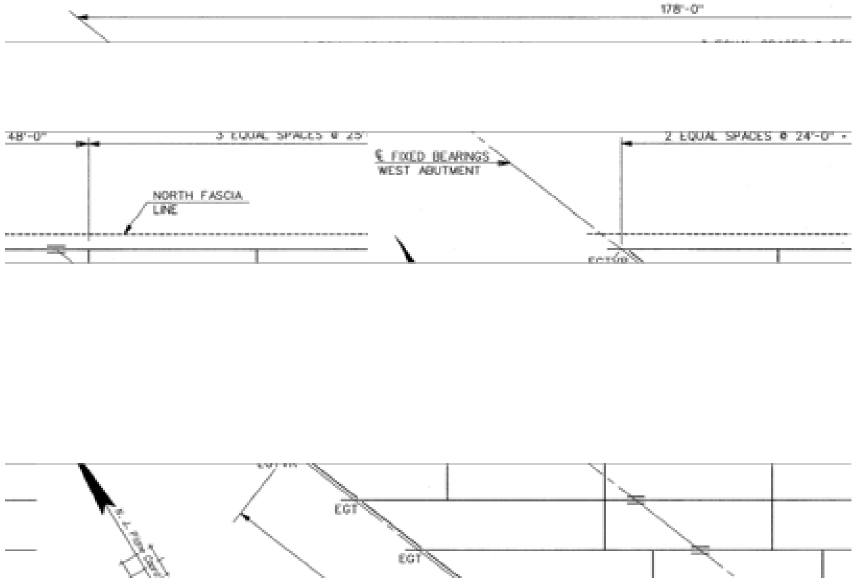


Figure 6: ROUTE 3 West Bearing Conditions

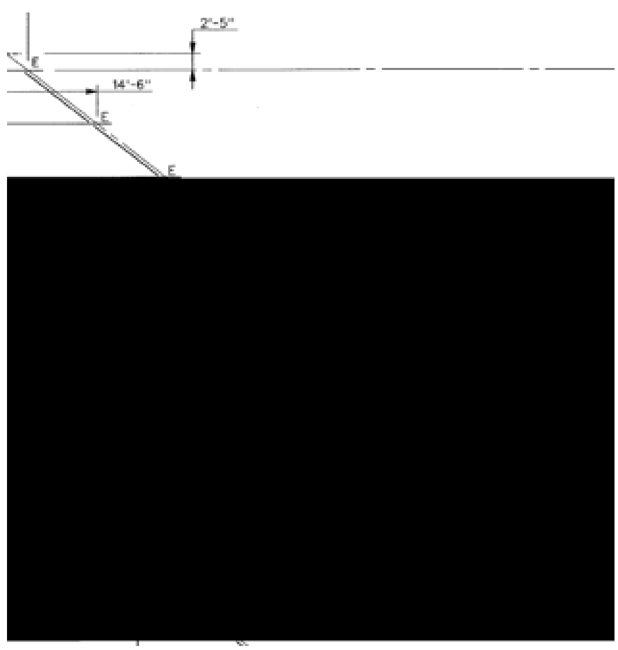


Figure 7: ROUTE 3 East Bearing Conditions

The bearing conditions shown in the drawings were replicated in the model by applying appropriate translational restraint to the model. The translational restraint was offset from the bottom flange of the girders to the bearing elevations specified in the contract drawings. The FE model was truncated at the bearing locations as shown in Figure 8.

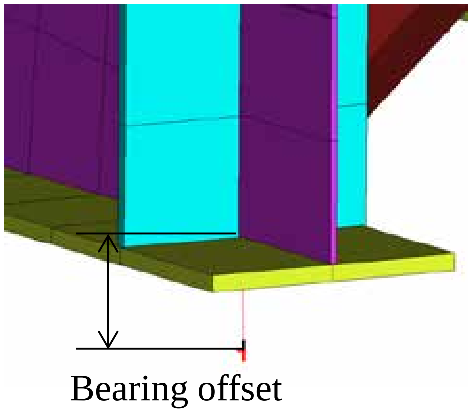


Figure 8: FE Model Bearing Offset

Member Cross Sections

The girders comprising the ROUTE 3 bridge superstructure have variable flange thicknesses along the girder length. These flange thickness transitions were modeled using different thickness shell elements as shown in Figure 5. The flanges were then offset half of their thickness to remove the overlap of the flange elements with the web elements.

Cross sections for beam element members were defined directly in Strand7 based upon information gathered from the design drawings. The cross sections of the beam elements are generated by selecting the proper section from the Strand7 cross section library. The library has a wide variety of standard steel shapes and also has the ability to develop custom cross sections. Once the geometry, cross sections, and material properties were developed and imported into the model, an error screening procedure was carried out to ensure the model was reasonable. This error screening procedure included the construction of a separate structural model where the girders were modeled as beam elements and the deck was modeled using shell elements. This reduced level model was used as a comparison for the responses predicted by the full 3D model and also as a model with which to implement different staged construction schemes without expending computational resources on the full 3D model.

Staged Construction

Using the nonlinear static solver within the Strand7 software, the girder erection procedure submitted by the contractor was replicated. Girder 1 was lifted into place and braced against the existing structure ROUTE 3 bridge structure. Once the girder was braced, girder 2 was lifted into place and the cross frames installed between the girders. The steel erection continued in this fashion until all girders were in place and all cross frames installed. At this point the bridge is considered in its steel dead load configuration. This process is shown graphically in Figure 9 through Figure 12. Field specific measures, such as the blocking of bearings, used to erect the bridge were not included in the modeling of the structure.

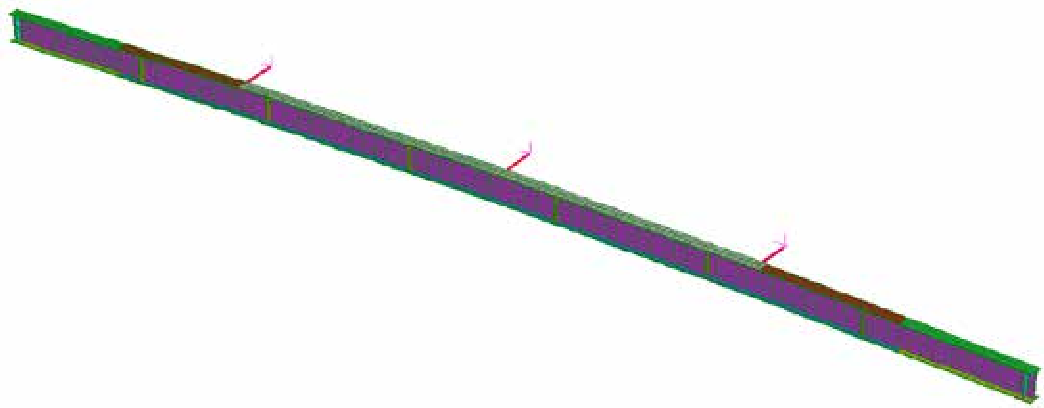


Figure 9: Construction Sequence – Stage 1 Braced To Existing Structure

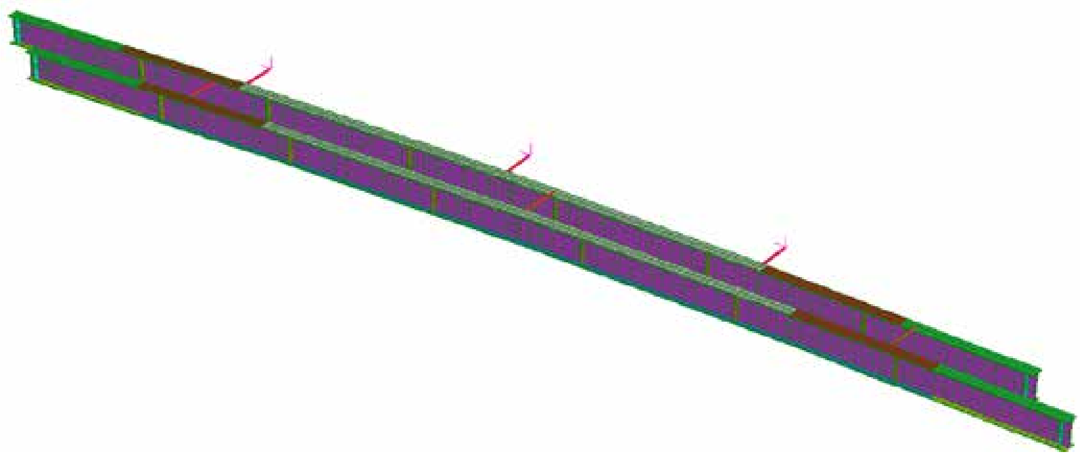


Figure 10: Construction Sequence – Stage 2

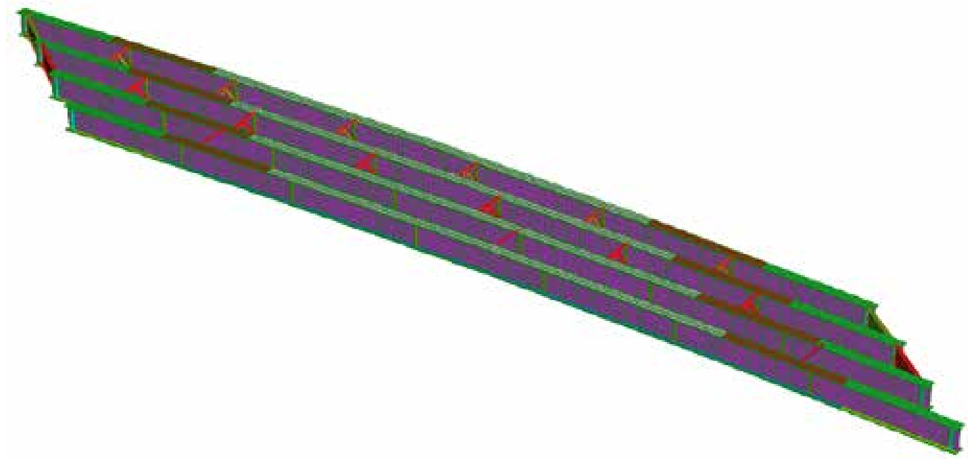


Figure 11: Construction Sequence – Stage 4

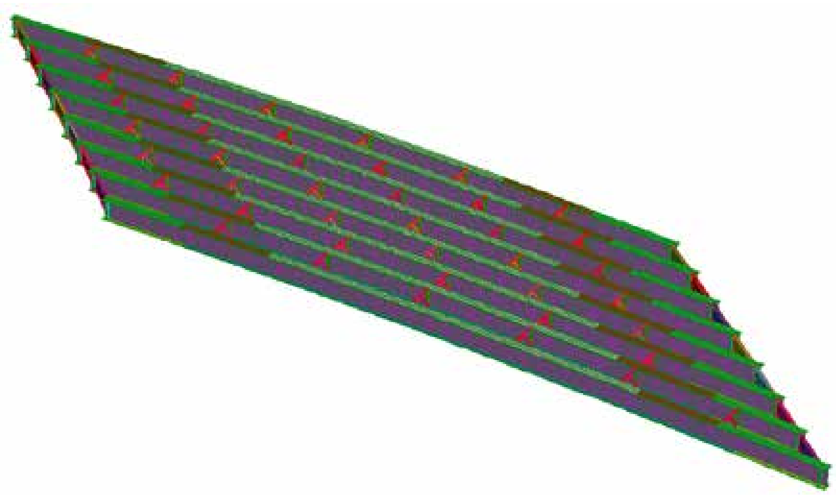


Figure 12: Construction Sequence – Stage 9 – Steel Dead Load

Sensitivity Analysis

A sensitivity analysis was conducted to identify the model parameters that directly influence the model responses of interest (girder strains, girder layover). During the sensitivity analysis these parameters were varied between their identified bounds to quantify the effect on the girder stresses. Since the bounds of the support conditions were assumed to be varied between free and fixed, a translation from qualitative values to quantitative values was required. The parameters were translated in the model as normalized stiffness values that were varied between 0 and 1. An example of a sensitive model parameters is given below to demonstrate the type of data generated during a simple sensitivity analysis.

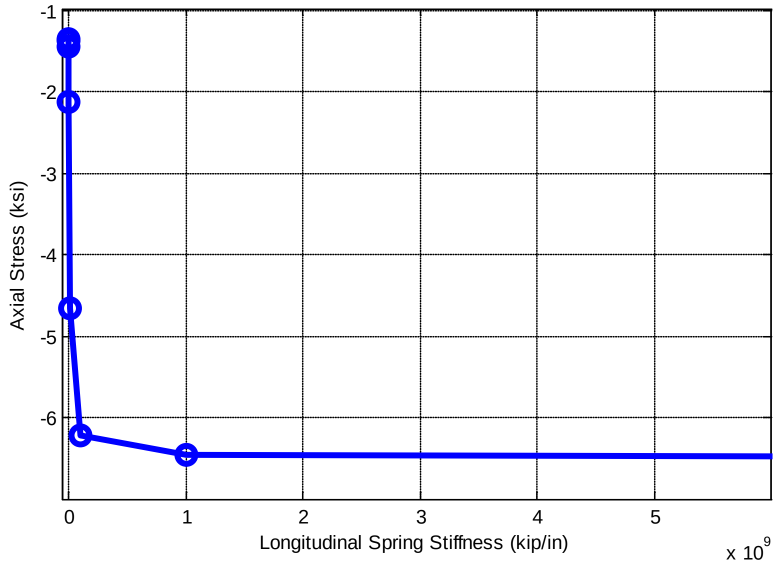


Figure 13: Longitudinal Support Stiffness vs. Axial Stress

Figure 13 shows the variation of axial stress in a typical girder as a function of the longitudinal spring stiffness. As the longitudinal restraint is increased the compressive axial stress in the girders increases. This effect would also be shown in measured data if a bearing that was free to move longitudinally was fixed during steel erection. This fixity would result in a buildup of compressive axial stress in the girders.

Instrumentation

Upon completion of the development of the finite element model, an instrumentation and monitoring program was developed to capture several key responses during the erection of the ROUTE 3 superstructure. The development of the monitoring program was driven by the following questions

- What are the primary bending stresses in the girders during erection?
- What are the flange bending stresses in the girders during erection?
- What is the layover of the girders during erection?

With these questions in mind, the following instrumentation layout for the bridge was developed (Figure 14). To measure strains in the girders over the long term, vibrating wire strain gages were chosen for their long term stability. The VW strain gages were installed on the girders before splicing and erection in order to capture the strains developed in the girders due to these operations. The girders arrived on site in three sections and at the time of installation, the 44.5' end sections were available for instrumentation. The 92' center sections arrived hours before being lifted into place and therefore instrumentation was not included on the center sections. Girders 1 and 2 were instrumented with four strain gages at approximately quarter span and three quarter span (measured from the east abutment). Girders 4, 5, 6, 7, 8 were instrumented with four strain gages each at approximately quarter span (measured from the west abutment). In addition to the strain gage measurements on the girders, girder web rotation measurements were made at the bearings before and after the deck pour to capture the layover of the girder at steel dead load and total dead load (minus superimposed dead load).

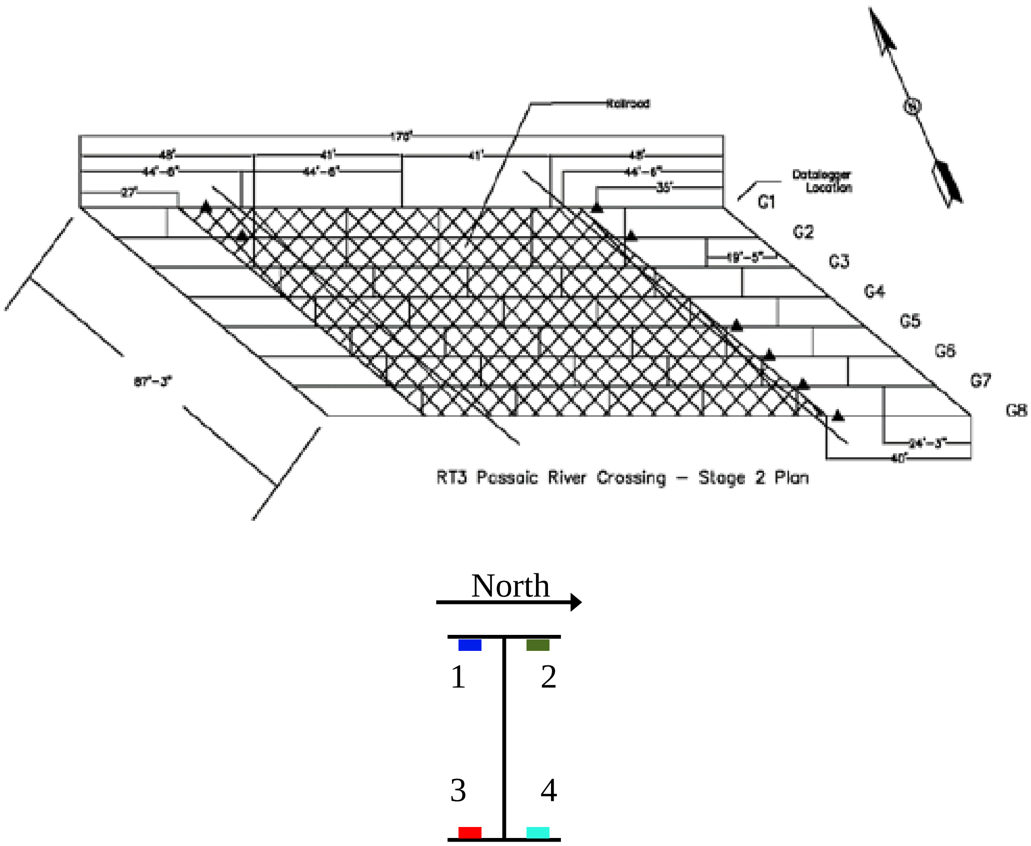


Figure 14: Strain Gage Instrumentation

Figure 15: Strain Gage Installation

The strain gages installed on the girders (triangles shown in Figure 14) were connected to a remote data acquisition system (shown in Figure 16) and monitored for a period of 160 days and the data presented in the next section details the results of the monitoring and instrumentation.

Figure 16: Remote Data Acquisition System

Data Interpretation, Results, and Observations

Following the conclusion of instrumentation and monitoring, there was a large database of measurements from the 32 sensors to reduce into usable data for comparison with the finite element model predictions. The first task carried out in data reduction was error screening. In this phase, the data was screened for anomalies or blatant errors which could be attributed to dead sensors or significant noise. Initially, two gages were damaged early in the monitoring and several others did not function properly after approximately 50 days. Those gages that ceased functioning after 50 days did contain data in the first 50 days that was useful in the data interpretation. Since the objectives of this project were to look at the stresses developed during erection of the bridge, only the first 40 days of data are presented in this report. The remaining data is archived and may be leveraged in subsequent years of the BRP.

Girder Stresses

Following the installation of the strain gages on the girders prior to steel erection, a set of readings were taken from each gage that served as a zero point for all future measurements. An example of the zeroed strain data for the gages located at the ¾ point of girder 1 is shown in Figure 17.

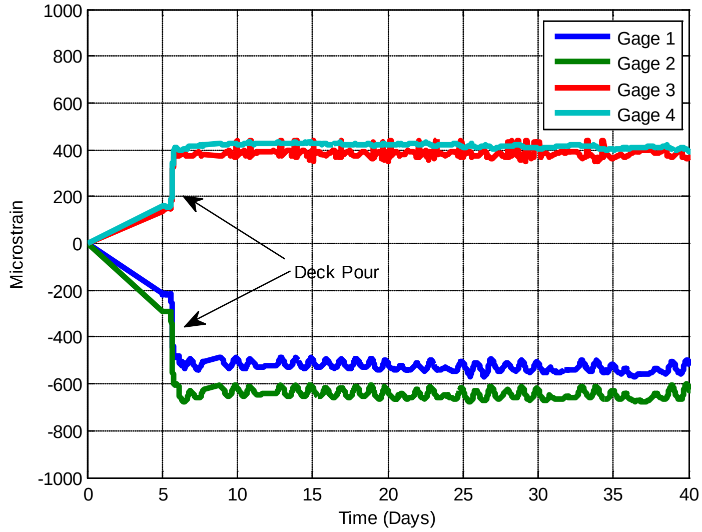


Figure 17: Example Strain Readings from Vibrating Wire Strain Gages

From each set of four gages, axial, primary, and secondary bending strains were calculated. For Girder 8 an example of these strains versus time is given in Figure 17. The measured strains were then converted to stresses to compare with the a-priori model. A tabulation of the axial, in plane bending, out of plane bending, and the top/bottom flange bending stresses following the deck pour is shown in Table 5.

Table 5: Field Measured Girder Stresses at Final Condition from ROUTE 3 Bridge Monitoring

	Field Measured Stresses				
	Axial (ksi)	In plane Bending (ksi)	Out of plane Bending (ksi)	Top Flange Bending (ksi)	Bottom Flange Bending (ksi)
G8 East	-2.1	12.5	0.7	1.5	-0.8
G7 East	-5.9	15.0	-1.2	-1.0	-0.2
G6 East	-3.2	13.7	-2.3	-2.5	0.2
G5 East	-1.6	12.2	-0.9	-0.4	-0.5
G2 East	-6.8	15.6		-0.7	
G1 East	-4.7	17.4			0.2
G2 West	-3.5	16.1	4.6	3.1	1.5
G1 West	-2.2	13.6	-1.2	-1.7	0.5

Girder Layover

The layover or the relative displacement between the top flange and bottom flange of the girder at the bearing lines was measured before and after the concrete deck pour to provide an indication of how much layover was in each girder at the steel dead load condition and at the final condition. Table 6 provides a summary of the girder layover measurements taken at steel dead load and also the final total dead load condition

Table 6: Field Measured Girder Layovers

	East - inches		**West - inches**	
	Steel DL	**Total DL**	**Steel DL**	**Total DL**
Girder 1	2.07	0.59	-1.50	-0.12
Girder 2	0.94	0.52	-1.59	-0.70
Girder 3	0.91	0.62	-1.16	-0.15
Girder 4	1.38	0.50	-1.40	-0.64
Girder 5	1.90	1.00	-1.14	-0.32
Girder 6	1.73	0.63	-0.81	-0.53
Girder 7	1.30	0.71	-1.19	-0.53
Girder 8	1.41	0.87	-0.81	-0.14
Average	**1.46**	**0.68**	**-1.20**	**-0.39**

Positive layovers shown in Table 6 indicate the girders rotated north while the negative layovers at the west bearing line indicate those girder ends rotated south. For the girder web depth of 54 inches, the specified AASHTO/NSBA developed plumb tolerance at the final condition is equal to 0.5625 inches. At the final condition for the Route 3 Bridge, the majority of west bearing line rotations fall under the specified tolerance while the majority of the east bearing line rotations exceed the plumb tolerance at the final condition. One explanation for this is the concrete pour started at the west end and proceeded linearly across the structure to the east end. There was a period where the pour was stopped due to an accident on Route 3 that prohibited concrete trucks from reaching the site. It is well known that concrete can gain strength quickly and set within hours. Therefore, it is possible the concrete deck began setting which restrained the girders towards the west end, reducing the ability of the east ends of the girders to rotate fully into the final plumb condition.

A second observation from the measured layovers is the variability of the measurements for each girder. Based on geometry and in plane bending rotations, the layover at the bearing line locations should be similar. However, there is variability in the actual layovers observed in the field. Two possible explanations for this phenomenon the use of oversize holes in the cross frames and the concrete pouring sequence. Due to different bearing elevations and the skew angle, there exists cross frame drop between adjacent girders resulting from the differential deflection of the girders. The cross frames are detailed for the cross frame drop at either the steel dead load stage (SDLF) or total dead load stage (TDLF). The ROUTE 3 Bridge was detailed for

TDLF and thus the cross frames would require applied force to fit at the steel dead load stage since the cross frames were detailed to fit without force at the total dead load stage. Since the cross frames require force to fit they have tendency to spring back after they are connected causing the girders to twist. This is due to the girders having less torsional rigidity than the cross frames which have large in plane stiffness and are resistant to racking. From the ROUTE 3 plans, it shows that the cross frame gusset plates have oversized holes and with oversized holes it is difficult to maintain geometry during construction. Therefore, it is difficult to expect consistency in girder layover.

The second possible explanation for the variability in girder layovers is the concrete pouring sequence utilized on the ROUTE 3 bridge structure. Previous research has indicated the importance of pouring and finishing the concrete deck aligned with the skew angle rather than perpendicular to the girders. Placing the concrete aligned with the skew angle provides a more even dead load distribution to the girders than placing the concrete perpendicular to the girders. Placing the concrete aligned with the skew angle will allow the girders of a skew structure to evenly rotate towards plumb. It is suggested that similar concrete pouring practices are put into use for skew bridges in New Jersey.

It should be noted that under actual conditions, girder webs will not be exactly plumb in the final condition. Therefore, the use of a plumb tolerance is appropriate. It is suggested as a check of the erection and construction, a tolerance is specified for the deviation from theoretical plumb such as the one provided by Article 9.2.2, Deviation from theoretical erected web position, of the AASHTO/NSBA Steel Bridge Erection Guide Specification (AASHTO/NSBA, 2007). Using a tolerance allows for field engineers to evaluate the deviation from theoretical plumb throughout construction. Since adjustments can be made only before the concrete is poured it is suggested the layover measurements be made after steel erection and excessive violation of the plumb tolerance be evaluated and rectified before the concrete deck is poured.

Comparison of A-Priori Model and Field Measured Responses

Girder Stresses and Layover

The error screened a-priori model was used to predict the stresses and girder layover at the instrumented locations. These values are compared with those measured in the field to determine the ability of an a-priori model to accurately predict the field measurements. A tabulation of the predicted stresses is shown in Table 7.

Table 7: ROUTE 3 FE Model Predicted Girder Stresses at Final Condition

	Model – Predicted Stresses				
	Axial (ksi)	In plane Bending (ksi)	Out of plane Bending (ksi)	Top Flange Bending (ksi)	Bottom Flange Bending (ksi)
G8 East	-1.3	12.7	0.2	-0.5	0.1
G7 East	-1.1	12.9	0.3	0.2	-0.8

G6 East	-1.1	13.0	0.4	0.7	-1.4
G5 East	-1.1	13.0	0.4	0.9	-1.7
G2 East	-1.2	13.2	0.4	-0.7	-0.2
G1 East	-0.4	14.1	-0.4	1.0	-0.2
G2 West	-1.3	13.8	-0.1	0.3	0.0
G1 West	-1.1	13.7	0.0	0.3	-0.2

The tabulated results show that at the instrumented locations, the model predicts the primary bending stresses dominate the stress state of the girders. The axial, out of plane, and flange bending stresses at the instrumented locations are predicted to be low. A visualization of the girder stresses is shown in Figure 18 which shows the maximum compressive longitudinal stress in the girders is 27 ksi near the midspan of the girder while the maximum tensile stress is 26 ksi.

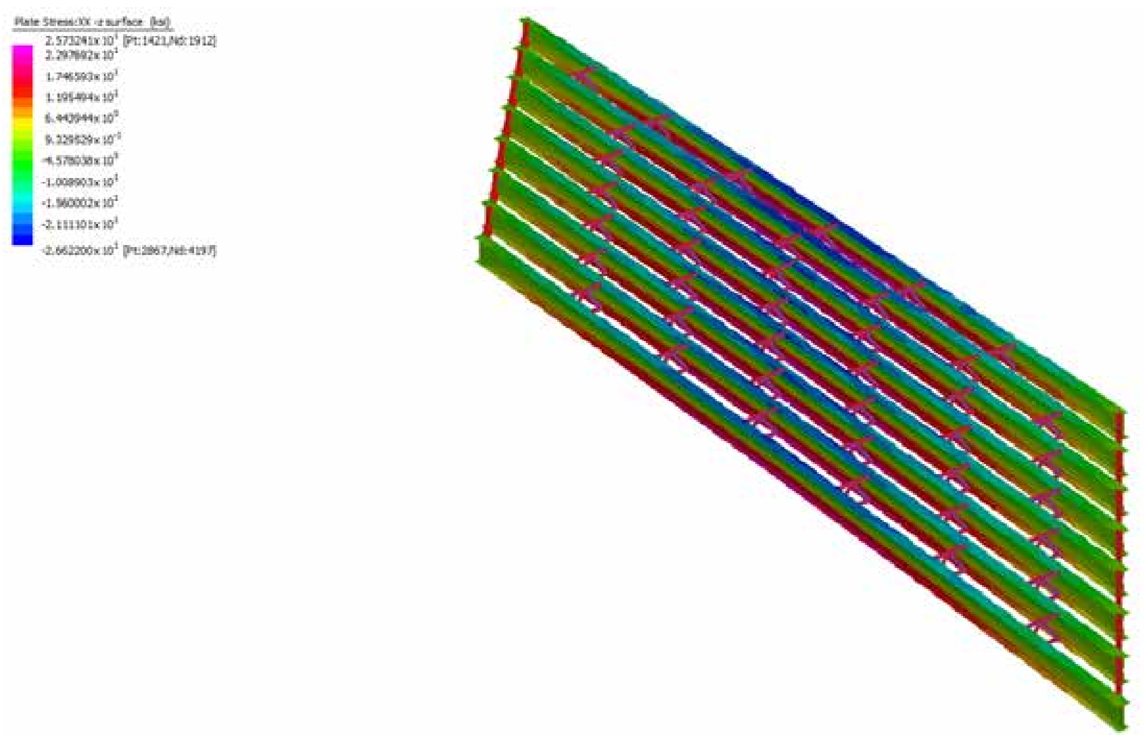


Figure 18: ROUTE 3 FE Model Longitudinal Stresses at Final Condition

A comparison of the a-priori model predicted and field measured stresses at the instrumented locations is given in Table 8. The a-priori model is able to predict the field measured primary bending stresses with less than 20% error while the model has difficulty predicting the axial, out of plane, and flange bending stresses. The predicted flange bending stresses are small which exacerbates the error calculations. It should be noted that the flange bending and out of plane stresses are a fraction of the total stress state in the girders and do not appear to be significant or detrimental to the strength and serviceability of the bridge. The axial stress in the girders are a significant portion of the total stress in the girders which the model does not predict. Since the predicted axial stresses are compressive, it is possible the longitudinal restraint at the bearings is more than what was provided in the model. The sensitivity analysis performed on the model shows the effect of the longitudinal girder restraint on the girder axial stress.

Table 8: Comparison of Model Predicted and Field Measured Girder Stresses

Stresses - Model/Experiment Error - Absolute					
	Axial (%)	**In plane Bending (%)**	**Out of plane Bending (%)**	**TF Bending (%)**	**BF Bending (%)**
G8 East	38.9%	1.5%	66.3%	134.1%	106.3%
G7 East	82.1%	14.1%	125.2%	119.7%	359.8%
G6 East	65.5%	5.7%	115.1%	125.9%	816.2%
G5 East	30.7%	6.3%	142.4%	295.4%	225.1%
G2 East	82.8%	15.4%		12.1%	
G1 East	90.9%	18.9%			194.0%
G2 West	62.4%	14.3%	102.7%	91.9%	100.0%
G1 West	49.8%	1.1%	97.9%	115.0%	143.1%
Average	**62.9%**	**9.7%**	**108.3%**	**127.7%**	**277.8%**

The tabulated results show that at the instrumented locations, the model predicts the primary bending stresses dominate the stress state of the girders. The axial, out of plane, and flange bending stresses at the instrumented locations are predicted to be low. The percent errors between the model and field for the flange bending and out of plane responses are large since the magnitudes of these stresses are low which amplifies the difference between the values.

One key observation regarding the top flange bending stresses is the stresses generated in the top flange near cross frame locations are characterized by a behavior where the stresses are amplified in the top flange at cross frame connections and are lower at the opposite side of the flange from the cross frames. The staggering of cross frames results in this behavior which is shown graphically in Figure 19.

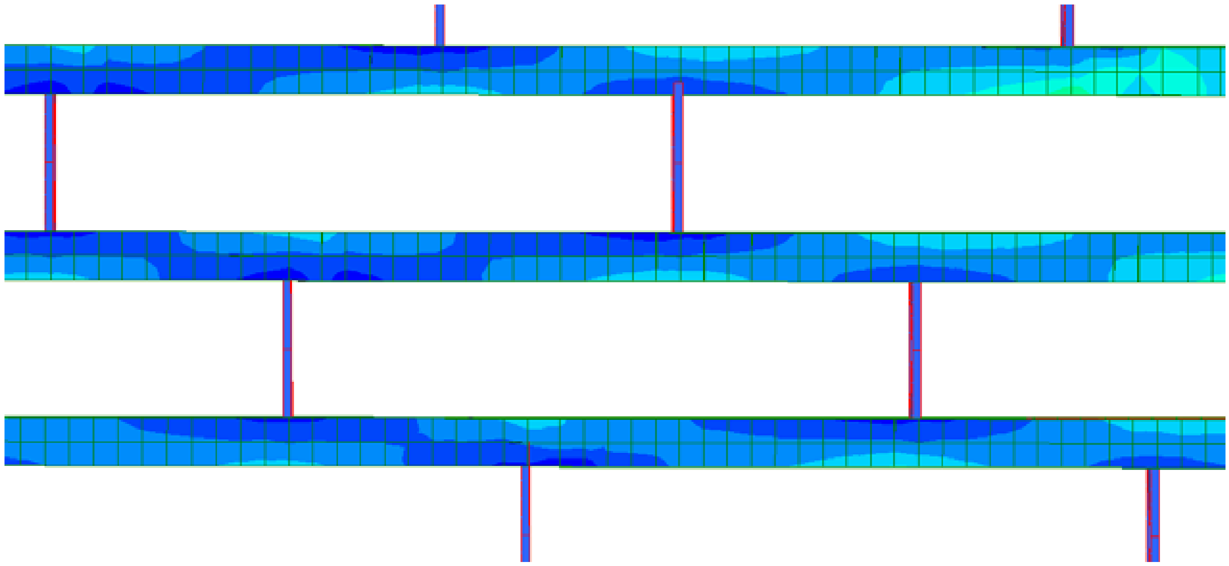


Figure 19: ROUTE 3 FE Model Predicted Top Flange Stresses

The 3D FE model was also used to predict the girder layover at the bearing locations. The model predicted layovers are compared with those measured in the field to determine the ability of an a-priori model to accurately predict the layover at the bearing locations. A tabulation of the 3D model predicted layovers is shown in Table 9 while a magnified visualization of the girder layovers is shown in Figure 20.

Table 9: Comparison of Layovers – Steel Dead Load

	East (inches)			West (inches)		
	Experiment	**Model**	**% Absolute Error**	**Experiment**	**Model**	**% Absolute Error**
Girder 1	2.07	1.41	31.52%	-1.50	-1.43	4.40%
Girder 2	0.94	1.33	-41.01%	-1.59	-1.38	13.62%
Girder 3	0.91	1.29	-41.25%	-1.16	-1.32	-13.83%
Girder 4	1.38	1.26	8.22%	-1.40	-1.28	8.11%
Girder 5	1.90	1.26	33.35%	-1.14	-1.26	-10.75%
Girder 6	1.73	1.26	27.19%	-0.81	-1.29	-59.32%
Girder 7	1.30	1.28	1.45%	-1.19	-1.26	-6.35%
Girder 8	1.41	1.37	3.33%	-0.81	-1.23	-52.34%
Average	**1.46**	**1.31**	**23.42%**	**-1.20**	**-1.31**	**21.09%**

The a-priori model is able to predict the girder layovers at the bearing locations with an average error of 25%. The a-priori model does not take into account specific actions taken in the field during the erection procedure such as blocking of bearings or other methods used to fit cross frames together that could affect the layover at the steel dead load condition. The a-priori model also does not take into the account the movement that could occur during erection due to the use of oversize holes. Therefore, it is reasonable to expect some error in the model predictions versus the measured as is field layovers.

From the model predicted layovers it can be seen that the predicted values are similar for each girder and have a variability of less than 0.1 inches. Whereas the field measured layovers exhibit significant variability of 0.42 inches at the east girder ends and 0.3 inches at the west end at the steel dead load condition. At the final dead load condition the field measured layovers at the east and west ends exhibit slightly less variability at 0.17 and 0.24 inches respectively.

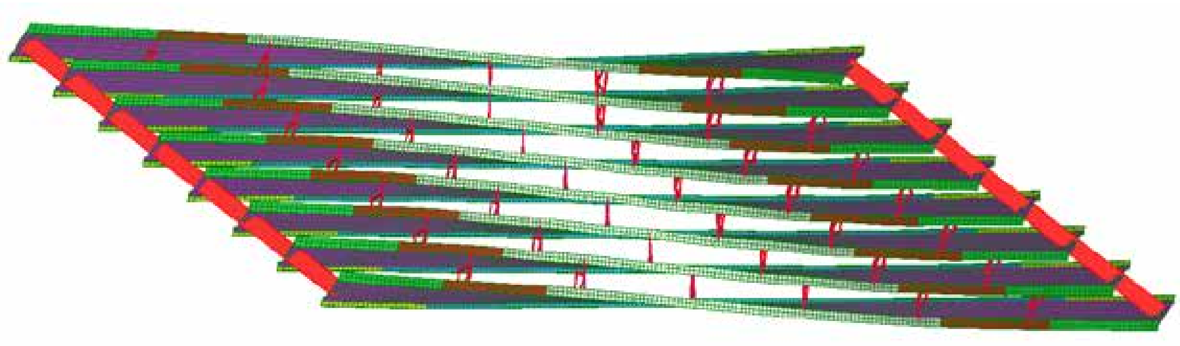


Figure 20: Plan View of Girder Layovers from ROUTE 3 Bridge Model (Rotations magnified for display)

6. Recommendations and guidance

Conclusions and Recommendations from Modeling and Instrumentation

The recommendations based on modeling and instrumentation are drawn from a single structure and may not be indicative of the results and conclusions for all structures. The main conclusions drawn from the instrumentation and monitoring of the Route 3 bridge are as follows:

- It is recommended that the skew index as described in NCHRP 725 is adopted for determining the skewness of a structure and that the skew index be used to select the appropriate detailing method. The NSBA subcommittee report on Skewed and/or Curved Steel I-Girder Bridge Fit recommends Total Dead Load Fit (TDLF) detailing for skewness indices less than or equal to 0.3 and Steel Dead Load Fit (SDLF) detailing for bridges with skewness indices greater than 0.3.
- The concrete pouring sequence should be carefully evaluated prior to the pour in order to develop a sequence that permits even distribution of dead load from the wet concrete to the girders.
    - It is suggested for simple span skewed bridges such as Route 3, the concrete be placed and finished along the skew angle rather than perpendicular to the girders

to promote a more even distribution of dead load due to the wet concrete as shown in Figure 21. An even distribution of dead load will allow the girders to rotate back to plumb in a more even fashion since the concrete supported by each girder will be similar. As shown in the qualitative example in Figure 21, placing the wet concrete perpendicular to the girders loads the girders near the acute corner of the skew early on in the placing procedure while the girders near the obtuse corner are not loaded until later in the pouring sequence. By placing the concrete aligned with the skew angle, the girders are evenly loaded throughout the pouring sequence if a linear pouring sequence is selected.

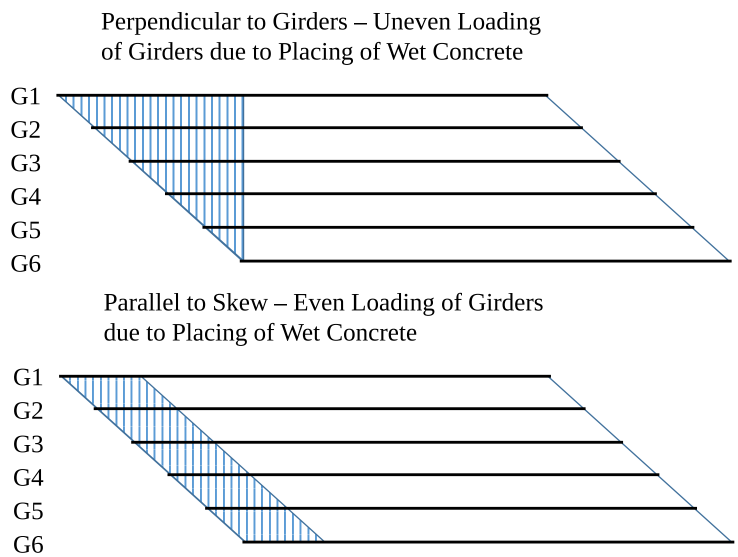


Figure 21: Qualitative Comparison of Dead Load Distribution due to Concrete Placing Procedure

- From the data garnered from the Route 3 Bridge monitoring, girders that are not plumb at the final condition do not appear to significantly impact the strength of the system, rather future investigations should focus on whether girders that are not plumb have a correlation with identified performance or serviceability issues. There has not been published documentation of a strength or performance issues due to girders that are not plumb.
- Compressive axial stress in the girders at the measured locations is approximately 60% larger than predicted by the a-priori model. An explanation for this may be that the bearings were blocked during girder erection which induced more compressive stress into

the girders due to the restraint. The bending and axial stresses of the girders are sensitive to the selection of the longitudinal boundary condition and therefore it is feasible that additional restraint at the boundary during erection resulted in an increase in the compressive stress in the girders.

  - It may be necessary for the fabricator/erector/contractor to perform an analysis of their erection procedures to ensure the erection methods are not introducing detrimental stresses, rotations, or deflections in the girders or cross frames.
  - It is also recommended that the actual erection procedures used in the field be documented for future reference and used in interpreting measured and/or model predicted data.

- It is suggested that a tolerance for evaluating deviation of girder webs from theoretical plumb be adopted in accordance with Article 9.2.2, Deviation from theoretical erected web position, of the AASHTO/NSBA Steel Bridge Erection Guide Specification (AASHTO/NSBA, 2007).
  - Using a tolerance for evaluating the plumb of girders webs is contingent upon the use of non-oversized holes in cross frames and cross frame connection plates. Oversized holes can allow movement during erection that will allow unintended deviation from the theoretical erected position. Therefore, as mentioned elsewhere in this report it is recommended that oversize holes are not used and all bolts are tightened to specified torque before the concrete deck is poured.

Excessive violation of the tolerance should be addressed at the steel dead load stage of construction when adjustments to the girder layover can be made with relative ease.

Appendix A – Straight, Skew Bridge Recommendations

Recommendations for revisions and additions to the NJDOT Design Manual for Bridges and Structures and the NJDOT Standard Specifications for Road and Bridge Construction are suggested regarding analysis, crossframe fit, girder plumbness and standard holes.

Analysis

Recommendation

Revise Article 3.2, Vehicular Bridge Structures, of the NJDOT Design Manual for Bridges and Structures as follows:

Section 4 - Structural Analysis and Evaluation

4.6.2 Approximate Methods of Analysis

4.6.2.2 Beam-Slab Bridges

4.6.2.2.1 Application

Add the following paragraph to the beginning of the Article:

Design all straight bridges with a skew index, I_s, less than or equal to 0.30 using the provisions of this Article. The skew index, I_s, is defined as:

$$I_s = \frac{w_g}{L} \tan q$$

where

w_g = width of steel framing between fascia girders (ft)

q = largest skew angle (radians)

L = average span length within a single span (ft)

Design all straight bridges with a skew index, I_s, greater than 0.30 using the provisions of Article 4.6.3.3.1.

C4.6.2.2.1

Add the following paragraph to the beginning of the commentary:

NCHRP Report 725 (White, et al., 2012) suggests that only marginally more accurate results than those from an appropriate analysis can be determined from a refined 2-D grid model for the proportioning and detailing of straight, skewed girders. However, out-of-plane distortions of interest in severely skewed bridges cannot be modeled with a distribution factor based approximate analysis.

4.6.3 Refined Methods of Analysis

4.6.3.3 Beam-Slab Bridges

4.6.3.3.1 General

Add the following paragraph to the end of the Article:

Indicate back-calculated distribution factors for moment and shear for each girder in the cross section on the contract drawings based upon the response to live load determined by the refined analysis.

4.9 References

Add the following reference:

White, D. W., D. Coletti, B, W, Chavel, A, Sanchez, C. Ozgur, J. M. J. Chong, R. T. Leon, R. D. Medlock, R.A. Cisneros, T. V. Galambos, J. M. Yadlosky, W. J. Gatti, and G. T. Kowatch. 2012. *Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges*, NCHRP Report 725, Transportation Research Board, National Research Council, Washington, DC.

Crossframe Fit

Recommendation

Revise Article 3.2, Vehicular Bridge Structures, of the NJDOT Design Manual for Bridges and Structures as follows:

Section 6 – Steel Structures

6.7.2 Dead Load Camber

The first sentence of the sixth paragraph is changed to:

For horizontally curved I-girder bridges with or without skewed supports, the contract documents should clearly state a theoretical erected web position of the girders and the crossframe fit condition under which that position is to be theoretically achieved.

Add the following to the end of the Article:

Design straight skewed I-girder bridges assuming the crossframe fit, either total dead load fit (TDLF) or steel dead load fit (SDLF) in accordance with Table 6.7.2-1 below:

Table 6.7.2-2 – Specified Crossframe Fit

Bridge Configuration	Crossframe Fit
straight, moderately skewed bridges ($I_s \leq 0.30$)	TDLF
straight, severely skewed bridges ($I_s > 0.30$)	SDLF

(adapted from NSBA technical subcommittee report on Skewed and/or Curved Steel I-Girder Bridge Fit)

where skew index, Is, is defined in Article 4.6.2.2.1.

For straight, skewed bridges, state the specified crossframe fit on the contract drawings.

For severely skewed girders with a specified steel dead load fit (SDLF), the theoretical web position under steel dead load only should be plumb. Under total dead load, the theoretical web position will not be plumb. Indicate the calculated theoretical out-of-plumbness at each crossframe location in the final constructed position on the contract drawings. Also, indicate the assumed bearing conditions; in other words, tie-downs or blocked bearings required to limit bearing loads or movements during erection; for the analysis under steel dead load only

For moderately skewed girders with a specified total dead load fit (TDLF), the theoretical web position under total dead load should be plumb.

Girder Plumbness

Recommendation

Revise Article 506.03.01, Structural Steel, of the NJDOT Standard Specifications for Road and Bridge Construction as follows:

506 – STRUCTURAL STEEL

506.03 CONSTRUCTION

506.03.01 Structural Steel

Add the following to the end of the Sub article B, Erection Plan:

8. Procedures to verify the erected web position of straight, skewed girders.

Add the following to the end of Sub article D, Erecting:

For straight, skewed girders, ensure the appropriate erected web position at crossframe locations throughout the length of the girder. Webs of girders with a specified steel dead load fit (SDLF) should be at the theoretical out-of-plumbness indicated on the contract drawings within a tolerance of ±1/8 inch × (web depth, in feet) in the final constructed position. Webs of girders with a specified total dead load fit (TDLF) should be plumb within a tolerance of ±1/8 inch × (web depth, in feet) in the final constructed position.

Standard Holes

Recommendation

Revise Article 3.2, Vehicular Bridge Structures, of the NJDOT Design Manual for Bridges and Structures as follows:

Section 6 – Steel Structures

6.13 Connections and splices

6.13.1 General

The sixth paragraph is changed to:

Unless otherwise permitted by the contract documents, standard-size bolt holes shall be used in connections in horizontally curved and straight, skewed bridges.

C6.13.1

The third paragraph is changed to:

Standard-size bolt holes in connections in horizontally curved and straight, skewed bridges ensure that the steel fits together in the field.

Article 24.9, Diaphragms and Crossframes, of the NJDOT Design Manual for Bridges and Structures is changed to:

Section 24 - Structural Steel

24.9 Diaphragms and Crossframes

1. The criteria of Subsection 6.7.4 – Diaphragms and Cross frames of the LRFD

Specifications and Section 3 of this Manual shall be followed in analyzing the need for their provision.

2. The structural steel layout should be examined to determine if the location of relatively stiff intermediate diaphragms placed normal to the stringers introduce detrimental stresses in diaphragms and stringers due to twisting. Detrimental stresses derived from 2-D refined analysis can be defined as elevated crossframe forces and /or flange lateral bending moments more than about 25% of those estimated through 1-D line-girder analysis. If this condition is anticipated, the staggering of the diaphragms should be considered.

Also, the following note should be included on the plans:

“Intermediate diaphragm connections to stringers shall be limited to-standard holes. The bolts shall be tightened prior to deck placement.”

3. Generally, the above note should be provided on final plans for all structural steel erection applications. For those projects where stage construction is involved in the construction process, oversized holes may be considered for the connections between stages only.

Appendix B – Checklist for the review of a contractor's erection plan and procedures submittal

Contract No.	Date
Bridge No.	Engineer

STRAIGHT, SKEWED STEEL-BRIDGE ERECTION PLAN AND PROCEDURES

Are the following items properly included in the erection plan and procedures submittal?	**yes**	**no**	**n/a**
Drawings supporting the plan and procedures			
Calculations supporting the plan and procedures			
PE seal			
Are the following items properly included on the plan of work area?	**yes**	**no**	**n/a**
Permanent and temporary structures			
All roads, railroads, waterways, clearances, utilities, potential conflicts			
Materials storage area			
Are the following items properly included in the erection sequence?	**yes**	**no**	**n/a**
Step-by-step procedure figures and accompanying narrative			
Component delivery location			
Crane locations			
Temporary support, hold cranes, blocking, tie-downs			
Load restrictions at certain stages (for example, wind)			
Are the following items properly included in the crane specification?	**yes**	**no**	**n/a**
Type, pick radii, boom length			
Approximate crane pick points			
Crane pick weights			
Hold crane loads			
Are the details for the following items included?	**yes**	**no**	**n/a**
Rigging (beam clamps, lifting lugs, etc.) and lifting devices (spreader and lifting beams, etc.)			
Bolting requirements (see Article 11.6.5 of the AASHTO *LRFD Bridge Construction Specifications*)			
Bearing blocking and tie-downs			
Load Restrictions (such as wind velocity and construction loads)			
Temporary supports			
Jacking devices			

Appendix C – Review of the State of Practice for Highly Skewed Bridges in Ohio

Introduction

The state of practice for highly skewed bridges was reviewed through studying the April 24, 2007 PowerPoint© presentation of the same name by the Ohio Department of Transportation (ODOT) and Burgess & Niple, Inc.[1] and the 2007 edition of the ODOT Bridge Design Manual (BDM).[2]

The topic of crossframe fit in skewed and/or horizontally curved highway bridges is a fertile topic for research of which much as been performed since the development of the Ohio provisions in 2007.

Characterization of Skew

ODOT classifies highway bridges with skew angles greater than 30° as highly skewed. NCHRP Report 725 demonstrated that the skew angle alone is not the best characterization of the effects of skew.[3] The aspect ratio of the bridge along with the skew angle as represented in the skew index characterizes skew effects better. Therefore, no changes are warranted to the recommendations with regard to the characterization of skew effects in the Synthesis Report on Construction Deformations Severely Skewed Structural Steel Bridge Construction.[4]

Analysis

ODOT requires refined analysis for steel bridges with skew angles greater than 45°. Again as discussed above, the skew index is a better characterization of skew effects. The CAIT Synthesis Report recommends that 2-D-grid refined analysis be performed for skew indices greater than 0.30. No changes are warranted to the recommendations with regard to refined analysis. However, for bridges with a skew angle between 30° and 45°, a refined analysis may be warranted where differential displacements between girders analyzed with a 1-D line-girder analysis is greater than S/100, where S equals girder spacing, according to ODOT practice. The limit of S/100 is an approximation based upon simple geometry of a single bay. The limit seems restrictive as a differential displacement of only 1 ¼ inches where S equals 120 inches or 10 feet would trigger a refined analysis. More research is required before a trigger for refined analysis

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[1] http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Pages/SkewedBridges.aspx

[2] http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/Pages/BDM2007.aspx

[3] White, D. W., D. Coletti, B, W, Chavel, A, Sanchez, C. Ozgur, J. M. J. Chong, R. T. Leon, R. D. Medlock, R.A. Cisneros, T. V. Galambos, J. M. Yadlosky, W. J. Gatti, and G. T. Kowatch, 2012. Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges, NCHRP Report 725, Transportation Research Board, National Research Council, Washington, DC.

[4] Szary, P., D. Mertz, J. Prader, A. Roda and C. Monopolis, 2014. Synthesis Report on Construction Deformations Severely Skewed Structural Steel Bridge Construction, Technical Memorandum, Report No. RU435056-2, Center for Advanced Infrastructure & Transportation (CAIT), Rutgers, the State University, Piscataway, NJ

such as this should be considered. No changes are warranted to the CAIT recommendations as the limit of S/100 is not well researched.

Out-of-Plumbness Tolerance

The limit on out-of-plumbness specified by ODOT is 0.6° or 1/8 inch per foot of web depth. This limit is also that recommended in the CAIT Synthesis Report.

Crossframe Fit

ODOT mandates that crossframes be detailed to fit and that girder webs be plumb under steel dead load. This practice is contrary to the latest research and recommendations from the National Steel Bridge Alliance (NSBA).[5] Further, the ODOT practice results in out-of-plumb girder webs in the final constructed condition. This out-of-plumbness is not necessary for all but the most severely skewed bridges as characterized by the skew index. No changes are warranted to the recommendations with regard to crossframe fit in the CAIT Synthesis Report.

Bolts and Bolt Holes

ODOT mandates standard bolt holes in crossframe connections where slotted holes were previously used, just as the CAIT recommendations. Further, the ODOT practice is to not place the deck until the crossframe connections are fully bolted or welded, again just as the CAIT recommendations. No changes are warranted to the recommendations with regard to bolting practices in the CAIT Synthesis Report.

Increased Superstructure Stiffness to Resist Twist

ODOT allows superstructure stiffening in an attempt to reduce girder twist. The suggested methods include:

- adding steel to the girders, either bigger flanges or deeper webs;
- adding a girder line; or
- using shored construction.

The increase in girder stiffness to resist twist is limited to 25%.

While this ODOT suggestion can result in less girder twist, it does not necessarily represent efficient design. With regard to the first bullet item, open sections such as plate girders do not resist twist well. Wisely, ODOT limits the additional weight of stiffening to 25%. Effective girder stiffening cannot be achieved with only a 25% increase in weight. Even in the ODOT example in their PowerPoint© presentation, a 60% increase in girder weight was required to produce acceptable levels of twist. Such an increase in weight is uneconomical and not within ODOT's specified limit. The suggestion to increase girder stiffness to resist twist but not increase girder weight more than 25% is usually unachievable.

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[5] NSBA, 2014. Skewed and/or Curved Steel I-Girder Bridge Fit, National Steel Bridge Alliance, Chicago, IL.

Finally, adding a girder line or employing shored construction, both represent inefficient steel-girder design especially when the CAIT recommendations will enhance NJDOT practice to the point where girder twist in skewed bridges is manageable.

Summary

Based upon the review of ODOT practice for skewed steel bridges, no changes are warranted to the recommendations with regard to bolting practices in the Synthesis Report on Construction Deformations Severely Skewed Structural Steel Bridge Construction. The recommendations of the synthesis are based upon up-to-date quantifiable research whereas the ODOT practices are based upon observation and good engineering judgment.

Appendix D – References

1. AASHTO, 2014. LRFD Bridge Design Specifications, 7th Edition, American Association of State Highway and Transportation Officials, Washington, DC.
2. AASHTO, 2010. LRFD Bridge Construction Specifications, 3rd Edition, American Association of State Highway and Transportation Officials, Washington, DC.
3. AASHTO/NSBA, 2007. Steel Bridge Erection Guide Specification, American Association of State Highway and Transportation Officials/National Steel Bridge Alliance Steel Bridge Collaboration Document S 10.1 – 2007.
4. NJDOT, 2009. Design Manual for Bridges and Structures, 5th Edition, New Jersey Department of Transportation, Trenton, NJ.
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11. Dykas, J.C. (2012). Behavior During Construction of Ramp B Bridge Over I-40 in Nashville, TN. Georgia Institute of Technology Masters Thesis.
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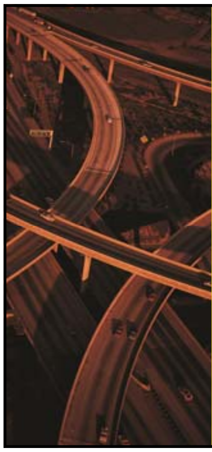
APPENDIX 2F
EVALUATION AND REPORTING ON NEW TECHNOLOGIES



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High Performance Concrete use in New Jersey: Internally Cured Concrete

May 15, 2013

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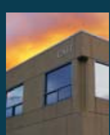

2012 Bridge Resource Program

Overview

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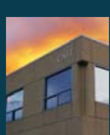
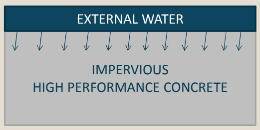


Internal Curing

Overview

Conventional curing

- The goal of conventional curing is to maintain the cement hydrated to prevent chemical (plastic) and autogenous shrinkage
- However, this method may only allow water to penetrate a few millimeters into the concrete.



Example of Chemical Shrinkage (CS)

Hydration of tricalcium silicate

$C_3S + 5.3\ H \rightarrow C_{1.7}SH_4 + 1.3\ CH$

Molar volumes

$71.1 + 95.8 \rightarrow 107.8 + 43$

CS = (150.8 – 166.9) / 166.9 = -0.096 mL/mL or -0.0704 mL/g cement

For each lb (g) of tricalcium silicate that reacts completely, we need to supply 0.07 lb (g) of *extra* curing water to maintain saturated conditions (In 1935, Powers measured a value of 0.053 for 28 d hydration – 75 %)

Slide courtesy of NIST – http://concrete.nist.gov/internalcuring.html

What is internal curing (IC)?

- Traditional concrete/HPC cures from the outside in
- Internal curing is a process that **cures concrete from the inside out**.
- Internal water is generally supplied via internal reservoirs,
  - ***saturated*** lightweight aggregates (LWA),
  - superabsorbent polymers (SAPs)
  - ***saturated*** wood fibers,
  - **saturated** crushed (returned) concrete aggregates (CCA).

Slide courtesy of NIST – http://concrete.nist.gov/internalcuring.html

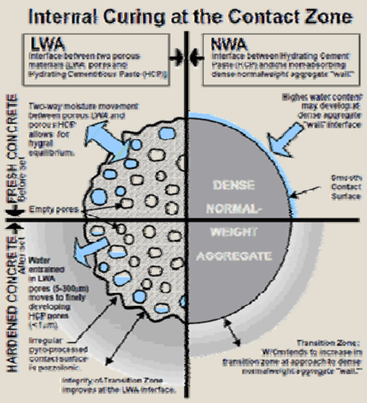
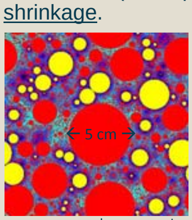


Image courtesy of the Expanded Shale, Clay and Slate Institute (ESCSI)

How does IC work?

- IC distributes the extra curing water (uniformly) throughout the entire concrete element, making it more readily available to keep the cement paste saturated throughout the cure, avoiding self-desiccation (in the paste) and reducing autogenous shrinkage.



Yellow – Saturated LWA
Red – Normal weight sand
Blues – Pastes within various distances of an LWA

Image courtesy of NIST – http://concrete.nist.gov/internalcuring.html

IC performance

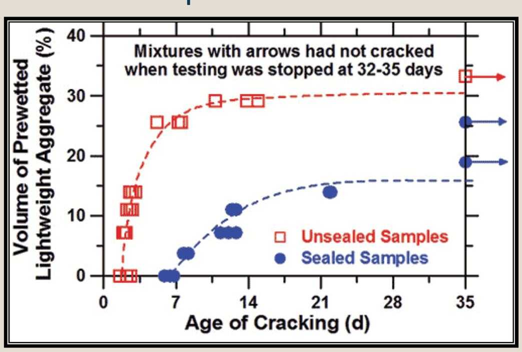


Image courtesy of Structure Magazine– http://www.structuremag.org/article.aspx?articleID=1372

A word on Conventional curing

- IC is NOT a replacement for Conventional Curing.
- Surface evaporation, windy conditions and other adverse environment will affect IC
- Wet burlap, misting, and addition of water should be done as per standard specifications.

LWA proportioning (supply/demand)

$$C_f*CS*\alpha_{max}=S*\Phi_{LWA}*M_{LWA}$$

$$M_{LWA}=\frac{C_f * CS * \alpha_{max}}{S * \Phi_{LWA}}$$

M_{LWA} = mass of (dry) LWA needed per unit volume of concrete (kg/m^3 or lb/yd^3);

C_f = cement factor (content) for concrete mixture (kg/m^3 or lb/yd^3);

CS = chemical shrinkage of cement (mass of water/mass of cement);

α_{max} = maximum expected degree of hydration of cement (0 to 1);

S = degree of saturation of aggregate (0 to 1);

Φ_{LWA} = desorption of lightweight aggregate from saturation down to 93 % RH (mass water/mass dry LWA).

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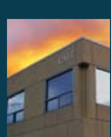

Case Studies

New York State ICHPC Inventory (2011)

Highway	Feature Spanned	Location
NY Route 9W	Vineyard Avenue	Lloyd
NY Route 96	Owego Creek	Owego
Interstate 81S	Tioughnioga River	Whitney Point
Interstate 81N	Tioughnioga River	Whitney Point
Court Street	Interstate 81	Syracuse
Bartell Road	Interstate 81	Cicero
Interstate 86 NY	Route 415	Painted Post
Interstate 84	Route 6	Brewster
Interstate 290	Ramp B Interstate 190	Tonawanda
Interstate 81N	East Hill Road	Lisle
Interstate 81S	East Hill Road	Lisle
NY Route 17 Exit 90 Ramp	East Branch Delaware River	East Branch
NY Route 38B	Crocker Creek	Endicott
NY Route 353	Allegheny River	Salamanca
Interstate 290	Ramp D Interstate 190	Tonawanda
Interstate 87	Route 9 and Trout Brook	Chestertown
Goulds Corners Road	Fort Drum Connector	Watertown

Interstate 190/Interstate 290

Tonawanda, NY

Concrete Batch Designs - Interstate 190/ Interstate 290 Interchange - Buffalo, NY		
	Class HP	Class HP-IC
Cement – Blended with 7% Silica Fume	540	540
Fly Ash – Type F	139	139
Fine Aggregate – Natural Sand	1150	813
Fine Aggregate – LWAF 22.0% moisture	0	244
Coarse Aggregate – No. 1 Stone	674	959
Coarse Aggregate – No. 2 Stone	1038	792
Water	272	273
Air Entrainment - BASF AE-100	16.3	17.7
Water Reducer - BASF 100 Xr	20.4	26.5

Concrete Properties - Interstate 190/ Interstate 290 Interchange - Buffalo, NY		
	Class HP	Class HP-IC
Average 7 day Compressive Strength	3040	3500
Average 28 day Compressive Strength	4677	4683
Average 56 day Compressive Strength	5343	5417
Concrete Density	140.2	135.2
Air Content	5.5%	6%
Slump	5	4.5

Structures constructed in 2010 – inspected 2011 and no cracking was found.

Court Street overpass - Interstate 81

Syracuse, NY

Concrete Batch Designs - Court Street over I-81		
	Class HP	Class HP-IC
Cement – Blended with 7% Silica Fume	540	540
Fly Ash – Type F	135	135
Fine Aggregate – Natural Sand	1130	782
Fine Aggregate – LWAF 22.0% moisture	0	239
Coarse Aggregate – Blended Stone	1720	1720
Water	270	262
Air Entrainment - BASF AE-100		
Water Reducer - BASF 100 Xr		

Concrete Properties - Court Street over I-81		
	Class HP	Class HP-IC
Average 7 day Compressive Strength	4727	4859
Average 14 day Compressive Strength	5917	6222
Average 21 day Compressive Strength	6077	6570
Average 28 day Compressive Strength	6309	6976

Structures constructed in 2009 – inspected 2010 and no cracking was found.

Comparison of HPC and ICHPC (NY State)

New York State HPC and ICHPC mix designs		
	HPC Mix	ICHPC MIX
Cement content (LB/CY)	500	500
Fly Ash Content (LB/CY)	135	135
Microsilica (silica fume) content (LB/CY)	40	40
Sand percent total aggregate (solid volume)	40	28
Lightweight fines percent total aggregate (solid volume) *	0	12
Design Water/Total Cementitious Content	0.4	0.4
Desired Air content	6.5	6.5
Allowable Air Content	5.0 - 8.0	5.0 - 8.0
Desired Slump	4	4
Allowable Slump	3.0 - 5.0	3.0 - 5.0
Type of Coarse Aggregate gradation	CA - 2	CA - 2

*Replaces 30% of N.W. Sand with pre-wetted LWAF (40% x 30% = 12%)

Comparison of HPC and ICHPC (Route 52)

NJDOT HPC and Equivalent ICHPC mix designs		
Mix Design No.	R7410937	Equivalent ICHPC
Cement (lb)	600	600
Pozzolan 2: Fly Ash (lb)	106	100
Total Cement (lb)	706	700
Sand (lb. / S.G. 2.8)	1208	1052
#57 Stone (lb)	1625	1625
LWA (lb)	0	156
Sand percent total aggregate (solid volume)	43%	30%
Lightweight fines percent total aggregate (solid volume) *	0	13%
Water (lb)	263	263
W/CC Ratio	0.37	0.38
Air Content %	6.0	6

*Replaces 30% of N.W. Sand with pre-wetted LWAF (40% x 30% = 12%)

Final Thoughts

- Overall goal to minimize/eliminate cracking
- Internal Curing mentioned in NCHRP Synthesis Report for HPC practices as one solution to early age cracking
- Ballpark estimates - $10-15/cy premium for ICHPC
- QA/QC of pre-wetting LWA is important
- Other thoughts/discussion?

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Thank you!

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APPENDIX 3A
NON-ROUTINE, NON-PLANNED STRUCTURAL MANAGEMENT, MATERIALS & TECHNOLOGY

RU435056-1

State-of-the-Art Practices of Mass Concrete
A Literature Review

Technical Memorandum
August, 2013

Submitted by
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U.S. Department of Transportation
Federal Highway Administration

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16. Abstract

NJDOT requested that Rutgers-CAIT perform a literature review on the state-of-the-art practice of mass concrete and use the findings to compare with the Thermal Control Plan for the Route 7 Wittpenn Bridge Pier 1W cap as well as the current mass concrete specifications included in the NJDOT 2007 Standard Specifications. The review focused on material composition, with description of each component's contribution to heat of hydration. The team observed that the literature focused on two areas of concern, maximum temperature reached during curing and thermal differentials between the core and surface of the mass concrete element.

The literature has extensively documented the urgency of maintaining the maximum curing temperature below 160°F. The adverse effects associated with exceeding the maximum temperature threshold are severe, but not visible for months or years after construction. This threshold should never be exceeded.

The literature also documents damages resulting from exceeding temperature differential thresholds, which are more immediate and can be identified during construction. The thermal-induced cracking that results may be repaired through industry accepted means, from seals, coatings for hairline cracking, to more comprehensive repairs.

During early stages of curing, the concrete has not developed sufficient strength to resist excessive thermal gradients. Thus, form insulation and other methods to protect the concrete surface from dissipating heat greatly or reach excessively high peak temperatures reduces the likelihood of deleterious effects. The results of this literature review suggest that current research and industry agree that temperature thresholds are critical to mass concrete. Proper controls must be established in order to ensure well-performing concrete elements to be constructed.

17. Key Words	18. Distribution Statement
Mass concrete, delayed ettringite formation, thermal cracking, adiabatic temperature rise, thermal gradient	

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Table of Contents

Introduction and Background

The Rutgers Center for Advanced Infrastructure and Transportation (CAIT) Bridge Resource Program (BRP) team has reviewed various technical reports, journals and codes to provide guidance to NJDOT regarding the construction of mass concrete piers associated with the Route 7 Wittpenn Bridge construction project. Concrete performance and quality has been studied extensively in recent years. ACI, AASHTO, NCHRP and other organizations have invested research funding to better understand the use of concrete in structures, including design, mixing, placement, curing and maintenance. This recent focus on understanding concrete design and construction has provided newfound capabilities to address the challenges of improving concrete performance.

Overview of mass concrete

Mass concrete is defined by the American Concrete Institute (ACI) as: *Any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.* The practice dates back to the turn of the 20th century. The technologies employed today provide much greater quality control and capability to predict the material's performance. Recent research[1] has better defined the processes affecting mass concrete and provided guidance in the temperature thresholds that trigger deleterious effects such as Delayed Ettringite Formation (DEF). This study, as well as other state DOTs practice, provides a solid basis to provide recommendations for mass concrete operations in New Jersey.

The objective of using mass concrete is primarily for durability and workability and secondarily for strength. Mix designs and curing practice should be developed to provide the concrete with a suitable environment to develop strength at a controlled pace, thereby maintaining controllable adiabatic temperature increases and protecting the concrete from sharp temperature contrasts between core and surface.

Adiabatic Temperature Rise (ATR) and thermal cracking

In the early phases of curing, it is critical to prevent large temperature contrasts between the core and the surface. The combination of Adiabatic Temperature Rise (ATR) and low thermal conductivity results in high core temperatures. Heat escaping at the surface induces tensile stresses in the concrete. The material properties can resist this tension, but only in a limited capacity. Once tension exceeds the material's capacity to resist, thermal cracking ensues. In New Jersey, other concrete operations have been limited in their allowance for tension in concrete. Thus, the tolerance for this phenomenon should be considered in relation to the DOTs practice to limit tension in concrete elements.

[1]Folliard, K., et al., "Preventing ASR/DEF in New Concrete: Final Report", FHWA/TX-06/0-4085-5, June 2006

Concrete overheating Delayed Ettringite Formation (DEF)

As the exothermic reaction of concrete hydration develops, it is critical to prevent the concrete core from overheating in order to prevent longer-term deleterious effects. During hydration, the cement releases high amounts of heat. In mass concrete, the heat is maintained internally, creating adiabatic temperature increases. As temperatures rise, the chemical reactions in cement change, causing the entrapment of sulfates and aluminates in the cement paste (C-S-H gel). Over time, sulfates (and aluminates) diffuse from the hardened paste and react with monosulfate hydrates to form ettringite, an expansive material that induces stress in concrete, causing cracks. This phenomenon, referred to as Delayed Ettringite Formation or DEF, will continue over the years, reducing the concrete's life. In contrast, when temperatures are controlled, ettringite is allowed to form as part of the early formation of cement, thus accommodating expansion while concrete is still green.

Strength gain is considered secondary in mass concrete, however its development should be considered in relation to short and long-term. In mass concrete, strength develops at lower rates than conventional concrete, but can continue to grow significantly up to one year. The level of strength gain can be between 30% and 200%[2]. During this time, hydration of cement particles continues to churn out the exothermic reaction within the core of the concrete. The rate and magnitude of heat of the concrete depends on the cement mix and pozzolanic content, the compound composition and fineness of cement, the shape of the concrete element and its volume to surface ratio, the initial temperature of the concrete, the ambient temperature and the other surrounding conditions[3]. The time for the core to reach ambient temperature is inversely proportional to the measure of the least dimension in the concrete element. Thus a 6 inch thick element can be thermally stable in a few hours, while a 50-foot thick dam wall would require two years. More common element, such as a 5-foot thick wall or pier cap would take approximately a week to reach comparable conditions.

Technical review of mass concrete composition

Mix Design – Cement

Thermal cracking and Delayed Ettringite Formation (DEF), which will be discussed in greater detail further in this report, can be addressed to an extent by the composition of the cement used in the design. Temperature control is important here, as entrapment of sulfates (SO_4) in the CSH gel is triggered once the concrete reaches a temperature of 160^{o}F or higher and thermal cracking occurs when the temperature gradient within the mass concrete causes sufficiently high tensile forces to exceed the concrete's stress limit, which develops over the curing period. The cement used in the mass concrete mix could be used as a means of controlling the temperature by specifying cement compositions that have a low heat of hydration or a longer set time to delay the hydration reaction and allow the heat generated in the reaction to develop over a more

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[2] ACI 207.1R-96, November 1996
[3] Chini, A., Parham, A., "Adiabatic Temperature Rise of Mass Concrete in Florida", BD 529, February 2005

prolonged period. Figure 5.3.1 in ACI 207.1R-96 indicates ATR of mass concrete as a function of time, for each type of cement (Figure 1), given a content of 376 LB/CY.

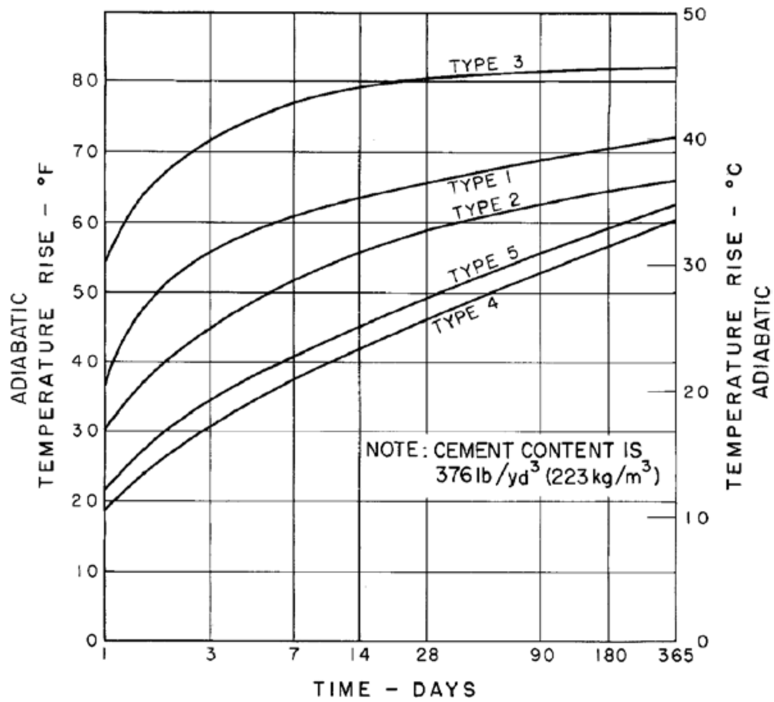

Figure 1 - Temperature rise of mass concrete (ACI 207.1R-96 Fig. 5.3.1)

Note that Type I and III cements generate the greatest ATR. Cements with low heat of hydration incorporate smaller percentages of tricalcium aluminate (C_3A) and tricalcium silicate (C_3S), since these components of cement contribute to a higher heat of hydration. In general, cement types II, IV and V provide reduced C_3A and C_3S content, making them suitable for mass concrete applications. NJDOT Qualified Products List (QPL) and Standard Specs only allow for the use of Type II cement.

In addition to the cement's heat of hydration, several studies have looked at Sulfite (SO_3) content in cement mixes in relation to DEF formation. A 2006 TXDOT study examined a Type V cement which had a SO_3 content of 1.9% (Types I and III tested alongside it ranged from 2.78% to 4.2%), which experienced DEF-induced expansions that were smaller than 0.1%, far less than the other types. The same TXDOT study also found that a type I cement with a lower percentage of C_3A and SO_3 experienced substantially less DEF expansion.[4] In comparison, a Cement and Concrete Research Report, composed in 2003, also finds that low SO_3 content in cement is able to prevent DEF. For comparison purposes, the Essroc Type I cement used in NJDOT projects contains 3.9% SO_3.[5]

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[4] Folliard, K., et al., "Preventing ASR/DEF in New Concrete: Final Report", FHWA/TX-06/0-4085-5, June 2006
[5] Ramlochan, T., Zacarias, P., Thomas, M. D., & Hooton, R. D. (2003). *The effect of pozzolans and slag on the expansion of mortars cured at elevated temperature Part I: Expansive behaviour. Cement and Concrete Research,* 33(6), 807-814.

Mix Design – SCMs

The Supplementary Cementitious Materials (SCMs) used in the concrete mix reduce the heat of hydration. Class-F fly ash and slag can be used for this purpose, although the percentage of replacement depends on several factors including environmental exposure and durability requirements.[6] This reduction in heat of hydration makes the possibility of DEF less likely by limiting the temperatures at the element core reach 160^{o}F or greater during the curing period. A 2005 FLDOT study examined the effect of fly ash and slag on the peak temperature of concrete, and found that there were reductions of 0.1% to 26.1% over ordinary Portland cement (OPC), as detailed in Table 1 below.[7]

Table 1 - Effect of Pozzolans on the Peak Temperature of Concrete (Chini & Parham, 2005)

Cement Source	**Placing Temperature**	**% Reduction in Peak Temperature after 14 days**			
		25% Fly Ash	35% Fly Ash	50% Slag	70% Slag
A	73^{o}F	12.7	17.2	8.2	21.2
B	73^{o}F	8.5	26.1	14.1	24.1
Average for Cements A & B		*10.6*	*21.7*	*11.2*	*22.7*
A	95^{o}F	1.9	8.0	0.1	23.4
B	95^{o}F	9.7	18.6	4.6	7.0
Average for Cements A & B		*5.8*	*13.3*	*2.4*	*15.2*

By lowering the peak temperature, the concrete core temperatures are less likely to reach the 160^{o}F temperature threshold during the critical period of curing, and less likely to have an extreme core-surface temperature difference. In addition, when cements are placed at lower temperatures, the peak temperature drops, thus colder placing temperatures significantly help thermal control of the curing concrete. In the *Materials and Structures* article, Breitenbücher advocates a lower placing temperature for fresh concrete, as it makes it less likely to experience thermal cracking.[8]

A similar FLDOT report, performed in 2003 investigated the effects of high curing temperatures on the phenomena of DEF by testing multiple cement mixes with varying levels of SCMs[9]. The study found that DEF did not occur in any samples cured at room temperature (73^{o}F). However, once concrete samples were cured at temperatures at or above 160^{o}F, DEF began to occur in all mixes. The OPC mix exhibited greater presence of DEF in comparison to mixes that incorporate SCMs. These mixes experienced a smaller decrease in compressive strength than the OPC mix, which lost 34% of its compressive strength (in comparison to the samples cured at room

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[6] Gajda, J., “Mass Concrete: How do you handle the heat?”, PCA
[7] Chini, A., Parham, A., “Adiabatic Temperature Rise of Mass Concrete in Florida”, BD 529, February 2005
[8] Breitenbücher, R. (1990). Investigation of thermal cracking within the cracking frame. *Materials and Structures*, (23), 172-177
[9] Chini, A. R., Muszynski, L. C., Acquaye, L., & Tarkhan, S. (2003, February). *Determination of the maximum placement and curing temperatures in mass concrete to avoid durability problems and DEF*

temperature, or 73°F) at the 28 day mark. A mix that incorporated an 18% fly ash replacement (by weight) only lost 8% of its compressive strength at 28 days when cured at 160°F. Similarly, a mix using a 50% weight replacement of ground granulated blast furnace slag, experienced a 7% reduction in its compressive strength[10]. Similar effects are reported by multiple studies[11]. Thus, using SCMs in the mix design lowers the incidence of DEF, and mitigates the cracking associated with the condition.

Slag is divided into three classifications based on its activity index, grade 80, 100 and 120. The grade reflects the strength of a mortar mix made with 50% slag and 50% Portland cement, and is reported as a percentage of the strength of mortar made with reference cement alone. NJDOT has approved grade 100 (medium activity) and grade 120 slag (high activity). Grade 80 slag has a low activity index thus generating less heat than Portland cement concrete, making it ideal for use in mass concrete applications. FHWA recommends avoiding the use of grade 80 slag unless warranted in special circumstances.

ASTM C989 indicates that the use of slag cement will decrease the C_3A content of the cementing materials, reducing concrete reactivity, and will decrease the permeability and calcium hydroxide content of concrete.

It should be noted that some SCMs are not suitable for use in mass concrete. The 2006 TXDOT report examined DEF and the impact of silica fume on related expansion, in addition to the effects of fly ash and slag replacements[12]. It was determined that at 10% replacement (by weight), silica fume was unable to successfully mitigate expansions caused by DEF, which lead to cracking. It did manage to delay the onset of DEF, which caused reduced expansions in the concrete as compared to a pure cement mix, however not within acceptable limits. Fly ash (20% and 40% replacements) and ground granulated blast furnace slag (35% and 50% replacements) on the other hand were able to reduce DEF-induced expansion to the point where extremely minor cracks occurred, or even none at all. Silica fume causes the concrete's heat of hydration to increase; increasing the risk that concrete reaches temperatures above 160°F, and the likelihood of DEF occurring. This also increases the risk of temperature differences between the core and surface to exceed the 35°F threshold, which could lead to thermally induced cracking. Other studies have reported similar issues with silica fume[13]. These results indicate that silica fume is less suited for use with mass concrete than fly ash or ground granulated blast furnace slag.

[10] Chini, A. R., Muszynski, L. C., Acquaye, L., & Tarkhan, S. (2003, February). *Determination of the maximum placement and curing temperatures in mass concrete to avoid durability problems and DEF*

[11] Siler, P., Kratky, J., & De Belie, N. (2011). Isothermal and solution calorimetry to assess the effect of superplasticizers and mineral admixtures on cement hydration. *Journal of Thermal Analysis and Calorimetry*.

[12] Folliard, K., et al., "Preventing ASR/DEF in New Concrete: Final Report", FHWA/TX-06/0-4085-5, June 2006

[13] Ramlochan, T., Zacarias, P., Thomas, M. D., & Hooton, R. D. (2003). *The effect of pozzolans and slag on the expansion of mortars cured at elevated temperature Part I: Expansive behavior. Cement and Concrete Research,* 33(6), 807-814.

Mix Design – Admixtures

The ACI 207.1R-13 guide for Mass Concrete states that admixtures do not have an effect on heat of hydration after the first few hours after mixing. Thus, their effects can be neglected in preliminary computations. However, when a design incorporates several million cubic yards of concrete, adiabatic temperature rise should be determined for the exact mixture used and compared with the proposed placing temperature to arrive at a proposed peak temperature.

Some admixtures may not be suited to use with mass concrete. An accelerating admixture will contribute to undesirable heat development, so it should not be used. Superplasticizers still require more research on their effects, as studies offer conflicting results on peak heat and total heat generated.

Mix Design – Aggregates

The aggregates incorporated in mass concrete will impact temperature control during curing. The aggregate's co-efficient of thermal expansion, and thermal conductivity determine the concrete's ability to manage temperature changes and maximum temperature achieved during curing. Table 2 shows typical coefficient of thermal expansion ranges for several widely used aggregates. Using aggregates with low coefficients of thermal expansion causes the aggregates to expand less as they increase in temperature, which reduces the risk of cracking due to thermally induced volume expansion. In addition to this, having a low thermal conductivity reduces the risk of thermal cracking, as the concrete on the outside will not cool off as quickly, decreasing the temperature differential between the outer portion and the core of the concrete.[14]

Table 2 – Typical thermal expansion ranges for common aggregates (FHWA)

	Coefficient of Thermal Expansion	
	10^{-6}/°C	**10^{-6}/°F**
Aggregate		
Granite	7-9	4-5
Basalt	6-8	3.3-4.4
Limestone	6	3.3
Dolomite	7-10	4-5.5
Sandstone	11-12	6.1-6.7
Quartzite	11-13	6.1-7.2
Marble	4-7	2.2-4

In addition to their thermal characteristics, the size of the aggregates, the volume used and the manner of their preparation prior to use all have impacts on the concrete's temperature. It is recommended that the largest aggregates compatible with the mix be used, in the largest volume possible. Prior to their usage, aggregates should be kept in a shaded area, in addition to being chilled or wetted to reduce the placing temperature of the concrete.

[14] Choktaweekarn, P.; Somnuk, T. (2010) Effect of aggregate type, casting, thickness and curing condition on restrained strain of mass concrete. *Songklanakarin Journal Of Science & Technology*, *32*(4), 391.

Technical review of typical mass concrete symptoms

Initial symptoms caused by thermal gradients

Thermal gradients in concrete lead to the development of tensile stresses. Large gradients during concrete curing can lead to early age cracking of mass concrete elements. Following initial placement of mass concrete, the ATR at the core of the mass reaches its peak and begins to diffuse its heat to the surface. If the surface is allowed to release the heat quickly, surface temperatures may drop beyond a threshold, allowing unacceptable tensile stress to develop at the surface. Ultimately, this gradient will act as a restraint, causing the surface to crack under the tensile stress.

According to ACI 207.1R-96, concrete tensile strength can be expressed as a relationship to its compressive strength as follows: $f_t = 1.7\ f_c^{2/3}$ (psi). Within a time-dependent analysis, the critical thermal gradient threshold can be determined. Various studies referred to in this report, as well as industry standards, indicate that a 35°F gradient threshold is sufficient to avoid thermal-induced cracking on the surface of mass concrete. Through additional modeling, it is anticipated that temperature differential thresholds can be tabulated as a function of curing period in order to provide a more stringent criteria for thermal control.

Symptoms of concrete overheating – Delayed Ettringite Formation

Delayed ettringite formation is a deleterious phenomenon that may occur in mass concrete resulting from elevated concrete curing temperatures. Although less common than other similar phenomena such as Alkali-Silica Reaction, it may be equally damaging to concrete elements. the phenomenon is directly linked to cement curing at temperatures exceeding 158°F threshold. Normally forming ettringite ($C_3A \cdot 3CaSO_4 \cdot 32H_2O$) in curing concrete is delayed at these higher temperatures due to a change in the hydration of the cement paste. Sulfates and aluminates in the cement become trapped in the Calcium Silicate Hydrate (C-S-H) paste and other hydrates produced during hydration.   Once the hydration process is complete and the concrete is exposed to moisture at ambient temperature for extended periods, these trapped sulfates and aluminates slowly diffuse through the C-S-H paste and react with monosulfate hydrates, forming ettringite. A material that normally expands in a cement paste expands in a hardened concrete. Expansive tensile forces cause cracking in the concrete. Sufficiently high sulfate and aluminate concentrations in mass concrete result in reduced durability and strength loss in concrete elements.

Ettringite may also fill in in pre-existing cracks, exacerbating the condition. The 2006 TXDOT report documents a 1993 investigation by Fu on delayed ettringite formation, incorporating fracture mechanics and thermodynamic considerations.[15] It was determined that ettringite nuclei will form near the tips of the cracks. After this nucleation, the ettringite crystals can grow, expanding the crack and further weakening the concrete.

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[15] Folliard, K., et al., "Preventing ASR/DEF in New Concrete: Final Report", FHWA/TX-06/0-4085-5, June 2006

Ettringite Formation in well-performing concrete

Despite the problems associated with DEF, ettringite presence in concrete is typically expected. Often, ettringite is found in mature concrete, especially in areas such as voids or air bubbles which give it necessary space to expand. If ettringite forms prior to concrete hardening, the material may expand within the "green" concrete, without creating tensile stresses. It is ultimately the time at which ettringite forms that determines whether it has a negative effect.

Wittpenn review and comparisons

Concrete mix design

According to the Thermal Control Plan (TCP) prepared by CTL Group, dated October 1, 2012, the Wittpenn Bridge pier cap design mix consisted of a Class P concrete consisting of 537 Lb/CY of Essroc Type I cement and 178 LB/CY Holcim Grade 100 Slag. Mill certs from local suppliers indicate that Essroc Type I cement consists of a 71.48% $C_3S + C_3A$ composition[16]. Using Table 3, the mix closely aligns with a 30% slag replacement, providing some reduction in heat of hydration.

Table 3 – Summary of concrete mixes tested by semi-adiabatic calorimetry (converted to cal/g)[17]

No.	Cement Type	Heat of Hydration at 100% Hydration (cal/g)
1	Type I Cement	114
2	Type I Cement + 15% Class C Fly Ash	113
3	Type I Cement + 25% Class C Fly Ash	112
4	Type I Cement + 35% Class C Fly Ash	111
5	Type I Cement + 45% Class C Fly Ash	110
6	Type I Cement + 15% Class F Fly Ash	106
7	Type I Cement + 25% Class F Fly Ash	101
8	Type I Cement + 35% Class F Fly Ash	95
9	Type I Cement + 45% Class F Fly Ash	88
10	Type I Cement + 30% Ground Granulated Blast Furnace Slag	113
11	Type I Cement + 50% Ground Granulated Blast Furnace Slag	112

[16] Quality Assurance Sample, Essroc Cement Co. Plant #1 – Nazareth, PA, dated October 18, 2011
[17] Schindler, A, Folliard, J., "Heat of Hydration Models for Cementitious Materials", ACI Materials Journal, Title no. 102-M04

ACI recommends limiting the use of cement to as small an amount as possible. Other optional recommendations include limiting the $C_3S + C_3A$ composition to 58% or limiting the heat of hydration to 70 cal/g at 7 days (ACI 207.1R-13, section 2.2).

Thermal Control Plan

The thermal control plan indicates that the specified 35°F temperature gradient may not prevent thermal cracking at early ages and can be too conservative at later ages because it does not consider the properties of the actual concrete mix design.[18] The concrete's ability to resist the temperature gradient is proportional to the strength gain during curing. It is a time-dependent behavior that may be best described by tabulating temperature thresholds as a function of time, which can be calculated through Finite Element Analysis of the element.

The TCP outlines the methods to be used to maintain both thresholds. Table A in the document indicates that only the final, 7 ¼-FT thick pier cap segment would require cooling pipes. The document also includes a graph that outlines temperatures as the concrete cures.

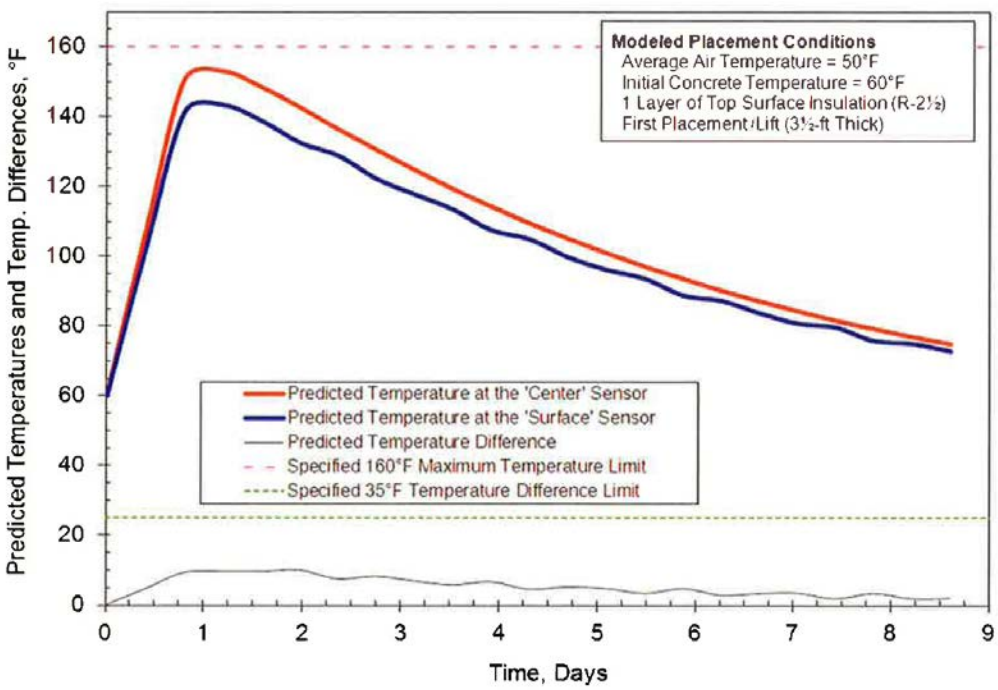

Figure 2 - Detailed Results of Thermal Modeling for the First Placement

As can be seen from the graph, the model predicted that both maximum temperature and temperature difference thresholds would be maintained. Further, the temperature differences would be maintained no higher than 10-15°F, which is well within the limits.

[18] Letter dated October 1, 2012 – titled "Thermal Control Plan for the mass concrete fill within the precast cofferdam at Pier 1W Wittpenn Bridge, Route 7 over the Hackensack River, Kearny NJ, CTLGroup Project No. 051622, TCP 1", Feld, J., Gajda, J., Smith, S., CTLGroup

Comparison with NJDOT 2007 Standard Specifications

The NJDOT 2007 Standard Specifications provide explicit direction to contractors about the required documentation and plans that must be submitted at least 30 days before placing concrete. The following is a point-by-point discussion of each requirement a contractor must include in the Thermal Control Plan:

1. Concrete mix design, including pozzolanic materials to control concrete temperature.

As described in the previous sections, it is critical for each concrete component to be accounted for in terms of heat of hydration. The contractor should be aware of materials that should be explicitly avoided, such as silica fume and accelerating agents.

2. Adjustments to form removal and loading times for slower strength gains for high pozzolan mixes.

Mass concrete will cure at a slower rate than other concrete methods. It is critical for the contractor to understand concrete maturation. This may be monitored through NDE evaluations such as Ultrasonic Surface Wave. The contractor should outline the steps taken to ensure timing of form removal. In addition, mass concrete pours may be "staged" to further control thermal effects. The contractor should identify the timing between form removal and placement of the subsequent mass concrete segment.

3. An analysis of the anticipated thermal developments within placements using proposed materials and casting methods.

This analysis can be accomplished via Finite Element Analysis. The contractor's engineer should be experienced in mass concrete modeling and be able to develop a proper analysis of the element being constructed.

4. A plan outlining specific measures to be taken to control the temperature differential within the limits.

This typically includes insulation, cooling pipes, and other methods to mitigate the tendency for concrete to dissipate heat from the surface. By maintaining a constant temperature throughout the element and minimizing ATR at the core, the contractor can best control temperature differentials. Modeling the mass concrete element prior to construction is critical to identifying the number and location of cooling pipes needed to maintain a consistent, acceptable temperature gradient through the element cross section.

5. The proposed monitoring system

The system should include temperature readings at the element's central core and surface. It should also include maturation data to determine strength as a function of the element's curing time. This could play a pivotal role in developing a more stringent threshold for temperature gradient, which relates concrete strength development with tensile resistance during curing.

6. Outline of corrective actions to control the temperature differential and maximum internal temperature.

In addition to precautionary steps outlined in item #4, the contractor should take necessary steps to maintain the differential below the 35°F threshold. Curing operations should take this into consideration, especially when wet curing. While water is the best option for mass concrete, the thermal control plan should account for this via maintaining an acceptable gradient through the element cross-section.

7. Proposed methods of repairs or corrective actions if the mass concrete member is not accepted.

The literature has extensively documented the urgency of maintaining the maximum curing temperature below 160°F. The adverse effects associated with exceeding the maximum temperature threshold are severe, but not visible for months or years after construction. This threshold should never be exceeded.

The literature also documents damages resulting from exceeding temperature differential thresholds, which are more immediate and can be identified during construction. The thermal-induced cracking that results may be repaired through industry accepted means, from seals, coatings for hairline cracking, to more comprehensive repairs.

It should also be noted that the standard specifications indicate that temperature control must be maintained for 15 days. This limit may not be sufficient to control the high core temperature that persists for significant periods beyond the 15 day limit. It is recommended that this limit be replaced with a requirement that the contractor's engineer submit an analysis indicating equilibrium between core and air temperatures that will result in temperature differences not exceeding the 35°F threshold.

In addition to the 15 day limit, the department should also consider the effects of thermal differentials on early age strength of mass concrete. Relying on tensile stresses is typically not acceptable by NJDOT. In considering other concrete placement practices such as prestressed concrete, in which no tensile stress is allowed, limiting tensile stresses in mass concrete should be a top priority. Thus, at minimum, maintaining the 35°F delta and 160°F maximum should be continued. The team recommends considering that a table be developed outlining temperature thresholds as a function of time after placement. This table should be mix design-specific, and account for the strength development and its ability to resist tensile forces developed through thermal effects.

Recommendations and conclusion

The information provided in this review is considered a synthesis of current research and practice, and guidance and recommendations are based on the literature reviewed. For more information on the publications reviewed for this study, please refer to the references section of this document.

Mass concrete placement requires strict thermal controls in order to ensure proper concrete performance. Thermal Control Plans that outline how the contractor will achieve a low temperature during concrete placing, limit ATR, maintain peak temperatures below 160^{o}F and insulate the curing concrete from exceeding the 35^{o}F temperature threshold are critical.

During early stages of curing, the concrete has not developed sufficient strength to resist excessive thermal gradients. Thus, form insulation and other methods to protect the concrete surface from dissipating heat greatly or reach excessively high peak temperatures reduces the likelihood of deleterious effects. The results of this literature review suggest that current research and industry agree that temperature thresholds are critical to mass concrete. Proper controls must be established in order to ensure well-performing concrete elements to be constructed.

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November 5, 2012

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RE: New Jersey Bridge Resource Program
I-195 over the New Jersey Turnpike
Final Report

Dear Mr. Roda:

Intelligent Infrastructure Systems (IIS), a Pennoni Company, is pleased to submit our final report for rapid load testing of the fire damaged bridge carrying Interstate 195 over the New Jersey Turnpike. As I'm sure you're aware, this damage occurred during a vehicular accident on October 3, 2012. This report summarizes the findings of our emergency field testing of the structure overnight on Thursday evening October 11th into Friday morning October 12th.

Again, thank you for including IIS on your team and for utilizing our services for this emergency evaluation. We certainly appreciate the opportunity. Please do not hesitate to contact me at 215-254-7729 if you have any questions or require additional information.

Best Regards,

INTELLIGENT INFRASTRUCTURE SYSTEMS

David S. Lowdermilk, PE
Principal
Regional Vice President

Introduction

On October 3, 2012, a dump truck traveling southbound on the New Jersey Turnpike (NJTPK) in the right lane had a mechanical failure, swerved across the middle and left lane and collided with a jersey barrier. The truck slid along the barrier, causing sparks which ignited a fire. The truck came to rest under an overpass carrying I-195. The structure had recently been widened to carry both directions of traffic as part of the NJTPK widening project. The fire burned directly under the structure for approximately one hour before firefighters could arrest it. Additionally, timber formwork that remained on the structure from the widening also burned. After the incident the lane has remained closed to traffic, causing substantial delays for the traveling public.

As part of New Jersey DOT's Bridge Resource Program with Rutgers University, Intelligent Infrastructure Systems (IIS) was asked to investigate the viability of the structure. The goal was to potentially open the structure to traffic for a finite period of six weeks, until the adjacent structure under construction can be put into service.

Assessment Approach

A brief visual assessment prior to testing was performed as part of the investigation. Fire damage was observed along the four exterior beams at the northeast corner of the eastbound structure. The most prevalent discoloration was present along the second interior beam (Girder #3). Heavy soot and debris was located along the flanges. In addition, the metal stay-in-place forms exhibited noticeable sag. The condition of the plate steel (flanges and webs) was in overall good condition. Moderate pitting and section loss was observed along G3 and G4. The general condition of the bridge at the time of testing can be seen in

Figure 1. Small out-of-plane deformation was observed in the girder webs. This may or may not be as a result of the fire.

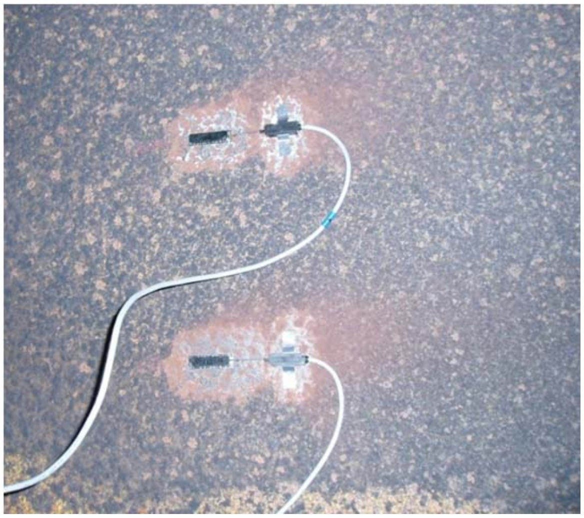
IIS approached the experimental assessment of the structure from two view points; 1) performance of the fire affected area and 2) overall performance of the structure. Both views are discussed in the subsequent sections. Background research into other bridge fires was conducted but the primary examples involved total collapse of the structure. Research was limited by the available time window.

The approach for this test was predominately based on direct interpretation of the data collected during the test. The goal was to validate the global and local behavior of the structure under known loads. The model was a secondary level of analysis used to provide pre-test insight and confidence, and to extrapolate other load scenarios after the test. The model was not calibrated in the traditional sense, meaning the parameters in the model were not altered to reduce the error between measurement and model. However, the model was refined to better represent behaviors or characteristics that were observed on site. The updated model was used to investigate a simple capacity-demand ratio as an indicator of the immediate reserve strength of the structure.

a. Fire-affected Area

b. Debris on Flanges

a. Sagging of Stay-in-Place Forms



b. Pitted Web

Figure 1a-d – Post-fire Condition

Finite Element Model Development

Even with the limited timeline for the test, IIS felt that a finite element (FE) model was required for instrumentation design and to provide a reliable frame of reference during and after the test. The model was element-level, meaning that the geometry was represented but prismatic members, like beams, were represented with beam elements, and 2D planar members, like the deck, were represented with shell elements. Continuity and compatibility were maintained with link elements. The connection between the existing and the new construction at the deck was comprised of link elements to allow for variation in the stiffness of the cold joint.

There were some simplifications made when constructing the model to help expedite the process without compromising the integrity of the results. These simplifications are stated below:

1. The flange transitions on the existing structure were ignored in favor of a single, smeared section that approximated the varied section adequately
2. Initially, all girders are fully composite with the deck
3. The structure is resting on idealized simple supports
4. The barrier is not composite with the deck (only one barrier included a priori)
5. The cold joint provides a solid connection, excluding rotation about the longitudinal axis of the joint
6. No specific material property or other types of changes were made to the model to represent the fire effects

Some of these assumptions were directly addressed via instrumentation and later changed to better represent the structure.

The A Priori model, as built, is shown in Figure 2. The model was used to verify approximate response magnitudes at proposed sensor locations, and to evaluate the importance of several parameters including composite action, barrier participation, and cold joint stiffness.

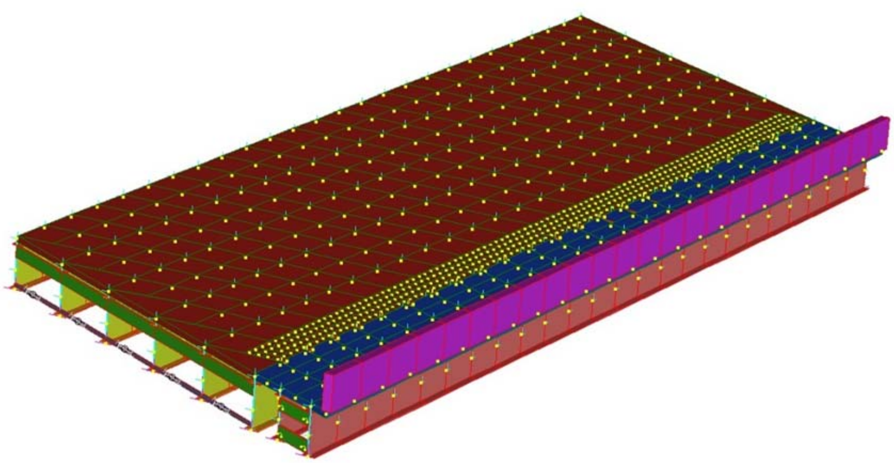
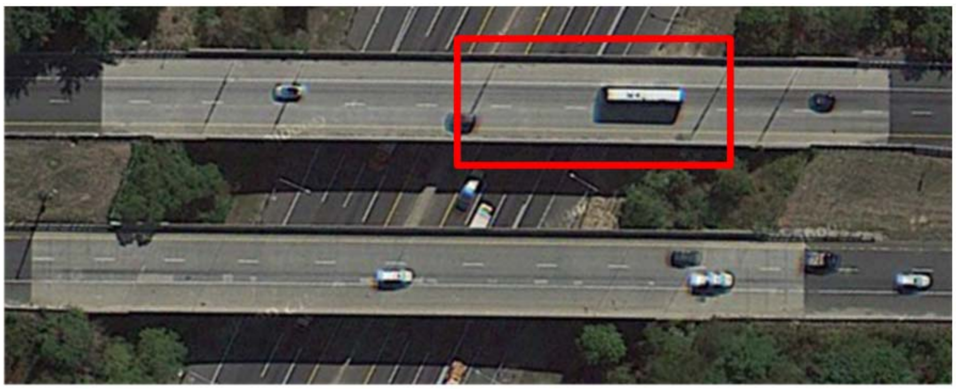


Figure 2 - A Priori Model

Instrumentation Design

Considering the rapid nature of the test and high priority situation for NJDOT, the experiment was prepared as quickly as possible over the span of two days and implemented on October 11, 2012 between 10PM and 5AM. The planned instrumentation consisted of 11 displacement measurements, three strain rosettes, and 16 longitudinal strain gages. The instrumentation plan was designed to capture both the global behavior of the structure as well as the local response in the fire-affected area, while meeting the constraints set forth in terms of access and schedule.

The displacements were included to provide both a global reference of how the entire structural system was performing, as well as relative information about lateral movement of the girders. Longitudinal strains provided similar information about the entire structural system including peak strain (i.e., stress), and also indicate if the section is composite with the deck. The strain rosettes measure the direction and magnitude of the peak tensile and compressive strains in the web at the critical location for shear response (approximately the depth of the beam section from the bearing).



Span 2 on Eastbound Structure (pre-widening)

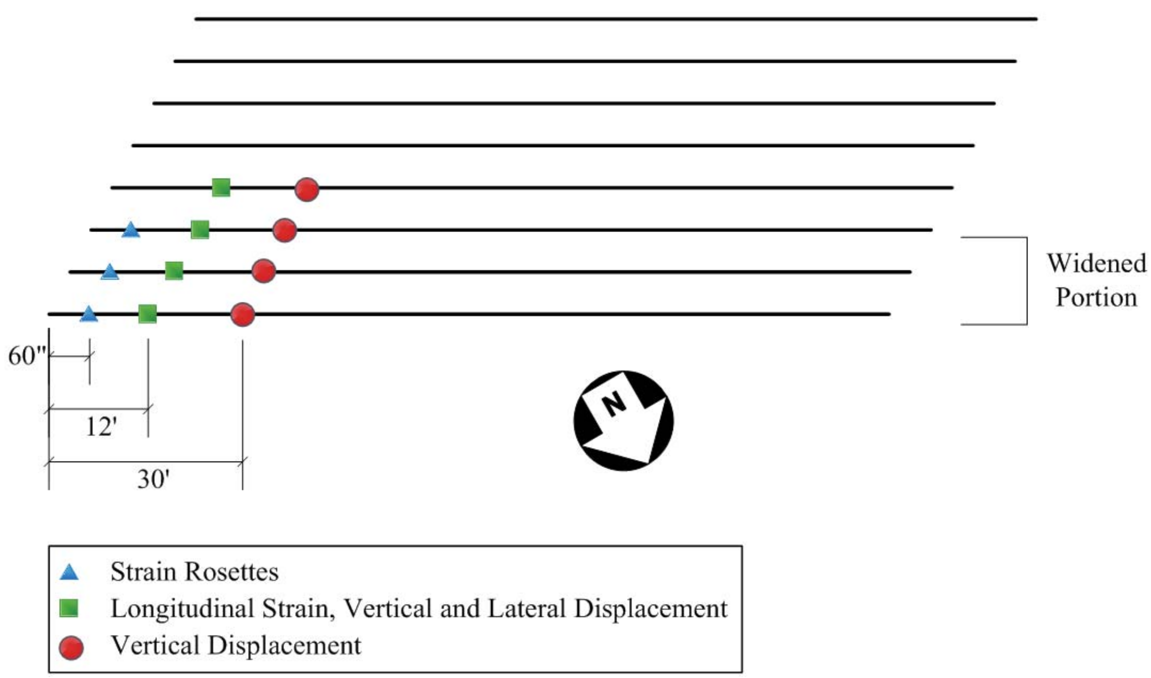


Figure 3 - Conceptual Instrumentation Plan

Test Execution

IIS personnel met with NJDOT and reviewed the available documentation of the original structure, the widening retrofit, and the fire incident. Access to the structure was limited as the bridge passes over the heavily traveled New Jersey Turnpike. A two lane closure of the turnpike would be provided, focusing on the area of the bridge where the fire was concentrated. Similarly, the lane of I-195 that was exposed to the fire had also been closed since the indicient and was available. The total length of time available for the closure was seven hours, overnight. Not all sensors were installed due to time constraints. The localized working area and some access equipment failures caused installation delays.

The actual loading of the structure occurred between 3AM and 4AM on Friday October 12, 2012. The span was loaded incrementally with two (2) Workstar International Dump Trucks, each weighing 22,000 lbs. The trucks were positioned at two primary locations; 1) midspan for maximum global response and 2) over the fire affected area for maximum local response. The following load stages were used:

LS1.1 - One truck at midspan
LS1.2 - One truck at the affected area
LS2.1 - One truck at midspan
LS2.2 - Two trucks at midspan
LS2.3 - One truck at affected area, one truck at midspan
LS2.4 - Two trucks at affected area

Direct Interpretation of Data

The primary concern related to the fire-affected region in particular was that one or more girders were overstressed due to damage, load redistribution, or unexpected global or relative movements (i.e. lateral movement). The instrumentation which directly related to the performance of the fire affected regions were the strain rosettes and a horizontal displacement between the existing fascia girder and the interior widening girder. The rosettes, which are groups of three sensors, were installed at approximately the depth of the web from the bearing (between 5 ft. and 6ft.) During LS2.4, the worst case scenario for these sensors, a maximum principle tensile stress of 0.49 ksi was observed on Girder #3, and a maximum principle compressive stress of -0.43 ksi on Girder #1. These sensors were located directly in the fire affected zones. The response magnitudes were very small indicating that there is substantial sharing of load across the two widening girders and the existing structure. Similarly, only negligible relative horizontal displacement was recorded across all load stages.

Strain gages were placed on both sides of the bottom flange of the first four girders to capture any out of plane movement. A maximum of only 0.15 ksi difference was recorded between these sensors on any load stage, indicating very little out of plane movement. Table 1 shows a summary of peak values of displacement and stress during LS2.2 and LS2.4.

Table 1 - Peak Responses

		Peak Stress in fire-affected region (ksi)	Peak Disp at 30 ft. from bearings (in.)
LS2.2	G1	0.408	-0.191
	G2	0.429	-0.156
	G3	0.519	-0.133
	G4	0.433	-
LS2.4	G1	0.767	-0.174
	G2	0.748	-0.147
	G3	0.833	-0.124
	G4	0.433	-

The global vertical displacement at a distance of 30 ft. from the bearings was measured on Girders 1, 2, and 3. Across the widened portion of the structure, the displacement behavior was linear, with a maximum displacement of 0.19 in. at the fascia girder under LS2.4. The deflection basin for the widened portion, seen in Figure 3, indicated that the doweled connection of the deck is still functioning as a load transfer mechanism between the widened portion and the existing structure. This response was observed under both trucks positioned at midspan, equal to approximately 44,000 lbs. in total load. Extrapolated to one legal HS-20 truck, this corresponds to L/4600 which is much more conservative than the traditional L/800 deflection limit. Note that this displacement was recorded at approximately one quarter of the span because of the closure limitation.

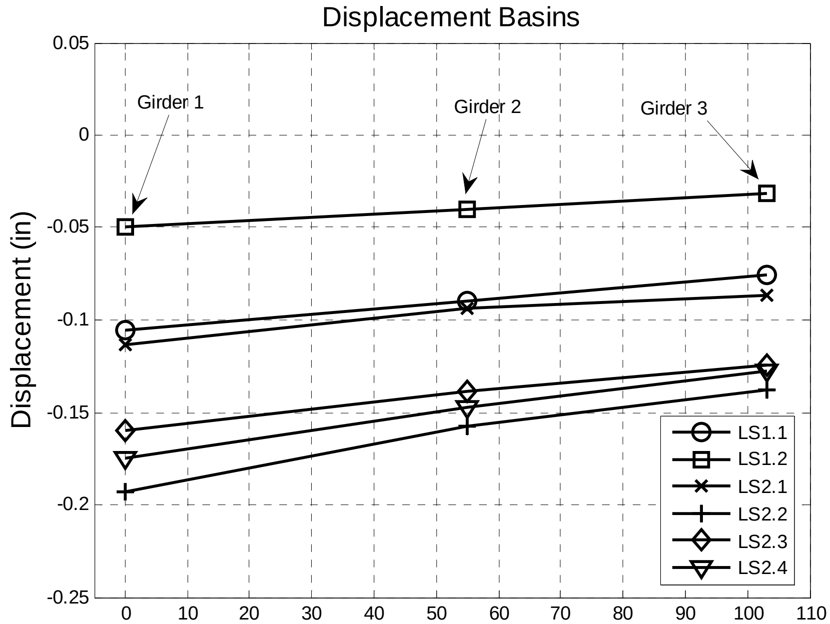


Figure 4 - Displacement Basins of Affected Area under All Load Stages

Comparing displacements across load levels showed an essentially linear behavior. The slight apparent softening that can be seen in Figure 4 was a result of the variance in loading configuration between one truck and two trucks.

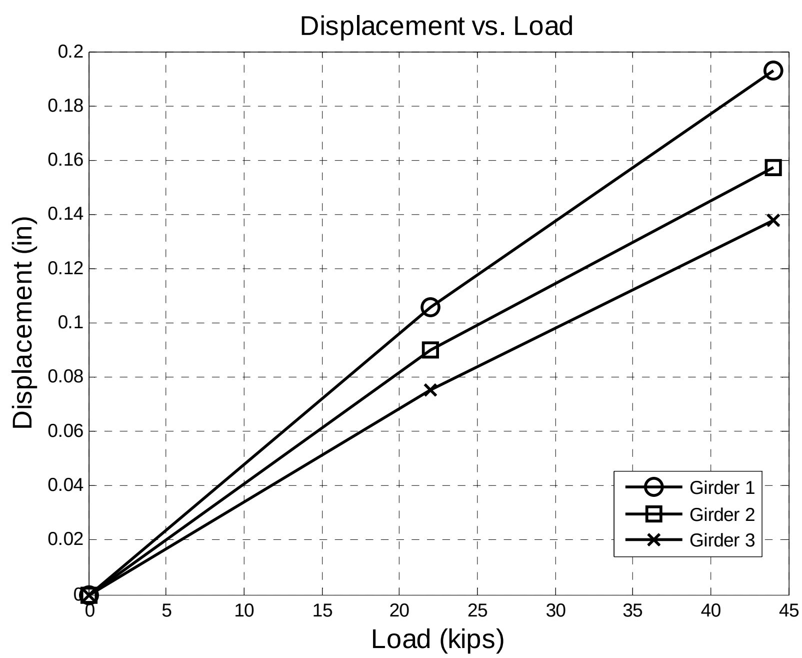
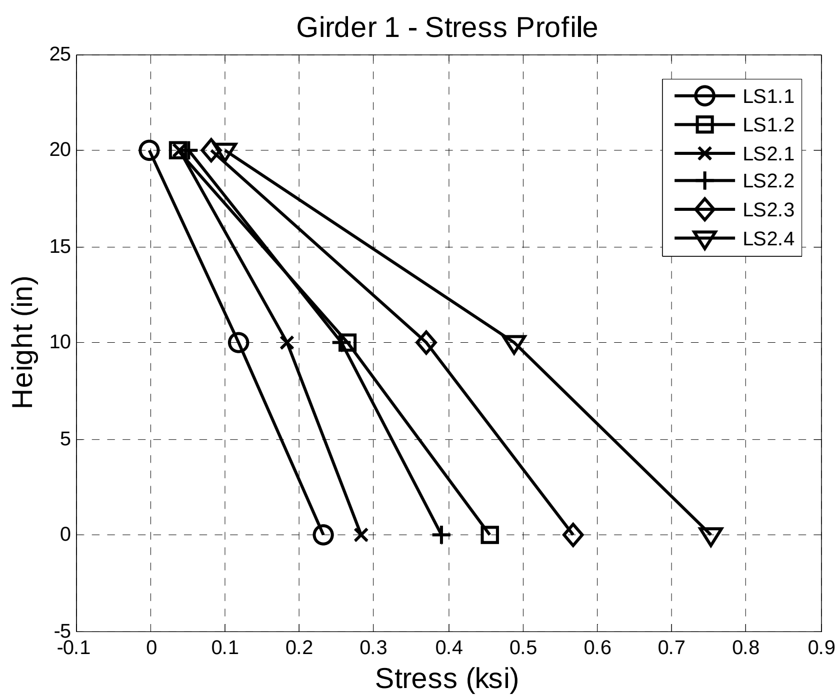


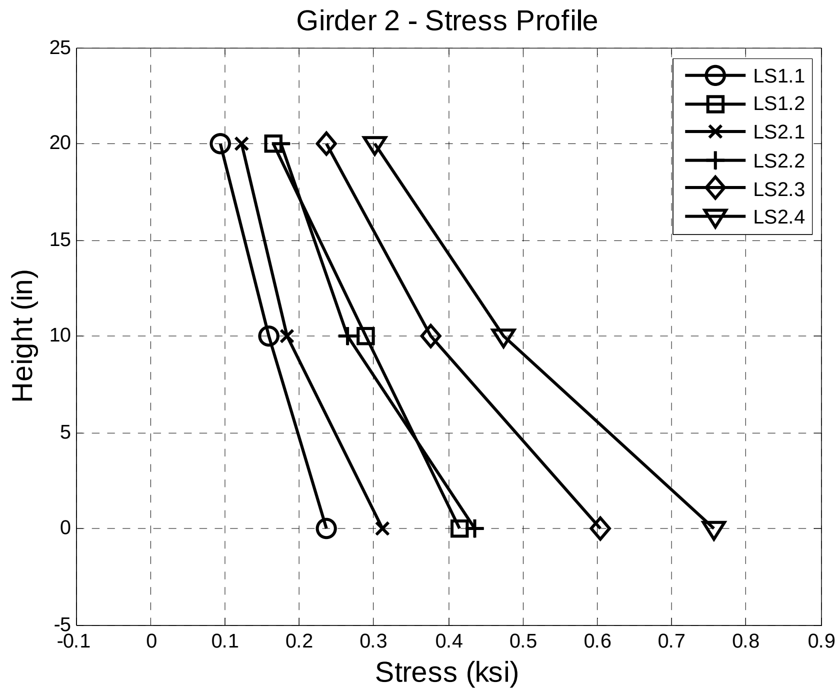
Figure 5 - Displacement vs. Load

From a global performance perspective, it was critical to determine if the structure was still behaving linearly. The most basic check on this is to compare the responses before and after loading to determine if there was any plastic deformation. All sensors returned to values very close to the original zero points after unloading indicating that the structure was globally elastic.

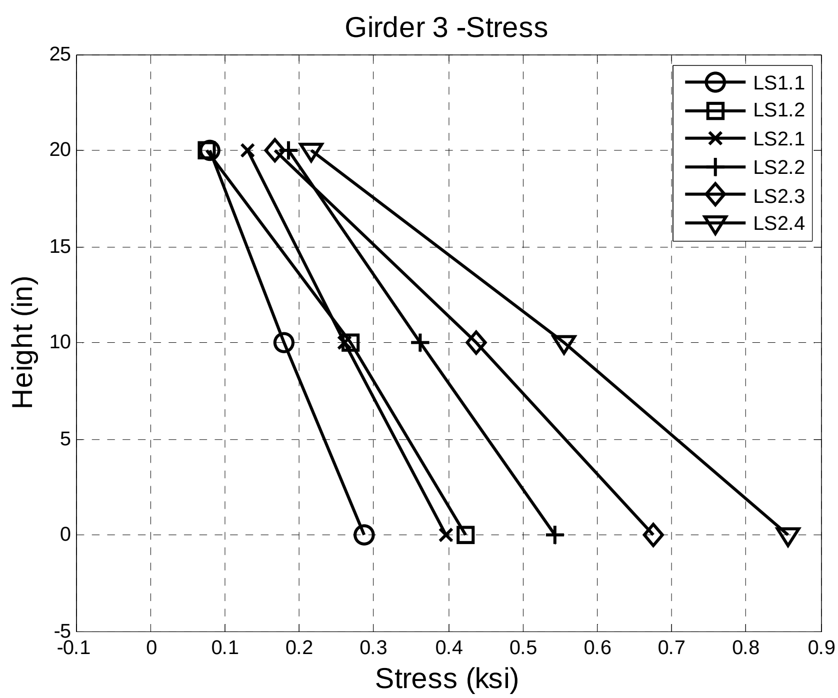
Four girders were instrumented to determine if plane sections remained planes, whether or not the girder was behaving compositely, and to establish the flexural demand under live load. At all load positions and levels all four girders have essentially linear strain profiles, indicating that plane sections do remain planes.



6a.


Girder 2 - Stress Profile
LS1.1
LS1.2
LS2.1
LS2.2
LS2.3
LS2.4
Height (in)
Stress (ksi)
25
20
15
10
5
0
-5
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9

6b.


Girder 3 -Stress
LS1.1
LS1.2
LS2.1
LS2.2
LS2.3
LS2.4
Height (in)
Stress (ksi)
25
20
15
10
5
0
-5
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9

6c.

Figure 6a - 6d - Stress Profiles on Girder #1 through #4

Girder #2 and #4 were both behaving composite with the deck. Interestingly it appears that the fascia girder of the widened portion (Girder #1) and the existing fascia girder (Girder #3) both were behaving non-compositely. However, even as non-composite sections, the response magnitudes are so low that capacity is not an issue under normal loading. Peak stress response under the two trucks was 0.83 ksi. While the trucks were positioned statically, the bridge was still open to traffic. During LS2.3, truck traffic in the open lane in conjunction with the static trucks produced a maximum overall measured stress response of 1.59 ksi in Girder #3 (Figure 6). At the same moment Girders #1 through #3 experienced 1.39 ksi, 0.97 ksi and 0.94 ksi respectively. These values, when compared to the baseline measurements for LS2.3 at 0.35 ksi, 0.65 ksi, 0.58 ksi and 0.58 ksi from Girder #4 to #1 respectively, indicated good load sharing across these girders even under a more extreme load event that had originally been planned for the test.

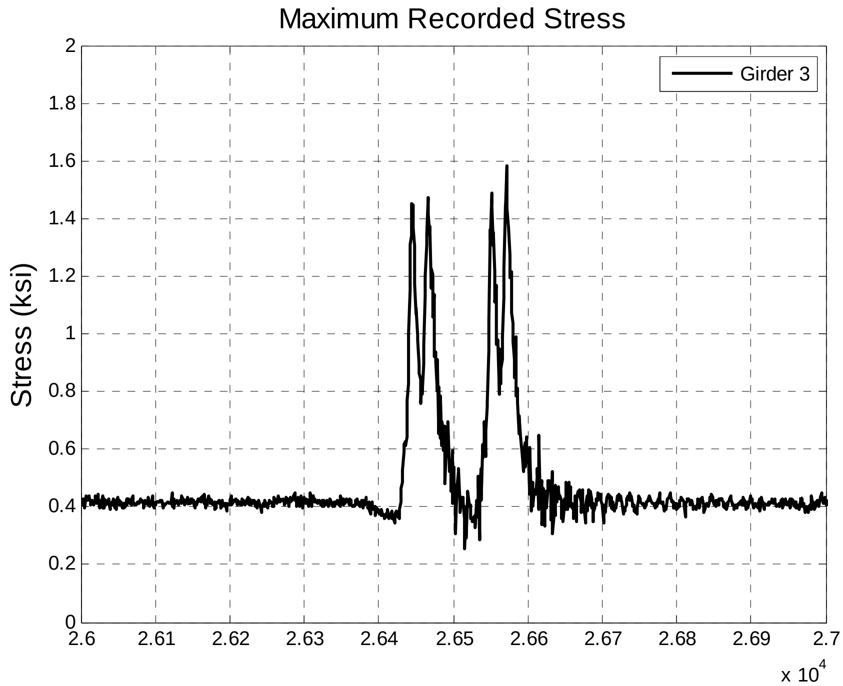


Figure 7 - Maximum Recorded Stress Event

Finite Element Model Refinement and Utilization

The A Priori FE model was revisited after the test to provide quantitative information on the performance of the structure. Several changes to the model were implemented that stemmed directly from observations made on site during the test. The joints in the barrier were observed to be soft and compressible, indicating that the barrier was not in active compression, and would not act as a continuous member under live load. The strain profiles indicated that Girder #1 and Girder #3 were not acting compositely with the deck. In the model, the connection element between the beam element and the shell element representing the girder and deck, respectively, was modified to allow relative translation in the longitudinal (along the roadway) direction. The cold joint appeared to be effectively linking the two decks both in terms of translation and rotation, so the rotational spring at the joint was removed. Finally, the two barriers in the middle and on the opposite edge of the roadway were added for completeness. The updated model is shown in Figure 8.

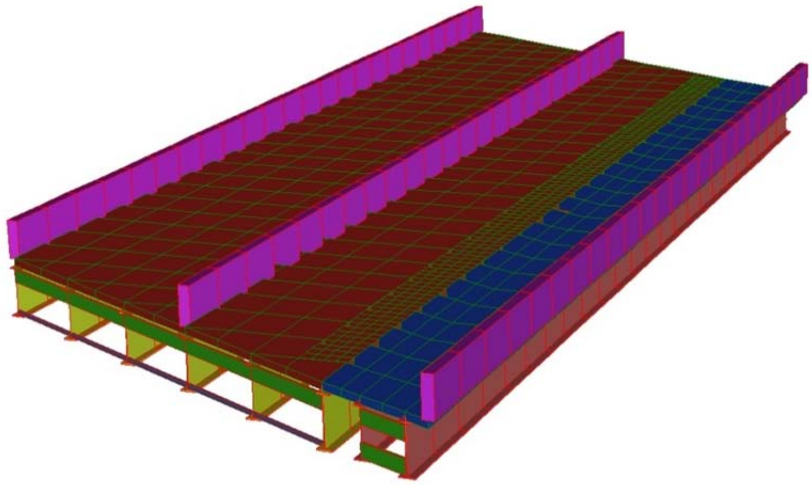


Figure 8 - Refined FE Model

The model was then coarsely validated by comparing the experimental and model responses. The testing trucks were input into the model for primary global load case; two trucks at midspan, or LS2.2. The responses that were compared, vertical displacements at approximately 30 feet from the east bearings, are shown in Table 2.

Table 2 - Model-Experiment Comparison after Model Refinement

	Exp Disp (in)	Model Disp (in)	Percent Error)
G1	-0.191	-0.204	-7%
G2	-0.156	-0.161	-3%
G3	-0.133	-0.124	7%

Judging by the small magnitude of error in displacement response between the model and the experiment, the model has been deemed validated for the purpose of developing some simple performance metrics that are indicative of the performance of the structure.

While a complete AASHTO load rating was out of the scope of this investigation, the model was used to develop factors indicative of the remaining capacity of the structural members. This capacity-demand ratio, the ratio of the capacity over the total demand (dead and live), provided reassurance regarding the factor of safety of the structure over the next several weeks before decommissioning.

The model was loaded with a single HS20 truck in various locations focused on the widened portion of the structure. Peak stress values were extracted from several critical locations. Similarly, dead load demand was extracted from the model, neglecting the cold joint interface between the decks with all girders non-composite.

For the flexural case, capacity was defined as the yield stress of the steel (50 ksi). For the shear case, capacity is defined by AASHTO 6.10.9.2. The resulting value was 9.7 ksi.

Table 3 shows the capacity-demand ratio for the midspan of Girder #1 through #5 for flexure, and Table 4 shows the similar value for all four girders for shear near the bearings (the fire-affected region). This assumes that material properties are typical steel properties and that capacity is defined by yielding.

Table 3 - Capacity/Demand Ratios for Flexure at Midspan - One HS-20 Truck

	Flexure		C/D
	DL (ksi)	LL (ksi)	
G1	13.2	1.81	3.33
G2	14	3.78	2.81
G3	11.7	1.55	3.77
G4	12.4	1.54	3.59

Table 4 - Capacity/Demand Ratios for Shear at 60 in. from the bearing - One HS-20 Truck

	Shear		C/D
	DL (ksi)	LL (ksi)	
G1	2.33	0.319	3.66
G2	2.22	0.968	3.04
G3	1.91	0.343	4.31
G4	2.07	0.3	4.09

These results indicate that there was substantial reserve capacity in the girders in the fire-affected region as well as globally for both flexure and shear. The structure shared load between girders very well, even across the widened portion, limiting the live load demand on any given girder. It should be noted that this calculation did not account for any potential residual stress in the fire affected region or otherwise as there is no definitive data as to whether these stresses exist or not.

Conclusions and Recommendations

Considering the results of the load test conducted on October 11, 2012 and the visual assessment of the condition of the I-195 structure, it is recommended that the bridge remain open to traffic for the duration of the construction of the adjacent replacement span (approximately six weeks). The risk associated with opening the structure to traffic is minimal. The span appears to have robust load sharing mechanisms in the diaphragms and the deck. The measured response magnitudes under the testing loads were low. Considering the global behavior of the span, the low responses can be extrapolated to higher load levels.

The A Priori FE model was refined to reflect some observations made regarding the structure through data analysis. Using the adjusted model, an HS20 truck load was applied to the widened region both at midspan and over the fire affected region. Comparing these demands with the available capacity of the members indicated substantial reserve capacity, further supporting the conclusion that the structure may be reopened for the prescribed time period of approximately six weeks.

It is critical to note that all of these analyses assume that there is no appreciable residual stress in the steel from the heating and quenching that occurred immediately after the fire. It is not possible to measure or estimate residual stress without expensive Non-Destructive Evaluation methods or destructive testing including removing samples from the structure. Typically, high residual stresses would exacerbate fatigue issues. Since the bridge is only remaining open for approximately six weeks, fatigue is not a concern and ignoring residual stresses is an acceptable assumption.

APPENDIX 3B
DEVELOPMENT OF NEW STANDARDS FOR NEW MATERIALS AND CONSTRUCTION TECHNIQUES

Proposed Internal Curing High Performance Concrete Specification

Replace subsection 901.06.03 with the following:

901.06.03 Lightweight Aggregate

Manufacture lightweight aggregate by expanding or sintering material such as slate or shale by the rotary kiln process.

1) Grade the lightweight coarse aggregate to the size designation requirements for ¾-inch to No. 4 sieves of Table 1 of ASTM C 330. Ensure that the lightweight coarse aggregate producer has at least 5-years of experience and a record of successful production and use of such product. Submit to the ME a certification of compliance as specified in 106.07. Ensure that the lightweight coarse aggregate conforms to ASTM C 330, Table 901.06.02-1 and Table 901.06.02-2; and the following requirements:

   a) **Sodium Sulfate Soundness.** Use a lightweight coarse aggregate that has a sodium sulfate soundness loss of weight that does not exceed 10 percent after 5 immersion and drying cycles when tested according to AASHTO T 104.

   Table 901.06.03-1 Sampling for Sodium Sulfate Soundness Testing

Sieve Size	Weight
1" to 3/4"	2.20 pounds
3/4" to 1/2"	1.65 pounds
1/2" to 3/8"	1.10 pounds
3/8" to No. 4	0.66 pounds

   b) **Percentage of Wear.** Use a lightweight coarse aggregate with a loss that does not exceed 40 percent when tested according to AASHTO T 96.

2) Grade the lightweight fine aggregate to the size designation requirements for No. 4 to No. 0 sieves of Table 1 of ASTM C 1761. Ensure that the lightweight fine aggregate producer has at least 5-years of experience and a record of successful production and use of such product. Submit to the ME a certification of compliance as specified in 106.07. Ensure that the lightweight fine aggregate conforms to ASTM C 1761.

The following subsections are added to the standard specification

903.11 Internally Cured High Performance Concrete (ICHPC)

903.11.01 Composition

Produce HPC conforming to the composition requirements specified in 903.03.01, except for the following:

1. When using more than 1 admixture, ensure that they are compatible. If using admixtures from different manufacturers, submit letters from each manufacturer certifying that their admixtures are compatible with all others in the mix design.
2. Pozzolanic material maximum percentage limitations specified in 903.01 and 903.02.03 are waived for HPC mix designs.
3. In order to achieve the desired resistance to chloride penetration, provide an appropriate pozzolanic or other cementitious material, such as silica fume, fly ash, or slag in the mix design.
4. Do not use silica fume as a sole material to achieve the desired resistance to chlorides. Do not use more than 5 percent of silica fume by weight of the total cementitious material. If using fly ash in conjunction with silica fume, use 10 to 15 percent fly ash by weight of the total cementitious materials. If using slag in conjunction with silica fume, use up to 40 percent slag by weight of the total cementitious materials.
5. Replace a portion of fine aggregates with prewetted lightweight fine aggregate to supply 7 pounds of internal curing water per hundred weight of cement. Do not include the internal curing water in the calculation of a water cement ratio. Determine the quantity of lightweight aggregate by the following equation:

W_{LWA} = 0.07 (total weight of cementitious material) (1 + absorption) / (desorption)

Where:

The total weight of cementitious material is expressed in lbs
The absorption and desorption values are expressed as decimal fractions, as determined by ASTM C1761-13b
Use absorption and desorption values to compute W_{LWA} for specific source of lightweight fine aggregate

6. Use lightweight fine aggregate as specified in 901.06.03 and ASTM C1761 Standard Specification for Lightweight Aggregate for Internal Curing of Concrete.

903.11.02 Mix Design and Verification

Design an ICHPC, mix that conforms to the requirements in Table 903.11.02-1. Use the design of an approved HPC mix and replace a portion of fine aggregates with prewetted lightweight fine aggregate to supply 7 pounds of internal curing water per hundred weight of cement.

1. Replace a volume of ordinary weight fine aggregate with an equal volume of prewetted lightweight fine aggregate which provides the quantity of internal curing water determined in 903.11.01-5.

2. At least 60 days prior to concrete placement, provide the ME with a copy of the final mix design with the following information:
    a. Fine and coarse aggregate content in lb/yd^3 based on saturated surface dry (SSD) condition of all normal weight aggregates.
    b. Fine lightweight aggregate content in lb/yd^3 based on wetted surface dry (WSD) or oven dry condition.

c. Cementitious content (lb/yd^3).
d. Water content (lb/yd^3).
e. 56-day compressive strengths (psi)
f. Batch weights/quantities of all intended materials including admixtures.
g. Results of required performance tests, which are outlined in Table 903.11.02-1

3. Reach a minimum 56 day compressive strength as specified in 903.11.02-1.

Design mixes according to the ICHPC-1 criteria for use in bridge decks, parapets, and bridge sidewalks.

Table 903.11.02-1 Design and Verification Requirements for ICHPC

Performance Characteristic	Test Method	Requirements ICHPC-1
Scaling Resistance[1] @ 50 cycles (visual rating of the surface, maximum)	ASTM C 672	3
Freeze-Thaw Durability (relative dynamic modulus of elasticity after 300 cycles, minimum)	ASTM C 666 Proc. A	80%
Chloride Permeability[2] @ 56-days (coulombs, maximum)	AASHTO T 277	1000
Compressive Strength[3] @ 56-days (pounds per square inch, minimum)	AASHTO T 22	5400
Water-Cement Ratio (maximum)	–	0.40

[1] For the scaling resistance testing, moist cure specimens for 14 days and then air cure for 14 days.

[2] If the chloride permeability requirement has been achieved in 28 days, consider the chloride permeability acceptable. If the required chloride permeability is not achieved in 28 days, test the ICHPC sample at 56 days.

[3] If the compressive strength requirement has been achieved in 28 days, consider the strength acceptable. If the required compressive strength is not achieved in 28 days, test the ICHPC samples at 56 days.

In addition to verifying the compressive strength of the ICHPC mix, the ME will verify the chloride permeability testing according to AASHTO T 277. Submit 4 additional cylindrical samples, having a 4-inch diameter and a length of at least 8 inches, to the ME for this verification testing. The ME will average the values of tests on 2 specimens for each mix design.

903.11.03 Mixing

Mix ICHPC concrete as specified in 903.03.03, with the following additional preparations for the lightweight fine aggregate:

1. Construct lightweight fine aggregate stockpile(s) at the production facility in order to maintain uniform moisture throughout the pile. Using a sprinkler system approved by the ME, sprinkle the stockpile(s) uniformly and continuously with water for a minimum of 48 hours, or until the “absorbed moisture content” of the stockpile is at

least the value determined using ASTM C 1761. Allow the stockpiles to drain for 12 to 15 hours prior to use, unless otherwise directed by the ME.

2. Determine the "absorbed moisture content" and the "surface moisture content" of the stockpile utilizing NJDOT Test Method A-7. Adjust the batch weights based on the moisture contents of the aggregates.

3. Ensure that the lightweight fine aggregate manufacturer has representative at the concrete ready-mix plant for the first day of concrete placement operations to assist in the control of ICHPC mixing and placement.

4. During production, do not change the volume of the components of the mix in any way from the approved mix design. If the components must be changed, redesign and re-verify the mix.

903.11.04 Control and Acceptance Testing Requirements

With the exception that the ME may perform compression testing at 56 days, the ME will enforce the requirements specified in 903.03.05 for control and acceptance testing of non-pay adjustment Class A concrete in the fabrication of the ICHPC elements.

Produce ICHPC that conforms to the acceptance testing criteria in Table 903.05.04-1.

Table 903.11.04-1 Acceptance Requirements for ICHPC

Performance Characteristic	Test Method	Requirement
Percent Air Entrainment[1]	AASHTO T 152	6.0 ± 1.5 (No. 57/67 Aggregate) 7.0 ± 1.5 (No. 8 Aggregate)
Slump (inches)[1, 2]	AASHTO T 119	3 ± 1
Chloride Permeability @ 56-days[3, 4] (coulombs, maximum)	AASHTO T 277	2000
Compressive Strength @ 56-days[5] (pounds per square inch, minimum)	AASHTO T 22	4400

[1] If using a Type F or G admixture, change the Slump and Air Content values for the ICHPC as follows:

[1.1] Slump: 6 ± 2 inches

[1.2] Air Content: increase both the target value and tolerance percentages by 0.5

[2] For slip-formed parapet, design and produce a mix with a slump of 1 ± 1/2 inch.

[3] The ME will not test for the chloride permeability requirements for ICHPC used for Items other than bridge decks.

[4] For chloride permeability testing, the ME will mold 4 additional cylinders, taking 2 cylinders each from 2 randomly selected delivery trucks for testing at 56-days.

[5] For compressive strength testing, the initial rate for the ICHPC is 6 per lot. The retest limit is 4400 pounds per square inch.

The ME will test 2 specimens for chloride permeability and will average the results of the 2 specimens to determine the test result. The ME will perform 2 tests on each lot from samples taken from 2 randomly selected delivery trucks. The lot is eligible for 100 percent payment provided that the test results are equal to or below 2000 coulombs.

If, upon testing at 56 days, 1 or more individual test results exceed 2000 coulombs, the RE may:

1. Require that the Contractor remove and replace the defective lot, or
2. Allow the Contractor to submit a corrective action plan for approval.

The following subsections are modified in the standard specification, as shown by tracked changes:

SECTION 507 – CONCRETE BRIDGE DECK AND APPROACHES

507.02 MATERIALS

507.02.01 Materials

Provide materials as specified:	
Concrete	903.03
HPC	903.05
Non-Shrink Grout	903.08.02.A
Epoxy Grout	903.08.02.B
Internally Cured High Performance Concrete	903.11
Reinforcement Steel	905.01
4-Bar Open Steel Parapet	906.07
Bearing Pads	907.03
Preformed Joint Filler	914.01
Preformed Elastomeric Joint Assemblies	914.04.01
Strip Seal Expansion Joint Assemblies	914.04.02
Modular Expansion Joint Assemblies	914.04.03

507.04 MEASUREMENT AND PAYMENT

The Department will measure and make payment for Items as follows:

Item	**Pay Unit**
___" BY ___ " PREFORMED ELASTOMERIC JOINT ASSEMBLY	LINEAR FOOT
STRIP SEAL EXPANSION JOINT ASSEMBLY	LINEAR FOOT
MODULAR EXPANSION JOINT ASSEMBLY	LINEAR FOOT
CONCRETE BRIDGE DECK	CUBIC YARD
CONCRETE BRIDGE DECK, HPC	CUBIC YARD
CONCRETE BRIDGE DECK, ICHPC	CUBIC YARD
DATE PANEL	UNIT
CONCRETE BRIDGE SIDEWALK	CUBIC YARD
CONCRETE BRIDGE SIDEWALK, HPC	CUBIC YARD
CONCRETE BRIDGE SIDEWALK, ICHPC	CUBIC YARD

CONCRETE BRIDGE PARAPET	LINEAR FOOT
CONCRETE BRIDGE PARAPET, HPC	LINEAR FOOT
CONCRETE BRIDGE PARAPET, ICHPC	LINEAR FOOT
4-BAR OPEN STEEL PARAPET	LINEAR FOOT
___" X___" CONCRETE BARRIER CURB, BRIDGE	LINEAR FOOT
CONCRETE BRIDGE APPROACH	CUBIC YARD

Additional Reference Material

Item Number List

Construction Details	CD-507-1, CD-507-2, CD-507-3, CD-507-4, CD-507-5, CD-507-6,CD-507-7, CD-507-8, CD-507-9, CD-507-10, CD-507-11

The Department will include payment for epoxy coated reinforcement steel for the bridge approach under the item CONCRETE BRIDGE APPROACH; for other concrete items, the Department will make payment for reinforcement steel under REINFORCEMENT STEEL, REINFORCEMENT STEEL, EPOXY-COATED, and REINFORCEMENT STEEL, GALVANIZED as specified in 504.04.

The Department will measure ___" BY ___ " PREFORMED ELASTOMERIC JOINT ASSEMBLY, STRIP SEAL EXPANSION JOINT ASSEMBLY, and MODULAR EXPANSION JOINT ASSEMBLY OF the various sizes by the linear foot along the centerline, including the vertical face of curbs and tops of sidewalks and brush curbs.

The Department will make pay adjustments for surface requirements as specified in Table 507.03.02-2 and will apply to the lot volume for concrete in deck slabs and approach.

The Department will make a payment adjustment for concrete surface requirement quality in deck slabs and approach, by the following formula:

$$\text{Pay Adjustment} = Q \times BP \times PR$$

Where:

BP = Bid Price.
Q = Surface Requirement Lot Quantity
PR = percent reduction as specified in Table 507.03.02-2

Referenced Sections:

903.05.01 Composition

Produce HPC conforming to the composition requirements specified in 903.03.01, except for the following:

1. When using more than 1 admixture, ensure that they are compatible. If using admixtures from different manufacturers, submit letters from each manufacturer certifying that their admixtures are compatible with all others in the mix design.
2. Pozzolanic material maximum percentage limitations specified in 903.01 and 903.02.03 are waived for HPC mix designs.
3. In the design of HPC, in order to achieve the desired resistance to chloride penetration, provide an appropriate pozzolanic or other cementitious material, such as silica fume, fly ash, or slag in the mix design.

Do not use silica fume as a sole material to achieve the desired resistance to chlorides. Do not use more than 5 percent of silica fume by weight of the total cementitious material. If using fly ash in conjunction with silica fume, use 10 to 15 percent fly ash by weight of the total cementitious materials. If using slag in conjunction with silica fume, use up to 40 percent slag by weight of the total cementitious materials.

903.03.02 Mix Design and Verification

Design at least 1 mix to equal or exceed the required verification strengths specified in Table 903.03.06-3 for each class of concrete included on the Project. A single mix design may satisfy the requirements for more than 1 class of concrete. Compute and set up the designs according to ACI Standard 211.1 or 211.2, as applicable.

At least 45 days before the start of concrete placement, submit each mix design on concrete mix design forms provided by the ME. Identify the sources of materials and test data on the forms.

The ME will be present at the time of verification batching to confirm that the proportions and ingredients batched are according to the proposed mix designs. If directed by the ME, mix at least 3 cubic yards of concrete in a central mix plant or transit truck for verification. The ME will direct that the verification batch be mixed in the top half of the allowable slump and air content ranges. Test for and report the slump and air content of the trial batch. The ME will reject the verification batch if the slump, air content, or yield is not acceptable. Prepare at least six 4×8-inch test cylinders from each acceptable batch and cure according to AASHTO T 23 or AASHTO R 39. Between 2 and 5 days after molding, deliver the cylinders to the ME for testing. The ME will test 3 cylinders at 7 days and 3 cylinders at 28 days to determine the 7-day and 28-day compressive strengths, respectively.

At the ME’s option, verification may be done on an annual basis for a concrete plant rather than on a project-to-project basis, provided the properties and proportions of the materials do not change. If the Contractor submits written verification that the same source and character of materials are to be used, the ME may waive the requirement for the design and verification of previously approved mixes.

Provide concrete conforming to the approved mix design. If using a previously approved mix design, notify the ME at least 1 day before making the change. Do not change the source, type, or proportions of materials until approved and the requirements for design and verification have been satisfied.

903.03.03 Mixing for Central-Plant and Transit Mixing

A. **Handling, Measuring, and Batching Materials.** Mix concrete at a concrete plant that is listed on the QPL and conforms to the requirements specified in 1010.01. Ensure that the plant's location, layout, equipment, and provisions for transporting material will ensure a continuous supply of concrete to the work.

   Stockpile aggregates as specified in 901.02. Separately weigh the fine aggregate and each size of coarse aggregate into hoppers according to the amounts in the job mix design.

   Measure cement by weight, using separate scales and hoppers with a device to indicate the complete discharge of the batch of cement into the batch box or container. Ensure that the weighing hopper and scale are of adequate size, completely encased, and have provisions for locking. Operate the weighing hopper discharge gate so as to not affect the scale balance. Suspend the discharge chute, boot, or other such device from the encasement, not from the weighing hopper. Discharge the cement so that it does not lodge in the weighing hopper and there is no loss of cement by air currents. Ensure that the required cement content is added to each batch.

   Store mineral admixtures, unless pre-blended cement is supplied, at the batching plant in a separate storage facility. Batch mineral admixtures to tolerances equivalent to those specified for cement. When mineral admixtures are weighed cumulatively with the cement, add the mineral admixtures last in the batching sequence.

   When silica fume and dyes are added, demonstrate, prior to production, that the batching sequence will produce a uniform mix. If using mineral admixtures packaged in bags, empty the bag into the mix. Do not put degradable bags in the mix.

   Add chemical, air-entraining, and corrosion inhibiting admixtures to the mixing water or sand. Use a water measuring device that automatically registers and stops the flow of the water when the designated quantity has been delivered into the mixing drum.
B. **Batch Tolerances.** For individual batches, conform to the following tolerances based on the required scale reading:

   1. Cement and Mineral Admixtures: ±1.0 percent of the required weight of material or ±0.3 percent of scale capacity, whichever is greater.
   2. Aggregates 1-1/2 inches or smaller: ±2.0 percent of the required weight of material or ±0.3 percent of the scale capacity, whichever is greater.
   3. Aggregates larger than 1-1/2 inches: ±3.0 percent of the required weight of material or ±0.3 percent of scale capacity, whichever is greater.

4. Water: ±1.0 percent of the required weight of material.
5. Chemical, Air-entraining, and Corrosion Inhibiting Admixtures: ±3.0 percent of the required weight of material or ±1 ounce, whichever is greater.

C. **Delivery Tickets.** Supply a delivery ticket for each load of concrete. Ensure that the delivery ticket contains the following information:

1. Use tickets that are serially numbered and bear the printed heading of the supplier and the location of the batch plant.
2. Show the name of the Project, the name of the Contractor, the quantity and class of concrete, the batch time as imprinted on the ticket by an automatic clock, the date, and the truck number.
3. After the truck has been discharged, fill in the time when the concrete was completely discharged, the amount of mixing water and the amount of tempering water, if used, and the total number of mixing revolutions for transit mix.
4. An authorized representative of the supplier shall sign each ticket and give copies to the ME and the RE.

In addition, for each truck or batch, provide a batching ticket to the ME, indicating the amount, brand name, and type of cementitious material; the amount and source of the fine aggregate; the amount, sizes, and sources of the coarse aggregates; the amount of mixing water; and the amounts, brand names, and types of admixtures.

D. **Mixing Requirements.** Do not allow the elapsed time from batching to the discharge of all the concrete from the mixer to exceed 90 minutes, except that under conditions contributing to quick stiffening of the concrete or when the temperature of the concrete is above 85 °F, the time limit is changed to 60 minutes. Under very severe conditions, the RE may further reduce the time limits. Measure batching time from the time cement is introduced to the mixer.

If the concrete cannot be entirely discharged within 10 minutes, keep the concrete in the drum plastic and workable by revolving the truck drum at the manufacturer's designated speed for agitation for at least 2 minutes in each 10 minute period.

Use one of the following mixing methods unless mixing on the Project as specified in 903.03.04:

1. **Mixing at a Central-Mixing Plant.** For central-mix concrete, proportion and mix concrete at a central plant and transport to the point of use in an agitator approved by the ME. If approved by the ME, non-agitating vehicles may be used to transport concrete at precast/prestressed concrete fabricators. Use central-mixing plant mixers that are of the type and capacity capable of combining the required materials into a thoroughly mixed and uniform mass within the specified mixing time and of discharging the mixture with a

satisfactory degree of uniformity. Operate the plant according to N.J.A.C 7:27-6.1 *et seq*.

Mix for at least 1 minute, with mixing time measured from the time all cement and aggregates are in the drum. Charge the batch into the mixer so that sufficient water enters in advance of cement and aggregates to prevent caking. Ensure that all water is in the drum by the end of the first quarter of the mixing time.

When the temperature of the mixing water exceeds 100 °F, modify the loading sequence by mixing all the water and the aggregates and then the cement. Begin mixing immediately following the complete charging of the drum, and continue for not less than 1 minute.

Restrict the volume of mixed concrete in the agitating truck to not exceed the manufacturer's rating or 80 percent of the gross drum volume, whichever is less.

Before acceptance testing, the Contractor may add mixing water, air entraining agent, or chemical admixture incrementally in order to achieve the proper slump or air content range as specified in Table 903.03.06-1 or Table 903.03.06-2.

2. **Transit Mixing.** For transit mix concrete, proportion materials, including water, into a truck mixer from a 1-stop or 2-stop batching plant and mix in the truck. A one-stop batching plant is a plant where the dry ingredients for each batch of concrete are loaded into the mixer truck while water is being introduced. A 2-stop batching plant is a plant where the ingredients for each batch of concrete are loaded into the mixer truck at 2 separate locations.

   When loaded for mixing concrete, restrict the volume of concrete to no more than 63 percent of the gross drum volume of the transit truck mixer.

   Immediately begin mixing after the complete charging of the drum and continue for not less than 50 or more than 100 revolutions of the drum at the mixing speed recommended by the manufacturer of the transit truck mixer. After completing the minimum number of mixing revolutions at the plant, reduce the speed of the drum to the agitation speed recommended by the manufacturer. When using Type F or G admixtures, mix the load at the minimum specified number of mixing revolutions as recommended by the manufacturer.

   Before acceptance testing, the Contractor may add mixing water, air entraining agent, or chemical admixture incrementally in order to achieve the proper slump or air content range as specified in Table 903.03.06-1 or Table 903.03.06-2.

E. **Rejection Criteria.** The RE will reject concrete for any of the following reasons:

1. The information for batching and delivery tickets is not complete, does not agree with the mix design, or is not supplied to the ME.
2. The mixer fails to maintain the manufacturer's stated speed of rotation for both mixing and agitation, or is not able to properly discharge the concrete.
3. The RE observes improper batching, lack of uniform distribution of constituents throughout the load, or balling of the cement and aggregates.
4. Water has been added while the truck is en route to the work site.
5. The concrete is not discharged within the specified time limit, or if the revolution counter shows a total of more than the 300 revolutions. However, if the load has been partially discharged and if the concrete yet to be discharged conforms to the specified ranges for slump and entrained air without further addition of water or admixtures, then the RE may allow the use of the concrete.
6. The slump or air content does not comply with requirements specified in 903.03.05.C.
7. The concrete has been tempered after the ME has performed the final acceptance testing.
8. Water is added after the truck has partially discharged regardless of ME testing.
9. The indicator on the revolution counter shows that the instrument has been turned off or tampered with.
10. The temperature of the concrete does not comply with requirements.
11. The water-cement ratio of the load is greater than the allowable maximum water-cement ratio for the class of concrete.

903.05.04 Control and Acceptance Testing Requirements

With the exception that the ME may perform compression testing at 56 days, the ME will enforce the requirements specified in 903.03.05 for control and acceptance testing of non-pay adjustment Class A concrete in the fabrication of the HPC elements.

Produce HPC that conforms to the acceptance testing criteria in Table 903.05.04-1.

Table 903.05.04-1 Acceptance Requirements for HPC		
Performance Characteristic	**Test Method**	**Requirement**
Percent Air Entrainment[1]	AASHTO T 152	6.0 ± 1.5 (No. 57/67 Aggregate) 7.0 ± 1.5 (No. 8 Aggregate)
Slump (inches)[1, 2]	AASHTO T 119	3 ± 1
Chloride Permeability @ 56-days[3, 4] (coulombs, maximum)	AASHTO T 277	2000
Compressive Strength @ 56-days[5] (pounds per square inch, minimum)	AASHTO T 22	4400

[1] If using a Type F or G admixture, change the Slump and Air Content values for the HPC as follows:

[1.1] Slump: 6 ± 2 inches

[1.2] Air Content: increase both the target value and tolerance percentages by 0.5

[2] For slip-formed parapet, design and produce a mix with a slump of 1 ± 1/2 inch.

[3] The ME will not test for the chloride permeability requirements for HPC used for Items other than bridge decks.

[4] For chloride permeability testing, the ME will mold 4 additional cylinders, taking 2 cylinders each from 2 randomly selected delivery trucks for testing at 56-days.

[5] For compressive strength testing, the initial rate for the HPC is 6 per lot. The retest limit is 4400 pounds per square inch.

The ME will test 2 specimens for chloride permeability and will average the results of the 2 specimens to determine the test result. The ME will perform 2 tests on each lot from samples taken from 2 randomly selected delivery trucks. The lot is eligible for 100 percent payment provided that the test results are equal to or below 2000 coulombs.

If, upon testing at 56 days, 1 or more individual test results exceed 2000 coulombs, the RE may:

1. Require that the Contractor remove and replace the defective lot, or
2. Allow the Contractor to submit a corrective action plan for approval.

NJDOT A-7 – DETERMINING MOISTURE CONTENT OF LIGHTWEIGHT FINE AGGREGATE

A. **Scope**. This test method is used to determine the total, absorbed and surface (free) moisture of lightweight fine aggregate to be used for internal curing of Portland cement concrete.

B. **Apparatus**.

1. Sampling container: Non-absorbent, sealable, bag or tub with a capacity sufficient for holding approximately 2000 grams of fine aggregate.
2. Scoop, shovel, or large spoon.
3. Sheet of non-absorbent cloth, canvas or polyethylene (approximate size: 24" (600 mm) x 24" (600 mm)).
4. Drying apparatus: A ventilated oven capable of maintaining temperature of 230±9ºF (110±5ºC) for 24 hours. In cases where the aggregate is not altered by overheating, other sources of heat, such as electric or gas hotplates, electric heat lamps, or a ventilated electric microwave oven may be used.
5. Disposable brown paper towels: Commercial grade, typically manufactured from post-consumer recycled paper.
6. Heat resistant pans: With sufficient capacity to hold a minimum of 500 grams of fine aggregate in an oven or on a hot plate at the specified temperature. If a microwave oven is used for drying, the container shall be non-metallic.

C. **Sampling.**

Stockpile sample: For determination of moisture and absorption content at the ready mix concrete plant, prior to mixing: After soaking and draining the stockpiles, obtain a representative sample from the stockpile or plant storage bin; minimum sample size of 1500 grams. Immediately upon obtaining the composite sample, place it in a non-absorbent container to prevent loss of moisture prior to testing. Quarter the sample into four sub-samples of approximately 350 grams each.

D. **Procedure**. Perform the following steps:

Total moisture content:

1. Weigh one sub-sample to the nearest 0.1 grams, to be known as "Sample #1".
2. Record weight of Sample #1 as W_T.
3. Dry Sample #1 to a constant mass to the nearest 0.1 percent.
4. Record weight of dried Sample #1 as W_{1OD}.

Absorbed moisture content:

1. Place another sub-sample, labeled as "Sample #2", on a 2-3 foot long sheet of clean, dry paper towel.
2. Spread Sample #2 uniformly across the paper towel while patting the sample with another paper towel. Continue patting and spreading the sample, replacing the sheets of paper towel whenever the paper becomes too damp or dirty to absorb moisture. Conduct this process as quickly and carefully as possible. Repeat the patting and spreading of the sample until no further moisture appears on the clean paper towels.
3. Weigh Sample #2 to the nearest 0.1 gram.
4. Record weight of Sample #2 as W_{WSD}.

5. Dry Sample #2 to a constant mass to the nearest 0.1 percent.
6. Record weight of Sample #2 as W_{2OD}.

E. **Calculations and Report**.

1. Calculate the "% Total Moisture" Content of Sample #1 (expressed as a percent of the oven dried weight) as follows:

$$\%\,Total\ Moisture: (M_T) = 100\% \text{ x } \frac{W_T - W_{1OD}}{W_{1OD}}$$

2. Calculate the "% Absorbed Moisture" Content of Sample #2 as follows:

$$\%\,Absorbed\ Moisture: (M_A) = 100\% \text{ x } \frac{W_{TD} - W2_{OD}}{W2_{OD}}$$

3. Calculate "% Free Water" Content as follows:

$$\%\,Free\ Water: (M_S) = Total\ Moisture\ (M_T) - Absorbed\ Moisture\ (M_A)$$

Where:
W_T = Total weight of Sample #1 measured in Step 2.
W_{1OD} = Oven-dry weight of Sample #1 measured in step 4
W_{WSD} = Wetted Surface dry weight of Sample #2 measured in step 4
W_{2OD} = Oven-dry weight of Sample #2 measured in step 6

F. **Report**. Report % total moisture, % absorbed moisture and % Free Water to the nearest 0.1%.