SEISMIC ANALYSIS OF RETAINING WALLS, BURIED STRUCTURES, EMBANKMENTS, AND INTEGRAL ABUTMENTS

FINAL REPORT
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Submitted by

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16. Abstract

This study evaluates the impact of the newly recommended seismic design guidelines from NCHRP 12-49 on seismic design of bridges in New Jersey. It also provides seismic design criteria and guidelines for integral abutments, retaining walls, embankments, and buried structures. The study provides an overall review of the recommended guidelines and compares them to the current AASHTO LRFD specifications. It provides recommendations on seismic hazard and performance objectives and soil site factors for New Jersey that incorporates design criteria from NCHRP 12-49 guidelines, AASHTO LRFD specifications, South Carolina seismic design criteria, and NYCDOT seismic design guidelines. The study also includes two design examples based on the NCHRP 12-49 guidelines and current AASHTO LRFD specifications. Research results showed that: (1) the MCE ground motion level adopted by NCHRP 12-49 which has a 2500 years return is acceptable for safety evaluation of 'critical bridges' in New Jersey, (2) a reduced (2/3 MCE) ground motion is acceptable for safety evaluation of 'non-critical' bridges, (3) soil-site factors have increased dramatically for soft soils subjected to small ground motions which will have an impact on seismic design in Southern Jersey, (4) the USGS National Seismic Hazard Maps adopted by NCHRP 12-49 for ground motion accelerations may not necessarily reflect the actual geological soil conditions and realistic hazard levels in New Jersey, and (5) NCHRP12-49 SDAP E (pushover analysis) is preferable for the seismic analysis and design of bridges in New Jersey, and (6) NCHRP12-49 SDAP C is a relatively simplified design procedure for many bridges and should be used when applicable. Recommendations from this study include adoption of NCHRP 12-49 subject to above conclusions. However, there is a need to: (1) predict extreme earthquake events for New Jersey and the Northeast United States, (2) prepare Seismic Hazard Maps for bridge design in New Jersey and re-evaluate NCHRP 12-49 soil-site factors proposed for New Jersey, and (3) quantify damage level by using structural capacity and demand.

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Acknowledgements

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SUMMARY

In 1998, the National Cooperative Highway Research Program (NCHRP) initiated a project to develop a new set of seismic design provisions for highway bridges intended to be compatible with the AASHTO LRFD Specifications [1]. This project, designated 12-49, was conducted by a joint venture of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER). This research project was needed to reflect the experience gained during recent damaging earthquakes, as well as the results of research work conducted in the United States, Japan, and other countries over the last decade [2]. Recommended LRFD Guidelines for the Seismic Design of Highway Bridges [3] were based on NCHRP Project 12-49. The purpose of the new NCHRP 12-49 provisions is to provide seismic design guidelines and performance objectives for bridges in order to ensure the safety of the public, and to minimize structural and non-structural damage. In recent years, several major bridges have collapsed and others have sustained significant damage during earthquakes [2].

The NCHRP 12-49 guidelines adopted the MCE (maximum considered earthquake or 2 percent PE on 50 years) as an upper level event for collapse-prevention and adopted the EXP (expected) earthquake (50 percent in 75 years) as a lower level event for which the structure essentially remains elastic. These changes in the newly recommended guidelines will have a major impact on seismic design of bridges in the Eastern United States. Several states, including New Jersey, are evaluating the impact of these changes on their local, state, and federal bridges. In addition, soil amplification factors F_a and F_v have increased dramatically for soft soils, especially when subject to small ground motions. These factors are not site-specific to the Eastern United States and were based on soils and earthquake records predominantly in the Western United States [See references 3,4,5, and 7]. These factors may vary for different soils, geographic locations, and ground motions. Among the other major changes in the new NCHRP 12-49 seismic design provisions are updated seismic maps, new response modification factors (R), detailed performance and hazard level criteria, and design incentives when performing "pushover" analysis. These provisions are intended to help bridge owners and state officials with current designs and provide designers more flexibility in the analysis and design.

The main objectives of this study are: 1) perform a comprehensive review of the new provisions proposed in NCHRP12-49 and examine its new guide documents and seismic design methods for abutments and retaining structures, 2) provide guidelines for the seismic design of retaining structures, such as walls and abutments, buried structures, and embankments, 3) provide procedures to analyze, design, and detail freestanding abutments and integral abutments for seismic design with examples, and 4) provide specifications for the seismic

design and detailing of abutments, retaining structures, embankments consistent with the proposed provisions in NCHRP 12-49 report.

The new seismic design provisions proposed in the NCHRP Report 12-49 are substantially different and more complex than the existing provisions. It includes many new concepts and several major modifications. The basic design philosophy behind the NCHRP 12-49 guidelines is to design explicitly for ground motion accelerations associated with larger events (2500-year event), but also to refine the current design provisions to reduce the conservatism. This approach is in contrast to the current AASHTO LRFD specifications which require design for a moderate ground motion event (500-year event) while using conservative design provisions and detailing, based on engineering judgment to protect against larger earthquake events. The margin of safety in the current AASHTO LRFD specifications is not likely to be more than 1.5. If the ground motion during an actual event were higher than the design event (500 years) by two to three times, the structure will suffer significant damage. Whether the risk of designing for the 500-year event is acceptable or not depends on the probability of occurrence and the consequences of a higher event on the performance of the structure. To evaluate and quantify these risks and consequences, the higher event and the extra margin of safety available in the structure must be known. In addition to adopting MCE as the upper level event, the NCHRP 12-49 provisions also require designing for a lower level event or the expected earthquake (EXP), which has 50 percent PE in 75 years (108 years return period). The design for this lower level earthquake requires that the structure remains basically elastic (minor to minimal damage). This requirement is consistent with AASHTO LRFD requirements for the 100-year wind and flood designs.

This study provides an overall review of the new recommended seismic design guidelines from the NCHRP Report 12-49; compares the guidelines to the current AASHTO LRFD specifications; and provides seismic hazards and performance objectives and soil site factors for the State of New Jersey. The study also provides seismic design criteria and guidelines for integral abutments, retaining walls, embankments, and buried structures consistent with the newly recommended guidelines. The study also includes two design examples based on the NCHRP 12-49 guidelines and current AASHTO LRFD specifications. The NCHRP 12-49 provisions have recommended significant changes to the current LRFD Specifications.

The main conclusions from this research include the following: 1) the MCE (Maximum Considered Earthquake) level adopted by NCHRP 12-49, which has a 2 percent PE in 50 years (2500 years return period) is an acceptable level for safety evaluation for all new bridges in New Jersey, 2) soil site factors have

increased dramatically for soft soils subjected small ground motions. These factors will have a major impact on the design response spectra and the selection of the seismic hazard level in Central and South Jersey, 3) new NCHRP 12-49 guidelines provide many options for seismic analysis and design procedures that will help bridge designers. For example, SDAP C, D, and E could be used in most cases and the reduction factors of elastic seismic loads (R) are tabulated in more detail for various cases. It also provides additional information on the analysis of integral and seat abutments, and foundation stiffness more than the current specs, 4) Transverse column reinforcement in plastic hinge zones is significantly affected by the longitudinal steel ratio. This reinforcement is independent of the longitudinal steel in the existing provisions.

Among the research recommendations from this study are the following: 1) Adopt seismic hazard and performance levels based on NCHRP 12-49 ground motions with modifications to be consistent with those adopted by NYCDOT and SCDOT. , 2) Safety Level design for 'critical bridges' shall be based on the MCE event, while Operational Level design shall be based on the 500-year event, 3) Safety level design for 'non-critical bridges' (Other bridges) shall be based on 2/3 of the MCE with minimum seat width at abutments and expansion piers be based on the MCE rather than 2/3 of the MCE, and 4) seismic design criteria and guidelines for integral abutments, retaining walls, embankments, and buried structures in New Jersey consistent with the provisions of NCHRP 12-49.

INTRODUCTION

In 1998, the National Cooperative Highway Research Program (NCHRP) initiated a project to develop a new set of seismic design provisions for highway bridges intended to be compatible with the AASHTO LRFD Specifications [1]. This project, designated 12-49, was conducted by a joint venture of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER). This project was initiated to reflect the experience gained during recent damaging earthquakes, as well as the results of research work conducted in the United States, Japan, and other countries over the last decade [2]. Recommended LRFD Guidelines for the Seismic Design of Highway Bridges [3] were based on NCHRP Project 12-49.   The purpose of this study is to evaluate the impact of the new seismic design provisions proposed in the NCHRP Project 12-49 for the seismic design and detailing of bridges in New Jersey. Two bridges on soft soils were designed in three different regions in New Jersey based on the NCHRP 12-49 recommended guidelines and the current AASHTO LRFD specifications. The purpose of the new NCHRP 12-49 provisions is to provide seismic design guidelines and performance objectives for bridges in order to ensure the safety of the public, and to minimize structural and non-structural damage. In recent years, several major bridges have collapsed and others have sustained significant damage during earthquakes [2].

The NCHRP 12-49 provisions have recommended significant changes to the current LRFD Specifications. Among the major changes in the new NCHRP 12-49 seismic design provisions are ground motion accelerations based on new USGS maps, new soil site factors, and new response modification factors (R). The new guidelines provide more detailed performance and hazard level criteria to help bridge owners and state officials with current designs; provide designers more flexibility in analysis and design; and, provide updated seismic maps.

Background

In 1997, a special Seismic Procedure Design Group (SPDG) was formed by NEHRP (National Earthquake Hazards Reduction Program) and BSSC (Building Seismic Safety Council) to work with USGS for the purpose of incorporating the latest seismic design procedures with new USGS maps. The SDPG determined that rather than designing for a uniform event or hazard nationwide, it makes more sense to design for a uniform margin of safety against an arbitrarily selected maximum earthquake. The SPDG selected the 2 percent PE (probability of exceedance) of a 50 years event as the most severe earthquake that is practical to design for. This earthquake also known as the MCE (Maximum Considered Earthquake). However, the SPDG had to address locations near major active faults where the predicted MCE was much higher than the commonly recorded events, and the buildings designed in those areas had a substantial margin of safety against collapse. This margin of safety was estimated by the SDPG to be 1.5, and it was decided that near major active faults, the MCE should not exceed 150 percent of the mean ground motion obtained from the deterministic characteristic earthquake near the active faults. It was also agreed that all buildings should be designed for 1/1.5 or (2/3) of the MCE for life-safety.

Although the 1997 and 2000 NEHRP Recommended Provisions for the Seismic Regulations of New Buildings and Other Structures [4,5,6] adopted the (2/3) MCE for design, the *NEHRP Guidelines* did not directly adopt the concept of the design at (2/3) of MCE, based on the fact that this would result in different probabilities of exceedance across the nation. The NCHRP 12-49 guidelines adopted the MCE (2 percent PE on 50 years or 3 percent PE in 75 years) as an upper level event for collapse-prevention and adopted the EXP earthquake (50 percent in 75 years) as a lower level event for which the structure essentially remains elastic. These changes in the newly recommended guidelines will have a major impact on seismic design of bridges in the Eastern United States. Several states, including New Jersey, are evaluating the impact of these changes on their local, state, and federal bridges. In addition, soil amplification factors F_a and F_v have increased dramatically for soft soils, especially when subject to small ground motions. These factors are not site-specific to the Eastern United States and were based on soils and earthquake records predominantly in the Western

United States [See references 3,4,5, and 7]. These factors may vary for different soils, geographic locations, and ground motions. The USGS seismic hazard map of the continental United States is shown in Figure 1. The map illustrates the earthquake hazard in New Jersey, which varies from small hazards in Southern New Jersey to a moderate one in Northern New Jersey and neighboring New York City.
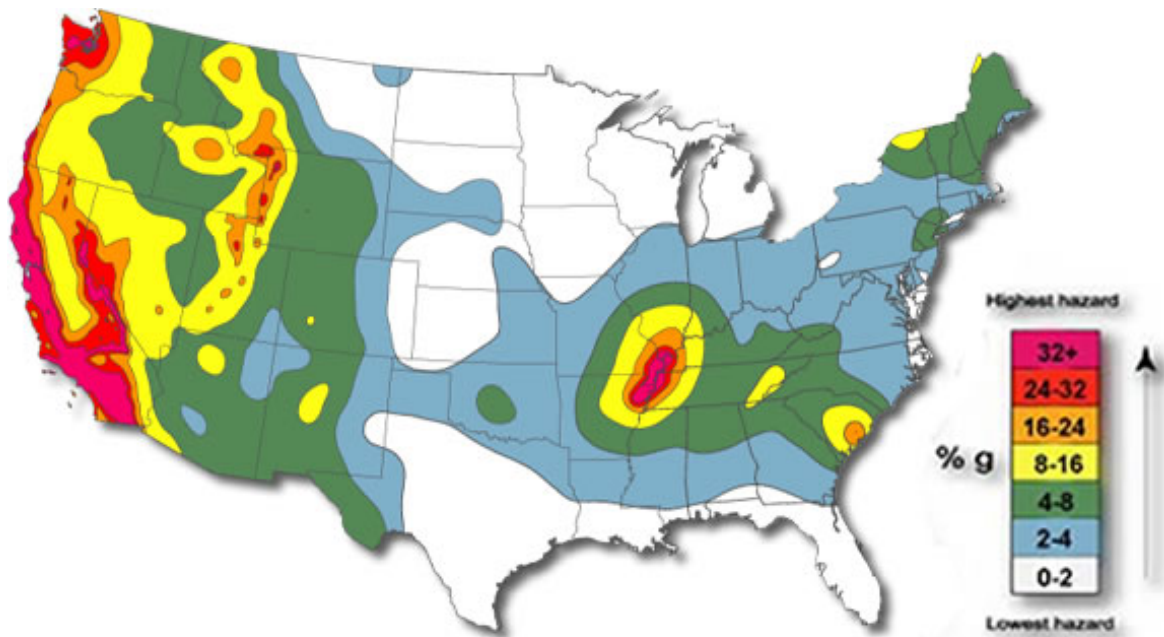
Figure 1. USGS Seismic Hazard Map for the Continental United States.

The seismic history of the Northeastern United States between 1638 and 1998 yields a fairly low to moderate seismic zone. More than 1,000 earthquakes have hit the Northeastern United States over the last 360 years according to the U.S. Geological Survey (USGS) and Northeast States Emergency Consortium (NESEC). There were several significant earthquakes that hit the Northeastern United States with a magnitude 5.0 or higher. In 1755, a 6.0 magnitude earthquake hit the Cape Ann area north of Boston, MA and in 1884 a 5.0 magnitude earthquake hit the shores between Brooklyn, NY and New Jersey. The more recent ones were the Pymatuning Reservoir earthquake in Pennsylvania in 1998, which had magnitude of 5.2 and the Plattsburg earthquake in Upstate New York, which had a magnitude of 5.1. Figure 2 shows the seismic history of New Jersey and the New York Metropolitan area.
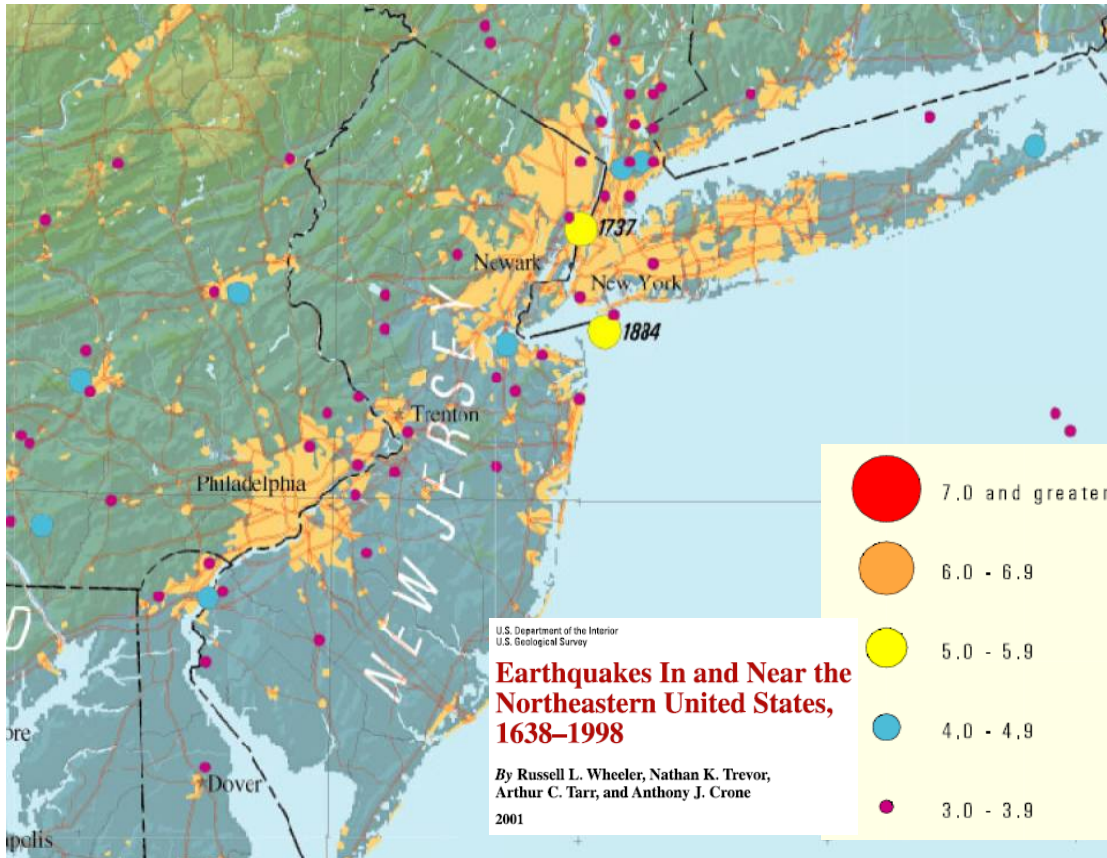
Figure 2. Seismic history of New Jersey, Metro New York, and Long Island from 1638 to 1998.

The current guidelines for seismic design of bridges in New Jersey [8] are based on the AASHTO LRFD Specifications. For seismic design of deep foundation, NJDOT requires the AASHTO Standard Specifications (16th edition) [9].

For Turnpike bridges, the specifications require two-level design: (1) Safety Evaluation Event or SEE (2500 years), and (2) Functional Evaluation Event or FEE (500 years). Local road bridges over the Turnpike, including State and Federal Highways, are designed based on the lesser of the two events. For the 500 years event (10 percent PE in 50 years), all substructure elements will be designed using an R-factor equal to 1.5. For the design of certain special bridges that are deemed "critical" by the Turnpike Authority, the design shall be based on the 2500 years event using an R-factor equal to 1.5 for all substructure elements. According to the proposed provisions in NCHRP 12-49, the lower level or expected earthquake (108- year return period) in New Jersey has a very small horizontal spectral acceleration S_s and S_1. This event will not control the design, and the non-seismic design of the bridge will be adequate to resist the expected earthquake loads. Therefore, no seismic design is required for the lower level

event or the expected earthquake in New Jersey. However, for very soft soils (NCHRP 12-49 Soil E and F) the liquefaction potential must be investigated. For stiff soils and for the short period range, the 2,500 years event (MCE) has larger design accelerations compared to the current LRFD specs (500 years event) in New Jersey. For small ground motions and very soft soil sites, the seismic design loads from NCHRP 12-49 are significantly increased, based on higher accelerations and higher soil site factors. This will have an impact of the seismic design of bridges in Southern New Jersey.

In New York City, which is in close proximity to Northern New Jersey, the NYCDOT seismic design guidelines include two-level design for earthquakes. Level one is an ODE (Operating Design Earthquake) and level two is a MDE (Maximum Design Earthquake). These levels are based on hard rock ground motion with a 10 percent probability of being exceeded in 50 years (500 year return period) for level one; and based on hard rock ground with a 2 percent probability of being exceeded in 50 years (2,500 year return period) for level two. The guidelines also specify a single hazard level depending on the performance level of the structure. This level is based on (2/3) of the MDE. i.e. (2/3) of the hard rock ground motion with a 2 percent probability of being exceeded in 50 years (2,500 year return period) and is applicable to bridges classified as "Essential" and "Others". The soil amplification factors specified by the New York City Department of Transportation (NYCDOT) are similar to those in the 1997 NEHRP Guidelines [4,5]. Similarly, the NCHRP Report 12-49 adopted the same soil site factors as they appear in the 1997 NEHRP Guidelines [4, 5].

In South Carolina, current seismic design criteria [10] require two seismic design levels for critical bridges and a single seismic design level for normal and essential bridges in the state. The two levels are a lower-level event or FEE, which is based on a design spectra for an earthquake with 10 percent PE (probability of exceedance) in 50 years; and an upper-level event, or SEE, which is based on a design spectra for an earthquake with 2 percent PE in 50 years. The single level for normal and essential bridges is the SEE event. The minimum performance level for service and damage expected will depend on the particular bridge, as given in SCDOT design criteria [10]. The SCDOT specs require the design to follow these requirements in conjunction with the AASHTO Standard Specifications (1996) Division I-A, some Caltrans Seismic Design Criteria [11], soil site factors similar to those in the NEHRP 1997 Guidelines, and Seismic Hazard Maps developed specifically for the state of South Carolina.

Objectives

The objectives of this study are:

1. Perform a comprehensive review of the new provisions proposed in NCHRP12-49 and examine its new guide documents and seismic design methods for abutments and retaining structures.

2. Provide guidelines for the seismic design of retaining structures, such as walls and abutments, buried structures, and embankments.

3. Provide procedures to analyze, design, and detail freestanding abutments and integral abutments for seismic design with examples.

4. Provide specifications for the seismic design and detailing of abutments, retaining structures, embankments consistent with the proposed provisions in NCHRP 12-49 report.

EVALUATION OF NCHRP PROJECT 12-49 GUIDELINES FOR SEISMIC DESIGN OF BRIDGES IN NEW JERSEY

Brief Review Of NCHRP 12-49 Guidelines

The new seismic design provisions proposed in the NCHRP Report 12-49 are substantially different and more complex than the existing provisions. It includes many new concepts and several major modifications. Among the new concepts and major changes in the proposed NCHRP 12-49 provisions are: (1) *adoption of the 2002 USGS Maps* as the basis for rock ground motion of seismic design. The parameters obtained from these maps include the peak ground acceleration (PGA), elastic response ground acceleration for 0.2 sec, 0.3 sec, and 1.0 sec periods of vibrations. These values are available in three different probabilities: 10 percent in 50 years, 5 percent in 50 years, and 2 percent in 50 years (2 percent in 50 years is approximately 3 percent in 75 years), (2) *design earthquakes and performance criteria*: the proposed provisions provide two design earthquakes with definite performance objectives and design checks: an upper level event termed the "rare" or Maximum Considered Earthquake (MCE) which has a 3 percent PE in 75 years; and a lower level event termed the "Expected" earthquake, which has a 50 percent PE in 75 years. The "Rare" earthquake, or MCE, governs the limits of inelastic deformations in the substructure and the design displacements in the superstructure, while the "expected" earthquake event essentially assures an elastic response of the structure with minimum or no damage, (3) *new soil site factors*, where soil sites are classified based on the average shear wave velocity, SPT blow-count, and untrained shear strength in the upper 30 m of the site profile, (4) *new spectral shapes,* where the long-portion of acceleration response spectrum is governed

by a spectral shape that decays as $1/T$ rather than $1/T^{2/3}$, (5) *allowing ERS and ERE* (Earthquake Resisting Element) that are not currently specified in current AASHTO LRFD, (6*) four seismic hazardous levels I, II, III, and IV* based on the design earthquake response spectral acceleration at short periods, $S_{DS}= F_v S_1$ and at long periods, $S_{D1}=F_a S_s$, (7) *five defined SDAP's (Seismic Design and Analysis Procedures) A, B, C, D, and E* that reflect the variation in seismic risk are based on seismic hazard level, performance objective, structural configuration, and types *of ERS (Earthquake Resisting System) and ERE (Earthquake Resisting Element)* used, (8) *six defined SDR's (Seismic Detailing Requirements)*, and (9) *design incentives when performing "pushover" analysis*, in which higher values of the response modification factors (R) are allowed.

NCHRP 12-49 Design Philosophy

The basic design philosophy behind the NCHRP 12-49 guidelines is to design explicitly for ground motion accelerations associated with larger events (2500-year event), but also to refine the current design provisions to reduce the conservatism. This approach is in contrast to the current AASHTO LRFD specifications which require design for a moderate ground motion event (500-year event) while using conservative design provisions and detailing, based on engineering judgment to protect against larger earthquake events. The margin of safety in the current AASHTO LRFD specifications is not likely to be more than 1.5. If the ground motion during an actual event were higher than the design event (500 years) by two to three times, the structure will suffer significant damage. Whether the risk of designing for the 500-year event is acceptable or not depends on the probability of occurrence and the consequences of a higher event on the performance of the structure. To evaluate and quantify these risks and consequences, the higher event and the extra margin of safety available in the structure must be known.

The MCE earthquake or the 2500-year event chosen by NCHRP 12-49[3] is similar to that of the 2000 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures and the 2000 International Building Code, as well as the NEHRP Guidelines for the Seismic Rehabilitation of Buildings. The difference in the ratio of the 2500-year to the 500-year ground motion accelerations between the Eastern United States and high seismic zones, like California and Alaska, was one of the main reasons cited by the NCHRP 12-49 research team to adopt the new MCE design earthquake. While the ratio of the 2500-year to 500-year ground motions in high seismic zones such as California is around 1.2 to 1.8, this ratio can be as high as 3.0 to 3.8 in the Northeast (New Jersey, New York, and New England). Similarly, the ratio of the MCE to EXP ground accelerations is about 2 to 2.5 in California, while it is in the range of 13 to 15 in the Eastern United States. The NCHRP rationale was that the factor of safety, or the reserve strength for the 500-year event design in the Eastern US (currently used in AASHTO LRFD), would not be sufficient to withstand the 2500-year event. However, it is important to mention here that

some of the ground motion accelerations from high seismic zones are likely to be controlled by the deterministic bounds (1.5 times the median ground motions near active faults) and, hence, some of these comparisons may not be accurate.

In addition, the NCHRP 12-49 Report uses ground motions in the Charleston and the New Madrid regions to validate the use of the 2500-year return period (MCE) for the safety or collapse-prevention design. In these two locations, the 2500-year event ground motions were in good agreement with ground accelerations recorded from historic earthquakes in those regions. The 2500-year event, which was based on seismic source models, ground motion attenuations, and updated and improved seismicity adopted for the Northeastern United States, seems to have less justification than what was given for high seismic zones like California and the New Madrid and the Charleston regions. The lack of historic major earthquake records in the Northeastern United States is one of the main reasons for insufficient justification. Although the NCHRP 12-49 provisions were consistent with the 2000 NEHRP Guidelines for Seismic Regulations in New Buildings [4,5] and the International Building Code (IBC)[12] in choosing the MCE as the no-collapse earthquake, they did not recommend using 2/3 of that earthquake for the design. The NCHRP 12-49 rationale is that the design provisions in those documents provide a factor of safety of 1.5 against collapse. Also, the 2/3 factor was not recommended in the NCHRP 12-49 provisions to directly address and incorporate design displacements associated with MCE collapse-prevention event.

In addition to adopting MCE as the upper level event, the NCHRP 12-49 provisions also require designing for a lower level event or the expected earthquake (EXP), which has 50 percent PE in 75 years (108 years return period). The design for this lower level earthquake requires that the structure remains basically elastic (minor to minimal damage). This requirement is consistent with AASHTO LRFD requirements for the 100-year wind and flood designs. In New York City, which is in close proximity to Northern New Jersey, the NYCDOT seismic design guidelines include two-level design for earthquakes. Level one is an ODE (Operating Design Earthquake) and level two is an MDE (Maximum Design Earthquake). These levels are based on hard rock ground motion with a 10 percent probability of being exceeded in 50 years (500 year return period) for level one, and based on hard rock ground with a two percent probability of being exceeded in 50 years (2,500 year return period) for level two. The guidelines also specify a single hazard level depending on the performance level of the structure. This level is based on (2/3) of the MDE. i.e. (2/3) of the hard rock ground motion with a two percent probability of being exceeded in 50 years (2,500 year return period), and is applicable to bridges classified as "Essential". The soil amplifications factors specified by NYCDOT are similar to those in 1997 NEHRP Guidelines [4,5]. NCHRP Report 12-49 adopted the same soil site factors as in the 1997 NEHRP Guidelines. South Carolina DOT uses similar events to those used by NYCDOT. It is worth noting here that the lower

level event in NCHRP 12-49 is smaller than the lower level event specified by the NYCDOT and SCDOT for critical bridges.

NCHRP 12-49 Design Response Spectra for Various Return Periods

The NCHRP 12-49 design response spectra for various return periods in Northern New Jersey in soft soil (class E) are shown in Figure 3. This figure compares the design response spectra from NCHRP 12-49 to that from the current AASHTO LRFD. Also shown in this figure are the design spectra for 500, 1000, and 1500 year return periods, and for two-thirds of the two percent PE in 50 years event. Figure 3 clearly shows that the 500-year return period event from NCHRP 12-49 has lower spectral accelerations than those of AASHTO LRFD with the same return period for the short-period and long-period range. It also illustrates the (two-thirds) of MCE spectra, which is used by the NEHRP for retrofit of buildings and by NYCDOT and NYSDOT as a single event for the seismic design of essential and other bridges. The spectra show that bridges with short periods will be penalized heavily according to the NCHRP 12-49 guidelines, while flexible bridges would have lower forces compared to the current AASHTO LRFD specifications. Although Figure 3 represents the northern region in New Jersey with soft soils, which is the most severe case, there is a similar trend in design response spectra observed for other regions and other soil conditions in the state.

NCHRP 12- 49 Design Earthquakes for New Jersey

According to the proposed provisions in NCHRP 12-49, the lower level or expected earthquake in New Jersey, which has about 108- year return period, had a very small peak in horizontal accelerations S_s and S_1. This event will not control the design and the non-seismic design of the bridge will be adequate to resist the expected earthquake loads. Hence, no seismic design is required for the lower level event or the expected earthquake in New Jersey. For very soft soils (NCHRP 12-49 Soil E and F) liquefaction potential, however, need to be investigated. On the other hand the NCHRP 12-49 maximum earthquake (MCE) has higher accelerations than those in the current specifications.

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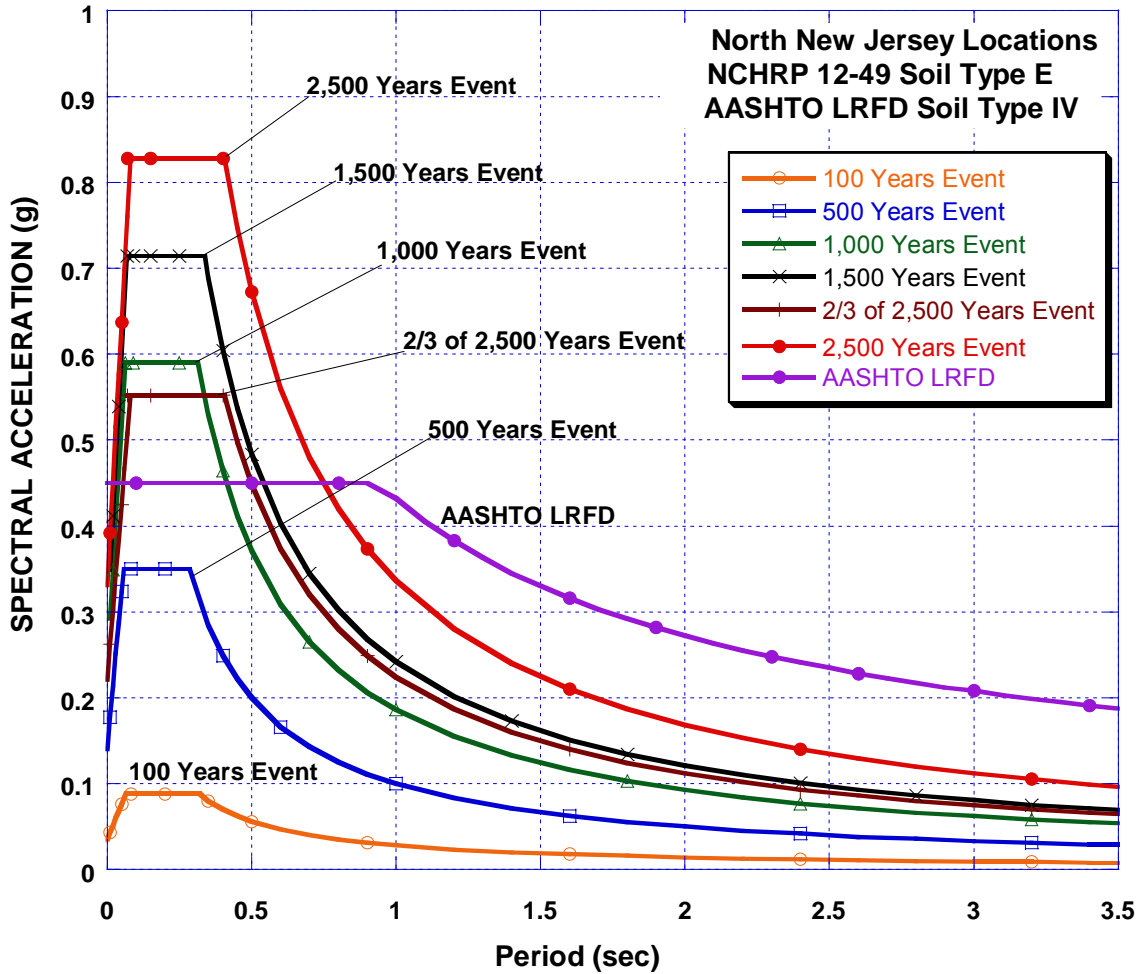
Figure 3. Design response spectra from AASHTO LRFD and NCHRP 12-49 with various return periods.

Figure 4 shows a comparison of the design response spectra of NCHRP 12-49 design earthquakes and AASHTO LRFD earthquake. This spectrum was plotted for Northern New Jersey with soft soil conditions (Soil class E). For bridges whose period of vibration is about 0.5 seconds or less, and are not "critical bridges" or Turnpike bridges (where the NJDOT requires designing for the 2500-years event), the design spectra plotted in Figure 4 clearly show that the 2500-years event in NCHRP 12-49 will require designing for accelerations that are about twice those required in the current specifications. For bridges with periods approximately between 0.5 and 0.7 seconds, the ratio of NCHRP 12-49 to AAASHTO LRFD design accelerations varies between 1.5 and 1.0.

The figure also shows that the108-year event is much smaller than the current specs and hence no seismic design is required for this event in New Jersey. The report includes design tables for various counties in New Jersey that clearly show no need for seismic design for this NCHRP 12-49 event.
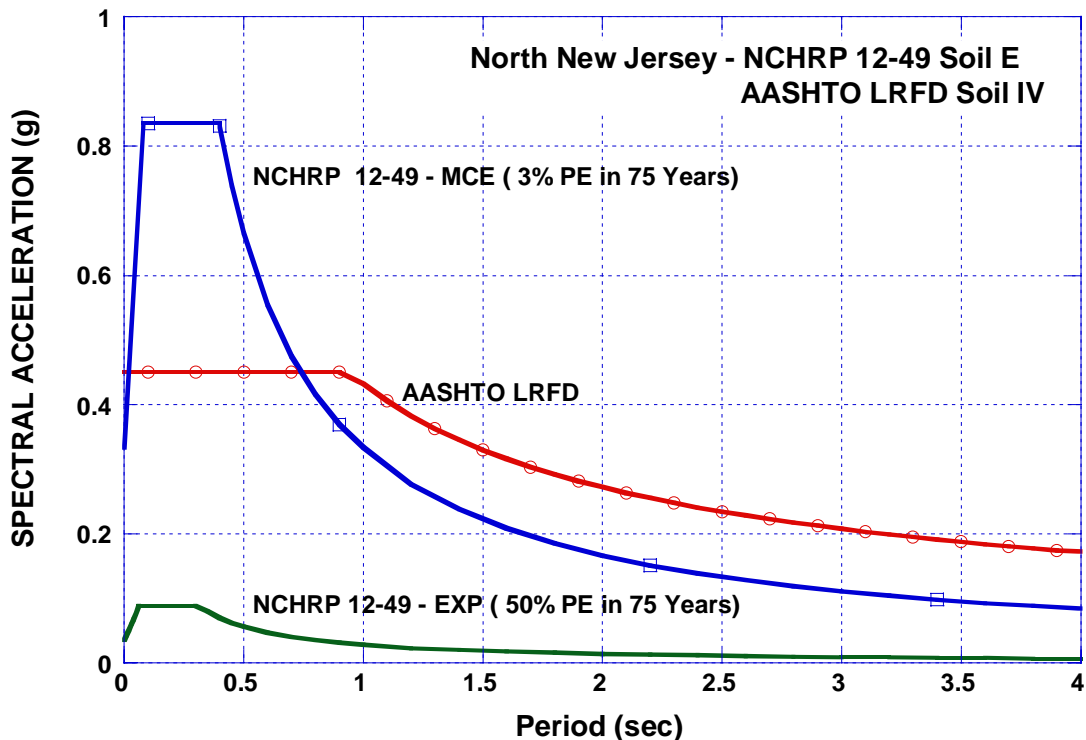
Figure 4. Design response spectra from AASHTO LRFD and NCHRP 12-49 MCE and EXP earthquakes.

Effect of Soil Type on NCHRP 12- 49 Design Earthquake

The geological map of New Jersey (1994) in Figure 5 shows the variation of rock and soil conditions across the state. The map shows stiff soil and rock conditions in the Northern region of the state and less stiff to softer soils in the Southern regions of the state. The soil site factors in NCHRP 12-49 are dependent on the soil classification and ground motion level. These factors have increased dramatically for soft soils subjected to small ground motions compared to the current specifications. These factors had a major impact on the design response spectra and the selection of the seismic hazard level. Additionally, they have an impact on the R-factors. The soil factors were based on soils from a specific region in the United States (mostly the Western United States). These soil site factors were first specified in NEHRP 1997 provisions for seismic design and retrofit of buildings. Generalized site classes and site factors were used with the general procedure for constructing response spectra. Six site classes (A, B, C, D,

E, and F) are specified depending on the average shear wave velocity for the top 100 ft of a site ($\overline{V}_S$), standard penetration test (SPT) blow count for the top 100 ft of a site ($\overline{N}$), and the undrained shear strength in the top 100 ft of a site ($\overline{S}_u$). Based on the soil class and the ground motion level, site coefficients Fa for the short-period acceleration and Fv for long-period acceleration are selected form NCHRP 12-49 Tables 3.4.2.3-1 and 3.4.2.3-2 respectively. The NYCEM (The New York City for Earthquake Loss and Mitigation) soil classification of the City of New York and the Northern region of New Jersey, according to NEHRP soil classification, are shown in Figure 6. Figure 6 illustrates that most of the northern region of New Jersey is composed of dense soils and soft and hard rock.
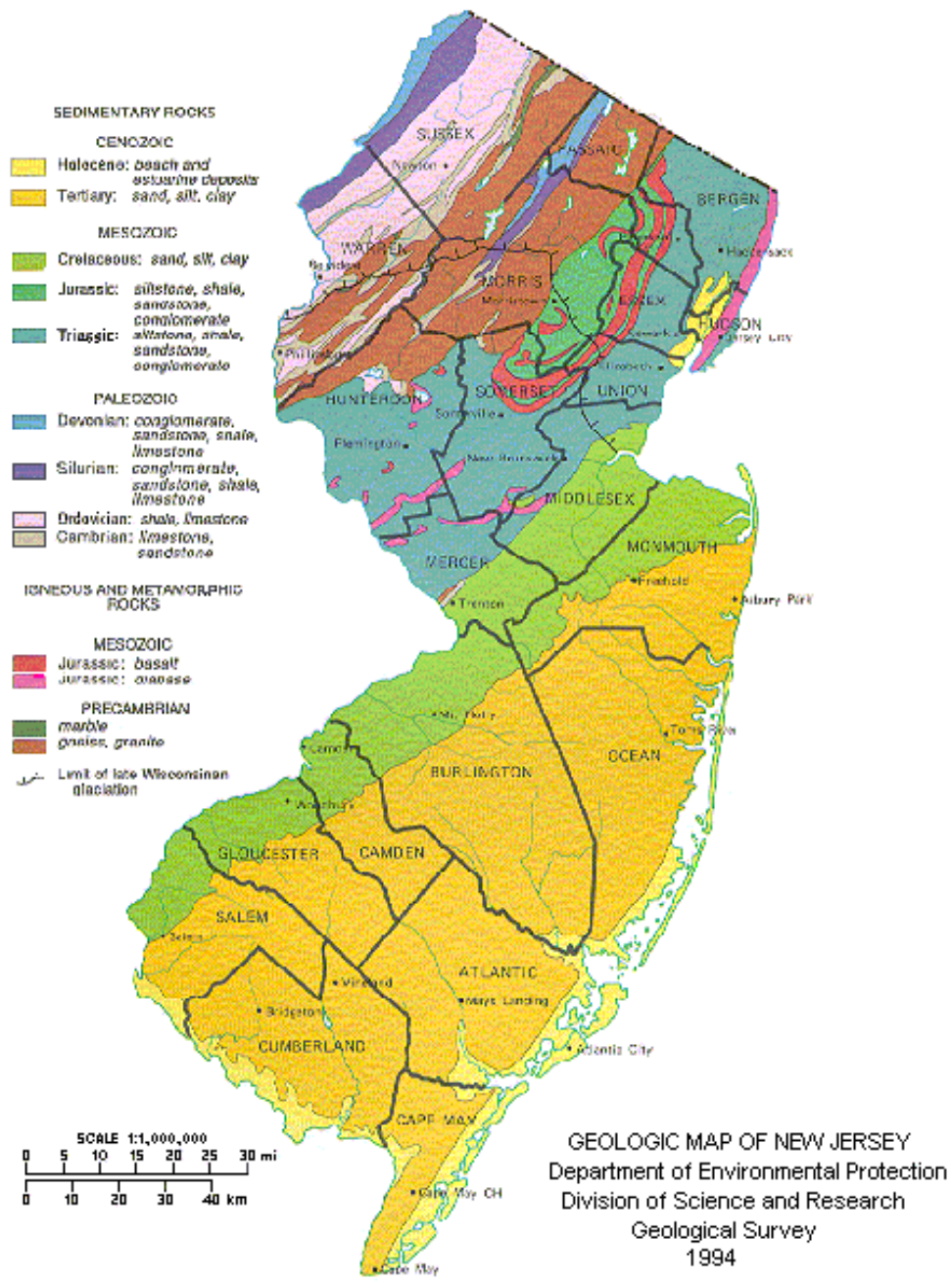
Figure 5. Geological map of New Jersey (1994).
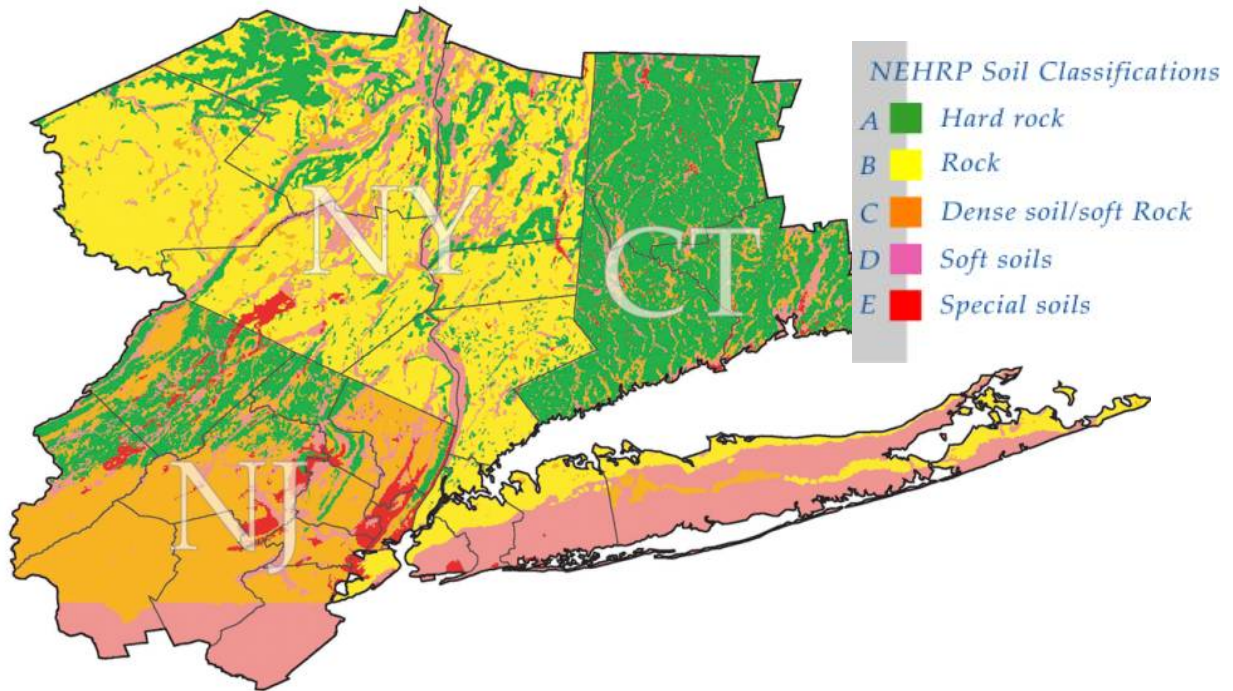
Figure 6. NYCEM Soil Characterization of NYC and Northern New Jersey
soil conditions according to NEHRP soil classification.

The effect of soil site factors in NCHRP 12-49 on the design response spectra for
the MCE is shown in Figures 7 and 8. Figure 7 shows the design response
spectra for various soil conditions in Northern New Jersey while Figure 8 shows
the design response spectra for various soil conditions in Southern New Jersey.
The spectral accelerations Ss and S_1 for North Jersey and South Jersey are 0.43
and 0.27 respectively. These values are obtained from the USGS maps. The
spectral acceleration for North Jersey is about 60 percent of that of South Jersey.
When the effect of the soil is included, the design spectral accelerations FaSs
become 0.83 and 0.65 for northern and southern New Jersey respectively. The
ratio of these two accelerations drops to 27 percent from the 60 percent without
including the effects of soils. The NCHRP 12-49 guidelines have increased the
design accelerations for low seismic zones by specifying higher soil factors for
those zones. The soil factors in the current AASHTO LRFD specifications are
independent from the spectral accelerations.
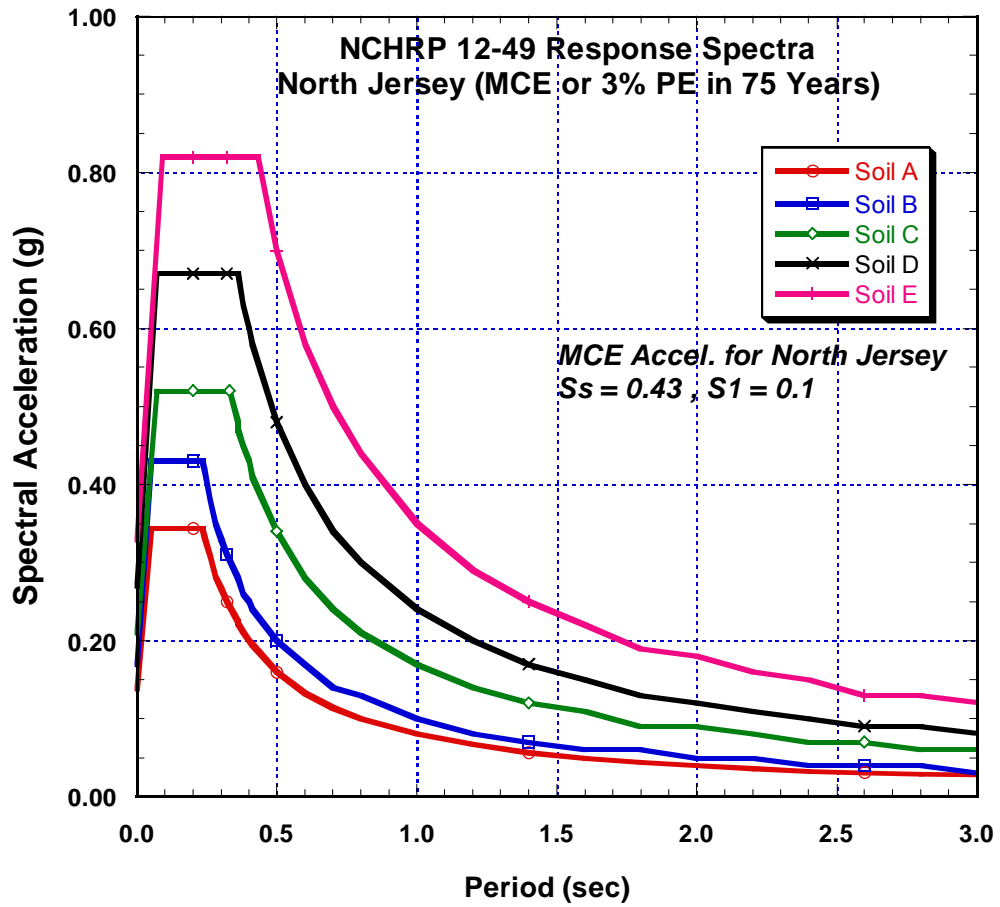
Figure 7. Design response spectra for NCHRP 12-49 MCE earthquake for various soil conditions in Northern New Jersey.
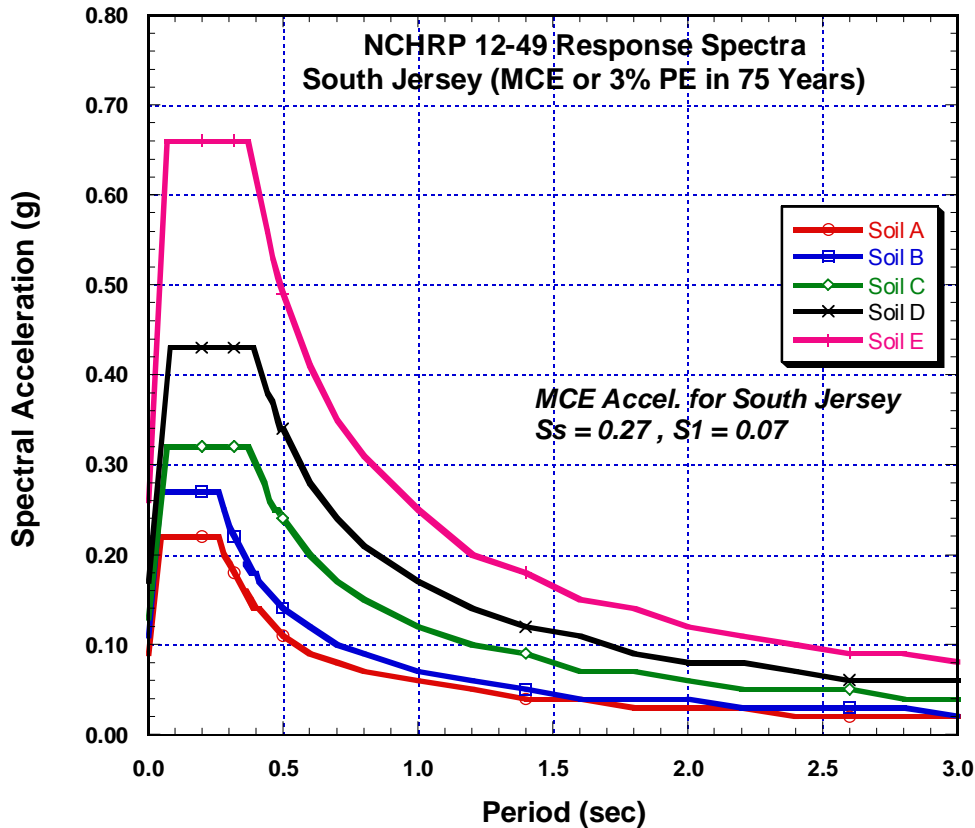
Figure 8. Design response spectra for NCHRP 12-49 MCE earthquake for various soil conditions in Southern New Jersey.

The effect of soil site factors in NCHRP 12-49 on the design response spectra for the EXP earthquake (108 years event) is shown in Figures 9 and 10 for Northern and Southern New Jersey, for various soil conditions respectively. Figures 9 and 10 show that the spectral accelerations for the EXP earthquake are small, except for bridges in Northern New Jersey in soft soils where the spectral accelerations F_aS_s (from Figure 9) are close to 0.32g. However, according to Table 3.7-1 in NCHRP 12-49, these accelerations will fall under Hazard Level II and for Life Safety performance level in Table 3.7-2, these accelerations do not require dynamic analysis. Thus, for all counties in New Jersey and for all soil conditions (except for Soil type F where site-specific spectra is required), no dynamic analysis is required but the design needs to satisfy specified minimum requirements according to Section 6 in NCHRP 12-49.
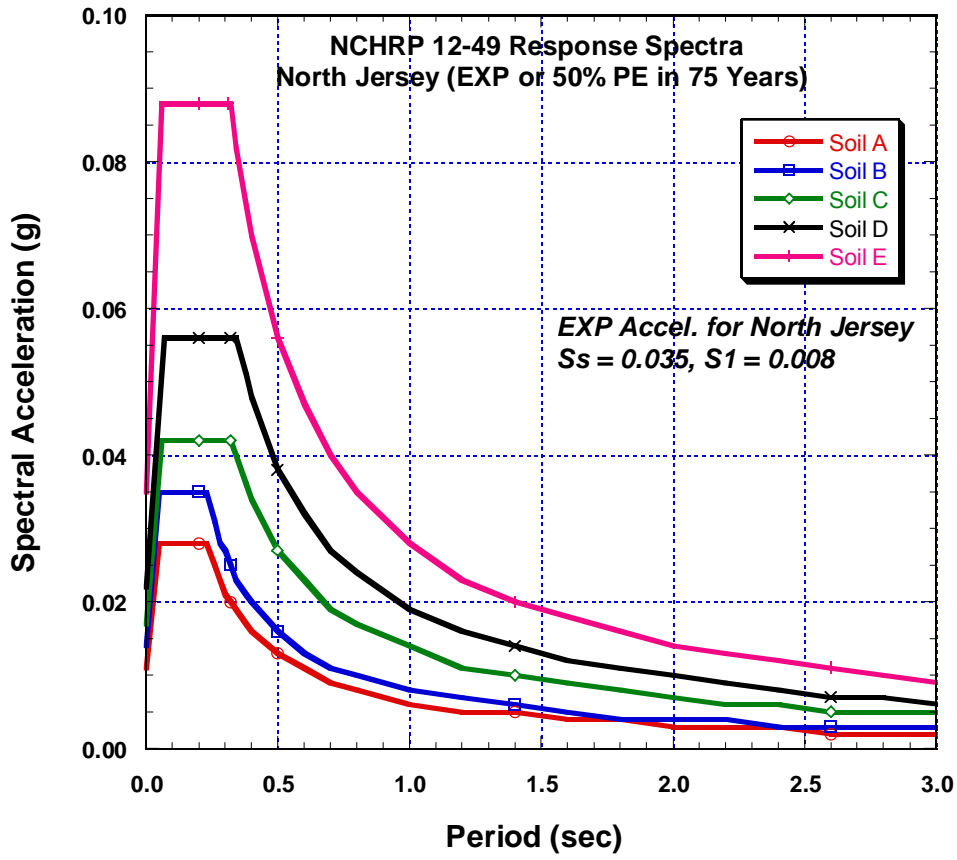
Figure 9. Design response spectra for NCHRP 12-49 EXP earthquake for various soil conditions in Northern New Jersey.
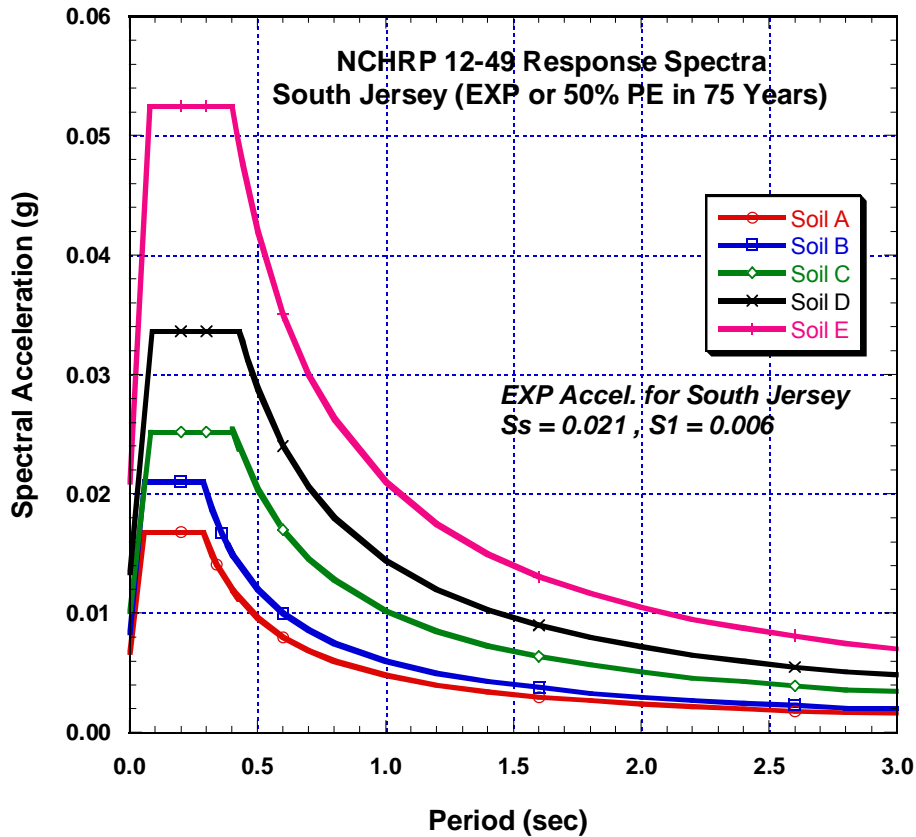
Figure 10. Design response spectra for NCHRP 12-49 EXP earthquake for various soil conditions in Southern New Jersey.

NCHRP 12- 49 Guidelines for Analysis and Design Procedures

Based on the soil factors, design spectral accelerations FvS_1 and FaSs, seismic hazard levels (I, II, III, and IV) can be determined from Table 3.7-1 in recommended NCHRP 12-49. The six SDAP (Seismic Design and Analysis Procedures) given in the new provisions are: A1, A2, B, C, D, and E depending on the seismic hazard and performance levels. Based on Table 3.7-2 in NCHRP 12-49, SDAP C, D, and E can be used for level IV for life safety evaluation and operational evaluation. The SDR (Seismic Detailing Requirements) was SDR 4 for life safety evaluation and SDR 6 for operational evaluation for all locations. SDAP C is a Capacity Design Spectrum Method (CDSM) in which demand and capacity analyses are combined. This SDAP may only be used in bridges that satisfy the requirements of Section 4.4.2 of the new provisions. SDAP D is an Elastic Response Spectrum Method (ERSM). This procedure is a one step procedure using elastic (cracked section properties) analysis. Either the Uniform Load or Multimode method of analysis may be used. The two examples in this study were analyzed and designed using SDAP D with multimode analysis.

20

SDAP E is an Elastic Response Spectrum Method with displacement capacity verification (pushover analysis). This SDAP is similar to SDAP D, except that the response modification factors R are increased which would lower the seismic design forces. Displacement-based design methods like pushover analysis are being used more and more in today's seismic of buildings and bridges and most structural analysis programs have that option. It is recommended to use SDAP E for bridge design because the seismic forces will be less (higher R-factors) and the design effort will not be significantly higher than for SDAP D. This SDAP (E) was not used in the two design examples. The examples analyzed here also satisfy the limitations for using SDAP C. Therefore; this procedure can be used for the analysis and design in both directions. The Capacity Design Spectrum Method is a relatively simple procedure and should be used whenever applicable.

Comparison of NCHRP 12- 49 Spectra and Site-Specific Response Spectra

The design response spectra from NCHRP 12-49 were compared to those from site-specific spectra for two bridge locations. The first bridge is the Victory Bridge over Route 35 in Perth Amboy and the second bridge is the Route 139 Bridge in Jersey City. The Victory bridge site-specific response spectra are shown in Figure 11 along with NCHRP 12-49 response spectra. Figure 12 shows the site specific spectra of the Route 139 Bridge in Jersey City compared to the NCHRP 12-49 spectra. In both cases, the NCHRP 12-49 spectra appear to be higher than the site-specific. These are only two examples; more examples are needed to make a larger comparison between the site-specific and the NCHRP 12-49 spectra. The research team attempted to collect available data, but received very few responses. This evaluation may need to be undertaken for all regions in New Jersey to establish a database for site-specific spectra.
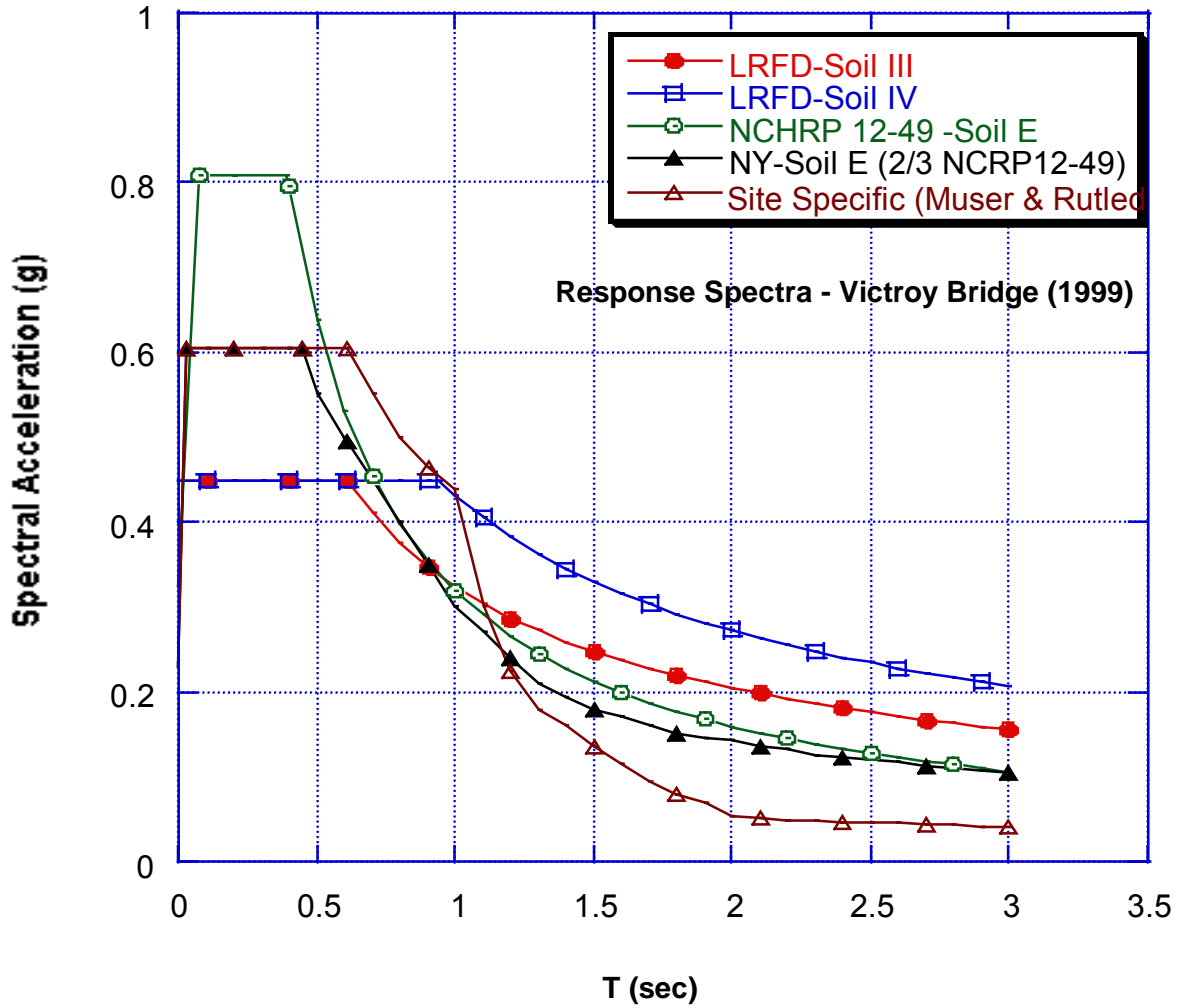
Figure 11. NCHRP 12-49 and site-specific response spectra for the Victory Bridge on Route 35 in Perth Amboy.

Figure 12. NCHRP 12-49 and site-specific response spectra for the Route 139 bridge in Jersey City.

PROPOSED SEISMIC HAZARD AND PERFORMNCE LEVELS FOR BRIDGES IN NEW JERSEY

Current Seismic Design Criteria in New Jersey

The current guidelines for seismic design of bridges in New Jersey are based on the AASHTO LRFD Specifications. For seismic design of deep foundation design, NJDOT requires the use of the 16th edition of AASHTO Standard Specifications. For Turnpike bridges, the specs require two-level design: (1) Safety Evaluation Event or SEE (2500 years), and (2) Functional Evaluation Event or FEE (500 years). Local road bridges over the Turnpike, including State and Federal Highways, are designed based on the lesser of the two events. For the 500 years event (10 percent PE in 50 years), all substructure elements will be designed using an R-factor equal to 1.5. For the design of certain special bridges that are deemed "critical" by the Turnpike Authority, the design shall be based on the 2500-year return period using an R-factor equal to 1.5 for all substructure elements. The FEE and the SEE design levels for the Turnpike bridges are somewhat similar to those levels specified by NYSDOT and SC DOT[9]. The SEE level is also similar to the MCE level proposed in NCHRP 12-49.

Current NYCDOT Seismic Design Criteria

The NYCDOT (2002) seismic design guidelines include a two-level design for earthquakes. Level one is an ODE (Operating Design Earthquake) and level two is an MDE (Maximum Design Earthquake). These levels are based on hard rock ground motion with a 10 percent probability of being exceeded in 50 years (500 year return period) for level one, and based on hard rock ground with a 2 percent probability of being exceeded in 50 years (2,500 year return period) for level two. The guidelines also specify a single hazard level dependant on the performance level of the structure. This level is based on (2/3) of the MDE. i.e. (2/3) of the hard rock ground motion with a two percent probability of being exceeded in 50 years (2,500 year return period) and is applicable to bridges classified as "Essential". The soil amplifications factors specified by NYCDOT are similar to those in 1997 NEHRP Guidelines[4,5]. NCHRP Report 12-49 adopted the same soil site factors as in the 1997 NEHRP Guidelines. The NYCDOT seismic design criteria are summarized in Table 1.

Table 1. NYCODT Seismic Performance and Hazard Levels.

Importance Category IC	Hazard Level	Seismic Event	Return Period	Performance Criteria (Service and Damage Criteria)
Critical Bridge	Upper Level (MDE)	2% PE in 50 Years	2500 Years	Service - Short disruption Damage - Repairable
	Lower Level (ODE)	10% PE in 50 Years	500 Years	Service - Immediate Damage – Minimal
Essential Bridge	Single-Level (2/3 of MDE)	2/3 (2% PE in 50 Years)		Service - Limited disruption Damage - Repairable
Other Bridges	Single Level (2/3 of MDE)	2/3 (2% PE in 50 Years)		Service - Significant disruption Damage - Significant

South Carolina Seismic Design Criteria

The current South Carolina DOT specifications [10] require two seismic design levels for critical bridges and a single seismic design level for normal and essential bridges in the state. The two levels are a lower-level event or FEE which is based on a design spectra for an earthquake with 10 percent PE in 50 years, and an upper-level event, or SEE which is based on a design spectra for an earthquake with two percent PE in 50 years. The single level for normal and essential bridges is the SEE event. The minimum performance levels level for service and damage expected will depend on the particular bridge as given in SCDOT Specifications [10] and shown in Table 2. The SCDOT specs requires the design to follow these requirements in conjunction with the AASHTO Standard Specifications (1996) Division I-A, some Caltrans Seismic Design Criteria [10], soil site factors similar to those in the NEHRP 1997 Guidelines, and Seismic Hazard Maps developed specifically for South Carolina.

Table 2. SCDOT Seismic Performance Criteria (SCDOT Specs, 2002).

Earthquake Level	Performance Level	Normal Bridge	Essential Bridge	Critical Bridge
Functional Evaluation	Service	Not Required	Not Required	Immediate
(FEE or 500-years)	Damage	Not Required	Not Required	Minimal
Safety Evaluation	Service	Impaired	Recoverable	Maintained
(SEE or 2500-years)	Damage	Significant	Repairable	Repairable

Proposed Seismic Hazards and Performance Levels in New Jersey

Based on the current criteria, NCHRP 12-49 recommended guidelines; the NYCDOT guidelines and the SCDOT seismic design criteria, the following seismic hazard and performance levels are proposed for New Jersey. The proposed criteria combines some of the NCHRP 12-49 recommended guidelines and some of the SCDOT guidelines, taking into account the seismicity level in New Jersey. Highway bridges in New Jersey are classified as 'critical' and 'others' (non-critical) to simplify the classification. The proposed classification requires "other bridges" to remain elastic with minimal damage when designed for the NCHRP 12-49 lower-level event (EXP or 50 percent PE in 75 years) and that these bridges do not collapse when designed for 2/3 of the MCE event in NCHRP 12-49, though they may suffer significant damage and may be closed to regular traffic for extended periods of time. Minimum seat widths at abutments and expansion piers shall be based on NCHRP 12-49 equations using the MCE spectral accelerations.

For critical bridges, the proposed ground motion levels are similar to those currently used for the Turnpike bridges in NJ, and NYCDOT bridges, and those of SCDOT. The lower-level event for critical bridges has a 500 years return period or 10 percent PE in 50 years. Following this event, the bridge should suffer minimal or no damage and the service level should be immediate. For the maximum event or the 2500-year event (two percent PE in 50 years), the damage should be repairable and the service should be maintained. A detailed description of service and damage levels is given in Table 3. In addition, to this

26

table, it is worth mentioning here that for certain bridges, NJDOT bridge officials may require specific provisions for bridge hazard and performance levels.

Table 3. Proposed Earthquake Hazard and Seismic Performance Levels for bridges in New Jersey.

Ground Motion Level	Performance Level	Critical Bridges	Other Bridges
Extreme Earthquake (EE)	**Earthquake**	**MCE (2500 Years Event)**	**(2/3) of MCE (2500 Years Event)**
	Service	Maintained	Impaired
	Damage	Repairable (No Collapse)	Significant (No Collapse)
Functional Earthquake (FE)	**Earthquake**	**10% PE in 50 Years (500 Years Event)**	**EXP (108 Years Event)**
	Service	Immediate	Immediate
	Damage	Minimal to None	Minimal

EXP = 50% PE in 75 Years, MCE = 2% PE in 50 Years,

Service Levels

Immediate Service: Full access to normal traffic should be available immediately following the earthquake and after inspection of bridge.
Maintained Service: Short periods of closure of traffic to the public. Immediately open to emergency traffic.
Impaired Service: Extended periods of closure of traffic to the public. Limited access may be possible after shoring (reduced lanes, light emergency vehicles).

Damage Levels

No Damage (None): Evidence of minor movements may be visible but no notable damage. Essentially an elastic behavior during the earthquake.
Minimal Damage: Minor inelastic response and some visible signs of damage. Damage will be limited to narrow flexural cracks and the beginning of yield of steel. Repair can be made under non-emergency conditions.
Repairable Damage: No collapse. Concrete cracking; spalling of concrete cover, some yielding of steel will occur. However, bridge damage should be limited such that the structure can be repaired to its pre-earthquake condition without replacement of structure members or reinforcement i.e. damage can be repaired without losing functionality.

Significant Damage: Although there is a minimum risk of collapse, permanent offsets may occur in elements other than foundations. Damage consisting of concrete cracking, extensive reinforcement yielding, major spalling of concrete, local and global buckling of steel braces, and deformations in minor bridge components may require closure for repair. Partial or complete demolition and replacement may be required.

Proposed Seismic Hazard Levels and Performance Levels For Bridge Retrofit In New Jersey

The proposed criteria for seismic retrofit of bridges in New Jersey combines some of the NCHRP 12-49 recommended guidelines and some of the SCDOT guidelines, taking into account the seismicity level in New Jersey. The bridge classification is divided into "Critical" and "Others". The proposed retrofit criteria requires Critical bridges be retrofitted for two-thirds of MCE (2500-year event) in which the bridge will not collapse and its service can be restored fairly quickly for this event. For Other bridges, seismic retrofit criteria requires that these bridges be retrofitted for the two-thirds of MCE with no collapse, but they may suffer significant damage and may be closed for regular traffic for extended period of time.

NCHRP 12- 49 Ground Motion Levels in New Jersey

According to the proposed provisions in NCHRP 12-49, the lower level or expected earthquake in New Jersey, which has about 108 year return period, had very small peak horizontal accelerations S_s and S_1. This event will not control the design and the non-seismic design of the bridge will be adequate to resist the expected earthquake loads. Consequently, no seismic design is required for the lower level event or the expected earthquake in New Jersey. For stiff soils and for the short period range, the 2,500-years event (MCE) had larger design accelerations compared to the current LRFD specs (500-years event) in New Jersey. For small ground motions and very soft soil sites, the seismic design loads from NCHRP 12-49 are significantly increased because of the much higher soil site factors. Table 4 shows the various spectral accelerations for three different regions in the state of New Jersey (North, Central, and South) based on various earthquake events without the effects of the soil site factors.

NCHRP 12-49 Design Accelerations and Analysis Procedures for the EXP and MCE Earthquakes in for Various Counties in New Jersey

The acceleration coefficients for the EXP (108-year return period) and the MCE (2500-year return period), the SDAP, and SDR requirements for various counties in New Jersey are discussed in this section. New Jersey counties are shown in Figure 13. Tables 5 and 6 show the seismic hazard levels, SDAP, and SDR for the various counties in New Jersey for bridges located in soft soil conditions (Soil type E). Table 5 shows that SDAP A1 and A2 and SDR 1 and 2 control the

analysis and design for the Expected earthquake in soil class E. According to NCHRP 12-49, SDAP A1 and A2 do not require any dynamic analysis of the structure. Except for minimum design forces and seat width requirements, SDR 1 and 2 do not require any special design requirements for seismic loads. Table 6 shows that for the MCE earthquake, the analysis and design procedures design require the use of SDAP C, D, or E and SDR 4 for life safety and SDR 6 for operational level in most locations in state with the exception of Cape May County where SDAP B can be used and SDR 3 and 5 would be sufficient. It is clear from Table 6 that non-seismic design will control the design for the lower-level earthquake (EXP) specified in NCHRP 12-49. The non-seismic design shall also satisfy the minimum confining steel in plastic hinge zones and minimum seat widths specified in Section of NCHRP 12-49.

Table 4. Acceleration Coefficients in New Jersey from NCHRP 12-49 and from AASHTO.

Location	Acceleration Coeff. S_s and $S1$	NCHRP 12-49 50% PE in 75 Years (108- Year Event) (Expected)	USGS MAPS 10% PE in 50 Years (500 Years Event)	USGS MAPS 5% PE in 50 Years (~1000 Years Event)	NCHRP 12-49 2% PE in 50 Years or 3% in 75 Years (2500-Year Event) (MCE)
North NJ	S_s	0.024 - 0.035	0.11 - 0.14	0.20 - 0.23	0.38 - 0.45
Central NJ	S_s	0.021 - 0.024	0.09 - 0.11	0.15 - 0.20	0.32 - 0.38
South NJ	S_s	0.015 - 0.021	0.06 - 0.09	0.10 - 0.15	0.18 - 0.32
North NJ	S_1	0.007 - 0.008	0.025 - 0.029	0.048 - 0.052	0.09 - 0.10
Central NJ	S_1	0.006 - 0.007	0.023 - 0.025	0.042 - 0.048	0.08 - 0.09
South NJ	S_1	0.004 - 0.006	0.02 - 0.023	0.03 - 0.042	0.06 - 0.08
Existing AASHTO LRFD Specifications (10% PE in 50 Years)**					
North NJ	A	0.18 - 0.20			
Central NJ	A	0.16 - 0.18			
South NJ	A	0.08 - 0.16			
Peak Horiz Accel. Maps at 0.2 sec and 1.0 sec, ** PGA Maps *Soil Site Factors are not included*					

Figure 13. County map of New Jersey.

Table 5. Accelerations, Hazard levels, SDAP, and SDR in New Jersey for
NCHRP 12-49 Expected earthquake (50% PE in 75 years).

#	County	ZIP CODE	$S_{s(g)}$	$S_{1(g)}$	F_v	F_a	$F_vS_{1(g)}$	$F_aS_{s(g)}$	Hazard Level	Life Safety		Operational	
										SDAP	SDR	SDAP	SDR
1	Atlantic	8302	0.018	0.005	3.5	2.50	0.02	0.05	I	A1	1	A2	2
2	Bergen	7662	0.035	0.008	3.5	2.50	0.03	0.09	I	A1	1	A2	2
3	Burlington	7504	0.030	0.008	3.5	2.50	0.03	0.08	I	A1	1	A2	2
4	Camden	8100	0.023	0.006	3.5	2.50	0.02	0.06	I	A1	1	A2	2
5	Cape May	8202	0.016	0.004	3.5	2.50	0.02	0.04	I	A1	1	A2	2
6	Cumberland	8353	0.019	0.005	3.5	2.50	0.02	0.05	I	A1	1	A2	2
7	Essex	7102	0.029	0.007	3.5	2.50	0.03	0.07	I	A1	1	A2	2
8	Gloucester	8030	0.023	0.006	3.5	2.50	0.02	0.06	I	A1	1	A2	2
9	Hudson	7305	0.029	0.007	3.5	2.50	0.03	0.07	I	A1	1	A2	2
10	Hunterdon	8801	0.028	0.007	3.5	2.50	0.03	0.07	I	A1	1	A2	2
11	Mercer	8504	0.025	0.006	3.5	2.50	0.02	0.06	I	A1	1	A2	2
12	Middlesex	8854	0.023	0.007	3.5	2.50	0.02	0.06	I	A1	1	A2	2
13	Monmouth	7721	0.025	0.007	3.5	2.50	0.02	0.06	I	A1	1	A2	2
14	Morris	7933	0.029	0.007	3.5	2.50	0.03	0.07	I	A1	1	A2	2
15	Ocean	8753	0.022	0.006	3.5	2.50	0.02	0.06	I	A1	1	A2	2
16	Passaic	7504	0.017	0.008	3.5	2.50	0.03	0.04	I	A1	1	A2	2
17	Salem	8098	0.021	0.006	3.5	2.50	0.02	0.05	I	A1	1	A2	2
18	Somerset	7059	0.029	0.007	3.5	2.50	0.03	0.07	I	A1	1	A2	2
19	Sussex	7462	0.030	0.008	3.5	2.50	0.03	0.08	I	A1	1	A2	2
20	Union	7201	0.029	0.007	3.5	2.50	0.02	0.07	I	A1	1	A2	2
21	Warren	7838	0.028	0.007	3.5	2.50	0.02	0.07	I	A1	1	A2	2

Expected Earthquake (50% PE in 75 Years) - Soil Class E

Table 6. Accelerations, Hazard levels, SDAP, and SDR in New Jersey for
NCHRP 12-49 MCE earthquake (2% PE in 50 years).

MCE (3% PE in 75 Years) - Soil Class E

#	County	ZIP CODE	$S_{s(g)}$	$S_{1(g)}$	F_v	F_a	$F_vS_{1(g)}$	$F_aS_{s(g)}$	Hazard Level	Life Safety		Operational	
										SDAP	SDR	SDAP	SDR
1	Atlantic	8302	0.249	0.067	3.5	2.50	0.23	0.62	IV	C,D,E	4	C,D,E	6
2	Bergen	7662	0.428	0.095	3.5	1.93	0.33	0.83	IV	C,D,E	4	C,D,E	6
3	Burlington	7504	0.424	0.095	3.5	1.94	0.33	0.82	IV	C,D,E	4	C,D,E	6
4	Camden	8100	0.328	0.081	3.5	2.25	0.28	0.74	IV	C,D,E	4	C,D,E	6
5	Cape May	8202	0.174	0.059	3.5	2.50	0.21	0.44	III	B,C,D,E	3	C,D,E	5
6	Cumberland	8353	0.269	0.071	3.5	2.44	0.25	0.66	IV	C,D,E	4	C,D,E	6
7	Essex	7102	0.420	0.093	3.5	1.96	0.33	0.82	IV	C,D,E	4	C,D,E	6
8	Gloucester	8030	0.304	0.078	3.5	2.33	0.27	0.71	IV	C,D,E	4	C,D,E	6
9	Hudson	7305	0.422	0.093	3.5	1.95	0.33	0.82	IV	C,D,E	4	C,D,E	6
10	Hunterdon	8801	0.357	0.087	3.5	2.16	0.30	0.77	IV	C,D,E	4	C,D,E	6
11	Mercer	8504	0.373	0.088	3.5	2.11	0.31	0.79	IV	C,D,E	4	C,D,E	6
12	Middlesex	8854	0.399	0.091	3.5	2.02	0.32	0.81	IV	C,D,E	4	C,D,E	6
13	Monmouth	7721	0.394	0.089	3.5	2.04	0.31	0.80	IV	C,D,E	4	C,D,E	6
14	Morris	7933	0.399	0.092	3.5	2.02	0.32	0.81	IV	C,D,E	4	C,D,E	6
15	Ocean	8753	0.291	0.078	3.5	2.37	0.27	0.69	IV	C,D,E	4	C,D,E	6
16	Passaic	7504	0.424	0.095	3.5	1.94	0.33	0.82	IV	C,D,E	4	C,D,E	6
17	Salem	8098	0.297	0.077	3.5	2.35	0.27	0.70	IV	C,D,E	4	C,D,E	6
18	Somerset	7059	0.399	0.091	3.5	2.02	0.32	0.81	IV	C,D,E	4	C,D,E	6
19	Sussex	7462	0.357	0.090	3.5	2.16	0.32	0.77	IV	C,D,E	4	C,D,E	6
20	Union	7201	0.420	0.093	3.5	1.96	0.33	0.82	IV	C,D,E	4	C,D,E	6
21	Warren	7838	0.336	0.087	3.5	2.22	0.30	0.75	IV	C,D,E	4	C,D,E	6

SEISMIC DESIGN CRITERIA AND GUIDELINES FOR ABUTMENTS, RETAINING WALLS, EMBANKMENTS, AND BURIED STRUCTURES

Seismic Performance and Hazard Levels

Seismic performance criteria and hazard levels for abutments, retaining walls, embankments, and buried structures shall be consistent with those proposed for NJDOT for bridge structures as shown in Table 1.

Seismic Design Criteria and Guidelines for Abutments

Seat Abutments

The seismic design criteria and guidelines for seat abutments are similar to those of retaining walls (see pages 36-40 of this report). However, when the seismic displacement at the top of the abutment exceeds the gap between the superstructure and the abutment back wall, the analysis and design methods of integral abutments shall be used.

Integral Abutments

Analysis

1. To analyze integral abutment bridges under seismic loads, the abutment stiffness need to be evaluated. There are several theoretical methods for estimating abutment longitudinal and transverse stiffness. Some of the methods are based on the application of the ultimate passive pressure on the abutment such as those given in NCHRP 12-49 guidelines while others are dependent on the structural and the geotechnical properties of abutment and soil. In general, the abutment stiffness is a function of height, soil type, abutment dimension, and movement. The various methods for calculating abutment stiffness are described in Appendix B of this report like the methods defined in NCHRP 12-49 guidelines and in the CALTRANS 2001 seismic design criteria. A comparison between these methods is included. Also other methods of measuring passive soil pressure and abutment stiffness are described. The abutment stiffness can be described either in terms of the passive pressure and displacement using various analytical approaches.

2. The transverse stiffness of the abutment piles, piers, and wing walls shall be included in the seismic model. Contribution from the embankments in the transverse direction may also be considered.

3. Isolated rigid piers or semi-rigid piers will participate in resisting seismic loads in the transverse direction. Shear keys and dowels for these piers shall be

designed to resist these forces using the appropriate R-factor for connections. In addition, for isolated rigid piers, it is important to provide the required seat widths to accommodate seismic movements (The NCHRP 12-49 guidelines require about 70 percent more seat width than the current AASHTO LRFD Specifications).

4. A three-dimensional seismic model using multi-modal analysis shall be used to model the integral abutment-pier-soil system. The design response spectra used to apply the earthquake loads shall be according the NCHRP 12-49 recommended design guidelines or the AASHTO LRFD taking into account soil factors. A site-specific response spectra may be used in-lieu of the AASHTO LRFD or the NCHRP 12-49 spectra. The soil behind the abutments can be modeled using discrete springs along the height and the width of the abutment wall. The stiffnesses of these springs are estimated using the methods described in Appendix B.

5. A pinned connection shall be assumed between the superstructure and the abutment for the seismic analysis. However, the designer need to check the seismic forces using fixed connection. Several researchers have questioned the rigidity of this connection in practice and the designer needs to use his or her judgment on whether the connection details are typical of a monolithic connection or a pin connection. The period of vibration of the integral bridge depends on the stiffness of the structure and hence its seismic loads will be dependent on the rigidity of the superstructure-abutment connection.

6. The piles supporting the abutment wall are modeled using the equivalent length of fixity. To estimate the length of fixity, several methods can be used such as the LPILE program, the MHD method, or approximate empirical formulas given in the literature. Alternatively, a more sophisticated analysis can be used including the soils surrounding the piles.

7. When small diameter drilled shaft are used under the abutment wall, the support condition at the top of the shaft may rotate depending on the size of the shaft and the width of the abutment wall. For small shaft diameters and large abutment walls, the rotation at the top of the shaft is assumed to be zero, and a length of fixity can be used to model the shaft. For larger shaft diameters, the soil around the shaft shall be modeled using discrete springs along the height.

Design

1. The abutment wall and piles will be designed for the controlling load case according to AASHTO LRFD Table 3.4.1-1. The abutment wall shall be designed using the proper pressure distribution behind the wall. The passive pressure distribution behind abutment walls given in Figure 14. The value of

Kp can be obtained from the Monobe-Okabe Method given in AASHTO LRFD. The NCHRP 12-49 guidelines suggest using $K_p = 0.67/\tilde{\gamma}$ The wall cross section should be designed for the maximum shear and moments acting on the wall in the vertical and horizontal directions as shown in Figure 15.

2. The R-factors for abutment wall and piles will be according to Table 3.10.7.1-1 of the AASHTO LRFD depending on the bridge category or to Table 4.7.1-1 of the NCHRP 12-49 guidelines depending on the design (Operational or Safety).

3. The soil pressure behind the abutment shall be checked and compared to maximum stress limits.

4. The girders should be checked for the additional axial stress due to seismic loads.

Detailing

No special details are required for the abutment wall or piles. If small diameter drilled shafts or concrete piles are used under the integral abutments, the confinement of the longitudinal steel should be designed and checked for minimum requirements. The drilled shaft or pile reinforcement should extend into the abutment wall to ensure proper connections.
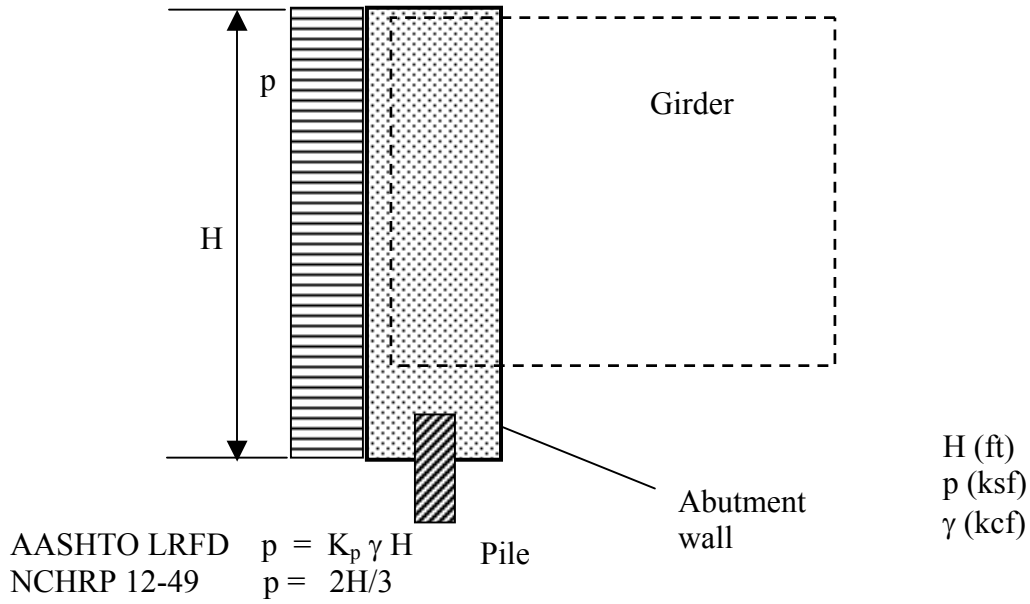
Figure 14. Passive pressure distribution behind abutment
wall under seismic loads.
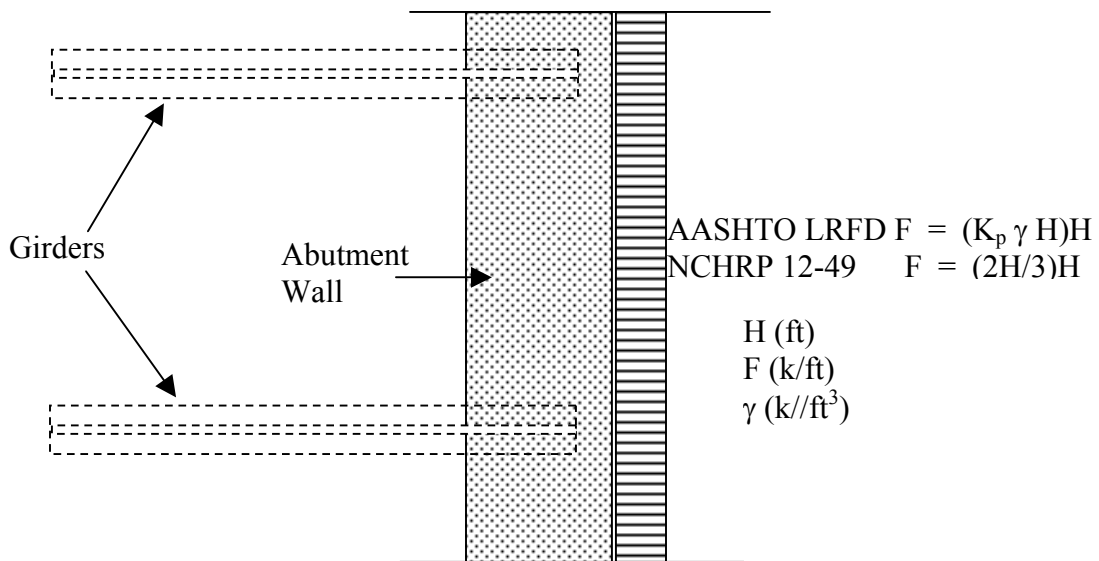
Figure 15. Horizontal bending in the abutment wall
between beams due seismic loads.

Seismic Design Criteria and Guidelines for Retaining Walls

Analysis

The two categories of retaining walls discussed for seismic design are: 1) Yielding walls and, 2) non-yielding walls. Examples of yielding walls are cantilever walls. Examples of non-yielding walls are buttress and basement walls. The amount of movement to develop minimum active pressure is estimated as 0.002 times the wall height. The design of a retaining wall is dependant upon the acceptable amount of movement: Independent walls may be able to move substantial amounts but retaining walls used in applications such as basements are required to remain in place without any or very little movement. The design solution is to make sure that the expected maximum pressure acting on the wall due to the earthquake loading and the static pressure does not exceed the design passive pressure. However, in some cases the earthquake loading will just be too large and the walls will fail, as in the Kobe earthquake.

The analysis of retaining walls under dynamic loads will be according to the Monobe-Okabe method for estimating dynamic loads on the walls. This is similar to what is currently specified in the AASHTO LRFD Specifications.

The active earthquake force E_{AE} is given by:

$$E_{AE} = \frac{1}{2}\gamma H^2 \left(1 - k_v\right) K_{AE} \tag{1}$$

Where the seismic active pressure coefficient K_{AE} is given by:

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta)} * \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \theta + \beta)\cos(i - \beta)}}\right]^{-2} \tag{2}$$

Where,
γ = unit weight of soil (kip/ft^3)
H = height of soil face (ft)
ϕ = angle of friction of soil (deg)
$\theta = \tan^{-1}\left[k_h/(1-k_v)\right]$
δ = angle of friction between soil and abutment (deg)
k_h = horizontal acceleration coefficient
k_v = vertical acceleration coefficient
i = backfill slope angle (deg)
β = slope of the wall to the vertical, +ve clockwise (deg)

Passive pressures will develop behind integral abutment walls due to thermal loads as well as seismic loads. Passive pressure resistance will also develop behind the walls of seat type abutments when the seismic displacements are larger than the expansion gap between the bridge superstructure and the back wall,

The equivalent expression for the passive earthquake force is given by:

$$E_{PE} = \frac{1}{2}\gamma H^2 \left(1 - k_v\right) K_{PE}$$

(3)

where the passive earth pressure coefficient, K_{PE} is given by:

$$K_{PE} = \frac{\cos^2\left(\phi - \theta + \beta\right)}{\cos\theta \cos^2\beta \cos\left(\delta - \beta + \theta\right)} * \left[1 + \sqrt{\frac{\sin\left(\phi + \delta\right)\sin\left(\phi - \theta + i\right)}{\cos\left(\delta - \theta + \beta\right)\cos\left(i - \beta\right)}}\right]^{-2}$$

(4)

For the static case with no earthquake effects, the location of the resultant of the soil pressure acting on the abutment is taken as H/3. Including earthquake effects, the additional dynamic earth pressure component is assumed to act approximately at 0.6H from the bottom. For most purposes, it will be sufficient to assume that the resultant of both components acts at half-height (H/2) with a uniformly distributed pressure.

Freestanding abutments can be designed more economically if small tolerable displacements are allowed rather than no displacements. By choosing a maximum wall displacement, a value of the seismic acceleration coefficient can be derived. Studies have shown that a value of the horizontal acceleration coefficient $k_h = 0.5A_{max}$ is adequate for most designs, provided that an allowance is made for an outward displacement of the abutment of up to 10 times the maximum earthquake acceleration A_{max} in inches. If no displacements are allowed (non-yielding walls and abutments or abutments with battered piles), the pressures from the soil on the abutment will be much higher than those predicted by the Monobe-Okabe method. Simplified models for rigid non-yielding abutments or walls indicate that a value of $k_h = 1.5A_{max}$ is reasonable and is suggested in this situation.

The horizontal acceleration k_h can also be estimated using the following formula:
For retained soils,

$$k_h = 0.67 A_{max} \sqrt[4]{\left(\frac{\left(A_{max}\right)\left(1\, in\right)}{d}\right)} \qquad \text{for } d > 1 \text{ in} \qquad (5)$$

$$k_h = A_{max} \qquad \text{for } d < 1 \text{ in} \qquad (6)$$

For infill soil,

$$k_h = \left(1.45 - A_{max}\right) A_{max} \qquad \text{for } d = 0 \qquad (7)$$

The allowable displacement d in the above equations represents the allowable lateral deflection that the wall can tolerate during a seismic event. This allowable value for d in the design is based on the engineer's judgment. A good approximation of d is given by:

$$d = (A_{\max})H \tag{8}$$

Where H is the height of the wall in feet and d is inches.
The value of $A_{\max}$ for the AASHTO LRFD Specs is obtained from Section 3.10.
For the NCHRP 12-49 guidelines, A max can be taken as the $0.4S_{DS}$

The active force acting on the wall (at rest conditions) is given by:

$$F_A = K_A(\gamma H^2)/2 \tag{9}$$

The total active plus dynamic force acting on the wall (at rest conditions) is given by:

$$F_{AE} = K_{AE}(\gamma H^2)/2 \tag{10}$$

Hence, the dynamic force acting on the wall is given by:

$$F_D = (K_{AE} - K_A)(\gamma H^2)/2 \tag{11}$$

The stresses from the active pressure on the wall have a triangular distribution whose resultant (F_A) is located at H/3 from the base of the wall. The stresses from the dynamic pressures only on the wall have an inverted triangular distribution whose resultant (F_D) is located at 0.6H form the bottom of wall. Figure 16 shows active and dynamic pressure distributions on the retaining wall.

Another alternative to calculate active and dynamic forces on retaining walls is given by Seed and Whitman [13]. Seed and Whitman [13] suggested simplified equations for yielding (flexible) and non-yielding (rigid) walls. For flexible walls, the dynamic component E_{AE} of the lateral earth pressure can be estimated as:

$$E_{AE} = \frac{3}{8} k_h \gamma H^2 \tag{9}$$

For rigid walls, the dynamic component E_{AE} of the lateral earth pressure can be estimated as:

$$E_{AE} = k_h \gamma H^2 \tag{10}$$

Where k_h is the horizontal ground acceleration. It is recommended that k_h taken equal to the site peak ground acceleration (that is $k_h = 0.4 S_{DS}$).
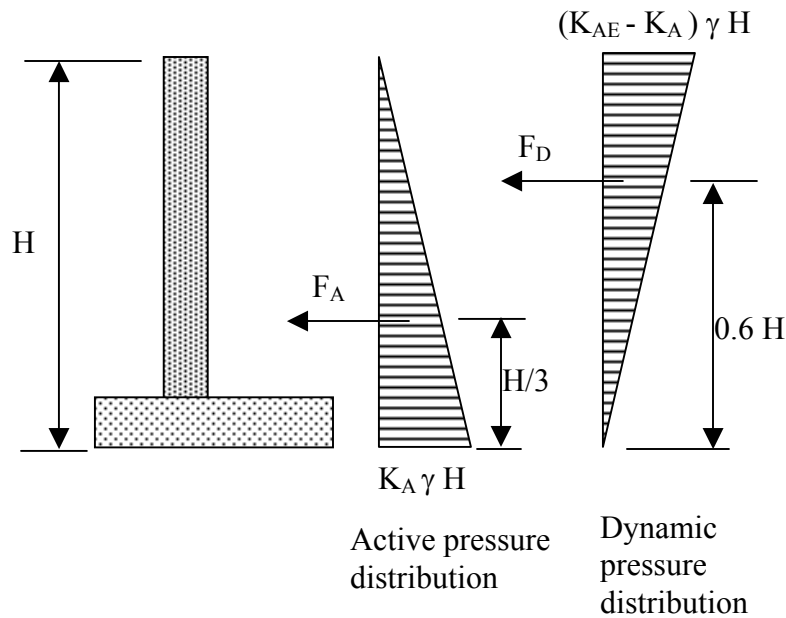
Figure 16. Active and dynamic pressure distribution on a retaining wall according to the Monobe-Okabe Method.

Design

The wall shall be designed for stability against sliding and overturning. The factor of safety against sliding under static loading should be larger than 1.5 and the factor of safety against overturning under static loads shall be larger than 2.0. Most government and state agencies recommend these factors. Because of the transient nature of the seismic loads, the minimum recommended factors of safety under seismic loads are 75 percent of the factors used for static loading. The wall and footing reinforcement and cross section shall also be designed for the maximum shear and bending moments resulting from seismic load combination and other combinations.

Detailing

No special seismic details are required for retaining walls. All other detailing requirements shall be according to AASHTO guidelines.

Seismic Design Criteria and Guidelines For Embankments

Embankments should be located away from potential faults and on foundation with liquefiable soft soils. Should the embankments be designed in active fault zones or on top pf potentially liquefiable soils, then they should be designed for all seismic displacements and forces and any failure under extreme earthquake should avoid loss of life.

Possible damage of embankments due to an earthquake could include:

1. Slope failures
2. Sliding failures of embankments having weak soils.

The material for all new embankments should be compacted to a density that will cause them to dilate rather than liquefy during earthquake shaking. It is recommended that the compacted density of the material should exceed 95 percent of the standard Proctor Maximum Dry Density. Cohesive embankment materials should be compacted with two to four percent higher than the optimum moisture content.

Analysis

The following methods of analysis can be used:

1. Pseudo-static stability analysis.
2. Sliding Block Method (Makdisi and Seed Method)
3. Dynamic Analysis

Well-compacted embankments founded on dense soils located in seismic zones may be evaluated by the pseudo-static method. Where foundation liquefaction is potential exists, the Sliding Block Method or the Simplified Dynamic Analysis will be necessary. Site-specific evaluations should be performed for all embankments located in active fault zones or where liquefaction potential exists for both, the embankment or its foundation.

Pseudo-Static Analysis Procedure

In this procedure the dynamic forces on the embankment are replaced by single constant unidirectional ground acceleration. The seismic design acceleration used in this method is much smaller than the peak ground acceleration corresponding to the design level earthquake. Figure 17 shows the forces on the sliding mass in the pseudo-static approach.

The design horizontal and vertical pseudo-static seismic design forces are given by:

$$F_H = \left(\frac{k_h}{3}\right) W \quad \text{for ordinary embankments} \tag{10}$$

$$F_H = \left(\frac{k_h}{2}\right) W \quad \text{for critical embankments} \tag{11}$$

$$F_V = \pm (2/3) W \tag{12}$$

Where k_h is peak ground accelerations and W is the weight of the sliding mass. The values of k_h in the AASHTO LRFD is equal to A_{max} and shall be taken equal to $0.4S_{DS}$ when using the NCHRP 12-49 guidelines, .The centroid of these forces will depend on the geometry of the sliding mass. A factor of safety against sliding of 1.5 is typical for non-seismic loads and a factor of safety of 1.1 is used for seismic loads (engineering judgment must still be applied as to the applicability of pseudo-static analyses and the acceptable factor of safety might be varied with the uncertainties involved in a particular analysis). Slope stability analysis including the pseudo-static design forces should be conducted to determine the available factor of safety. An alternative method to estimate the seismic forces on the embankment is to obtain a seismic coefficient corresponding to the earthquake magnitude from Figure 18. The magnitude of the horizontal force is calculated by multiplying the sliding weight W with the seismic coefficient.
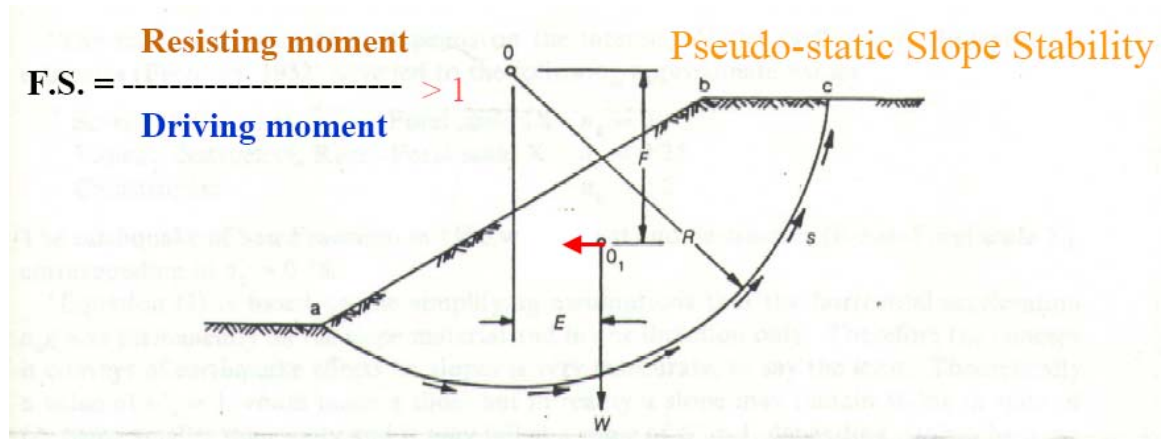
Figure 17. Pseudo-static slope stability approach.
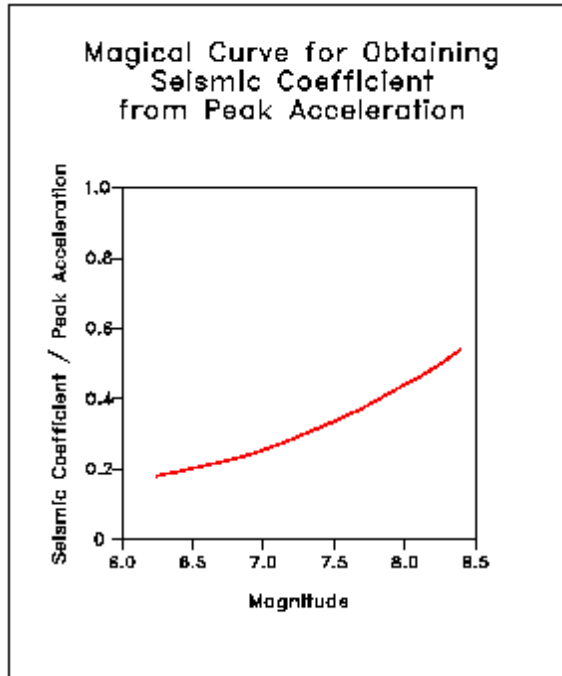
Figure 18. Estimate of seismic coefficient versus earthquake magnitude.

Sliding Block Method of Analysis

This method involves evaluation of the permanent deformations during an earthquake and comparing it with an acceptable deformation. The method follows the Newmark's sliding block analysis wherein the potential failure mass is treated as a rigid body with a rigid base with the contact in between as rigid plastic. The acceleration of the rigid body is assumed to correspond to the average acceleration time history of the failure mass. Deformations accumulate when the when the rigid body acceleration exceeds the yield acceleration.

The sliding block method is a relatively simple and in expensive method of analysis. It assesses the embankment stability and performance in terms of the deformations they produce rather than the minimum factor of safety. The deformations calculated along the failure surface using this method should not exceed 600 mm or it can be established by the design engineer on a case-by-case basis. Several alternative empirical approaches are available for determining permanent displacements such as the Makdisi and Seed Method for Sliding Block Analysis.

The Makdisi and Seed (1978) for calculating permanent slope deformation of earth dams produced by earthquake shaking is based on the sliding block method but uses average accelerations computed with the procedure of Chopra

(1966) and the shear beam method. The method uses a plot that relates the average maximum acceleration with the depth of the potential failure surface and a plot of normalized permanent displacement with yield acceleration for different earthquake magnitudes. The following procedure is used in order to evaluate the permanent slope deformation of a potential failure surface.

1. Determine the yield acceleration (K_y). This acceleration corresponds to a factor of safety equal to 1.
2. Determine U_{max} and K_{max}, the max crest acceleration and the maximum average acceleration of the potential sliding mass from Makdisi and Seed design charts (Appendix Figures 7 and 8).
3. Knowing the yield acceleration and the max acceleration, the permanent displacement of the embankment can be obtained using Makdisi and Seed design chart (Appendix Figure 9).
4. This permanent deformation is compared to 600 mm or to values set by the design engineer.

Dynamic Analysis

The dynamic analysis method estimates the deformation in the embankments using finite element analysis (FLAC, PLAXIS, TELDYN…) or finite different method. This method requires stress-strain relations of soils representative of the soil in-situ behavior. It also requires suitable earthquake time histories representing design earthquakes. The finite element method is powerful tool, which can cope with irregular geometries, complex boundary conditions and pore water pressure regimes and can simulate complicated construction operations.

The method can predict stresses, movements and poor water pressures due to construction procedures and also predict the most critically stressed zones within a slope. In this way the most likely mode of failure can be identified and deformations up to and sometimes beyond the point of failure can be calculated. All embankment stability analysis should consider liquefaction potential.

Seismic Design Criteria and Guidelines for Buried Structures

The design of superstructures is typically based on inertial forces. Buried structures, on the hand, are designed for strain and deformation caused by soil and soil-structure interaction. There are several approaches for the analysis and design of underground structures. One of the approaches is free field deformation. It is a simple approach that ignores the interaction between soil and structure. It assumes no presence of the structure to affect the soil deformation due to an earthquake. The structure is designed to accommodate this deformation. This method is used for low-level earthquakes or when the surrounding soil is much stiffer than the structure. For example, this would be for

tunnels within rock, where the stiffness of the structure cannot affect the surrounding deformation considerably. In other methods the soil structure interaction is considered. Several design examples are presented in the Appendix C.

The design of underground structures differs from the design of a super-structure mostly because:(1) they are completely enclosed in soil or rock, and (2) they are significantly longer (i.e. tunnels). So, their design criteria are very different from those for superstructures. The buried structures discussed here can be divided into three types:

1. Bored and mined tunnels
2. Cut-and-cover tunnels (culverts)
3. Immersed tube tunnels

These buried structures are commonly used for metro structures, high way tunnels, and large water and sewage transportation ducts. Methods of calculating seismic forces induced in these substructures are outlined in the following sections.

Safety Level Evaluation (MCE)

For cut-and-cover tunnel structures:

$$U = D + L + E1 + E2 + EQ \tag{13}$$

Where U = required structural strength capacity
      D = effects due to dead loads of structural components
      L = effects due to live loads
      E1 = effects due to vertical loads of earth and water
      E2 = effects due to horizontal loads of earth and water
      EQ= effects due to design earthquake (MCE) or 2/3 of (MCE)

For circular tunnel lining:

$$U = D + L + EX + H + EQ \tag{14}$$

      EX = effects of static loads due to excavation
      H   = effects due to hydrostatic water pressure

Functional Earthquake (FE)

The functional earthquake is defined in Table 3 (Page 32) for critical and ordinary buried structures. For the functional earthquake (FE), the seismic design loading

combination depends on the performance requirements of the structural members. Generally speaking, if the members are to experience little to no damage during the functional earthquake, the inelastic deformation in the structural members should be kept low. The following loading criteria, based on the load factor design, are recommended:

For cut-and-cover tunnel structures:

$$U = 1.05D + 1.3L + \beta_1(E1 + E2) + 1.3EQ \qquad (15)$$

Where D, L, E1, E2, EQ, and U are as defined in Eq (5.1)
$\beta_1 = 1.05$ if extreme loads are assumed for E1 and E2 with little uncertainty.
($\beta_1 = 1.3$ otherwise)

For circular tunnel lining:

$$U = 1.05D + 1.3L + \beta_2(EX + H) + 1.3EQ \qquad (16)$$

Where D, L, EX, H, EQ, and U are as defined in Eq (5.2)
$\beta_2 = 1.05$ if extreme loads are assumed for EX and H with little uncertainty.
($\beta_2 = 1.3$ otherwise)

Analysis Methods

Four methods to compute earthquake forces in buried structures are given in Table 7 with the advantages and disadvantages of each method. Two of these methods are described in details in the Appendix D the free-field deformation method and the soil-structure interaction method.

Table 7. Methods of computing earthquake forces in tunnels.

Approaches	Advantages	Disadvantages	Applicability
Dynamic earth pressure method	1.Used with reasonable results in the past 2.Require minimal parameters and computation error 3.Serves as an additional safety measure against seismic loading	1.Lack of rigorous theoretical basis 2. Resulting in excessive racking deformations for tunnels with significant burial 3. Use limited to certain types of ground properties	For tunnels with minimal soil cover thickness
Free-field racking deformation method	1.Conservative for tunnel structure stiffer than ground 2.Comparatively easy to formulate 3.Used with reasonable results in the past	1.Non-conservative for tunnel structure more flexible than ground 2.Overly conservative for tunnel structures significantly stiffer than ground 3.Less precision with highly variable ground conditions	For tunnel structures with equal stiffness to ground
Soil structure interaction method	1.Best representation of soil-structure system 2.Best accuracy in determining structure response 3.Capable of solving problems with complicated tunnel geometry and ground condition	1.Requires complex and time consuming computer analysis 2.Uncertainity of design seismic input parameters may be several times the uncertainty of the analysis	All conditions
Simplified frame analysis model	1.Good approximation of soil-structure interaction 2.Comperatively easy to formulate 3.Reasonable accuracy in determining structure response	1.Less precision with highly variable ground	All conditions except for compacted subsurface ground profiles

Design

Buried structures should be designed for the maximum axial load, shear, and bending moments obtained from all seismic load cases. In addition to checking the loads from each loading direction separately, a seismic combination of 70% strains and stresses from the longitudinal direction and 70% strains and stresses from the transverse direction should also be checked. The R-factors for these structures are not given in any code or specifications of seismic design. The fact these structures are below ground and thus are not easy accessible, higher R-factors should be used to minimize damage. An R-factor equal 1.0 is recommended for critical buried structures. A higher R-factor for ordinary buried structures could be used if justified. Plastic hinge analysis can be sued for safety evaluation to check stability.

Design Procedures

1. The structure should first be designed with an adequate structural capacity under static loading conditions.
2. The structure should be checked for ductility when earthquake effects, EQ, are considered. For tunnel structures, the earthquake effect is governed by the displacements/deformation imposed on the tunnel by the ground.
3. In checking the strength capacity, effects of earthquake loading should be expressed in terms of internal moments and forces, which can be calculated according to the lining deformation (distortion) imposed by the surrounding ground. If the 'strength' criteria expressed by Eqs. (13), (14), (15), or (16) can be satisfied based on elastic structural analysis, no further provisions for safety level need to be considered. Generally, the strength criteria can easily be met when the earthquake loading intensity is low (i.e., in low seismic risk areas) and/or the soil is very stiff.
4. If the flexural strength of the tunnel lining, using elastic analysis and Eq. (15) or (16) is found to be exceeded, the structure should be checked for its ductility to ensure that the resulting inelastic deformation, if any, is small. If necessary, the structure should be redesigned to ensure the intended performance goals during the FE level.
5. If the flexural strength of the tunnel lining for the MCE level using elastic analysis and Eqs. (13) or (14) is exceeded (e.g., at certain joints of a cut-and-cover tunnel frame), one of the following two design procedures should be followed:
   a) Provide sufficient ductility (using proper detailing procedure) at the critical locations of the lining to accommodate the deformation imposed by the ground in addition to those caused by other loading effects. The intent is to ensure that the structural strength does not degrade as a result of inelastic deformation and the damage can be controlled at an acceptable level. However, since inelastic shear deformations may result in strength

degradation, it should always be prevented by providing sufficient shear strength in structural members, particularly in the cut-and-cover rectangular frame.

b) Re-analyze the structure response by assuming the formation of plastic hinges at the joints that are strained into inelastic action. Based on the plastic-hinge analysis, a redistribution of moments and internal forces will result. If new plastic hinges are developed based on the results, the analysis is re-run by incorporating new hinges (i.e., an iterative procedure) until all potential plastic hinges are properly accounted for. Proper detailing at the hinges is then carried out to provide adequate ductility. The structural design in terms of the required strength (Eqs.13 and 14) can then be based on the results from the plastic-hinge analysis. The overall stability of a tunnel structure during and after the MCE has to be maintained. Realizing that the structure also must have a sufficient capacity (besides the earthquake effect) to carry static loads (e.g., D, L, E1, E2 and H terms), the potential modes of instability due to the development of plastic hinges (or regions of inelastic deformation) should be identified and prevented.

6. The strength reduction factor, φ, used in the conventional design practice may be too conservative, due to the inherently more stable nature of underground structures (compared to aboveground structures), and the transient nature of the earthquake loading.

7. For cut-and-cover tunnel structures, the evaluation of capacity using Eq (13) or (15) should consider the uncertainties associated with the loads E1 and E2, and their worst combination. For circular lined tunnels, similar consideration should be given to the loads EX and H.

8. In many cases, the absence of live load L may present a more critical condition than when a full live load is considered. Therefore, a live load equal to zero should also be used in checking the structural strength capacity using Eq (13) and (14).

Detailing

Reinforcing steel in buried structure should be designed and detailed to provide the required strength and ductility. Where plastic hinge analysis is used, sufficient ductility should be provided at plastic hinge locations.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study is to evaluate the impact of the new recommended seismic design guidelines from NCHRP 12-49 on seismic design of bridges in New Jersey and to provide seismic design criteria and guidelines based on these guidelines. The study provides an overall review of the recommended guidelines; compares the guidelines to the current AASHTO LRFD specifications; and provides recommendations on seismic hazard and performance objectives and

soil site factors for the State of New Jersey. The proposed seismic design guidelines for seismic design of bridges in New Jersey are based on the design criteria from NCHRP 12-49 and they are consistent with the guidelines from other eastern states such as New York and South Carolina. Two examples were designed based on NCHRP 12-49 recommended guidelines and the current AASHTO LRFD specs showed that the soil site factors and the return period had a major effect on the deign as well as the stiffness of the structure. The examples also showed that transverse reinforcement requirements for large-diameter columns with heavy reinforcement in plastic hinge zones were higher in the new provisions compared to current AASHTO LRFD design. The report also provides seismic design criteria and guidelines for abutments, retaining walls, embankments, and buried structure in New Jersey.

Among the main issues that need to be addressed by states in the Northeastern United States are the design earthquake for collapse prevention and the soil site factors. The NCHRP 12-49 specified the 2 percent in 50 years (MCE) as the design event nationwide. This decision seems to be justified for the high seismic zones such as California and Alaska where many records of moderate-to-large earthquakes are available. The decision also seems to be justified for regions where very infrequent but very large earthquakes took place in the past such as the New Madrid region in the Mississippi embayment and Charleston, South Carolina. However, the adoption of the 2 percent PE in 50 years for the northeastern United States does not seem to have enough justification due to the absence of rare large historic earthquakes in the region. Moreover, the absence of such large historic earthquake would make difficult to convince the owners and bridge officials to adopt such a large design earthquake. Current research on blast resistance and security of bridges and the evaluation of critical structures in big cities for security concerns may lead to adopt larger events or much smaller probability of exceedance. These studies are going to confront the same issues NCHRP 12-49 and the earthquake engineering community has faced in terms of acceptable risks and the consequences of those risks.

The soil site factors adopted in NCHRP 12-49 were largely based on earthquake and soil data primarily in California. These soil site factors are much higher than those in the current AASHTO LRFD specs especially in regions subjected to very small ground motions. Soil site factors for the eastern United States need to be reevaluated based on ground motion and soil data characteristic of the eastern United States.

Conclusions

Based on the results of this study, the following conclusions can be drawn:

1. Seismic performance criteria and seismic design criteria and guidelines consistent with NCHRP 12-49 guidelines are provided for integral abutments, retaining walls, embankments, and buried structures in New Jersey.

2. MCE (Maximum Considered Earthquake) level adopted by NCHRP 12-49, which has a 2 percent PE in 50 years (2500 years return period) is an acceptable level for safety evaluation for all new bridges in New Jersey. Seismic performance criteria consistent with those of NCHRP 12-49 are proposed. This ground motion level was first proposed by NEHRP for seismic design of buildings. It is also used by SCDOT, and NYCDOT. Although there are no records of a major earthquake in the Northeastern United States, for the time being, this hazard level provides a uniform level of safety.

3. Soil site factors have increased dramatically for soft soils subjected small ground motions. These factors will have a major impact on the design response spectra and the selection of the seismic hazard level in Central and South Jersey.

4. All counties in New Jersey that have soft soil conditions do not require seismic analysis and design for the lower-level (EXP) earthquake specified in NCHRP 12-49. Seismic hazard levels in all counties are Level I, hence SDAP A is used. SDAP A does not require seismic design; rather it specifies minimum design forces, transverse confining steel, and minimum seat width at piers and abutments.

5. New NCHRP 12-49 guidelines provide many options in seismic analysis and design procedures that will help bridge designers. For example, SDAP C, D, and E could be used in most cases and the reduction factors of elastic seismic loads (R) are tabulated in more detail for various cases. It also provides additional information on the analysis of integral and seat abutments, and foundation stiffness more than the current specs.

6. The Capacity Design Spectrum Analysis (SDAP C) design and analysis procedure given in NCHRP 12-49 is relatively simple and can be used for seismic design of many bridges in New Jersey. Design aids for this procedure can also be prepared to further simplify the procedure.

7. Transverse column reinforcement in plastic hinge zones is significantly affected by the longitudinal steel ratio. This reinforcement is independent of the longitudinal steel in the existing provisions. For column diameters

between 3 ft and 6 ft with 1 and 2 percent longitudinal steel, the existing specifications require more transverse reinforcement. For column diameters between 5 ft and 6 ft with 3 percent steel or more, the new NCHRP 12-49 provisions require more transverse reinforcement.

Based on the results of the two design examples, the following conclusions can be drawn:

1. The longitudinal steel requirements in example 2 (integral abutment bridge) were lower than those for example 1 (seat-type abutment). This was true for the AASHTO LRFD specifications and the NCHRP 12-49 specifications. This trend was observed for all regions in New Jersey. The same observation was true for pile requirements. Because of the integral abutments, less seismic loads were transferred to the piers, hence reducing its column steel and pile requirements.

2. For Operational Level design, the 4 percent maximum limit on the longitudinal steel ratio in concrete columns in NCHRP 12-49 will result in larger size columns in Central and Southern New Jersey compared to AASHTO LRFD current provisions.

3. Transverse column reinforcement in plastic hinge zones is significantly affected by the longitudinal steel ratio. This reinforcement is independent of the longitudinal steel in the existing provisions. For column diameters between 3 ft (0.91 m) and 6 ft (1.83 m) with 1 and 2 percent longitudinal steel, the existing specifications require more transverse reinforcement. For column diameters between 5 ft (1.52 m) and 6 ft (1.83 m) with 3 percent steel or more, the new NCHRP provisions require more transverse reinforcement.

4. The minimum seat width required in NCHRP 12-49 is higher than those in AASHTO LRFD. This means that wider abutment walls are needed according to the NCHRP 12-49 provisions. The design examples showed that the NCHRP 12-49 minimum seat widths were about 60 to 70 percent higher than those required in the current AASHTO LRFD specifications.

Recommendations

Based on the results of this study, the conclusions presented above, and the principal investigators discussions and private communications with members of seismic committees of various engineering societies and organizations, experts, and other researchers in the area of earthquake engineering and seismic design of bridges, the following recommendations are made:

1. Adopt seismic hazard and performance levels based on NCHRP 12-49 ground motions with modifications to be consistent with those adopted by NYCDOT and SCDOT. Safety Level design for 'Critical bridges' shall be based on the MCE event, while Operational Level design shall be based on the 500-year event. Safety level design for 'non-critical bridges' (Other bridges) shall be based on 2/3 of the MCE. Minimum seat width at abutments and expansion piers shall be based on the MCE rather than 2/3 of the MCE. The Operational Level design for "Other bridges" will be based on the EXP earthquake in NHCRP 12-49.

2. Use of SDAP E for seismic analysis and design of bridges is recommended. SDAP E allows designers to use higher R-factors compared to SDAP D, but requires verification through use of a pushover analysis. Pushover analysis is being used more frequently in design and most computer analysis programs have this feature.

3. SDAP C is permitted in NCHRP 12-49 for bridges meeting certain criteria. This procedure is simple and should be recommended for design because it covers a large number of bridges.

4. There is a need for research to predict large, infrequent earthquakes or extreme earthquake events for New Jersey and to prepare seismic hazard maps for seismic design in New Jersey. Also, there is a need to re-evaluate soil-site factors proposed by NCHRP 12-49 for New Jersey and the northeastern United States. This is a large undertaking and it may require the formation of a consortium of universities, agencies, and organizations in the northeast with expertise in seismology, geology, soil dynamics, risk analysis and management, and seismic design.

4. Current service and damage levels adopted in NCHRP 12-49, current AASHTO LRFD, SCDOT, and NYCDOT are qualitative and are open to interpretation on what constitutes minimal damage, significant damage, etc…. A quantitative assessment of damage similar to those proposed in the SEAOC Bluebook [9] for buildings that use the concepts of capacity and demand, and the ratios between, provide a more objective approach. These need to be considered.

APPENDIX A

ABUTMENT DESIGN EXAMPLES

EXAMPLE ONE – SEAT TYPE ABUTMENT BRIDGE

This example is a seat-type abutment non-skewed (straight) bridge with two column bents designed for three different regions in New Jersey: north, central, and south. The example was designed using both specifications: NCHRP 12-49 and current AASHTO LRFD. A typical two-column bent highway bridge was considered, idealized and modeled for each of the aforementioned regions, the seismic coefficients for each bridge were established and the response spectrum curves were developed. An Elastic Response Spectrum Analysis was performed using the SAP2000 Nonlinear analysis program and the results from the different analyses were tabulated. Elastic analysis forces were combined as per the provision requirements, and were adjusted with the proper response modification factors (R) stipulated by the provisions. The modified elastic design forces were then used to evaluate the substructure column and pile requirements.

Bridge Description

The bridge being considered in example one is a three-span prestressed concrete structure. The three spans are 80 ft, 100ft, and 80 ft, with two intermediate bents. The columns are 20 ft high and 4 ft in diameter circular section. The structure is founded on precast concrete piles, and the abutments are assumed to provide no restraint in the longitudinal direction. In the transverse direction, the transverse stiffness included the piles and a portion of the abutment wing walls. The superstructure was assumed to allow free rotation about a vertical axis at each of the supports and the spans were
continuous over the bents (fixed supports). Bridge elevation, plan, and cross sections of example one are shown in Figure 19.

Loads and Section Properties

Precast Prestressed concrete girders:
AASHTO Bulb-Tee Sections, 0.8 x 4 = 3.2 k/ft
Concrete Slab 8 inch thick = (8/12) x 44 x 0.15 = 4.4 k/ft
Barriers = 2 x 0.3 k/ft = 0.6 k/ft
Future Wearing Surface = 1.0 k/ft

Total dead weight = 3.2 + 4.4 + 0.6 + 1.0 = 9.2 k/ft

CL Abut   CL Pier   CL Pier   80 ft
    80 ft   100 ft
    20 ft   Elevatio   4 ft

44 ft
Plan

44 ft
8 in
AASHTO-PCI
Type IV
3 spaces @ 12'-
Section

20 ft   4 ft
5 ft
30 ft
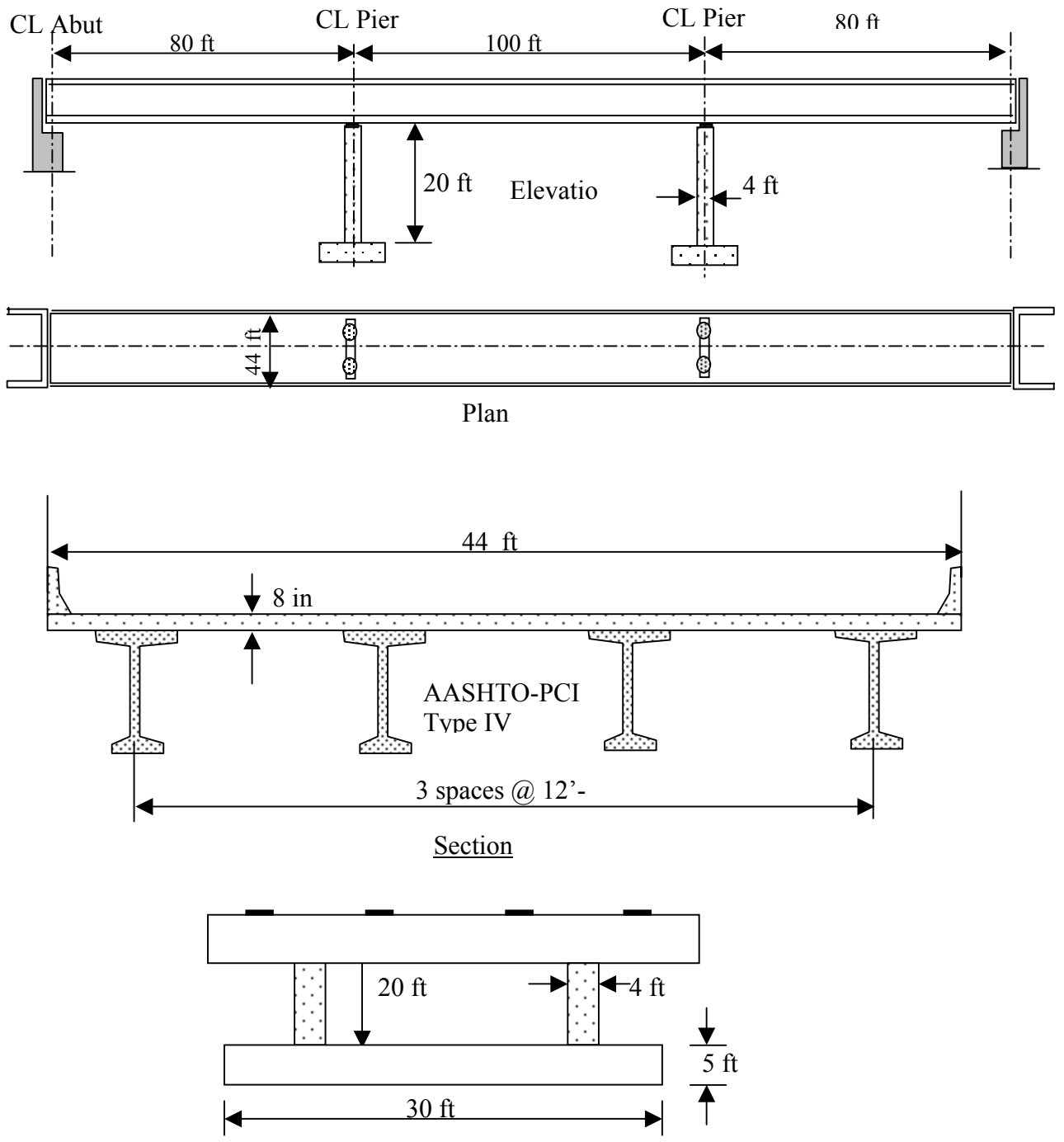
Figure 19. Plan, elevation, and sections of seat-type abutment example.

Properties of Superstructure:
W = 9.2 k/ft
Area = 50 ft^2
Ix-x = 235 ft^4
Mass = 0.285 k-s^2/ft

Column Properties:
W = 3.75 k/ft
Area = 25.1 ft^2
Ix-x = 25.1 ft^4
Mass = 0.11 k-s^2/ft

Footing Properties
Footing Size 30' x 15'
W = 68 k/ft
Area = 450 ft^2
Ix-x = 8400 ft^4
Iy-y = 33750 ft^4
Mass = 2.1 k-s^2/ft

Substructure Stiffness (AAHSTO LRFD)

For the seat-type abutment, the **substructure stiffness includes that of the piers and the abutments in the transverse direction. The current AASHTO LRFD provisions do not have guidelines on obtaining pier longitudinal and pier transverse stiffness neither on calculating abutment transverse stiffness. However, it is reasonable to use the following equation for foundation stiffness at the piers:**

$$K_{pier} = K_{pile-bending} + K_{soil} + K_{cap}$$

$$K_{pier} \cong K_{pile-bending} + K_{soil} + K_{cap} = (N_{piles})(40\,k\,/\,in\ per\ pile)$$

The 40 k/in is a reasonable value for piles based on pile load tests [11].

Assuming 16 piles per bent,

$K_{pier\ L} = K_{pier\ T} = $ (16) x (40 k/in) = 640 k/in = 7680 k/ft

<u>Abutment transverse stiffness:</u>

Current AASHTO LRFD provisions do not have guidelines on obtaining abutment transverse stiffness, however, the 1999 Caltrans [11] provide the following equation for calculating $K_{Transverse}$ at the abutment:

K_{Teff} = K wingwalls + Kpiles
Assuming 10 piles at the abutment.
Use 5 ft effective wing walls
Kwing walls = (1.33) x (20 k/in) x 5 x (5/5.5) = 1450 k/ft
K piles = (10) x (40 k/in per pile) = 4800 k/ft
Use $K_{T\ eff}$ = 1450+4800 = 6250 k/ft

Substructure Stiffness (NCHRP 12-49)

For piers – Stiffness is similar to those of AASHTO LRFD

For abutment transverse stiffness

$K_{T\ eff}$ = K wingwalls + Kpiles

Assuming 5 ft effective wing walls

K wingwalls = (1.33)x(2H/3)x5x5/(0.02H) = 1100 k/ft
K piles = (10)x (40 k/in per pile) = 4800 k/ft
Use $K_{T\ eff}$ = 1100+4800 = 5900 k/ft

<u>Modeling</u>

The seat abutment bridge was modeled using the SAP2000 program Version 7.1. The program is capable of performing a three-dimensional structural analysis using response spectrum or time history methods. A three-dimensional mathematical model was created to reflect the geometry, boundary conditions, and material behavior of the considered bridge. A continuous mass approach was used instead of lumped masses, hence increasing the accuracy of the dynamic analysis results. The pile foundation was modeled by restraining vertical displacement and rotations but provide translational springs in the longitudinal and transverse directions, consistent with the expected deformations of the pile cap and the surrounding soil. This pier stiffness was represented by two orthogonal springs as shown in Figure 20. The stiffness of these springs was calculated including the stiffness of the passive pressure on the pile cap and the piles. These springs account for pile lateral stiffness and soil passive pressures in their respective directions. The gross moment of inertia for the columns was used in the analysis based on the current AASHTP LRFD specifications. A multi

modal linear elastic response spectrum analysis was performed using response spectra curves developed in accordance with each set of specifications. The damping ratio for the model was taken as 0.05 (5 percent). Three load cases were defined for the structure; the first is a static gravity dead load, while the other two incorporate the seismic excitations in the longitudinal and transverse directions respectively. Load combinations were done according AASHTO LRFD as explained later in the example.
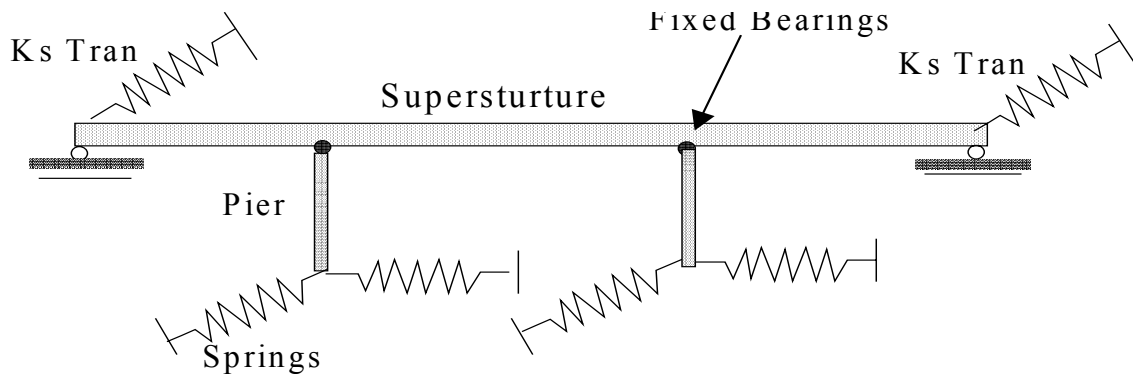
Figure 20   Model of Seat-Type Abutment Bridge

EXAMPLE TWO – INTEGRAL ABUTMENT BRIDGE

This example is an integral abutment non-skewed (straight) bridge with two column bents designed for three different regions in New Jersey: north, central, and south. The example was designed using both specifications: NCHRP 12-49 and current AASHTO LRFD. A typical two-column bent highway bridge was considered, idealized and modeled for each of the aforementioned regions, the seismic coefficients for each bridge were established and the response spectrum curves developed. An Elastic Response Spectrum Analysis was performed using the SAP2000 Nonlinear program and the results from the different analyses were tabulated.

Elastic analysis forces were combined as per the code requirements, and were adjusted with the proper response modification factors stipulated by the code provisions. The modified elastic design forces were then used to evaluate the substructure column and pile requirements.

Bridge Description

The bridge being considered in example 2 is a three-span prestressed concrete structure. The three spans are 80 ft, 100ft, and 80 ft, with two intermediate bents and two integral abutments 50 ft wide each. The columns are 20 ft high and have 4 ft diameter circular section. The structure is founded on precast concrete piles. In the transverse direction, the transverse stiffness included the piles and portion of the abutment wing walls. The superstructure was assumed to have monolithic connections with the abutment walls, and spans were considered continuous over the bents (fixed supports). Bridge elevation and plan are shown in Figure 21 and the integral abutment cross section is shown in Figure 22. The bridge carries a dead load and a superimposed dead load equal to 9.2 kips/ft.
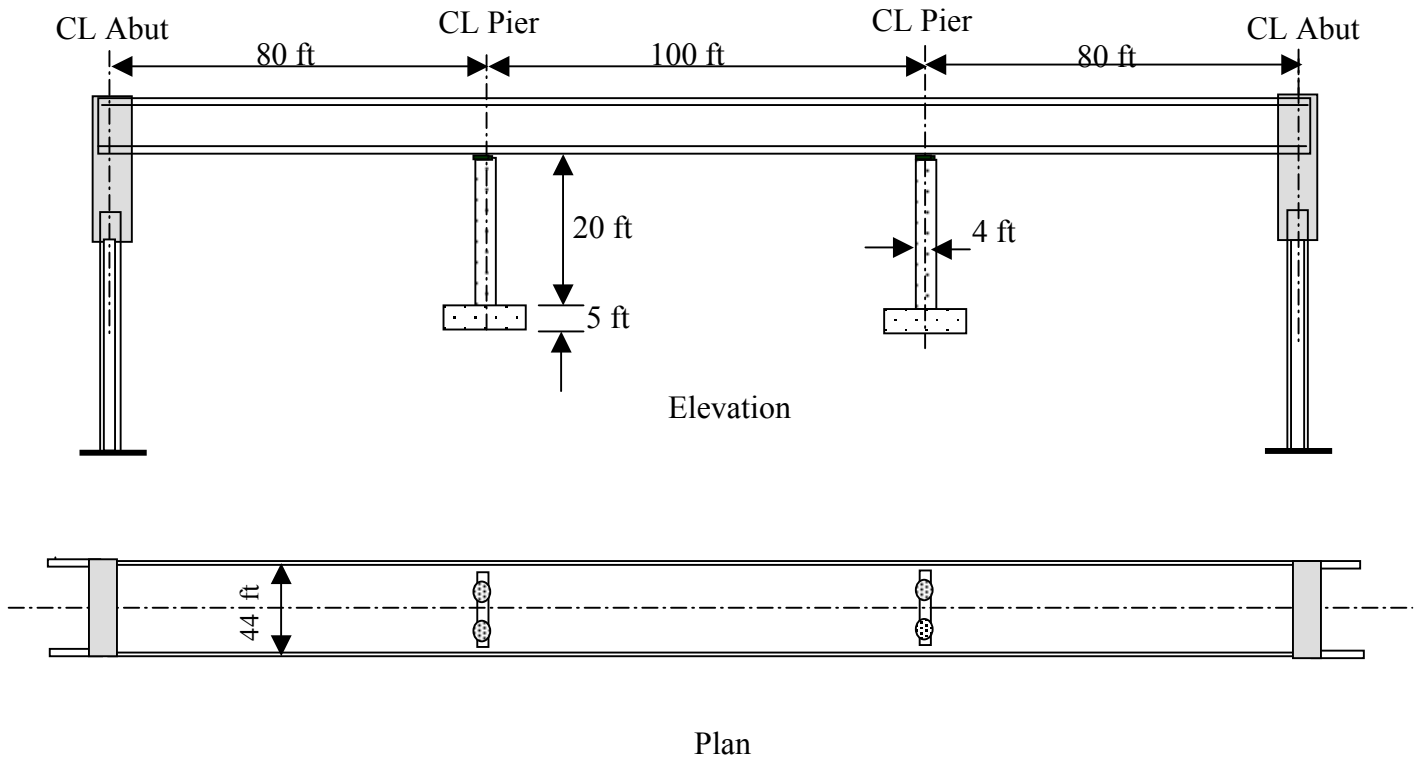
Figure 21.   Plan and elevation of integral abutment example.
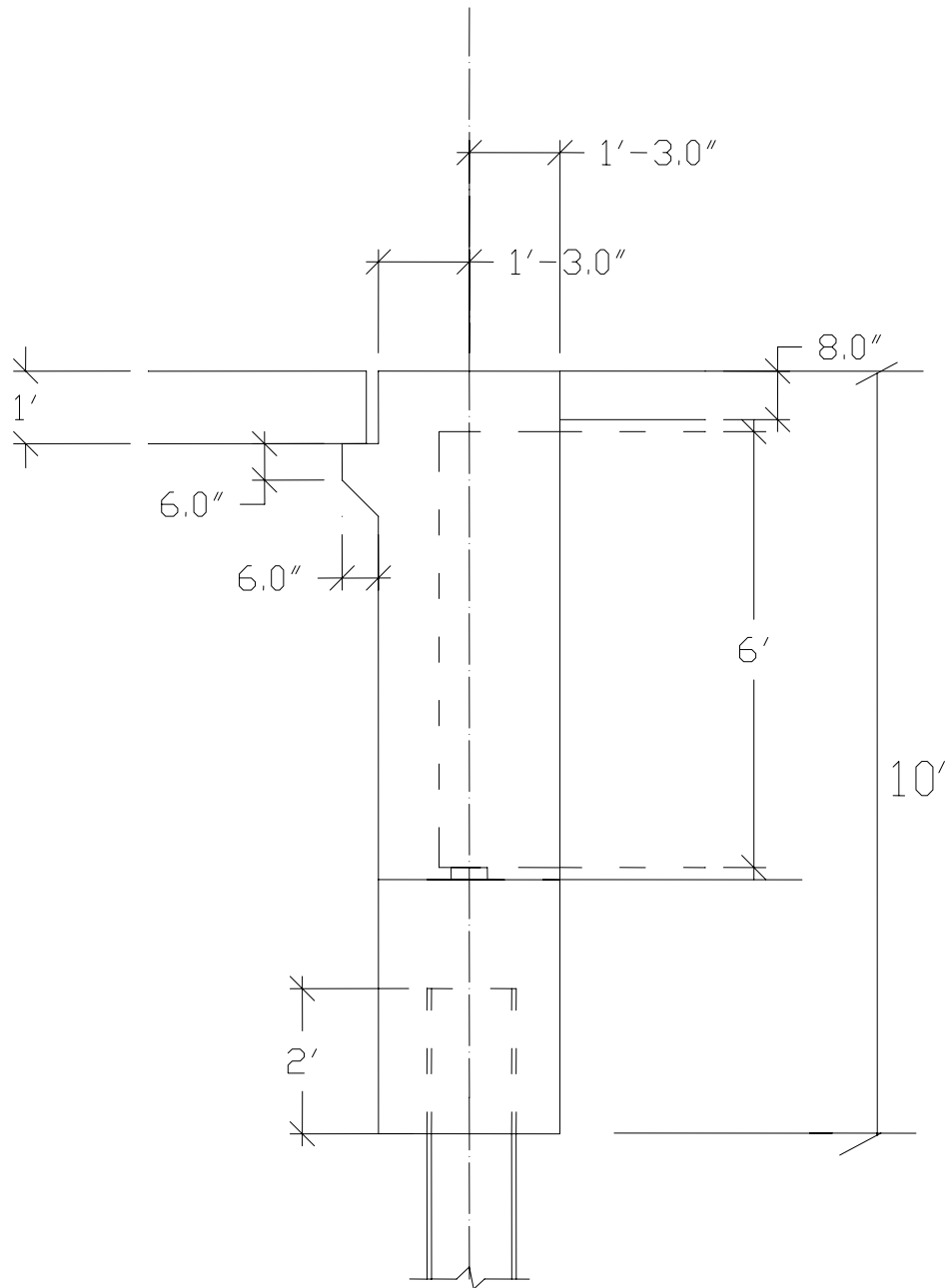
Figure 22.   Integral abutment cross-section.

Loads and Section Properties

<u>Precast Prestressed concrete girders:</u>
AASHTO Bulb-Tee Sections, 0.8 x 4 = 3.2 k/ft
Concrete Slab 8 inch thick = (8/12) x 44 x 0.15 = 4.4 k/ft
Barriers = 2 x 0.3 k/ft = 0.6 k/ft
Future Wearing Surface = 1.0 k/ft

<u>Total dead weight = 3.2 + 4.4 + 0.6 + 1.0 = 9.2 k/ft</u>

<u>Properties of Superstructure</u>:
W = 9.2 k/ft
Area = 50 ft^2
Ix-x = 235 ft^4
Mass = 0.285 k-s^2/ft

<u>Column Properties:</u>
W = 3.75 k/ft
Area = 25.1 ft^2
Ix-x = 25.1 ft^4
Mass = 0.11 k-s^2/ft

<u>Footing Properties</u>
Footing Size 30' x 15'
W = 68 k/ft
Area = 450 ft^2
Ix-x = 8400 ft^4
Iy-y = 33750 ft^4
Mass = 2.1 k-s^2/ft

<u>Αbutment Wall Properties</u>
Size 50' x 10'x 2'-6"
W = 18.75 k/ft (unit height)
Area = 125 ft^2
Ix-x = 65.1 ft^4
Iy-y = 26041 ft^4
Mass = 0.582 k-s^2/ft

Substructure Stiffness (AASHTO LRFD)

For the integral abutments, **substructure stiffness includes that of the piers and the abutment in the transverse and longitudinal directions. The current AASHTO LRFD provisions do not have guidelines on obtaining pier longitudinal and pier transverse stiffness neither on calculating abutment transverse and longitudinal stiffness. However, it is reasonable to use the following equation for foundation stiffness at the piers:**

$$K_{pier} = K_{pile-bending} + K_{soil} + K_{cap}$$

$$K_{pier} \cong K_{pile-bending} + K_{soil} + K_{cap} = \left(N_{piles}\right)\left(40\,k\,/\,in\ per\ pile\right)$$

The 40 k/in is a reasonable value for piles from Caltrans [11] based on pile load tests. Assuming 16 piles per bent:

$K_{pier\ L}$ = $K_{pier\ T}$ = (16) x (40 k/in) = 640 k/in = 7680 k/ft

Abutment transverse stiffness:
The current AASHTO LRFD provisions do not have guidelines on obtaining abutment transverse stiffness, however, Caltrans (1999) provides the following equation for calculating $T_{ransverse}$ at the abutment:

K_{Teff} = K wing walls + Kpiles
Assuming 10 piles at the abutment.
Use 5 ft effective wing walls
Kwing walls = (1.33) x (20 k/in) x 5 x (5/5.5) = 1450 k/ft
K piles = (10) x (40 k/in per pile) = 4800 k/ft
Use $K_{T\ eff}$ = 1450+4800 = 6250 k/ft

For longitudinal abutment stiffness (passive pressure)
Use Caltrans (2001)/ ATC (1996)
$K_{L\ eff}$ = (20 k/in) x (W) x (H/5.5) + K piles (neglected)
For W = 50 ft, H = 10 ft,
Use $K_{L\ eff}$ = 21,800 k/ft

Substructure Stiffness (NCHRP 12-49)

For piers – Stiffness is similar to those of AASHTO LRFD

For abutment transverse stiffness

$K_{T\ eff}$ = K wing walls + Kpiles
Assuming 5 ft effective wing walls

K wing walls = (1.33) x (2H/3) x5x5/(0.02H) = 1100 k/ft
K piles = (10) x (40 k/in per pile) = 4800 k/ft
Use $K_{T\ eff}$ = 1100+4800 = 5900 k/ft

For abutment longitudinal stiffness for granular Soils

$K_{L\ eff}$ = (2H/3) x (H) x (W)/(0.02H) + K piles (neglected)
For W = 50 ft, H = 10 ft,
Use $K_{L\ eff}$ = 16,667 k/ft

Modeling

The integral abutment bridge was modeled using the SAP2000 Version 7.1 program. The program is capable of performing a three-dimensional structural analysis using response spectrum or time history methods. A three-dimensional mathematical model was created to reflect the geometry, boundary conditions, and material behavior of the considered bridge. A continuous mass approach was used instead of lumped masses, hence increasing the accuracy of the dynamic analysis results. The pile foundation was modeled by restraining vertical displacement and rotations but provide translational springs in the longitudinal and transverse directions, consistent with the expected deformations of the pile cap and the surrounding soil. This pier stiffness was represented by two orthogonal springs as shown in Figure 23. The stiffness of these springs was calculated including stiffness of the passive pressure on the pile cap and the piles. These springs account for pile lateral stiffness and soil passive pressures in their respective directions. The gross moment of inertia for the columns was used in the analysis based on the current AASHTP LRFD specifications. An effective moment of inertia was used based on NCHRP 12-49. The longitudinal and transverse stiffness of the integral abutment were calculated and idealized as longitudinal and transverse springs at each abutment location. The vertical piles below the abutment were modeled using the length of fixity approach. The length of fixity for steel piles was estimated to be 20 ft. The structural model for the integral abutment example is shown in Figure 23. A multi modal linear elastic response spectrum analysis was performed using response spectra curves developed in accordance with each set of specifications. The damping ration for the model was taken as 0.05 (5 percent). Three load cases were defined for the structure; the first is a static gravity dead load, while the other two incorporate the seismic excitations in the longitudinal and transverse directions respectively. Load combinations were done according AASHTO LRFD as explained later in the example.
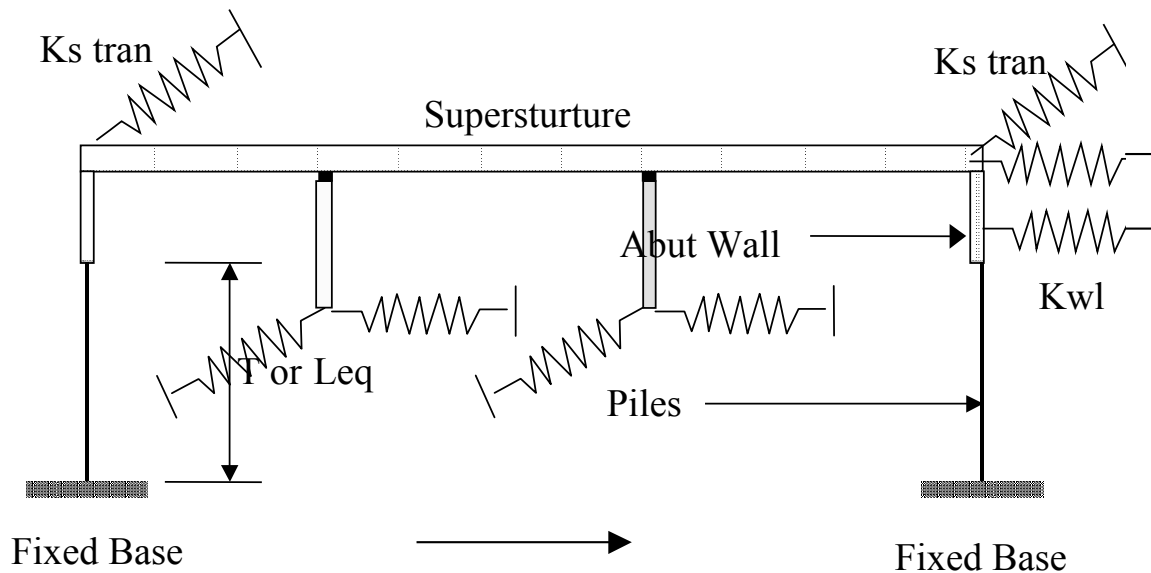
Leq = Pile Equiv Length to Point of Fixity

Figure 23. Model of integral abutment example.

DESIGN GROUND MOTION AND RESPONSE SPECTRA

AASHTO LRFD Specifications

The current AASHTO-LRFD specifications establish the effect of the design ground motion through the elastic seismic response coefficient C_{sm}, which is used to build the response spectrum curve. The elastic seismic response coefficient C_{sm} is defined by the formula:

$$C_{sm} = (1.2 \ A \ S / T^{2/3}) \leq 2.5A \qquad\qquad (21)$$

Where:
T = Structural Period in seconds
A = Acceleration Coefficient
S = Site coefficient
the acceleration coefficient A is determined from contour maps prepared by the U.S Geological Survey USGS, and adopted by the code. These maps are based on a uniform risk model of seismic hazard, in which the probability of the design earthquake being exceeded during a period of 50 years is 10%. This probability of exceedance implies a return period of the design earthquake of about 475

years. Figure 24 shows the USGS seismic contour map for the PGA (Peak Ground Acceleration) in New Jersey. The PGA coefficients for various locations in New Jersey are taken from the NJDOT Seismic Design guidelines and are shown in the Table 8.

Table 8. Design PGA for Various Regions in New Jersey [8]

Geographical Region	Acceleration coefficient A
North Jersey	0.18
Central Jersey	0.15
South Jersey	0.10

For all regions in New Jersey
0.09 < A < 0.19, then Seismic Zone 2
Soil Profile Type   = IV   (Soft Clays, or silts Greater than 40.0ft in depth)
Site Coefficient S = 2.0
Response Spectrum: (% of g)
$C_{sm} = 1.2*A*S / Tm^{(2/3)} < 2.5A$
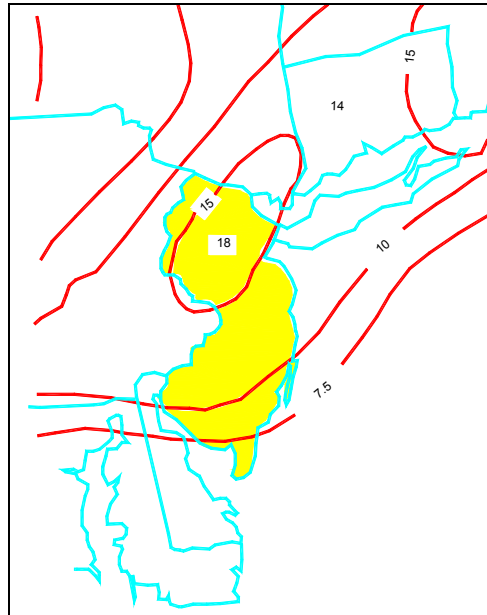
Figure 24. AASHTO-LRFD Acceleration Coefficient Map for the State of New Jersey (USGS Maps)
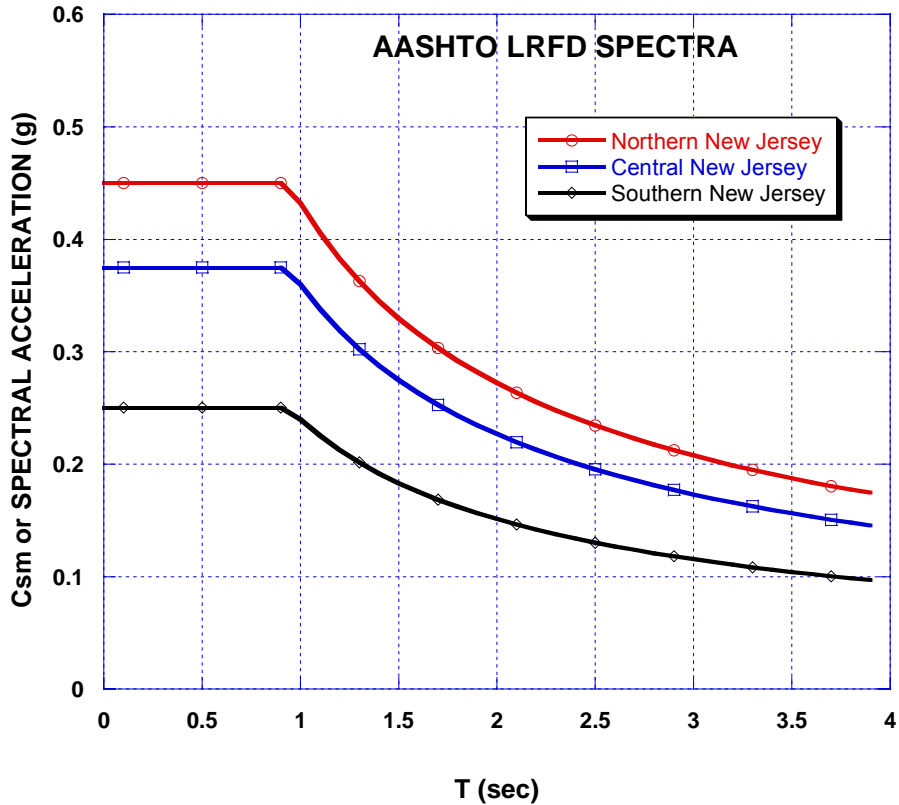
Figure 25. AASHTO LRFD design response spectra for northern, central, and southern New Jersey

The effect of soil conditions at the site is considered through the site coefficient S, the determination of which depends on the soil profile type. Four soil profile types are used in the AASHTO LRFD specifications to define a site coefficient S that is used to modify the acceleration coefficient. These soil profiles are representative of different subsurface conditions, the description of which was given in the specifications in article 3.10.5. For the purpose of this study, a soil profile type IV was assumed to correspond to the site sub grade conditions of the considered bridge structure. This would represent the worst condition soil profile that may be encountered for seismic effect considerations, and is characterized by a site coefficient value of: S = 2.0. Figure 25 shows the design response spectra according to the AASHTO LRFD provisions.

According to the AASHTO LRFD specifications, each bridge structure shall be assigned to one of four seismic zones. The definition of these zones relies on the value of the acceleration coefficient A. These zones reflect the variation in the seismic risk across the country, and are used to permit different requirements for methods of analysis, design, and detailing of the structural elements of the bridge. Under the current AASHTO LRFD, all bridges built in the state of New Jersey will likely fall in seismic Zone 2. All bridges considered in this study fell in seismic Zone 2. Based on this information, the response spectra curves were developed for all three New Jersey state geographic regions. The different

response spectra based on the AASHTO LRFD specifications for North, Central and South New Jersey were plotted on the same chart for comparative review as shown in Figure 25.

NCHRP 12- 49 Guidelines

The NCHRP 12-49 is a performance based set of guidelines that defines seismic performance level objectives in association with the design earthquake ground motions. Bridges shall be designed to satisfy the performance criteria requirements given by the guidelines upon enduring the design seismic earthquake levels. Two distinctively different earthquake levels are considered for the design purposes under the new provisions, an upper level, and a lower level. The upper level earthquake considered is designated as the *Rare event earthquake* or the *Maximum Considered Earthquake* (MCE), which is defined as the event with a probability of exceedance of approximately 3% in 75 years, reflecting an approximate return period of 2500 years. The lower level design earthquake is designated as the *Expected Earthquake* (EXP), and is based on a 50% probability of exceedance in 75 Years, which corresponds to an approximate return period of 108 years.

The two seismic performance objectives set by NCHRP 12-49 are Life Safety performance level, and Operational. The Operational performance level implies that there will be no disruption to functionality, and that there will be immediate service and minimal damage for both the MCE and the EXP level earthquakes. The Life safety means that the bridge will not collapse but may suffer some damage requiring partial or complete replacement in the MCE event. While the EXP level event will inflect minimal damage that will not affect the bridge availability for immediate service.

Construction of NCHRP 12- 49 Response Spectrum Curves

Design earthquake response spectral acceleration curves, under the new proposed NCHRP 12-49 specifications, are constructed using the accelerations from the national ground motion maps and site coefficients given by the specifications for each soil class type.
The design earthquake response spectral acceleration for the short periods S_{DS}, and for the long period S_{D1}, are given by:

$$S_{DS} = F_a. S_S \qquad\qquad (22)$$
$$S_{D1} = F_v. S_1 \qquad\qquad (23)$$

Where, S_S and S_1 are the 0.2 second (short period) spectral acceleration, and the 1.0 second (long period) spectral acceleration respectively. These accelerations are obtained from the USGS national ground motion maps (Uniform Hazard response spectra for the United States, by Frankel and

Leyendecker, USGS 2002). Figures 26 and 27 show USGS Maps for New Jersey for the MCE and EXP earthquakes in NCHRP 12-49.

F_a and F_v are the site coefficients given in Table 3.4.2.3-1 and 3.4.2.3-2 in NCHRP 12-49 for each site class and mapped short period and long period spectral accelerations respectively. The accelerations for the EXP earthquake are not available in contour maps similar to the MCE; however, these accelerations can be obtained using the hazard curves for the Annual Frequency of Exceedance (AFEX) given in the USGS Maps. These accelerations were tabulated for various counties in New Jersey in Table 6 in this report.

The design response spectrum curve under the NCHRP 12-49 is developed as follows:

For periods $T \leq T_0$, the curve is given by the equation:

$$S_a = 0.60 (S_{DS} / T_0) * T + 0.40 S_{DS} \qquad (24)$$

Where, $T_s = S_{D1} / S_{DS}$ and $T_0 = 0.2 T_s$

T = Period of Vibration (seconds)

For $T_0 \leq T \leq T_s$, $Sa = S_{DS}$ $\qquad (25)$

For $T \geq T_s$, $Sa = S_{D1} / T$ $\qquad (26)$
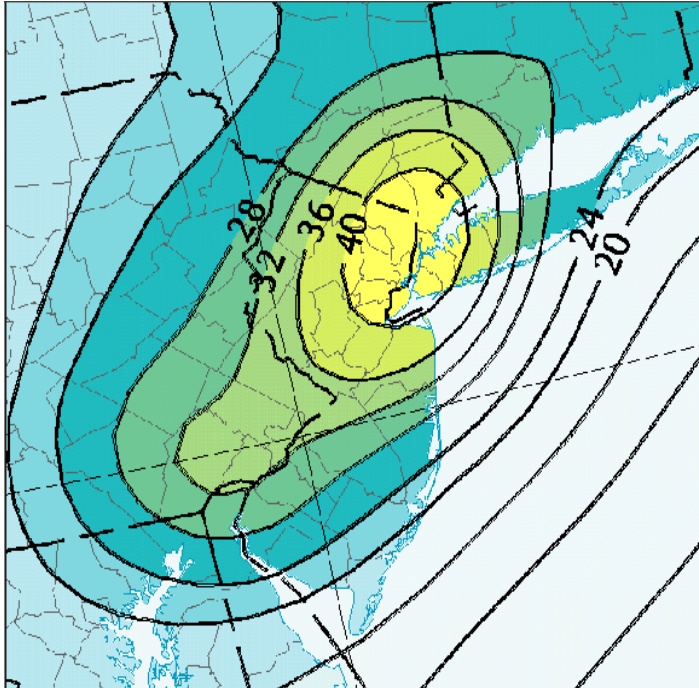
Figure 26.   USGS seismic hazard map of New Jersey for spectral acceleration
at 0.2 seconds for 3% PE in 75 years (MCE) ground motion.
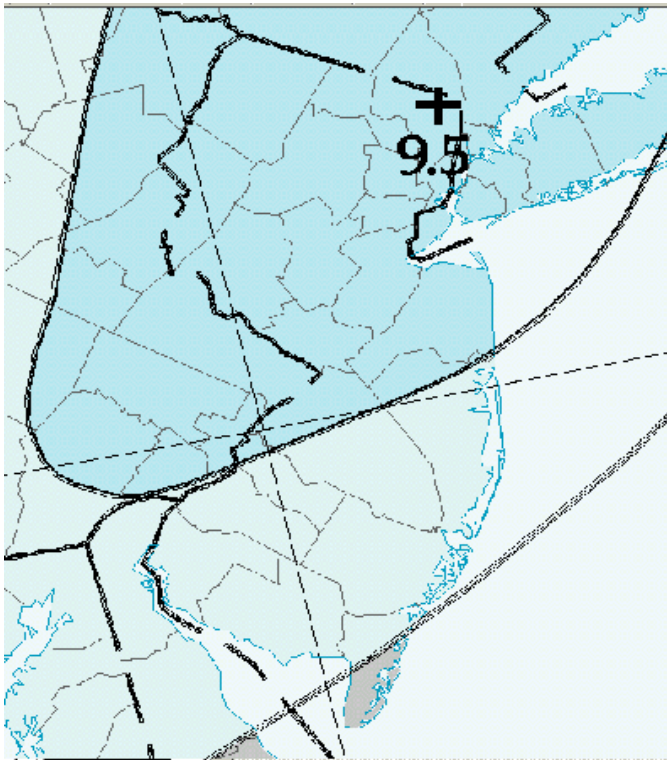
Figure 27. USGS seismic hazard map of New Jersey for spectral acceleration at
1.0 second period for 3% PE in 75 years (MCE) ground motion.

The NCHRP-12-49 set of guidelines does not provide cross referencing for the site class soil classifications with those of the AASHTO LRFD, therefore, site class E was considered equivalent to soil type IV in the AASHTO LRFD code, since both classifications indicate soft soil profile, and are representatives of the soft soil conditions. Based of the preceding discussion, the design response spectra curves were developed for the same geographical regions, i.e. North, Central, and South Jersey, for both, the MCE and the EXP earthquakes in NCHRP 12-49.

North, Central, and South New Jersey Spectral Accelerations and Soil Site Coefficients for MCE earthquakes

North New Jersey
NCHRP Project 12-49 Maximum Considered Earthquake (MCE)
Site Classification E
g = 32.2 ft/sec^2
S_s = 0.4g (from USGS Maps)
= 12.88 ft/sec^2
S_1 0.095g (from USGS Maps)
= 3.059 ft/sec^2
F_a = 2.02 (Interpolation from Table 3.4.2.3-1)
F_v = 3.50 Table 3.4.2.3-2, S_1<0.1
S_{DS} = $F_a. S_s$ = 0.808g = 26.018 ft/sec^2
S_{D1} = $F_v. S_1$ = 0.332g = 10.707 ft/sec^2
T_s = S_{D1}/S_{DS} = 0.411 Sec
T_0 = 0.2 T_s = 0.082 Sec
For T=0
S_a = 0.4* S_{DS} = 0.323g = 10.407 ft/sec^2

Central New Jersey
NCHRP Project 12-49 Maximum Considered Earthquake (MCE)
Site Classification E
g = 32.2 ft/sec^2
S_s = 0.36g (USGS Maps)
= 11.592 ft/sec^2
S_1 = 0.09g From USGS Maps, In terms of g
= 2.898 ft/sec^2
F_a = 2.15 (Interpolation in Table 3.4.2.3-1)
F_v = 3.50 Table 3.4.2.3-2, S_1<0.1
S_{DS} = $F_a. S_s$ = 0.775g = 24.89 ft/sec^2
S_{D1} = $F_v. S_1$ = 0.315g = 10.14 ft/sec^2
T_s = S_{D1}/S_{DS} = 0.407 Sec
T_0 = 0.2 T_s = 0.082 Sec.
For T=0,
S_a = 0.4* S_{DS} = 0.31g = 9.96 ft/sec^2

South New Jersey
NCHRP Project 12-49 Maximum Considered Earthquake (MCE)
Site Classification E

g = 32.2 ft/sec^2
S_s = 0.32g (USGS Maps)
= 10.31 ft/sec^2
S_1 = 0.08g (USGS Maps)
= 2.576 ft/sec^2
F_a = 2.28 (Interpolation in Table 3.4.2.3-1)
F_v = 3.50 Table 3.4.2.3-2, $S_1 < 0.1$
S_{DS} = $F_a \cdot S_s$ = 0.73g = 23.452 ft/sec^2
S_{D1} = $F_v \cdot S_1$ = 0.28g = 9.016 ft/sec^2
T_s = S_{D1}/S_{DS} = 0.384 sec
T_0 = 0.2 T_s = 0.077 sec
For: T=0 ;
S_a = 0.4* S_{DS} 0.292g = 9.38 ft/sec^2

North, Central, and South New Jersey Spectral Accelerations and Soil Site Coefficients for EXP earthquakes

North New Jersey
NCHRP Project 12-49 Expected Earthquake (EXP)
Site Classification E
g = 32.2 ft/sec^2
S_s = 0.127g (USGS Maps)
= 4.089 ft/sec^2
S_1 = 0.027g (USGS Maps)
= 0.8694 ft/sec^2
F_a = 2.50 (Interpolation in Table 3.4.2.3-1)
F_v = 3.50 Table 3.4.2.3-2, $S_1 < 0.1$
S_{DS} = $F_a \cdot S_s$ = 0.317g = 10.2235 ft/sec^2
S_{D1} = $F_v \cdot S_1$ = 0.095g = 3.0429 ft/sec^2
T_s = S_{D1}/S_{DS} = 0.298 Sec.
T_0 = 0.2 T_s =0.060 Sec.
For T=0,
S_a = 0.4* S_{DS} = 4.089 ft/sec^2

Central New Jersey
NCHRP Project 12-49 Expected Earthquake (EXP)
Site Classification E
g = 32.2 ft/sec^2
S_s = 0.11g (USGS Maps)

$= \quad 3.542 \ \text{ft/sec}^2$

$S_1 \quad = \quad 0.025\text{g} \ \text{(USGS Maps)}$

$= \quad 0.8694 \qquad \text{ft/sec}^2$

$F_a \quad = \quad 2.50 \quad \text{(Interpolation in Table 3.4.2.3-1)}$

$F_v \quad = \quad 3.50 \quad \text{Table 3.4.2.3-2, } S_1 < 0.1$

$S_{DS} \quad = \quad F_a . S_s \qquad = 0.275\text{g} = 8.855 \ \text{ft/sec}^2$

$S_{D1} \quad = \quad F_v . S_1 = 0.088\text{g} = 2.823 \ \text{ft/sec}^2$

$T_s \quad = \quad S_{D1}/S_{DS} \quad = 0.325 \ \text{sec}$

$T_0 \quad = \quad 0.2 \ T_s = 0.065 \ \text{sec}$

For: T=0 ;

$S_a \quad = \quad 0.4^* \ S_{DS} = 0.11\text{g} = 3.54 \ \text{ft/sec}^2$

South New Jersey
NCHRP Project 12-49 Maximum Considered Earthquake (MCE)
Site Classification E

$g \quad = \quad 32.2 \quad \text{ft/sec}^2$

$S_s \quad = \quad 0.09\text{g} \ \text{(USGS Maps)}$

$= \quad 3.22 \quad \text{ft/sec}^2$

$S_1 \quad = \quad 0.025\text{g} \ \text{(USGS Maps)}$

$= \quad 0.8694 \qquad \text{ft/sec}^2$

$F_a \quad = \quad 2.50 \quad \text{By Interpolation of table 3.4.2.3-1}$

$F_v \quad = \quad 3.50 \quad \text{Table 3.4.2.3-2, } S_1 < 0.1$

$S_{DS} \quad = \quad F_a . S_s \qquad = 0.225\text{g} = 8.05 \ \text{ft/sec}^2$

$S_{D1} \quad = \quad F_v . S_1 = 0.081\text{g} = 2.623 \ \text{ft/sec}^2$

$T_s \quad = \quad S_{D1}/S_{DS} \quad = 0.365 \ \text{Sec}$

$T_0 \quad = \quad 0.2 \ T_s = 0.073 \ \text{Sec}$

For: T=0 ;

$S_a \quad = \quad 0.4^* \ S_{DS} = 0.09\text{g} = 2.90 \ \text{ft/sec}^2$
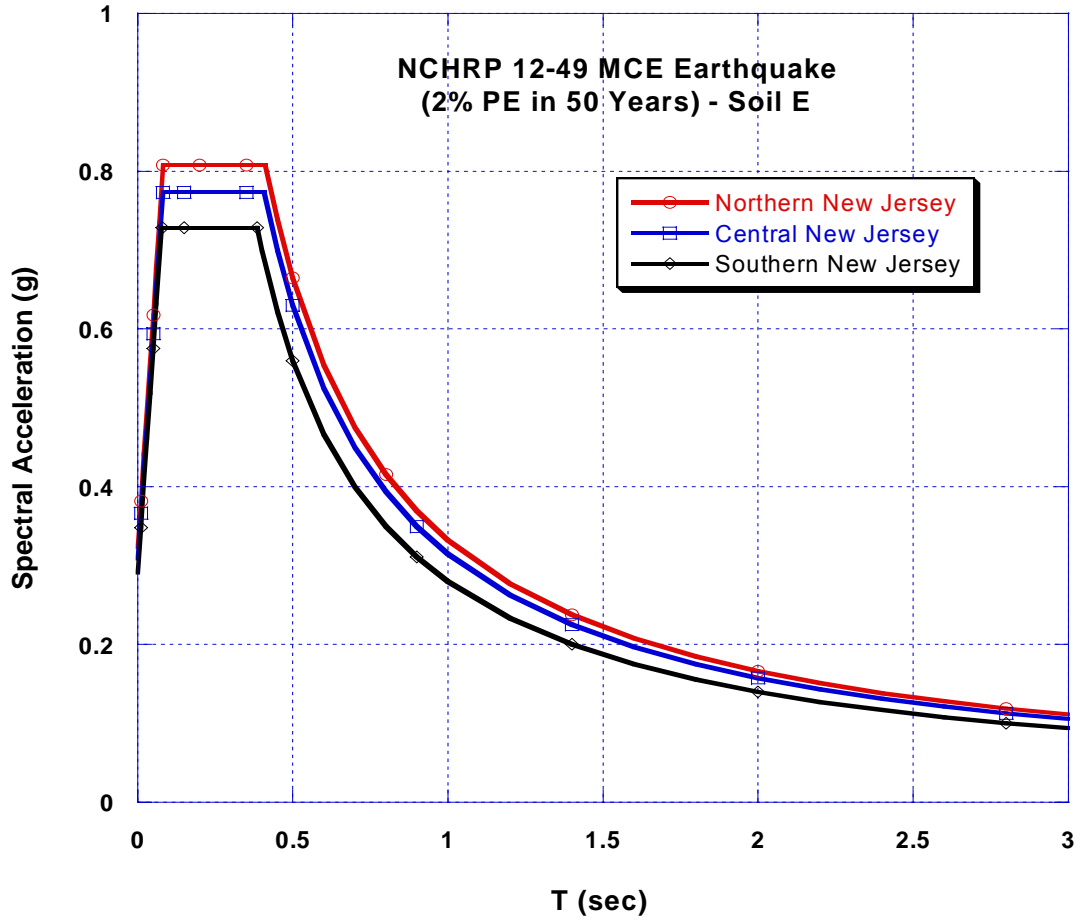
Figure 28. NCHRP 12-49 Design Response Spectra for the MCE
earthquake in soil class E in New Jersey.
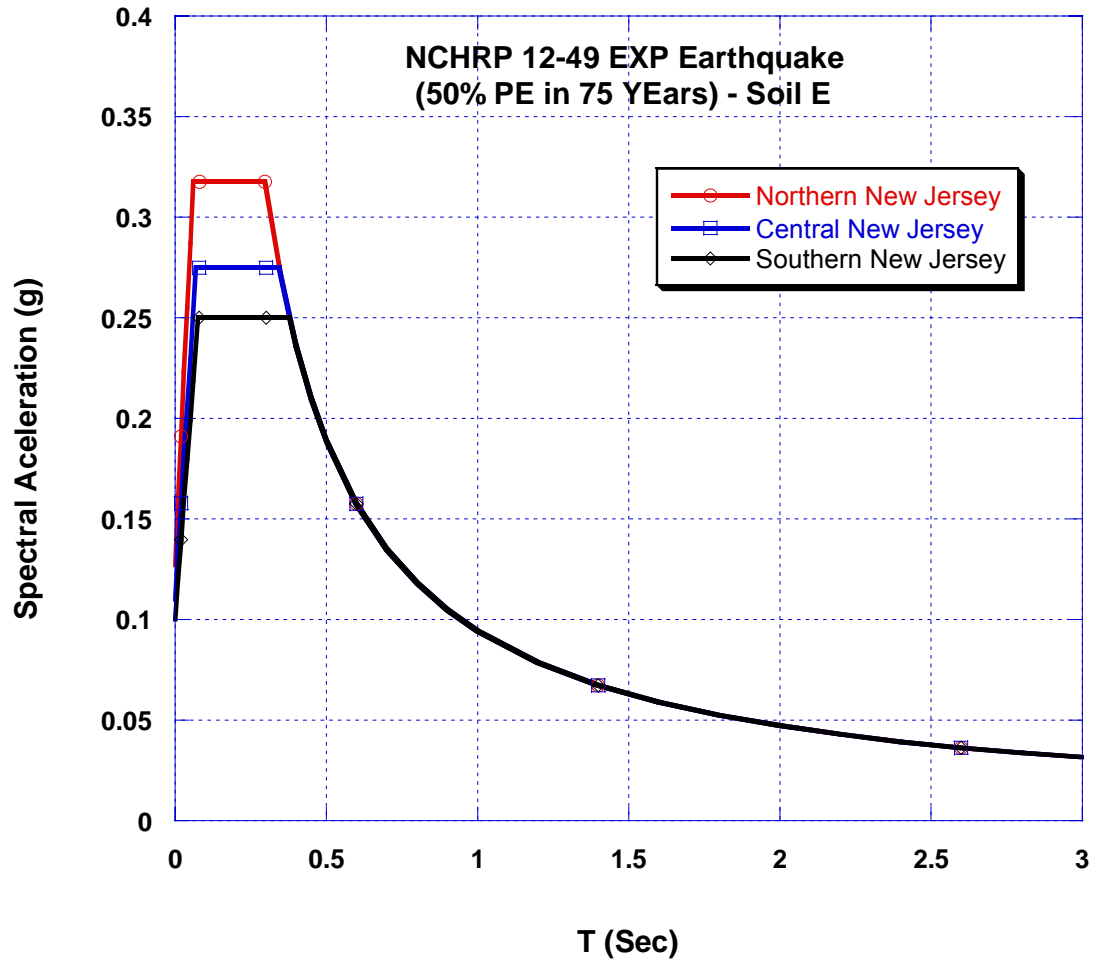
Figure 29. NCHRP 12-49 Design Response Spectra for the EXP earthquake in soil class E in New Jersey

Earthquake Load Combinations and Factored Loads

From AASHTO LRFD Specifications

The two load cases of orthogonal seismic force combinations specified by AASHTO LRFD to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions are LC1 and LC2. LC1 is Load Case 1 consisting of 100% of forces of the longitudinal motion plus 30 percent of forces of the transverse motion   (primarily longitudinal loading) and LC2 is Load Case 2 consisting of 100 percent of forces of the transverse motion plus 30 percent of forces of the longitudinal motion (primarily transverse loading).

The LC1 and LC2 for the elastic earthquake moments in the x-direction M_x are determined as follows:

$$M_x^{LC1} = 1.0 M_x^L + 0.3 M_x^T \tag{27}$$

$$M_x^{LC2} = 1.0 M_x^T + 0.3 M_x^L \tag{28}$$

Where M_x^L and M_x^T are the x-component from a longitudinal and transverse analysis. Similarly axial loads and shears are combined according to equations (27) and (28)   For biaxial moments, the maximum response quantities in the two directions from each load case shall be combined to determine the maximum vector moment from:

$$M = \sqrt{\left(M_x^{LC1}\right)^2 + \left(M_y^{LC1}\right)^2} \ \ \text{Or} \ = \sqrt{\left(M_x^{LC2}\right)^2 + \left(M_y^{LC2}\right)^2} \tag{29}$$

The LRFD load combination for extreme events including dead loads and earthquake loads only is given by

$$\text{Group Load} = 1.0 \ (D + EQ/R_{column}) \tag{30}$$

where
D = dead load
EQ/R_{column} = elastic seismic force from LC1 or LC2 divided by R_{column}.
For foundations, the design load combination is:

$$\text{Group Load} = 1.0 \ (D + EQ/R_{footing}) \tag{31}$$

The R-factors for columns and foundations for both, AASHTO LRFD and NCHRP 12-49 are given in Table 9. The forces and moments from the analysis are tabulated in Table 11 for both AASHTO LRFD and NCHRP 12-49.

From NCHRP 12-49 Guidelines

Elastic seismic design force effects shall be determined either by the Square Root of the Sum of the Squares (SRSS) combination or the 100 percent - 40 percent combination forces due to the individual seismic loads.

The SRSS combination of the response quantity is:

$$M_X = \sqrt{\left(M_X^L\right)^2 + \left(M_X^T\right)^2} \quad \text{and} \quad My = \sqrt{\left(M_y^L\right)^2 + \left(M_y^T\right)^2} \tag{32}$$

The two load cases of orthogonal seismic force combination specified by NCHRP 12-49 to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions are LC1 and LC2. LC1 is Load Case 1 consisting of 100 percent of forces of the longitudinal motion plus 40 percent of forces of the transverse motion   (primarily longitudinal loading) and LC2 is Load Case 2 consisting of 100 percent of forces of the transverse motion plus 40 percent of forces of the longitudinal motion (primarily transverse loading).

The LC1 and LC2 are determined as following:

$$M_X^{LC1} = 1.0M_X^T + 0.4M_X^L \tag{33}$$

$$M_X^{LC2} = 1.0M_X^L + 0.4M_X^T \tag{34}$$

If the biaxial design of an element is necessary, then for modified design the maximum response quantities shall be combined to determine the maximum vector moment according to eq. (32). Note that the definition of LC1 and LC2 are reversed from the definitions used in AASHTO Division I-A of the Standard Specifications. LC1 is now primarily transverse loading and LC2 is primarily longitudinal loading. Also, the contribution from the orthogonal earthquake component has been increased from 30 to 40 percent in the proposed provisions. This provides better accuracy in predicting elastic forces and displacements with actual time results than does the 30 percent value.

Load Groups

Extreme Event in Table 3.5-1 in NCHRP 12-49 gives load combination for seismic loads. Extreme Event I covers both events of the MCE and the EXP earthquakes. This is reasonable because both earthquake return periods exceed the nominal 75-year design life assumed for new bridges. For

primary member design of the earthquake resisting system (i.e., those members that will experience inelastic action), modified design forces are developed. The elastic seismic forces EQ are modified by the R-Factor combined with the other loads of the Extreme Event I combination. These modified forces, along with the forces associated with plastic hinging in the columns, are used in the seismic design of the various components of the bridge.

Extreme Event I Load for the modified design forces per column for dead loads and seismic loads is determined as follows:

$$\text{Extreme Event I Load} = 1.0 \, (DC + DW) + 1.0 \, (EQ/R) \qquad (35)$$

where

DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surface and utilities
EQ = elastic seismic force either Load Case 1 or Load Case 2 divided by the Response Modification R-Factor.

The forces and moments from the analysis are tabulated in Table 11 for both NCHRP 12-49 and AASHTO LRFD.

Table 9. Comparison of R-Factors for AASHTO LRFD and NCHRP 12-49.

Location	Substructure Element	Existing LRFD			New NCHRP 12-49 SDAP D					
		Critica	Essen	Others	T	T_s	$R_{B \, Safety}$	$R_{B \, Operat}$	R_{Safety}	R_{Operat}
Seat Type Abutment										
South NJ	Multipl Column l	1.5	3.5	5.0	0.967	0.443	4.0	1.5	**4.0**	**1.5**
	Vertical Piles	1.0	1.75	2.5	0.967	0.443	2.0	1	**2.0**	**1.0**
Central NJ	Multipl Column l	1.5	3.5	5.0	0.967	0.409	4.0	1.5	**4.0**	**1.5**
	Vertical Piles	1.0	1.75	2.5	0.967	0.409	2.0	1	**2.0**	**1.0**
North NJ	Multipl Column l	1.5	3.5	5.0	0.967	0.412	4.0	1.5	**4.0**	**1.5**
	Vertical Piles	1.0	1.75	2.5	0.967	0.412	2.0	1	**2.0**	**1.0**
Integral Abutments										
South NJ	Multipl Column l	1.5	3.5	5.0	0.485	0.443	4.0	1.5	**3.6**	**1.44**
	Vertical Piles	1.0	1.75	2.5	0.485	0.443	2.0	1	**1.9**	**1.00**
Central NJ	Multipl Column l	1.5	3.5	5.0	0.485	0.409	4.0	1.5	**3.8**	**1.47**
	Vertical Piles	1.0	1.75	2.5	0.485	0.409	2.0	1	**1.9**	**1.00**
North NJ	Multipl Column l	1.5	3.5	5.0	0.485	0.412	4.0	1.5	**3.8**	**1.47**
	Vertical Piles	1.0	1.75	2.5	0.485	0.412	2.0	1	**1.9**	**1.00**

Response Modification Factors (R)

Elastic seismic design force effects for individual members in substructures shall be determined by dividing the elastic force effects resulting from elastic analysis by the appropriate R-Factor given by eq. (36).

$$R = 1 + (R_B - 1)\frac{T}{1.25T_S} \leq R_B \qquad (36)$$

where T is the shorter vibration period of longitudinal and transverse direction in the bridge (sec), and T_s is equal to S_{D1}/S_{DS} (sec). To account for this increase, therefore, the R_B-Factor is decreased for periods shorter than $1.25T_s$, where this period is based on the break point in the response spectrum. The R-factors for NCHRP 12-49 in these examples are given in Table 9.

Minimum Seat Width and Columns Steel Requirements

<u>From AASHTO LRFD Specifications</u>

Minimum Seat Length

Bearing seats supporting the expansion ends of girders in Seismic Zone 2 shall be designed to provide a minimum support length N (in or mm) measured normal to the face of an abutment or pier due to the longitudinal earthquake loading, not less than that of specified below.

The minimum support length at the abutment bearing seat is calculated by:

$$N \text{ (inches)} = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \qquad (37)$$

where

> L = length of the bridge deck to the adjacent expansion joint, or the end of the bridge deck in feet
> H = column or pier height in feet
> S = skew angle in degrees

Column Longitudinal Steel

The longitudinal reinforcement area (A_s) shall not be less than 0.01 or more than 0.06 times the gross cross-section area (A_g) as described below.

$$0.01\ A_g \leq A_{st} \leq 0.06A_g \qquad (38)$$

Transverse Reinforcement for Confinement at Plastic Hinges

The volumetric ratio of spiral reinforcement (ρ_s) for circular columns is the larger value of:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \quad \text{or} \quad \rho_s = 0.12\frac{f_c'}{f_{yh}} \qquad (39)$$

where
  A_c = area of column core (in^2)
  f_{yh} = yield strength of hoop or spiral reinforcement (ksi)

From NCHRP 12-49 Guidelines

Minimum Seat Width

The minimum seat width shall not be less than 1.5 times the displacement of the superstructure at the seat nor the value of N as described in the following equation:

$$N = \left(0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1+\left(\frac{2B}{L}\right)^2}\right)\frac{1+1.25F_v S_1}{\cos\alpha} \qquad (40)$$

Where
        N = minimum seat width in meter
        L = distance between joints in meter (total length of span)
        H = tallest pier between the joints in meter
        B = width of the superstructure in meter
        α = skew angle
        B/L max = 3/8

P-Δ Requirements

The displacement capacity verification analysis shall be applied to individual piers or bents to determine the lateral load-displacement behavior of the pier or bent. The capacity evaluation shall be performed for individual piers or bents in the longitudinal and transverse directions, separately. The modified seismic displacement demand (Δ) of a pier or bent in the longitudinal and transverse directions must satisfy:

$$\Delta \leq 0.25 \, C_c \, H \tag{41}$$

$$\Delta = R_d \, \Delta_e \tag{42}$$

where

C_c = seismic coefficient based on the lateral strength of the pier or bent
= V/W (where V is the lateral strength and W is weight of bridge)
H = height of the pier from the point of fixity for the foundation
Δ_e = displacement demand from the seismic analysis in meter
R_d = ratio of estimated displacement to displacement determined from elastic analysis

$$= \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \quad \text{for T<1.25T}_s$$
$$= 1.0 \text{ otherwise}$$

Note that the ratio B/L need not be taken greater than 3/8.

Minimum Displacement Requirement for Lateral Load Resisting Pier and Bents

For SDAP E the minimum permitted displacement capacity from the displacement Verification must be greater than the displacement demand as follows:

$$1.5\Delta \leq \Delta_{capacity} \tag{43}$$

where Δ is the modified seismic displacement demand and $\Delta_{capacity}$ is the maximum displacement capacity.

Column Pier Requirements for Reinforced Concrete Design

Vertical Reinforcement

The area of longitudinal reinforcement (A_{st}) shall not be less than 0.008 or more than 0.04 times the gross cross-section area (A_g) such that:

$$0.008A_g \leq A_{st} \leq 0.04A_g \tag{44}$$

Transverse Reinforcement for Confinement at Plastic Hinges

The core concrete and piles bents shall be checked by transverse reinforcement in the expected plastic hinge regions. For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall not be less than:

For circular sections:

$$\rho_s = 0.008 \cdot \frac{fc'}{U_{sf}}\left[12 \cdot \left(\frac{Pe}{fc'Ag} + \rho_t \cdot \frac{fy}{fc'}\right)^2 \cdot \left(\frac{Ag}{Acc}\right)^2 - 1\right] \tag{45}$$

where

f_c' = compressive strength of concrete at 28 days in MPa.

f_y = yield strength of reinforcing bars in MPa.

U_{sf} = strain energy capacity of the transverse reinforcement = 110 MPa.

A_{cc} = area of column core concrete, measured to the centerline of the perimeter hoop or spiral $= \dfrac{\pi}{4} \cdot D''^2$

The longitudinal reinforcement in the potential plastic zone shall be restrained by lateral steel spaced as such to avoid local buckling. The minimum S is given by:

$$s \le 6 \cdot d_b \tag{46}$$

where d_b is the bar diameter of column for longitudinal reinforcement.

Also S should not exceed M/V (1-My/Mλo)

Calculations of Column Steel and Minimum Seat Width for Examples

From AASHTO LRFD

Minimum seat width N$_{min}$

N min = [8+0.02*(100+80+80)+0.08*20]*(1+0) = 14.8 in

The seismic displacements from Table 11 are much less than Nmin, [OK]

Column longitudinal and transverse steel

Column Longitudinal steel was designed using the PCA-Column program for all seismic design forces in the three regions. Column longitudinal steel is summarized in Table 12. For the case of "Critical Bridge" where the ratio of the longitudinal steel is close or exceed 6 percent, the column size in this example

need to be increased to 4'-6" or 5'-0" to satisfy the limitation on the maximum steel ratio in AASHTO LRFD given in eq. (38).

Column transverse steel was calculated using eq. (39).

f'c = 4 ksi
fy = 60 ksi
Ag = 1810 in^2
Ac = 1520 in^2

$$\rho_s = 0.12\frac{f_c'}{f_{yh}} = 0.008 \qquad \textbf{\textit{(Controls)}}$$

$$\rho_s = \frac{4A_{sp}}{sDc}$$

5 spiral, Asp = 0.31 in^2, then S min = 3.5 in
4 spiral, Asp = 0.31 in^2, then S min = 5.4 in

From NCHRP 12-49 guidelines

Minimum seat width N$_{min}$

$$Nmin = \left(0.10 + 0.0017(260/3.28) + 0.007(20/3.28) + 0.05\sqrt{(20/3.28)}\cdot\sqrt{1+\left(\frac{2x44}{260}\right)^2}\right)\frac{(1+1.25x3.5xS_1)}{\cos 0}$$

North New Jersey, S$_1$ = 0.125, therefore *N min* = 0.65 m = 25.5 in
Central New Jersey, S$_1$ = 0.10, therefore *N min* = 0.60 m = 23.7 in
Southern New Jersey, S$_1$ = 0.125, therefore *N min* = 0.58 m = 23.0 in

The seismic displacements from Table 11 multiplied by 1.5 are much less than the values of N$_{min}$ given in Table 10. **[OK]**

Note that the minimum seat width in NCHRP 12-49 is about 60% to 70% higher than that from AASHTO LRFD.

Table 10. Displacements, Minimum Seat Width, and P-Δ (1% steel ratio in columns).

Location	AASHTO LRFD				NCHRP 12-49			
	Δ_{el}	Δ_{dem}	P-Δ	N_{min}	Δ_{el}	Δ_{dem}	P-Δ	N_{min}
South New Jersey	2.31	N/A	N/A	14.8	2.72	2.72	[OK]	23.3
Central New Jersey	3.41	N/A	N/A	14.8	3.09	3.09	[OK]	24.0
North New Jersey	4.16	N/A	N/A	14.8	3.55	3.55	[OK]	25.5

(inches)

The seismic displacements obtained from the seismic analysis are given in Table 11. For AASHTO LRFD design, these displacements were obtained using the gross moment of inertia of the columns. NCHRP 12-49 design guidelines require that an effective moment of inertia of the columns be used in the analysis. As a result of this difference between the two provisions, the seismic displacements from NCHRP 12-49 were higher as shown in Table 11. The current AASHTO LRFD specifications do not include guidelines on calculating the displacement demand due to seismic loads and the *P-Δ* requirements. Rather, it specifies a minimum seat width. The NCHRP 12-49 provisions, on the other hand, do provide procedure for calculating the displacement demand and *P-Δ* requirements. In addition it requires the calculation of a minimum seat width that should also be at least 150 percent the displacement demand. For the seat abutment example evaluated in this study, the displacements and minimum seat widths are summarized in Table 10.

Calculation of Column longitudinal and transverse steel according to NCHRP 12- 49

Column Longitudinal steel was designed using the PCA-Column program for all seismic design forces in the three regions. Column longitudinal steel is summarized in Table 12. For the case of "Operational Performance" where the ratio of the longitudinal steel is close or exceed 4%, the column size in this example need to be increased to at least 5'-0" to satisfy the limitation on the 4% maximum steel ratio in NCHRP 12-49 given in eq. (44). The column transverse steel was calculated using the implicit and explicit approach outlined in NCHRP 12-49 guidelines as shown in the following calculations:

NCHRP 12-49 Column Shear Reinforcement - Implicit Shear Detailing Approach

North Jersey- MCE - Safety Level for 2-Column Bent

a- In Potential Plastic hinge zones:

For Circular / Rectangular Sections:

$\rho_v = K_{shape} * \Lambda (\rho_t / \phi) * (f_{su} / f_{yh}) * (A_g / A_{cc}) * \tan(\alpha) * \tan(\theta)$

ρ_v = Ratio of transverse steel. For circular sections,

$\rho_v = \rho_s / 2 = 2A_{bh} / (S*D")$

where:

f'c =	**4** ksi		
f_{yh} =	**60** ksi	f_{su} = Ultimate tensile strength of longitudinal steel	
f_{su} =	**90** ksi	f_{su} = 1.5*f_y = 1.5*60,000	
D =	**48** in		
P_e =	**900** kips		
cover =	**2** in		
d spiral =	**0.75** in	# 6 spirals	
ρ_t =	**0.011**	from column longitudinal design	
d bar $_{long}$ =	**1** in	# 9 bars	
column L =	**240** in	column height in inches	

A_{bh} = Area of one Spiral bar or hoop in a circular section
S = Spiral Pitch or hoop spacing
D"= Spiral Diameter in circular sections
D"= 43.25   in
Dc = 44

K_{shape} = **0.32**   for Circular sections

Λ = Fixity Factor
Λ = 1   for Fix-Free columns
Λ = 2   for Fix-Fix columns
for this case
Λ = **1**
ϕ = **1**

Column outside diameter =   48   in.
A_g =   1809.56 in^2.
Cover to Spiral =   2   in.
A_{cc} =   1520.53 in^2.   , Area of concrete core
Ag/Acc =   1.19008

θ = Angle of Principal crack plane

$\tan \theta = [(1.6 / \Lambda) * (\rho_v / \rho_t) * (A_v / A_g)]^{0.25}$

where, $\theta \geq 25°$, and $\theta \geq \alpha$

$\alpha=$ Geometric aspect ratio angle
tan $\alpha=$ D' / L

D' = Pitch circle Diameter of longitudinal steel in circular sections
Spiral Diameter =   0.75   in.
Longitudinal steel diameter =   1   in.   (#11 bars)
D' = D -(2*dsp) - (2*clr)-d b long =   41.5 in.
L column =   20   ft   =   240   in.
tan $\alpha=$ D' / L
tan $\alpha=$ 0.17292
$\alpha=$ 9.81039 °

$A_v=$ Shear area of concrete .

$A_v=$ 0.8 Ag   for circular sections

$A_v=$ 1447.65

Assume: $\theta =$ **30**
tan $\theta =$ 0.5774

$\rho_v = K_{shape} * \Lambda\ (\rho_t/\phi) * (\ f_{su}/f_{yh}) * (A_g/A_{cc})* tan(\alpha)* tan(\theta)$

$\rho_v =$ 0.00063   NCHRP-LRFD 8.8.2.3-1

$tan\ \theta = [\ (\ 1.6\ /\ \Lambda\ ) * (\rho_v\ /\ \rho_t\ ) * (A_v/A_g)]^{0.25}$ NCHRP-LRFD 8.8.2.3-4
tan $\theta =$ 0.520

revise θ to: $\theta =$ 27.4649

tan $\theta =$ 0.520

$\rho_v =$ 0.00056

tan $\theta =$ 0.506   Solution has converged

$\rho_v =$ $2A_{bh}\ /\ (\ S*D")$
Assuming: #4 spiral
$A_{bh}=$ **0.2**   in^2.
D"=   43.25   in
S=   16.376   in   *#4 Spiral with 8.0" pitch*

b- *Outside Potential Plastic Hinge Zones* :

$f_c =$ 4   ksi

$\rho^*_v = \rho_v - 0.17 (\ f'_c{}^{0.5}/f_{yh})$ $\rho^*_v = \rho_v - 0.4462 (\ f'_c{}^{0.5}/f_{yh}$ US
Hence:
$\rho^*_v=$ -0.01431   If $\rho^*_v =$ negative ==>   use minimum transverse r/f
for #4 Spiral:
S=   -0.646 in   *Use #4 Spiral with 10.0" pitch*

85

c- Transverse Reinforcement for Concrete Confinement at Plastic Hinges:

The volumetric ratio of spiral reinforcement ρ_s

$$\rho_s = 0.008*(f'_c/U_{sf})*[12*(P_e/f'_c A_g + \rho_t * f_y/f'_c)^2 (A_g/A_{cc})^2 -1]$$

U_{sf} = Strain Energy Capacity of the transverse reinforcement

U_{sf} =   16   ksi   or U_{sf} =   110   Mpa

P_e = Factored Axial Load including seismic effects

P_e =   900   kips

d- Transverse Reinforcement for longitudinal bar restraint in Plastic Hinges :

2- $$\rho_s = 0.008 \cdot \frac{fc'}{U_{sf}} \left[12 \cdot \left(\frac{Pe}{fc'Ag} + \rho_t \cdot \frac{fy}{fc'} \right)^2 \cdot \left(\frac{Ag}{Acc} \right)^2 - 1 \right]$$

= 0.008*(4*6.89/110) *[12(600/4*1810 + 0.011*60/4)2 * (1810/1520)2 - 1]

Therefore:

ρ_s =   0.00085   Assuming #4 Spiral : A_{bh}=   0.2   in^2

ρ_s = $4A_{bh}/sD'$ The spiral Step would then be:

S=   19.70865 in.

S < M/V (1- My/Mpo) = 20 (1 - /1.5) x 12 = 90 in   (conservative approximation)

S≤ 6db =

S ≤ 6 d b long" =   6   in

Therefore,
#4 Spiral with 6'' spacing

Column Transverse Steel using the NCHRP 12-49 Explicit Approach:

For the column designed above in Southern New Jersey (ρ_t = 0.012),

$Vn = Vp+Vc+Vs$

$Vp = \Lambda/2 \; Pe \; \tan \; \alpha$

$Vc = 0.019 \; sqrt(f'c)Av$ (minimum contribution)

$Vs = (p/2)*(Abh/2)*fyh*D''*\cot \; \theta$

$Vu \; max = \Phi \; Vn \qquad \Phi = \qquad 0.9$

Pe =	**900**	kips
tan a=	0.17	
s =	6.00	in
Av =	1447.65	in2
A bh =	0.2	in2
f yh =	60	ksi
ρ v =	0.00057	
ρ t =	0.012	
Av/Ag =	0.8	
Λ =	1	
tan θ =	0.50	rad
cot θ =	2.01	in
D'' =	43.25	in
Vp =	**78**	kips
Vc =	**55**	kips
Vs =	**273**	kips
Vn =	**406**	kips
Vu max =	**365**	kips

Cc = Vn/W =		0.45
H =		240 in
25*Cc*H =		**27.1** in

Vu from Table 12 for Southern New Jersey (NCHRP 12-49 Safety Level)
Vu = 121 kips < Vu max = 365 [OK]
Similarly for columns in other regions [OK]

Table 11. Summary of Forces, Moments, and Deflections From Elastic Multi-modal Spectrum Analysis From NCHRP 12-49 and AASHTO LRFD.

Seat -Type Abutment - Summary of Forces, Moments, and Deflections

NCHRP 12-49 SDAP D - Elastic Response Spectrum Ananlysis (Ieff = 0.5 Ig)

T 1 =   0.967 sec   T 2 =   0.485 sec

Provisions	Location	Column shears and moments				Piles shears and moments				Displacements				Abutment forces		Long Soil Pressure
		VL	VT	MT	ML	VL	VT	MT	ML	Abut x	Abut y	Pile x	Pile y	Px	Py	
Exisiting	South NJ	308	79	6094	1580	313	85	7728	1995	2.31	0.56	0.38	0.11	-	227	0
LRFD	Central NJ	462	118	9252	2370	470	128	11597	2992	3.41	0.88	0.57	0.167	-	341	0
I eff = Ig	North NJ	555	142	11099	2843	564	154	13912	3390	4.16	1.07	0.68	0.2	-	410	0
New	South NJ	367	184	7342	3620	374	200	9198	4643	2.72	1.37	0.46	0.26	-	525	0
NCHRP	Central NJ	418	206	8353	4127	427	224	10462	5206	3.09	1.47	0.53	0.3	-	590	0
12-49	North NJ	460	230	9190	4611	470	251	11510	5817	3.55	1.65	0.58	0.33	-	658	0

Integral Abutment - Summary of Forces, Moments, and Deflections

NCHRP 12-49 SDAP D - Elastic Response Spectrum Ananlysis (Ieff = 0.5 Ig)

T 1 =   0.485 sec   T 2 =   0.358 sec

Provisions	Location	Column shears and moments				Piles shears and moments				Displacements				Abutment forces		Long Soil Pressure
		VL	VT	MT	ML	VL	VT	MT	ML	Abut x	Abut y	Pile x	Pile y	Px	Py	
Exisiting	South NJ	41	79	827	1590	51	86	1087	2006	0.33	0.57	0.07	0.11	539	227	1.08
LRFD	Central NJ	62	119	1250	2385	76	129	1642	3010	0.49	0.85	0.11	0.17	807	340	1.62
I eff = Ig	North NJ	75	143	1495	2860	92	155	1971	3612	0.59	1.02	0.13	0.21	970	409	1.95
New	South NJ	135	184	2690	3680	152	200	3411	4643	1.02	1.34	0.21	0.26	1304	525	2.61
NCHRP	Central NJ	161	206	3288	4127	187	224	4169	5207	1.25	1.49	0.26	0.3	1593	590	3.19
12-49	North NJ	182	231	3536	4611	206	251	4602	5817	1.38	1.74	0.29	0.33	1760	658	3.52

Units: Shears in kip, Moments in kip-ft, Displacemnts in inch, Soil Pressure in ksf

Discussion of Analysis Results

The two bridges were analyzed using both the new NCHRP provisions and the existing AASHTO LRFD provisions. The bridges were also analyzed for three different geographic locations in the state of New Jersey – North, Central, and South. Results of the seismic analysis of the two models are shown in Table 11. Table 11 shows base shears, bending moments, and displacements in the longitudinal and the transverse directions. Also tabulated are abutment forces and soil pressures. Table 9 shows the response modification factors from both

provisions utilized to modify the earthquake loads obtained from the analysis. For the bridge configuration chosen in this study, the maximum displacement under the maximum earthquake (2500 years event) for the structure located in North New Jersey was about 90 mm (3.54 in) using the new NCHRP provisions and about 105 mm using the current AASHTO LRFD specifications. For the integral abutment bridge, these displacements were 15 mm (0.6 in) and 35 mm (1.7 in), respectively. The passive soil pressure behind the abutment wall increased the stiffness of the bridge and reduced its period of vibration (T_1 = 0.485 sec) compared to the seat-type abutment bridge (T_1 = 0.967 seconds). The maximum soil pressure behind the integral abutment wall was 1.45 KN/m^2 (3.5 ksf) for the 2500 years event. This value was less than the maximum value specified by NCHRP 12-49 provisions, however, it was about three times the pressure resulting from temperature variations.

Seat Abutment Example

For the bridge configuration considered, and for this abutment type, the maximum long seismic displacement for soil class E in North New Jersey was approximately 105 mm (4.13 in) without abutment resistance (K = 0). For a typical expansion gap, the soil behind the abutment wall is not likely to be mobilized and the passive pressure resistance will not be significant. In the Central and South regions of New Jersey, the maximum seismic displacements in the longitudinal direction for soil class E in were 58 mm (2.3 in) and 86 mm (3.4 in) respectively. Thus, no passive soil pressure behind the abutment and the only resistance to lateral loads will be provided by the piers. For this bridge configuration, both the existing AASHTO LRFD and the new NCHRP 12-49 provisions provide comparable displacements for North and Central NJ; however, for South New Jersey, the new NCHRP 12-49 gave higher forces and displacements.

Integral Abutment Example

For the bridge configuration and abutment type considered in this example, the soil is engaged in resisting the seismic loads and subsequently, longitudinal as well as transverse springs are used. The displacements and forces from the new NCHRP 12-49 provisions were higher than those of existing LRFD provisions. This can be attributed to higher accelerations and soil amplification factors. The higher displacements from NCHRP will result in higher forces on the embankment behind the diaphragm. In both provisions, the soil seismic loads did not exceed the 2.75 KN/m^2 (6.67 ksf) ultimate passive pressure specified in the new NCHRP 12-49 provisions for this abutment height. Because of the larger displacements from new NCHRP 12-49 provisions, more soil movement behind abutment wall is expected. The longitudinal forces in the piers of the integral abutment were less than those of a seat-type abutment because of soil participation in resisting seismic loads. Engaging the soil behind the abutment

will make the structure stiffer because of the added soil stiffness. This increased stiffness reduces the period of bridge and resulted in higher seismic loads.

Integral Abutment Wall Design (NCHRP 12-49)

The integral abutment wall is designed according to the equations given on Figures 14 and 15 on page 35 of this report. The design will be shown for NCHRP 12-49 in North Jersey. The design for other cases is similar. The passive pressure from the seismic analysis given in Table 11 is 3.52 ksf (note that this value is much lower than the maximum pressure pmax = 0.67 H = 0.67 x 10 = 6.67 ksf suggested in NCHRP 12-49 Sections 7.5.2.2 and 8.5.2.2).

The passive Force F = 3.52 x 10 = 35.2 k/ft of wall

The vertical bending moment, $M = \dfrac{2FH}{9\sqrt{3}} = \dfrac{2(35.2)(10)}{15.6} = 45.1$ k-ft /f t of wall

6 vertical at 1'-0" with 3 in cover, d = (2' – 6") – 3 in – 0.25 in = 26.75 in

Therefore ΦM_n = 46.6 k-ft/ft of wall > 45.1 [OK]

For the horizontal direction,

The horizontal bending moment, $M = \dfrac{FS^2}{12} = \dfrac{(35.2)(12)^2}{12} = 422.4$ k-ft /f t of wall

b = H = 10 x 12 = 120 in
d = 26.75 in
# 6 horizontal bars at 1'-0" ==> A_s = 11 bars x 0.44 = 4.84 in^2

Therefore ΦM_n = 512.7 k-ft/ft of wall > 422.1 [OK]

Shear V = FS/2 = 35.2 x 12/2 = 211.2 k/ ft of wall

$Vc = 2\sqrt{f'_c}\; b_w d = 2\sqrt{4000}\,(120)(26.75)/1000 = 406\text{ kip}$

ΦV_c = 0.85x406 = 345.1 k > 211.2 k [ok]

(Note that the maximum moment should be the one from Extreme Event I, in this case, the moments from DL and other loads were very small)

Column Longitudinal Steel and Confining Reinforcement in Plastic Hinge Zones

The requirements for column design in the new provisions are more complicated than those in the current AASHTO LRFD provisions. Column design based on the new NCHRP 12-49 provisions showed that the design was controlled by the requirements to restrain longitudinal bars in plastic hinge zones. In the current AASHTO LRFD provisions, column transverse reinforcement was also controlled by confinement in the plastic hinge zones. Column longitudinal steel requirements and number of piles are summarized in Table 12 for the two examples. For the integral abutment bridge, NCHRP 12-49 safety level and all AASHTO LRFD bridge categories (critical, essential, and others) required 1 percent longitudinal steel, while the NCHRP 12-49 operational level required 2.2 percent steel in southern New Jersey, 2.5 percent steel in central New Jersey, and 2.8 percent steel in northern New Jersey. For the seat-type abutment bridge, the columns longitudinal steel ratio went up to 1.5 percent and 2 percent for AASHTO LRFD essential bridges. NCHRP 12-49 safety level steel requirements for columns in seat-type abutment bridges were between those of AASHTO LRFD essential bridges and other bridges. Comparing critical bridges in AASHTO LRFD and operational level in NCHRP 12-49, steel requirements in northern New Jersey were higher for AASHTO LRFD while they were lower for central and southern New Jersey. This is due to high soil coefficients in NCHRP 12-49 for small ground motions in Central and South Jersey. The effect of the longitudinal steel ratio on transverse steel is shown in Figure 30. This figure also shows this effect for several common sizes of columns. When the two examples are designed for NHCRP 12-49 operational level, the steel requirements were significantly higher for both longitudinal and transverse in central and southern New Jersey. Table 12 shows that 6.5 percent longitudinal steel is required in Northern NJ for seat-type abutment bridges for NCHRP 12-49 operational level, compared to 1.3 percent for safety level. The difference between the two levels is due to the R-factors. The transverse steel ratio required for the 6 percent longitudinal steel would be 0.03. For the integral abutment bridge, this ratio increased from 1 to 2.7 percent.

It is interesting to see the effect of maximum longitudinal steel ratio from both provisions on the column design of the seat abutment example. Table 12 shows that designing the seat abutment example in Northern New Jersey, according to AASHTO LRFD for Critical bridges, requires 7 percent longitudinal steel ratio in the columns. This ratio exceeds the 6 percent maximum ratio specified in AASHTO. Hence, the column size (diameter) in this case needs to be increased to at least 4'-6" or 5'-0" (1.4 m or 1.52 m). On the other hand, designing the seat abutment example in North New Jersey according to NCHRP 12-49 for Operational level requires 6.5 percent longitudinal steel ratio in the columns. This ratio exceeds the 4 percent maximum ratio specified in NCHRP 12-49. Therefore, the column size (diameter) in this case must be increased to at least 5'-6" (1.67 m). Table 12 shows that the column diameter need to be increased in Central and Southern New Jersey when using the NCHRP 12-49 provisions for the Operational level, while the column size in those regions need not be

increased when designing the bridge as Critical according to the AASHTO LRFD provisions.

In the new provisions, the confining (transverse) steel requirements are usually governed by the need to restrain longitudinal reinforcement in plastic hinge zones. The required transverse reinforcement is dependent on the longitudinal steel ratio, diameter of the column, and strength of steel but independent of the strength of concrete. In the current AASHTO LRFD provisions, the transverse reinforcement in plastic hinge zones is independent of the longitudinal steel ratio and column diameter but dependent on the strength of concrete. The plot in Figure 30 shows that for column diameters between 0.91 m (3'-0") and 1.83 m (6'-0") with 1 and 2 percent steel ratio, the existing provisions require more confining reinforcement than the new provisions. For 1.52 m (5'-0") and 1.83 m (6'-0") diameter columns with 3 percent longitudinal steel or more, the new provisions require more confining reinforcement than the existing provisions.

Table 12. Column and Pile Design From NCHRP 12-49 SDAP D and AASHTO LRFD.

Provisions	Location	Column Design					Piles	
Seat Type Abutment								
		ρ_{long}	Pu	Vu	Mu	Φ Mn	V_{EQ}/R	N_{piles}
Exisiting	South NJ	3%	921	212	4197	4370	324	33
LRFD	Central NJ	5.7%	921	318	6367	6340	487	49
Critical	North NJ	7%	921	382	7638	7640	585	59
Exisiting	South NJ	1%	921	91	1799	2360	185	19
LRFD	Central NJ	1.5%	921	136	2729	2729	278	28
Essential	North NJ	2.2%	921	164	3274	3400	334	34
Exisiting	South NJ	1%	921	64	1259	2360	130	13
LRFD	Central NJ	1%	921	95	1910	2360	195	20
Others	North NJ	1%	921	115	2291	2360	234	24
New	South NJ	1%	920	106	2050	2360	212	20
NCHRP	Central NJ	1.1%	920	121	2330	2400	241	24
12-49	North NJ	1.3%	920	133	2570	2580	266	26
Safety Level								
New	South NJ	5.1%	920	283	5453	5852	424	40
NCHRP	Central NJ	6.3%	920	321	6198	6650	482	48
12-49	North NJ	6.5%	920	355	6836	7980	532	52
Operational								
Integral Abutment								
Exisiting	South NJ	1%	510	59	1195	2360	100	10
LRFD	Central NJ	1%	510	89	1795	2360	150	15
Critical	North NJ	1%	510	108	2151	2360	180	18
Exisiting	South NJ	1%	510	25	512	2360	57	6
LRFD	Central NJ	1%	510	38	769	2360	86	10
Essential	North NJ	1%	510	46	922	2360	103	12
Exisiting	South NJ	1%	510	18	358	2360	40	4
LRFD	Central NJ	1%	510	27	539	2360	60	6
Others	North NJ	1%	510	32	645	2360	72	8
New	South NJ	1%	510	66	1267	2360	132	14
NCHRP	Central NJ	1%	510	77	1388	2360	154	16
12-49	North NJ	1%	510	86	1530	2360	171	18
Safety Level								
New	South NJ	2.2%	1275	167	3168	3400	251	27
NCHRP	Central NJ	2.5%	1275	195	3470	3600	292	31
12-49	North NJ	2.8%	1275	217	3825	3900	325	35
Operational								

Units: Axial Loads and Shears in kips, Moments in kip-ft

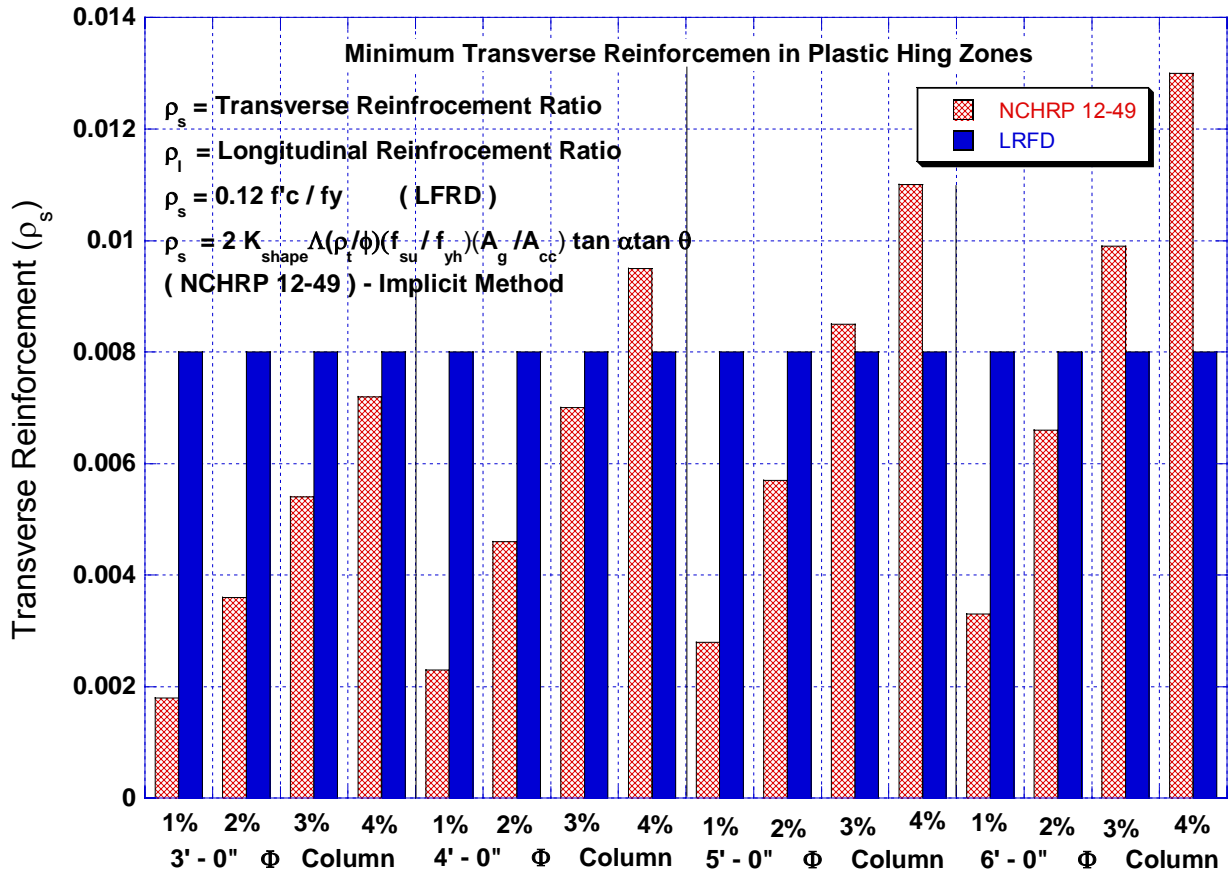V_{EQ}/R = design earthquake load on piles

Figure 30.   Column Transverse Reinforcement for AASHTO LRFD and NCHRP 12-49.

Conclusions From Design Examples

1. Soil site factors have increased dramatically for soft soils subjected to small ground motions. These factors had a major impact on the design response spectra and the selection of the seismic hazard level. Based on the results of the two examples used in this study, NCHRP 12-49 new provisions gave significantly higher seismic loads for stiff bridges on soft soils, also NCHRP 12-49 gave significantly higher seismic loads on bridges in Southern Jersey because of the soil factors.

2. Design for the MCE event for Operational Performance level in NCHRP 12-49 in Northern New Jersey in soft soils would result in significantly higher loads due to higher site factors and lower R-factors. The chosen performance levels, whether for safety or operational will have to be made by bridge owners and state and city officials depending on the bridge importance and location. In these examples, when the bridges are designed in Northern and Central Jersey, the AASHTO LFRD Essential bridges required about 50 percent more

94

column longitudinal steel than NCHRP 12-49 Safety level while LRFD Other bridges category required about 15 to 20 percent less steel. For South Jersey, column steel requirements for AASHTO LRFD Essential and Other bridges were similar to those of NCHRP 12-49 Safety level. In South Jersey, there was about 70 percent increase in column steel and for Central New Jersey about 15 percent when using the NCHRP 12-49 Operational Level compared to AASHTO LRFD critical bridge. The increase was higher in the case of integral abutments. Pile foundation requirements have also increased.

3. The longitudinal steel requirements in example 2 (integral abutment bridge) were lower than those for example 1 (seat-type abutment). This was true for the AASHTO LRFD specifications and the NCHRP 12-49 guidelines. This trend was observed for all regions in New Jersey. The same observation was true for pile requirements. Because of the integral abutments, less seismic loads are transferred to the piers, hence reducing its steel requirements and pile foundation requirements.

4. For bridges designed in Central and Southern New Jersey for the Operational Level, the 4 percent maximum limit on the longitudinal steel ratio in concrete columns specified in NCHRP 12-49 will result in larger size columns compared with AASHTO LRFD current provisions.

5. Transverse column reinforcement in plastic hinge zones is significantly affected by the longitudinal steel ratio. This reinforcement is independent of the longitudinal steel in the existing provisions. For column diameters between 3 ft (0.91 m) and 6 ft (1.83 m) with 1 and 2 percent longitudinal steel, the existing specifications require more transverse reinforcement. For column diameters between 5 ft (1.52 m) and 6 ft (1.83 m) with 3 percent steel or more, the new NCHRP provisions require more transverse reinforcement.

6. The new NCHRP 12-49 guidelines give additional options in seismic analysis and design procedures. For example, SDAP C, D, and E could be used in most cases, while the reduction factors of elastic seismic loads (R) are tabulated in more details for various cases. It also provides more information on the analysis of integral and seat abutments and on their foundation stiffness more than the current specs.

7. Because a large number of bridges can qualify under the category where the Capacity Design Spectrum Analysis (SDAP C) may be used, this procedure should be explored in more details and utilized because it is relatively simple. Design aids for this procedure can also be prepared to further simplify the procedure.

APPENDIX B

STIFFNESS OF ABUTMENTS AND RETAINING WALLS FOR SEISMIC DESIGN

Introduction

There are several methods for estimating the stiffness of abutments and retaining walls. Some of these methods are based on the application of the ultimate passive pressure on the abutment while others are based on empirical approaches for estimating passive pressures. Therefore having an adequate concept of soil pressure is necessary. Many of these methods are based on the structural and geotechnical properties of the abutment and the soil. In general, the abutment stiffness is a function of abutment height, soil type, and abutment movement. Several methods for estimating abutment stiffness are described in this report including those from the NCHRP 12-49 guidelines and the CALTRANS 2001 seismic design criteria. A comparison between these two approaches is also included. The abutment stiffness can be described either in terms of the passive pressure and displacement or using various analytical approaches.

Abutment Stiffness Using The Ultimate Passive Pressure

In this method the stiffness is dependent on the ultimate passive pressure induced on the wall and the maximum displacement produced by this force.

$$K_{abutment} = \frac{Force}{Displacement} = \frac{UltimatePassivePressure*Area_{wall}}{MaximumDisplacement} \qquad (47)$$

To find the ultimate pressure and displacement in eq. (47), conventional methods based on Coulomb and Rankin theories are being used. Later in this section, recent methods based on Geoffrey results will be used to find the ultimate passive pressure and displacement.

Abutment Stiffness Using The Analytical Approach

The abutment stiffness has three degrees of freedom (longitudinal, transverse and vertical). It is calculated using the geotechnical and the structural properties of the soil and the abutment. Several theoretical methods were proposed to obtain the abutment stiffness, however the one conducted by Geoffrey seems to provide the most accurate results. Results of Geoffrey's method (1995) are compared to the results of a study conducted at UC Davis (1995) later in this Appendix.

96

METHODS FOR DETERMINING PASSIVE PRESSURE

Geoffrey et al. (1995)

There are several methods to determine the static passive pressure induced on abutment walls such as Rankine's, Coulomb's, Caquot & Kerisel's and the Slice theory. Geoffrey et al. [29] used a numerical method to evaluate the passive pressure and compared it to the above conventional methods. He showed that that each of these methods could be conservative in some cases. Figures 31, 32, and 33 show a comparison of the k_p values versus the friction angle from these methods and from Geoffrey's numerical method. The graphs in Figures 34 and 35 show a comparison between the Coulomb's and Geoffrey's numerical method.

The parameter N_p that is used in the graphs in Figures 34 and 35 is a Normal Passive Force defined as:

$$N_p, \text{ Normalized passive force} = \text{Total force} / 0.5 \gamma H^2 \tag{48}$$

For sand (c=0) $N_p = K_p$ and for sand (φ=0) $N_p = (\cos \delta)^{-1} + \dfrac{4c}{\gamma H}(\cos \delta)^{-0.5}$ (49)

Comparison of Coulomb's Theory and Geoffrey's Numerical Method

For cohesionless soils, like sand, and for smaller backfills and wall friction angles, the Coulomb's theory coefficient appears to underestimate the passive earth pressure compared to those from Geoffrey's numerical method. For larger backfill and wall friction angles, the Coulomb theory overestimates the passive earth pressure.

For cohesive soils, like clay, and for smaller backfills, the Coulomb's theory slightly underestimates the passive earth pressure as compared to Geoffrey's numerical method. This underestimation becomes more severe when the wall friction angle becomes large. For a larger backfill, the Coulomb's theory overestimate the passive earth pressure compared to Geoffrey's numerical method.

The results show that a displacement of about 6% of the wall height mobilizes the ultimate passive pressure in sand backfills in most cases, however in clay backfills, the results show that the displacement needed for mobilization is about 10% of the wall height.
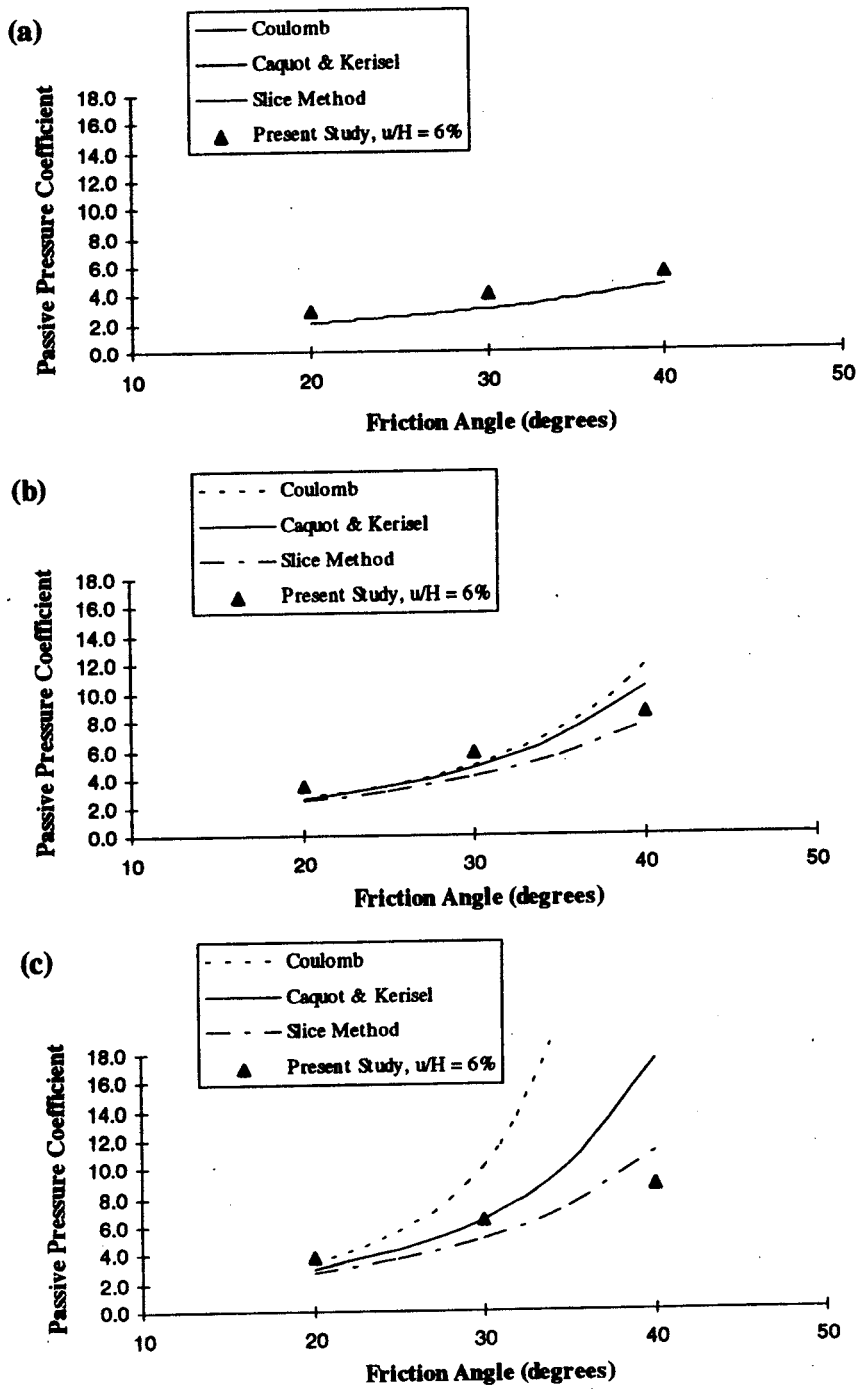
Figure 31. Passive Pressure Coefficient K_p vs. Friction Angle ϕ: γ=120 pcf, c=0, E=600 ksf, ν=0.35; H=10 ft. (a) δ=0; (b) δ=0.5φ; and (c) δ=φ.
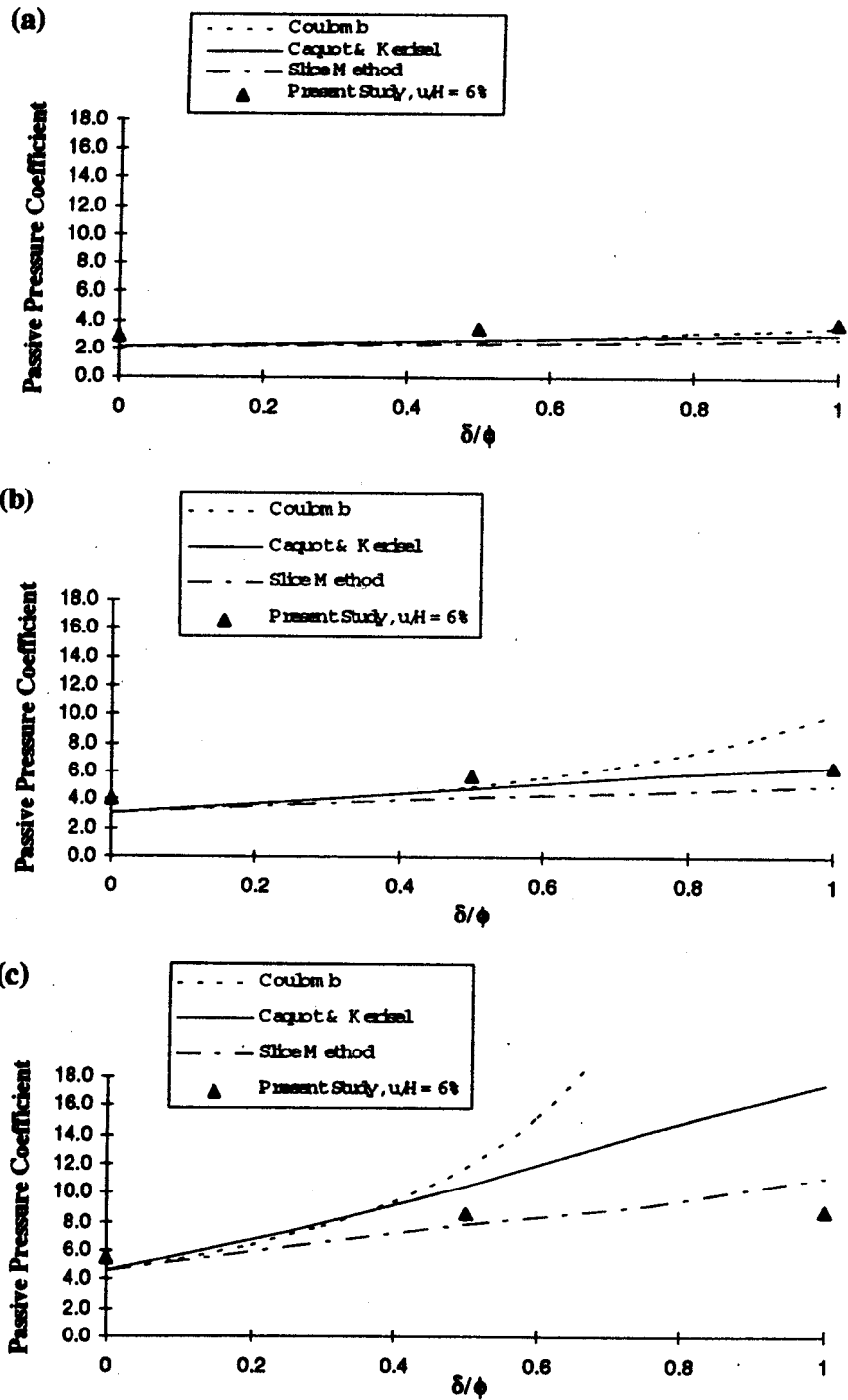
Figure 32.   Passive Pressure Coefficient K_p vs. δ/ϕ: γ=120 pcf, c=0, E=600 ksf,
v=0.35; H=10 ft. (a) φ=20; (b) φ=30; and (c) φ=40.
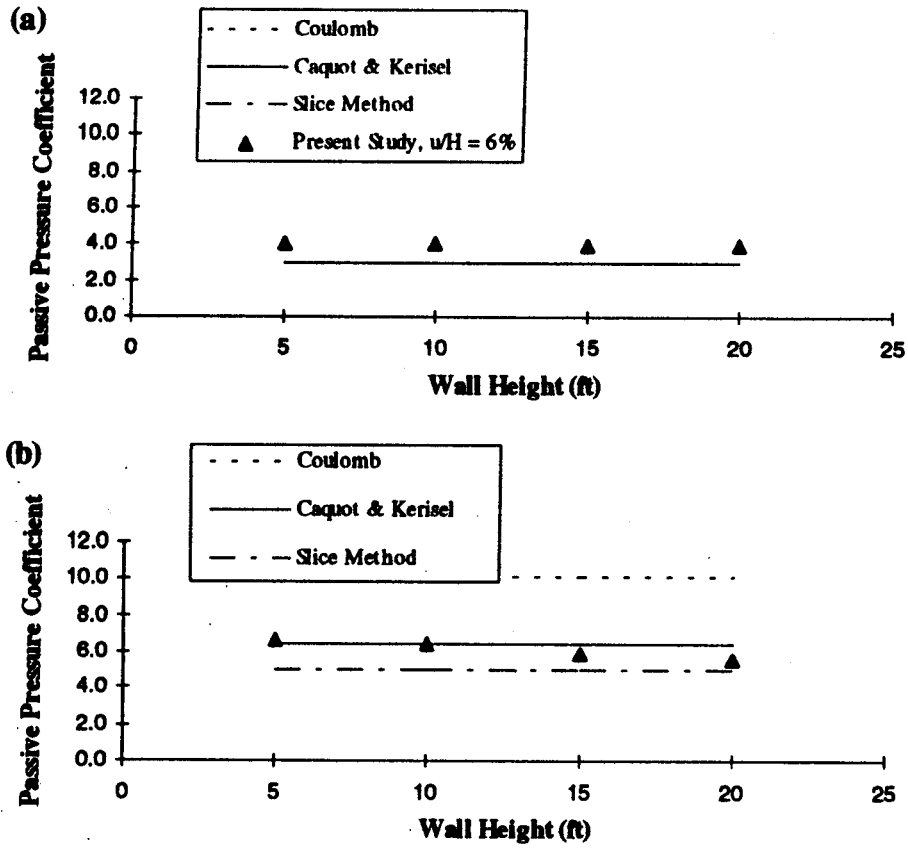
Figure 33. Passive Pressure Coefficient K_p vs. wall height H: γ=120 pcf, c=0, φ=30; E=600 ksf, ν=0.35; (a) δ=0; and (b) δ=φ.
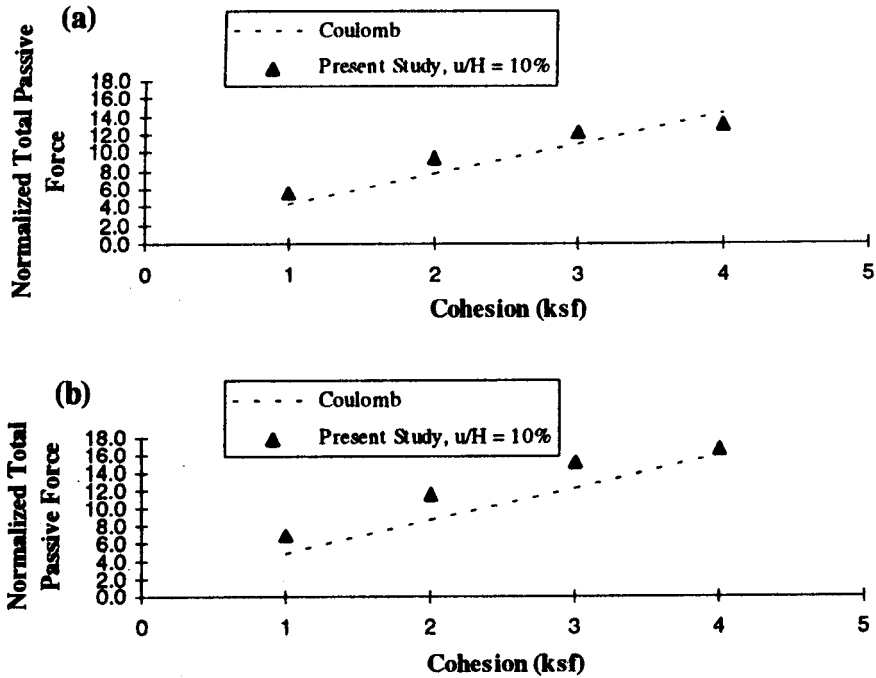
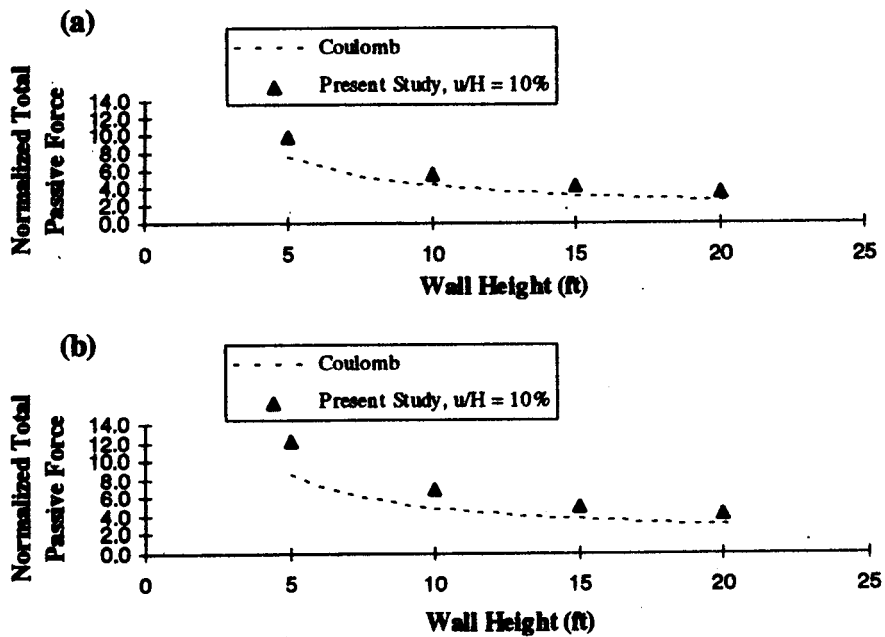Figure 34. Normalized total passive force N_p vs. cohesion c:γ=120 pcf, φ=0; E=200 ksf, ν=0.45; H=10 ft. (a) δ=0; and (b) δ=35.

Figure 35. Normalized total passive force N_p vs. wall height H: γ=120 pcf, c=1.0, ksf, φ=0, E=200 ksf, ν=0.45. (a) δ=0; and (b) δ=35.

Siddharthan et al Method (1997)

In this method, the longitudinal, transverse, and vertical stiffness values are calculated using several important factors such as nonlinear soil behavior, abutment dimensions, superstructures loads, and the difference in soil behavior under active and passive conditions. The agreement between the predicted stiffness from the Siddharthan Method [29] and the measured stiffness from the UC Davis experiments is very good. Figure 36 shows a comparison between the computed and the measured values. Subsequently, this approach was been used to develop normalized relationships for abutment stiffness.

The stiffness is a function of the displacement, abutment dimensions and the soil type given by [29]:

$$S_i = D_i E_i \left(\frac{x_i}{H} \right)^{-0.96} \quad kN/m/m \tag{50}$$

Where S_i is the stiffness in i direction, x is displacement, D_i is dimensionless stiffness coefficient and E_i represents the normalizing factors given by

$$E_L = E_V = \frac{\gamma B^3}{H^2} \text{ and } E_T = \frac{\gamma B W^2}{H^2} \tag{51}$$

(*L, V, T* represent longitudinal, vertical and transverse, respectively)

The difference between this formula and CALTRANS formula is that this formula can be used for any kind of soil. This method can also be used for abutments on spread footings. For abutments supported on piles, the stiffness of the pile group need to added to the stiffness from eq. (50).
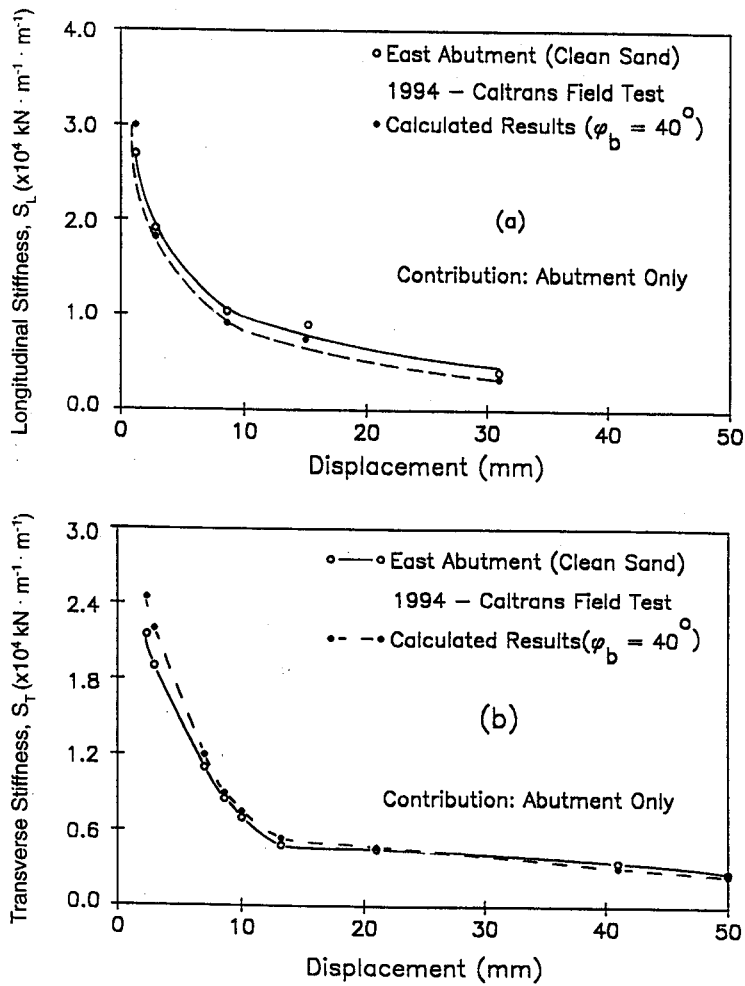
Figure 36. Comparison between computed (Siddharthan et al. 1997) and measured abutment stiffnesses. (a) Longitudinal stiffness. (b) Transverse Stiffness.

The CALTRANS Method (2001)

This method is based on the results for the experiments conducted at UC Davis (California) by Maroney in 1995. According to these results, the stiffness of a tested abutment (5.5 ft x 10ft) is 200 k/in or 20 k/in per ft of wall (for the 5.5 ft height wall). The properties of the soils used in the UC Davis tests were as follows:

Unit weight of soil, γ (kN/m^3)	8.5
Backfill friction angel, φ_b	40
Backfill-abutment interface friction angel, δ_b and δ_a	15
Foundation soil friction angel, φ_f	35
Foundation-based friction angel, δ_f	21
Poisson ratio, ν	0.3

103

The effective areas of the abutment Ae are defined as:

$$A_e = \begin{cases} h_{bw} \times w_{bw} \\ h_{dia} \times w_{dia} \end{cases} \qquad \text{Seat \& Diaphragm abutment} \qquad (52)$$

where:

$h_{dia} = h_{dia}^{*}$ =effective height when the diaphragm is not designed for seismic soil resistance
$h_{dia} = h_{dia}^{**}$ =effective height when the diaphragm is designed for seismic soil resistance

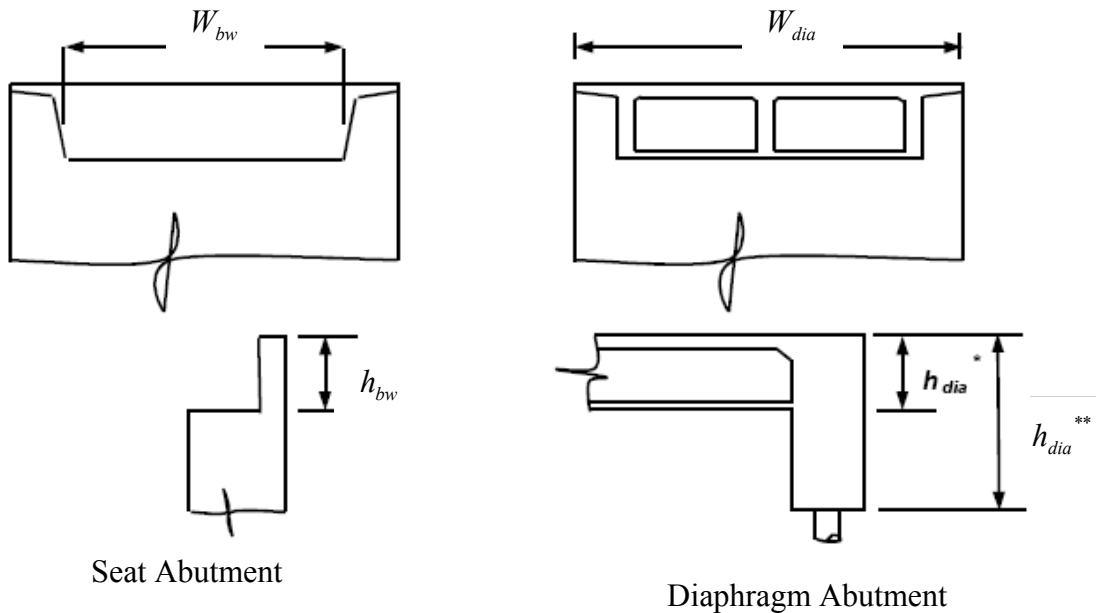h_{bw}, h_{dia}, h_{dia}, W_{bw}, W_{dia} , and W_{abut} are shown in Figures 37 and 38.

Seat Abutment

Diaphragm Abutment

Figure 37. Seat and diaphragm abutments dimensions from the Caltrans Manual [11].
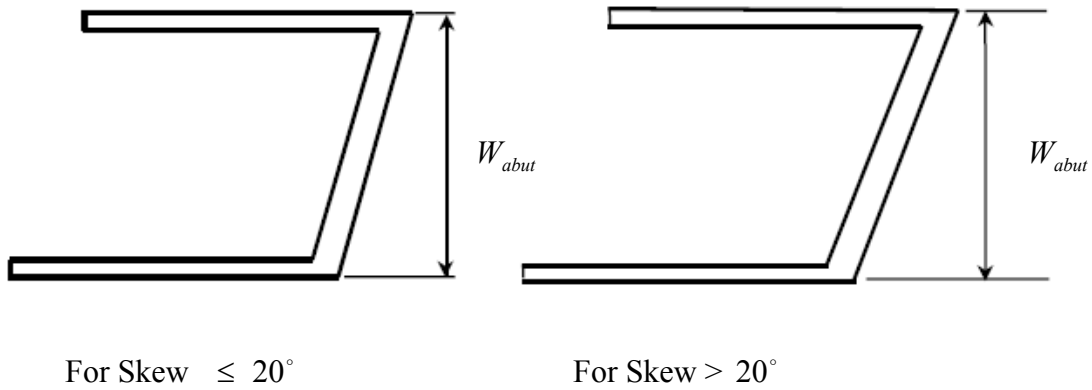
For Skew ≤ 20°   For Skew > 20°

Figure 38. Skew angles and abutment widths from the Caltrans Manual [11].

The passive resistance of the soil according to the UC Davis test results can be determined according to the following formula:

$$P_{bw} or P_{dia} = \begin{cases} A_e \times 5.0ksf \times \left(\dfrac{h_{bw} or h_{dia}}{5.5} \right)(ft, kip) \\ A_e \times 239kPa \times \left(\dfrac{h_{bw} or h_{dia}}{1.7} \right)(m, kN) \end{cases}$$ (53)

To use the UC Davis test results calculation of the stiffness of an abutment of an arbitrary height, CALTRANS [11] defined the stiffness as follows:

$$K_{abut} = \begin{cases} K_i \times w \times \left(\dfrac{h}{5.5} \right) & \text{(English Units)} \\ K_i \times w \times \left(\dfrac{h}{1.7} \right) & \text{(SIUnits)} \end{cases}$$ (54)

Where

K_i=20 *kip/in/ft* (11.49 *kN/mm/m*) (see Figure 39)
w = width of the abutment
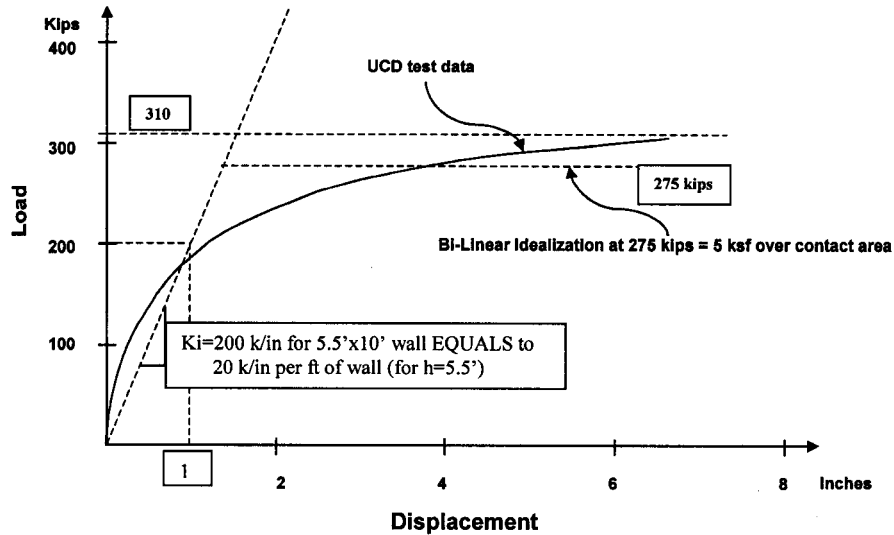h = height of the abutment

Figure 39. Estimated bi-linear abutment stiffness (UC, Davis Tests 1995).

The NCHRP 12-49 Method

The abutment stiffness in NCHRP 12-49 is calculated using the ultimate passive pressure induced on the abutment wall according to the following formula:

$$P_p = p_p \times H \qquad\qquad (55)$$

Where H = wall height in meter

p_p = passive pressure behind the abutment wall

P_p = passive force behind the abutment wall

For a cohesionless, non-plastic backfill (fines content less than 30 percent), the passive pressure may be assumed to be equal to H/10 MPa per meter of the length of the wall (2H/3 ksf per foot length of the wall).

For cohesive backfill (clay fraction > 15 percent), the passive pressure p_p may be assumed to be equal to 0.25 MPa (5 ksf) provided the estimated unconfined compressive strength is greater than 0.20 MPa (4 ksf).

The abutment stiffness in the integral abutment and seat abutment examples, K$_{abut}$ were calculated as follows:

106

Integral Abutment $$K_{eff1} = \frac{P_p}{0.02H} \qquad (56)$$

Seat Abutment $$K_{eff1} = \frac{P_p}{(0.02H + D_g)} \qquad (57)$$

Where D_g is the gap width.

It is worth noting here the according to the formula (56), the abutment stiffness from NCHRP 12-49 is independent of the height of the abutment (H will be canceled in the equation). However, according to CALTRANS, the abutment height has an effect on its stiffness as shown in CALTRANS eq. (54). Also the height is a parameter in the Siddharthan et al. (1997) Method [29]. The NCHRP 12-49 method uses 0.02H in the formula as the displacement that induces the ultimate passive pressure. This does not seem to be accurate according to Geoffrey et al. (1995), who has shown that the displacement necessary to mobilize the ultimate passive pressure for sand and clay backfills are about 6% and 10% respectively. According to Geoffrey's method, the abutment stiffness would be 3 to 5 times less than that from the NCHRP 12-49 or Caltrans methods.

COMPARISON OF THE NCHRP 12-49 AND CALTRANS METHODS

The CALTRANS equations for calculating abutment stiffness are independent of the soil type. In NCHRP the stiffness depends on the seismicity of the area (SDAP), and whether the soil is cohesionless or cohesive. Tests conducted for CALTRANS at UC Davis (1995) illustrate the effect of wall displacement on the abutment stiffness as shown in Figure 39. As seen in the formula, the pressure measured by CALTRANS depends only on the dimensions of the abutment and the assumed wall displacement. In NCHRP 12-49, it also depends on the soil type. The ultimate resistance of 5 ksf specified by CALTRANS is similar to that given in NCHRP 12-49.

APPENDIX C

SEISMIC DESIGN OF EMBANKMENTS

Seismic Design of Embankments

The design of slopes and embankments is controlled by the materials available to build them-there is usually no choice as to which material must be used for construction hence, an inevitable compromise must be made between the uniformity or reliability of those materials and the contractors ability to build. Permeability of the material is another attribute to be considered during placement and compaction. In difficult ground (weak rock or particularly weathered slopes) or on poor foundation soils, a good understanding of the geological processes enables a sensible line to be drawn about the theoretical slope angle and the ease of build and maintenance.

An embankment design must address: 1) the in-situ and as-placed character of the material to be used for construction, 2) the character of foundation materials 3) the local hydrological regime, 4) the strength and consolidation characteristics of filling and foundation soils, and 5) the safe slope angles for construction.

Static slope stability

Slopes become unstable when the shear stresses on a potential failure surface exceed the shearing resistance of the soil. In the case of slopes where stresses on the potential failure surface are high the additional earthquake induced stresses needed to trigger failure are low. In this sense the seismic slope stability is dependent on the static slope stability. The most commonly used methods of slope stability analysis are the limit equilibrium methods such as the Culman method (plane failures), the Wedge methods (failure on two or three planes), the Fellenius method (circular and log spiral failures- homogeneous soils), the Bishops simplified and Bishops modified (circular and log spiral failures – homogeneous soils) and others.

In practice, a slope is considered stable if the factor of safety produced from the stability analysis is greater than 1. Factors of safety are introduced in order to allow for certain uncertainties which relate to the accuracy of the method of analysis (how accurate is the failure mechanism), the accuracy of input parameters and also other important factors such as the potential consequences of a slope failure. For permanent slopes the minimum factor of safety is usually 1.5 and for temporary slopes 1.3. In cases of permanent structures such as motorway embankments where after completion, the factor of safety increase with time due to dissipation of pore water pressures, a minimum factor of safety of 1.3 may be acceptable at the end of the construction period.

An important aspect, which needs to be given proper attention in a slope stability calculation, is the possibility of a progressive failure mechanism. The various limit equilibrium methods treat the soil as a rigid perfectly plastic material but in reality many soils exhibit brittle, strain-softening stress-strain behavior.

Seismic slope stability

Seismic slope stability analyses are further complicated by two additional factors: 1) the dynamic stresses induced by earthquake shaking and, 2) the effect of dynamic stresses on the stress strain behavior and strength of slope materials.

Depending on the behavior of the soil during seismic shaking, seismic instabilities may be grouped into two categories: 1) inertial instabilities and 2) weakening instabilities [19].

In the case of inertial instabilities the strength of the soil remains relatively unaffected by the earthquake shaking and any permanent deformations are produced when the strength of the soil is exceeded during small intervals of time by the dynamic stresses. In the case of weakening instabilities the earthquake shaking produces a substantial loss of strength, which gives rise to very large displacements and instability. The most common causes of weakening instability are flow liquefaction and cyclic mobility. There are numerous analytical techniques that deal with the above two categories and these are either based on limit equilibrium or stress-deformation analyses.

Analysis of inertial instability

When the dynamic normal and shear stresses on a potential failure surface are superimposed upon the corresponding static stresses, these may produce inertial instability of the slope if the shear stresses exceed the shear strength of the soil. The problem is approached either by performing a pseudo-static analysis that produces a factor of safety against slope failure or by attempting to calculate permanent slope displacements produced by earthquake shaking using the sliding block method [23, 25].

Pseudo-static Analysis

The pseudo-static approach has been used by engineers to analyze the seismic stability of earth structures since the 1920s. This method of analysis involves the computation of the minimum factor of safety against sliding by including in the analysis static horizontal and vertical forces of some magnitude. These horizontal and vertical forces are usually expressed as a product of horizontal or vertical seismic coefficients and the weight of the potential sliding mass. The horizontal

pseudostatic force decreases the factor of safety by reducing the resisting force and increasing the driving force. The vertical pseudo-static force typically has less influence on the factor of safety since it affects positively (or negatively) both the driving and resisting forces and for this reason this is ignored by many engineers.

The factor of safety of a slope critically depends on the value of seismic coefficient Kh. In the mid 60s when pseudo-static analyses were widely used, one of the biggest problems facing the engineer was that of selecting a value of the seismic coefficient to be used for design purposes [20]. At the time the selection of values for K was mostly empirical and in typical U.S. practice the values used varied between 0.10 and 0.15. In Japan the earth dam code specified values between 0.15 and 0.25. Ambraseys (1960) was the first to make specific suggestions regarding a rational selection of seismic coefficients. He recommended the use of seismic coefficients based on maximum and root-mean square values as determined by elastic response analyses for 20% critical dumping. Typical seismic coefficients and factors of safety used in practice today are given in Table 13.

Table 13 - Typical Seismic Coefficients & Factors of Safety used in Practice [19].

Seismic Coefficient	Remarks
0.10	Major Earthquake, FOS>1.0 Corps of Engineers Manual EM-111-2-1902 (1982)
0.15	Great Earthquake, FOS>1.0 Corps of Engineers Manual EM-111-2-1902(1982)
0.15-0.25	Japan, FOS>1.0
0.05-0.15	State of California
0.15	Seed (1979), with FOS>1.15 and a 20 percent strength reduction
⅓-½ PGA	Marcuson and Franklin (1983), FOS>1.0
½ PGA	Hynes and Franklin (1984)>1.0 and a 20 percent strength reduction

The recommendation by Seed (1979) was based on a study of earth dams constructed of ductile soils (those that do not generate pore water pressures and show no more than 15% loss of strength upon cyclic loading) with crest accelerations less than $0.75g$[27]. He indicated that in these cases deformations would be acceptably small if the earthquake coefficients are 0.10-0.15 with factors of safety greater than 1.0. The recommendation by Hynes-Griffith and Franklin (1984) were based on deformation calculations using 354 accelerograms (see Figure 46) which showed that the use of horizontal

earthquake coefficients equal to 50% of peak ground acceleration and factors of safely greater than 1.0 would not develop dangerously large deformations [21].

Although the pseudo-static approach to stability analysis is simple and straightforward producing an index of stability (F.S.), which engineers are used to appreciating, it suffers from many limitations as it can not really simulate the complex dynamic effects of earthquake shaking through a constant unidirectional pseudo-static acceleration.

The pseudo-static approach should not be used at all when the materials involved might undergo a significant loss of strength under earthquake shaking and should always be used with caution. The most common mistake made in using such analyses does not in fact yield unconservative results, but rather the opposite. This mistake consists of using the expected peak horizontal acceleration as the seismic coefficient [20]

These limitations were recognized by many researchers including Terzaghi (1950), Seed (1966), Seed et al. (1969), Marcuson and Hynes (1980) etc. More specifically, in case of soils that build up large pore water pressures or have a degradation in strength of more than say 15% due to the earthquake shaking, the analysis can be unreliable. As shown by Seed (1979) a number of dams such as the upper & lower San Fernando Dams, Sheffield Dam etc have in fact failed due to earthquake shaking although the calculated factors of safety were well above 1.In the last couple of decades, methods based on the assessment of the permanent slope deformations induced by seismic shaking found increasing applications. These methods are particularly suited to the case of embankments where the magnitude of the induced deformations is used as a measure of the stability of the embankment.

Makdisi and Seed Method (1978)

The method proposed by Makdisi and Seed (1978) for calculating permanent slope deformation of earth dams produced by earthquake shaking is based on the sliding block method (Figures 40, 41, and 42) but uses average accelerations computed with the procedure of Chopra (1996) and the shear beam method. The method uses a plot that relates the peak crest acceleration (Umax) to the peak ground acceleration (Figure 43). In addition, the method uses a plot that relates the average maximum acceleration Kmax with the depth of the potential failure surface (Figure 44) and a plot of normalized permanent displacement with yield acceleration for different earthquake magnitudes (Figure 45). The latter was produced by subjecting several real and hypothetical embankments to several actual and synthetic ground motions, scaled to represent different earthquake magnitudes.

111

The Hynes-Griffin and Franklin [21] method uses the plot in Figure 46 to relate upper bound permanent, mean plus sigma, and mean displacements to the yield acceleration to the design peak ground accelerations.

Stress – deformation analyses (Finite element analysis)

Stress-deformation analyses can be performed mainly using dynamic finite element models, which allow the simulation of the complicated stress-strain behavior of soils. The finite element method is a powerful tool, which can cope with irregular geometries, complex boundary conditions and pore water pressure regimes and can simulate complicated construction operations. The method can predict stresses, movements and pore water pressures due to construction procedures and also predict the most critically stressed zones within a slope. In this way the most likely mode of failure can be identified and deformations up to and sometimes beyond the point of failure can be calculated.

Various methods have been used for calculating permanent strain within individual finite elements namely:

(1) the strain potential approach (Seed et al., 1973)
(2) Stiffness reduction approach (Lee, 1974; Serf et at., 1976)
(3) Non linear analysis approach (Finn et al., 1986)

The stain potential and stiffness reduction approaches are very approximate. Most accurate results may be obtained with the non-linear analysis approach, which employ non linear soil models (stress-strain relationships). The biggest difficulty in employing these models is to obtain soil stress-strain models that are representative of the soil in-situ behavior.
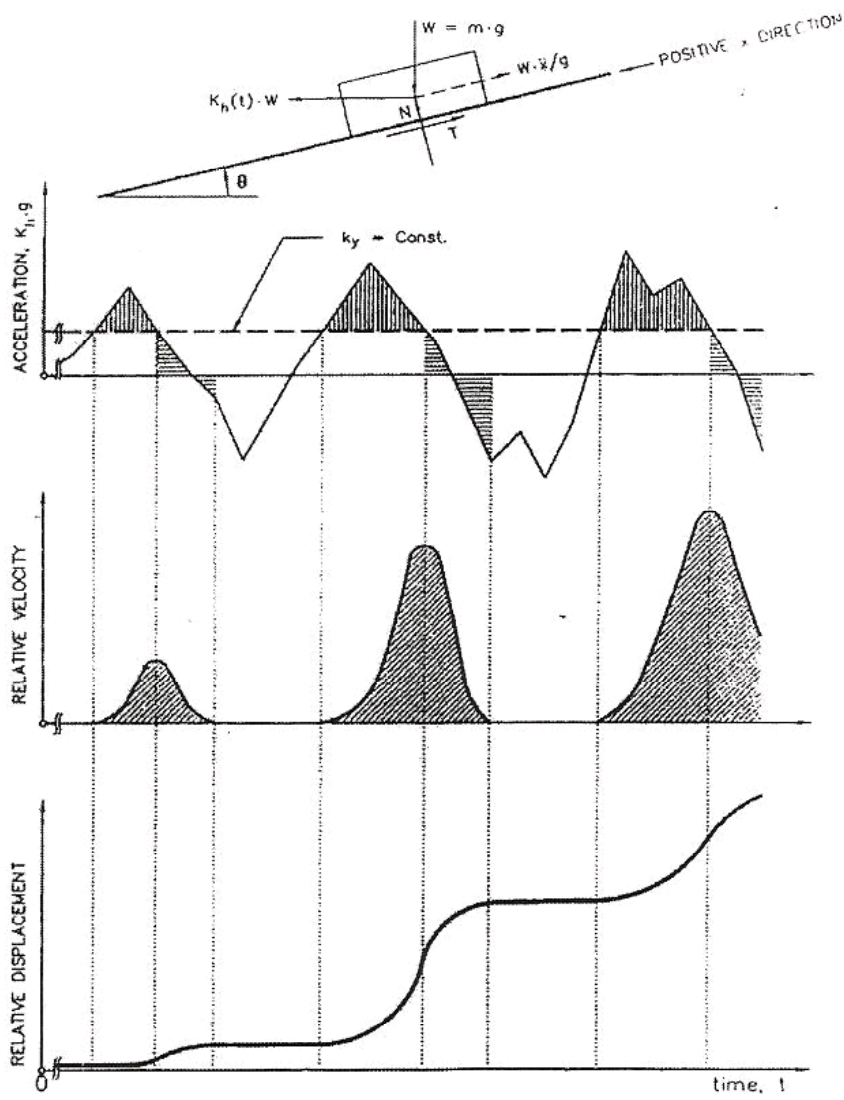
Figure 40. Basic Concept of the Newmark Sliding Block Model
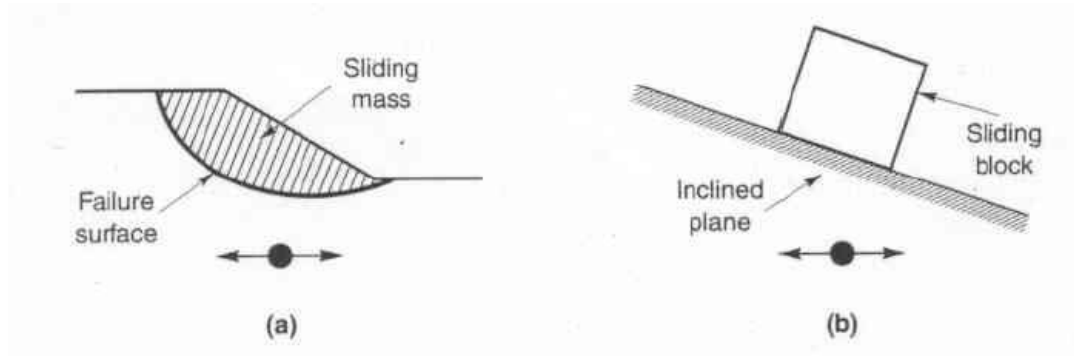(Matasovic et al, 1997).

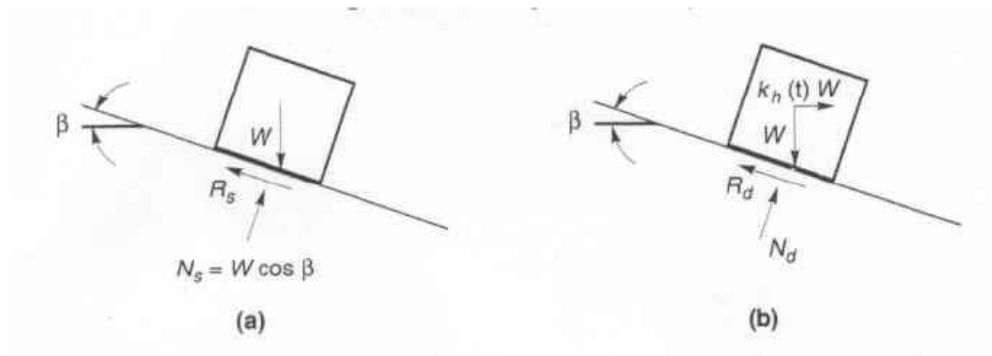Figure 41. Analogy between (a) potential landslide, (b) block resting on an inclined plane [19].

Figure 42. Forces acting on a block resting on an inclined plane, (a) static conditions, (b) dynamic conditions [19].
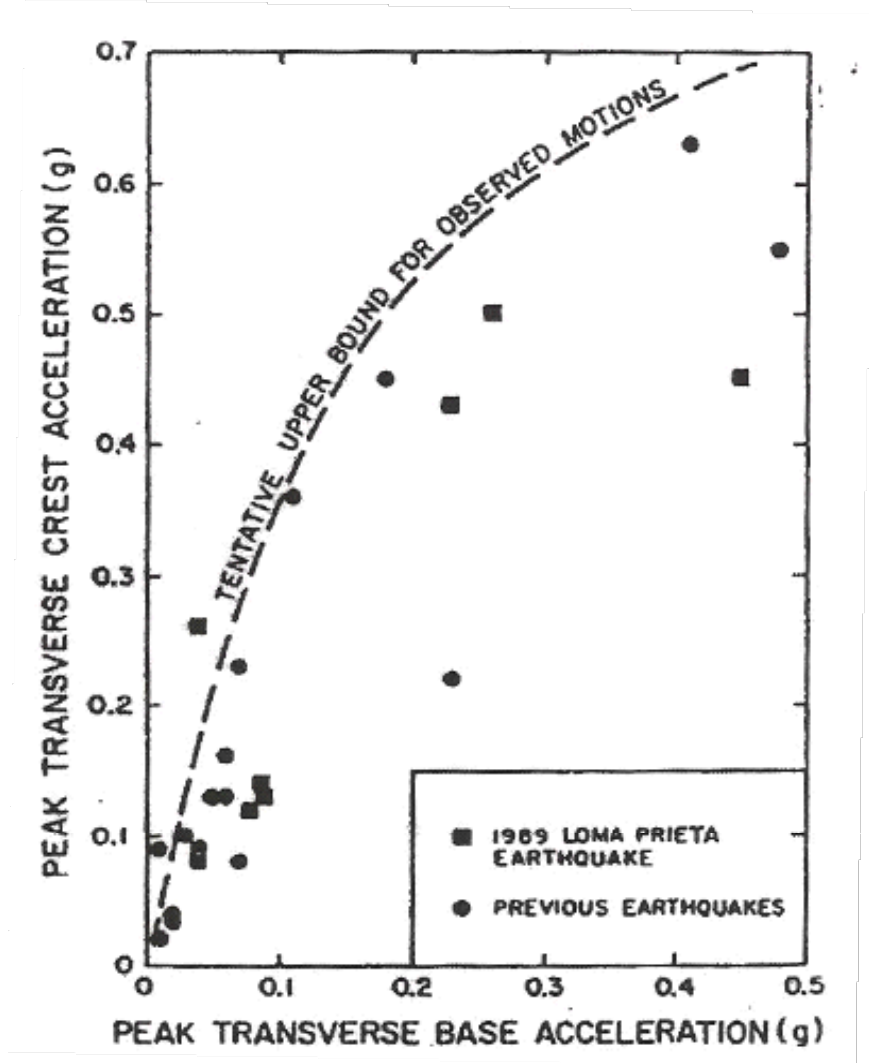
Figure 43. Peak crest acceleration U_{max} versus peak horizontal ground acceleration (Harder, 1991)
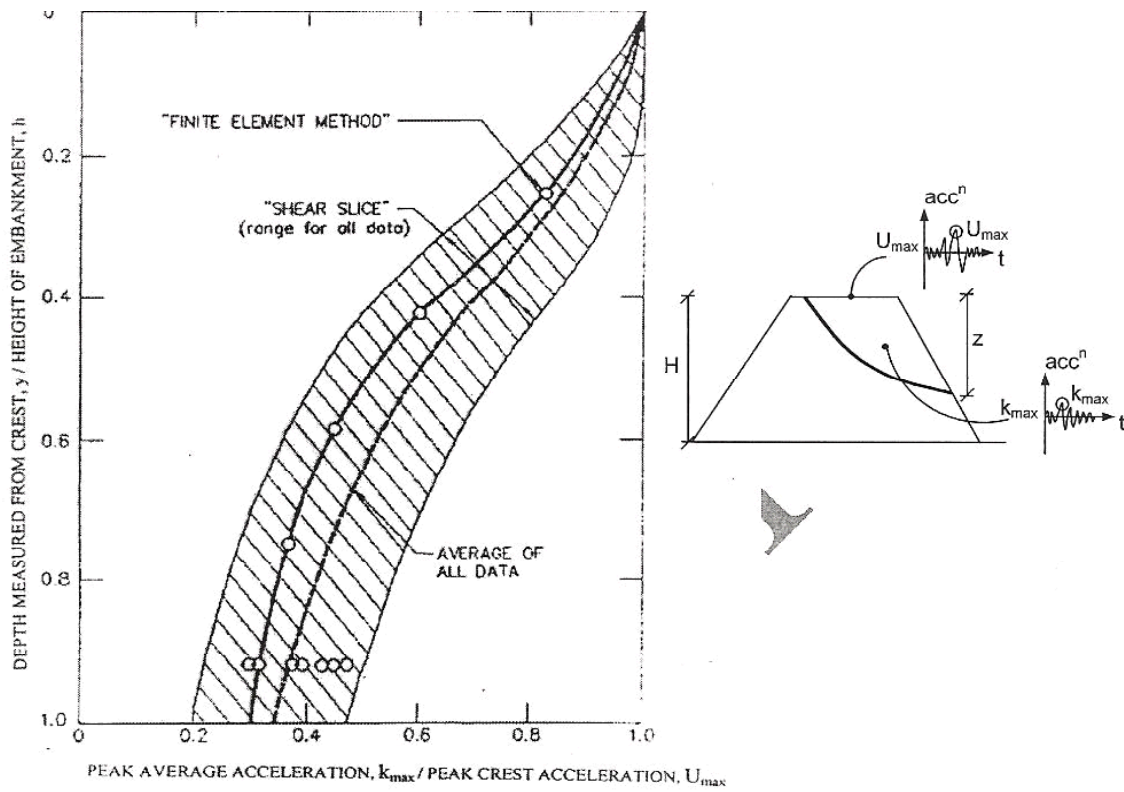
Figure 44. Ratio of K_{max}/U_{max} versus depth of sliding mass (Makdisi and Seed, 1978)
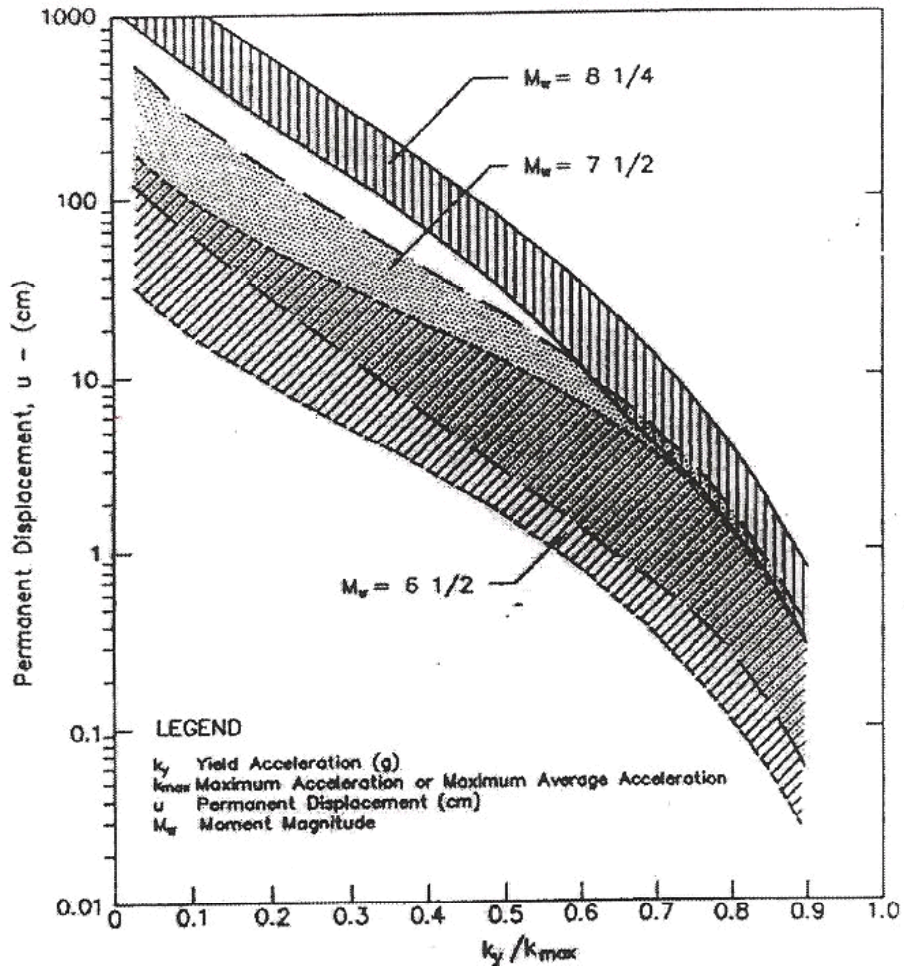
Figure 45. Permanent displacement versus normalized yield accelerations
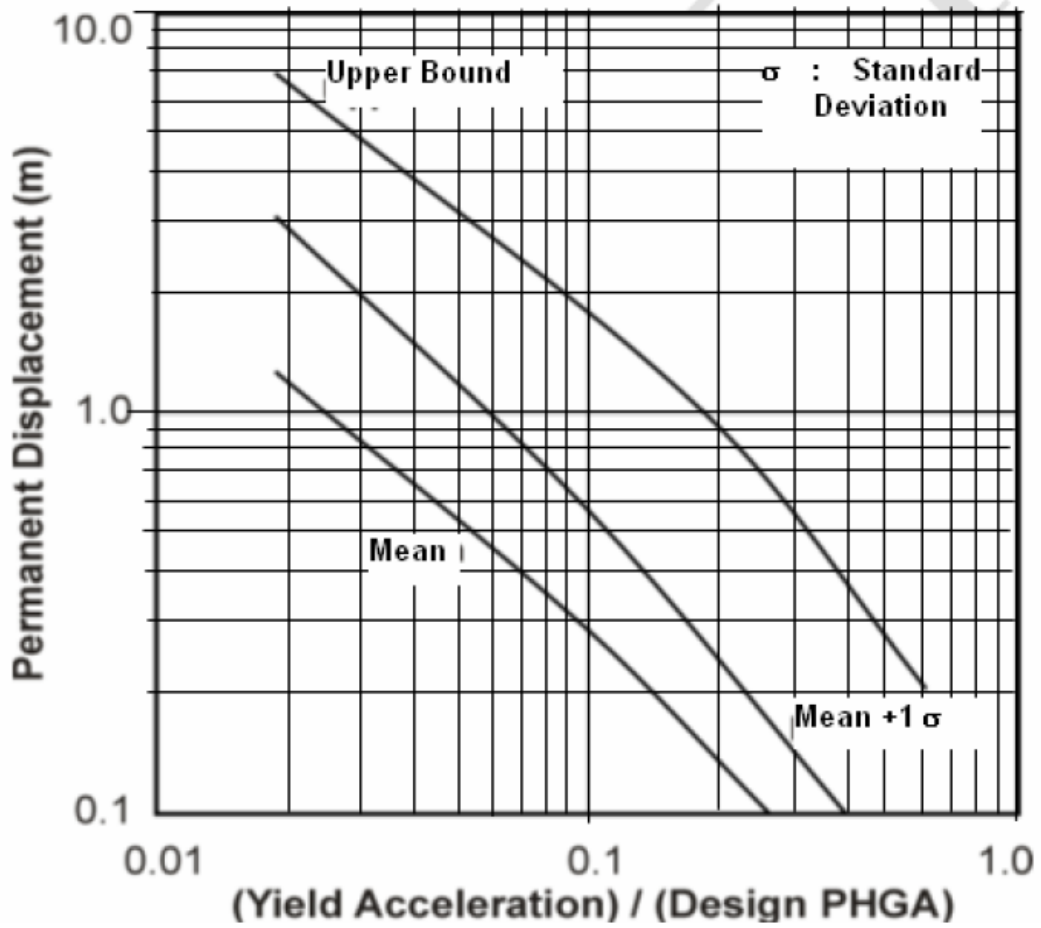(Makdisi and Seed, 1978)

Figure 46. Relation between yield acceleration and permanent displacement
(Hynes-Griffin and Franklin, 1984)[21].

APPENDIX D

ADDITIONAL GUIDELINES ON THE SEIMIC DESIGN OF BURIED STRUCTURES WITH DESIGN EXAMPLES

Free-field Axial and Curvature Deformations

The term 'free-field deformations' describes ground strains caused by seismic waves in the absence of structures or excavations. These deformations ignore the interaction between the underground structure and the surrounding ground, but can provide a first-order estimate of the anticipated deformation of the structure. A designer may choose to impose these deformations directly on the structure. This approach may overestimate or underestimate structure deformations depending on the rigidity of the structure relative to the ground [16]. Using the simplified approach, the free-field axial strains and curvature due to shear waves and Rayleigh waves (surface waves) can be expressed as a function of angle of incidence, as shown in Table 14. The most critical angle of incidence and the maximum values of the strains are also included in the table [17]. Newmark in 1968 and Kuesel in 1969 proposed a simplified method for calculating free-field ground strains caused by a harmonic wave propagating at a given angle of incidence in a homogeneous, isotropic, elastic medium shown in Figure 47. It is often difficult to determine which type of wave will dominate a design. Strains produced by Rayleigh waves tend to govern only in shallow structures and at sites far from the seismic source [13].

Table 14 - Free-Field Ground Strains for Shear and Rayleigh waves.

Wave Type		Longitudinal strain (Axial)	Curvature
Shear Wave	General Form	$\varepsilon = \dfrac{V_s}{C_s}\sin\theta\cos\theta$	$\left(\dfrac{1}{r}\right) = \dfrac{A_s}{C_s^2}\cos^3\theta$
	Maximum Value	$\varepsilon_{max} = \dfrac{V_s}{2C_s}$ For $\theta = 45°$	$\left(\dfrac{1}{r}\right)_{max} = \dfrac{A_s}{C_s^2}$ For $\theta = 0°$
Rayleigh Wave	General Form	$\varepsilon = \dfrac{V_R}{C_R}\cos^2\theta$	$\left(\dfrac{1}{r}\right) = \dfrac{A_R}{C_R^2}\cos^2\theta$
	Maximum Value	$\varepsilon_{max} = \dfrac{V_R}{C_R}$ For $\theta = 45°$	$\left(\dfrac{1}{r}\right)_{max} = \dfrac{A_R}{C_R^2}$ For $\theta = 0°$

Where

θ = Angle of incidence with respect to Tunnel Axis
r = Radius of Curvature
V_s, V_r, = Peak Particle Velocity for Shear Wave and Rayleigh Wave
C_s C_r, = Effective Propagation Velocity for Shear Wave and Rayleigh Wave
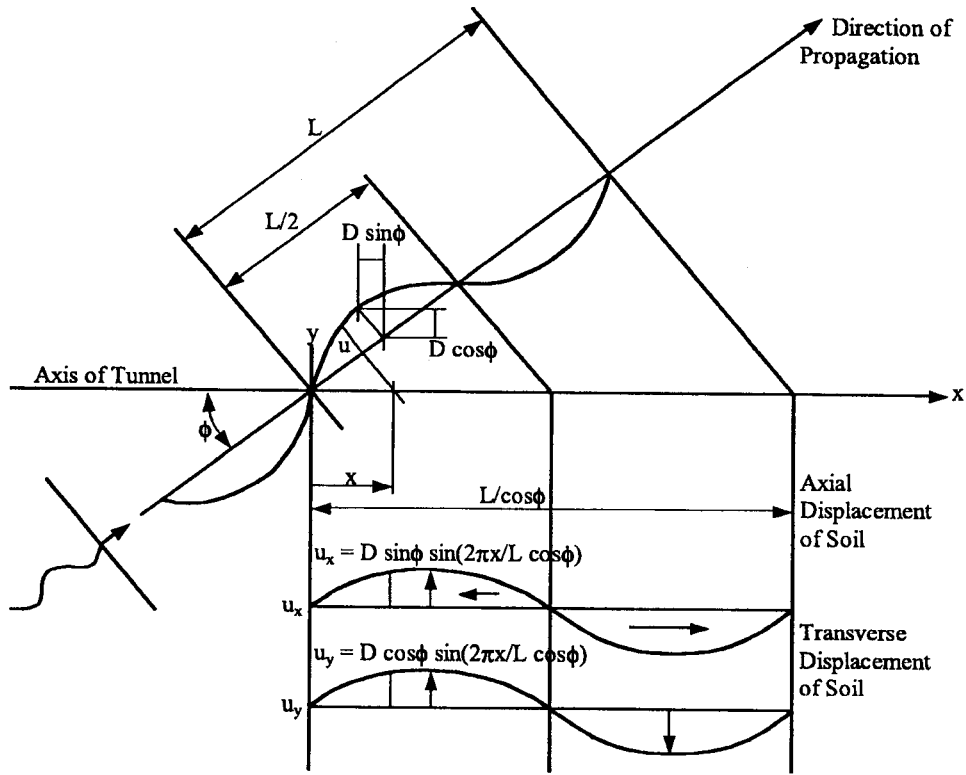A_s , A_r = Peak Particle Acceleration for Shear Wave and Rayleigh Wave

Figure 47. Geometry of a sinusoidal shear wave oblique to axis of tunnel.

Table 15. Combined Axial And Curvature Deformation.

P-Waves	$\varepsilon^{ab} = \left[\dfrac{V_p}{C_p} \cos^2 \phi + r \dfrac{a_p}{C_p^2} \sin \phi \cos^2 \phi \right]$ (5)
S-Waves	$\varepsilon^{ab} = \left[\dfrac{V_S}{C_S} \sin \phi \cos \phi + r \dfrac{a_S}{C_S^2} \cos^3 \phi \right]$ (6)
R-Waves	$\varepsilon^{ab} = \left[\dfrac{V_R}{C_R} \cos^2 \phi + r \dfrac{a_R}{C_R^2} \sin \phi \cos^2 \phi \right]$ (7)

Where

r   = radius of circular tunnel or half height of rectangular tunnel

a_p = peak particle acceleration associated with P-Waves

a_s = peak particle acceleration associated with S-Waves

120

a_R = peak particle acceleration associated with Rayleigh Waves

ϕ = angle of incidence of wave with respect to tunnel axle

V_P = peak particle velocity associated with P-Waves

C_P = apparent velocity of P-Waves propagation

V_S = peak particle velocity associated with S-Waves

C_S = apparent velocity of S-Waves propagation

V_R = peak particle velocity associated with Rayleigh Waves

C_R = apparent velocity of Rayleigh Waves propagation

When these equations are used, it is assumed that the structures experience the same strains as the ground in the free-field. Presence of the structure and the disturbance due to an excavation are ignored. This simplified approach usually provides an upper-bound estimate of the strains that may be induced in the structure by traveling waves. The greatest advantage of this approach is that it requires the least amount of input. Underground pipelines, for which this method of analysis was originally developed, are flexible because of their small diameters (i.e., low flexural rigidity), making the free-field deformation method a simple and reasonable design tool. For large underground structures such as tunnels, the importance of the structure's stiffness sometimes cannot be overlooked. Some field data indicate that stiff tunnels in soft soils rarely experience strains that are equal to the soil strains [16].

SOIL-STRUCTURE INTERACTION APPROACH [17]

Analysis of the tunnel-ground interaction that considers both the tunnel stiffness and ground stiffness is necessary in finding the true tunnel response. In general, the tunnel-ground system is simulated as an elastic beam on an elastic foundation, with the theory of wave propagating in an infinite, homogeneous, isotropic medium. When subjected to the axial and flexural deformations caused by waves traveling in the ground, the tunnel will experience the following sectional forces: 1) Axial forces, Q, on the cross-section due to the axial deformation, 2) Bending moments, M, and, 3) shear forces, V, on the cross-section due to the curvature deformation. These forces are shown in Figure 48.
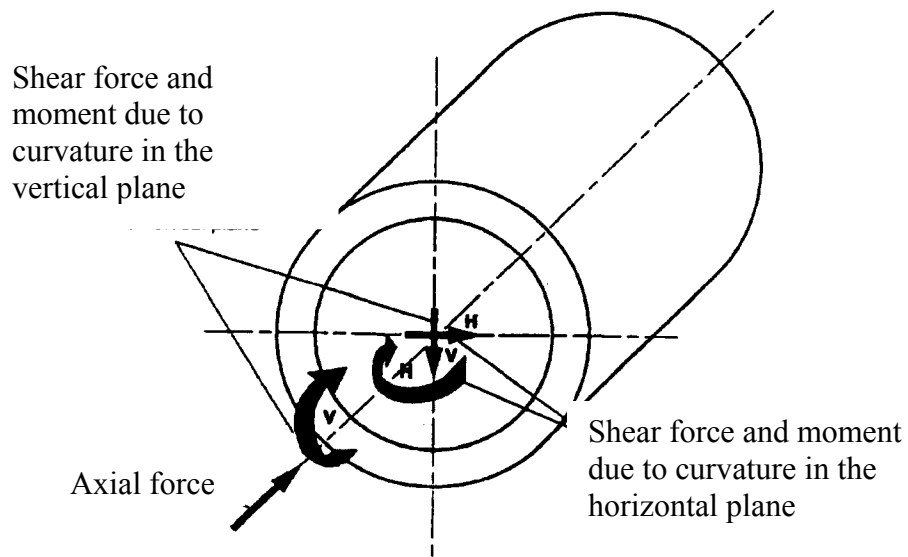
Figure 48. Induced forces and moments caused by waves propagating along tunnel axis.

Simplified Interaction Equations

Maximum Axial Force V_{max}

Through theoretical derivations, the resulting maximum sectional axial forces caused by a shear wave with a 45-degree angle of incidence can be obtained as:

$$Q_{max} = \frac{\dfrac{K_a L}{2\pi}}{1 + 2\left(\dfrac{K_a}{E_l A_c}\right)\left(\dfrac{L}{2\pi}\right)^2} A_a$$

(58)

Where
L   = wavelength of an ideal sinusoidal shear wave
K_a = longitudinal spring coefficient of medium in force per unit deformation per unit length of tunnel from eq. (66)
A_a = free-field displacement response amplitude of an ideal sinusoidal shear wave
E_l = modulus of elasticity of tunnel lining
A_c = cross-section area of tunnel lining

The calculated maximum axial force Q_{max} shall not exceed the upper limit defined by the ultimate soil drag resistance in the longitudinal direction. This upper limit is expressed as:

$$Q_{max} = \frac{fL}{4} \qquad (59)$$

Where f is the ultimate friction force (per unit length of tunnel) between the tunnel and the surrounding medium

Maximum Bending Moment Mmax

The bending moment resulting from curvature deformations is maximized when a shear wave is traveling parallel to the tunnel axis (i.e., with an angle of incidence equal to zero). The mathematical expression of the maximum bending moment is:

$$M_{max} = \frac{K_t \left(\dfrac{L}{2\pi}\right)^2}{1 + \left(\dfrac{K_t}{E_l I_c}\right)\left(\dfrac{L}{2\pi}\right)^4} A_b \qquad (60)$$

Where
Ic = moment inertia of the tunnel section
Kt = transverse spring coefficient of medium in force per unit deformation per unit length of tunnel in eq. (66)

Hashash et al. (2001) method for Finding Bending Strain, Moment and Axial Strain

In this method, the maximum strain is calculated first; then, from the maximum strain, the maximum moment is calculated:

$$\varepsilon_{max}^b = \frac{\left(\dfrac{2\pi}{L}\right)^2 A_b}{1 + \dfrac{E_l I_c}{K_t}\left(\dfrac{2\pi}{L}\right)^4} r \qquad (61)$$

Where
I_c = moment of inertia of the tunnel section *c*
K_t = transverse spring coefficient of the medium in force per unit deformation per unit length of tunnel, eq. (66)

r = radius of circular tunnel or half height of a rectangular tunnel

ε_{max}^a = maximum axial strain

ε_{max}^b = maximum bending strain

$$M_{max} = \frac{E_l I_c \varepsilon_{max}^b}{r} \tag{62}$$

$$\varepsilon_{max}^a = \frac{\left(\dfrac{2\pi}{L}\right)A}{2 + \dfrac{E_l A_c}{K_a}\left(\dfrac{2\pi}{L}\right)^2} \tag{63}$$

Maximum Shear Force, V_{max}

The maximum shear force corresponding to the maximum bending moment is given by:

$$V_{max} = M_{max}\frac{2\pi}{L} = \frac{\dfrac{L}{2\pi}K_t}{1 + \left(\dfrac{K_t}{E_l I_c}\right)\left(\dfrac{L}{2\pi}\right)^4}A_b \tag{64}$$

Comments on the Interaction Equations

Application of these equations is necessary only when the tunnel structure is built in soft ground. For structures in rock or stiff soils, the evaluation based on the free field ground deformation approach, in general, is satisfactory.

A reasonable estimate of the wavelength can be obtained as follows:

$$L = T.C_s \tag{65}$$

Where T is the predominant natural period of the shear wave traveling in the soil deposit in which the tunnel is built, and C_s is the shear wave propagation velocity within the soil deposit. The ground displacement response amplitude, A_b, should be derived based on the site-specific subsurface conditions by earthquake engineers. The displacement amplitude represents the spatial variations of

ground motions along a horizontal alignment. Generally, the displacement amplitude increases as the wavelength, L, increases.

Derivations of spring coefficients Ka and Kt differ from those for the conventional beam on elastic foundation problems in that the spring coefficients should be representative of the dynamic modulus of the ground under seismic loads. Also the derivations should consider the fact that loading felt by the surrounding soil (medium) is alternately positive and negative due to an assumed sinusoidal seismic wave.

For a preliminary design, the expression suggested by St. John and Zahrah (1987) should serve the purpose:

$$K_a = K_t = \frac{16\pi G_m (1 - v_m)}{(3 - 4v_m)} \frac{d}{L} \tag{66}$$

> Where
> Gm = shear modulus of the medium
> vm = Poisson's ratio of the medium
> d = diameter (or equivalent diameter) of the tunnel
> L = wavelength

A review of eqs. (58), (60) and (64) reveals that increasing the stiffness of the structure (i.e., EcAc and EcIc), although it may increase the strength capacity of the structure, will not result in reduced forces. In fact, the structure may attract higher forces as a result of an increased stiffness. Therefore, the designer should realize that strengthening of an overstressed section by increasing its sectional dimensions (e.g., the lining thickness) might not always provide an efficient solution in the seismic design of tunnels. Sometimes, a more flexible configuration with an adequate reinforcement to provide sufficient ductility is a more desirable measure.

RACKING DEFORMATION OF RECTANGULAR TUNNELS

Rectangular tunnels are usually built in shallow soils and are mostly cut and over. Design of such tunnels requires careful consideration of soil structure interaction for two reasons:

1. There is a higher deformation during an earthquake in shallow soils than in deeper soils because of the decreased stiffness of the surrounding soils due to a lower overburden pressure, and the site amplification effect is higher.
2. Box shape tunnels, unlike the circular ones, can not transfer static load completely to soil because of their shape

Therefore, rectangular tunnel lining is usually stiffer than circular tunnel lining.

Calculation of Racking Deformation in Rectangular Tunnels

Consider a rectangular soil element in a soil column under a simple shear condition, as shown in Figure 49. When subjected to simple shear stress the shear strain, or angular distortion of the soil element is given by:

$$\gamma_s = \frac{\Delta}{H} = \frac{\tau}{G_m} \tag{67}$$

$$\frac{\tau}{\gamma_s} = \frac{\tau}{\Delta/H} = G_m \tag{68}$$

$$\gamma_s = \frac{\Delta}{H} = \frac{P}{HS_1} = \frac{\tau W}{HS_1} \tag{69}$$

$$\frac{\tau}{\gamma_s} = \frac{\tau}{\Delta/H} = \frac{S_1 H}{W} \tag{70}$$

$$F = \frac{G_m W}{S_1 H} \tag{71}$$

Where W=width of structure
S_1= the force required to cause a unit racking deflection of the structure
F=flexibility ratio of the structure

In the expression above, the unit racking stiffness, S_1, is simply the reciprocal of lateral racking deflection, $S_1 = 1/\Delta_1$, caused by a unit concentrated force.

Special Case 1: For some one-barrel frames, it is possible to derive the flexibility ratio without resorting to computer analysis. The expression of F developed for a one-barrel frame of equal moment inertia, I_L, for roof and invert slabs, and equal moment inertia, I_H, for side walls is given by:

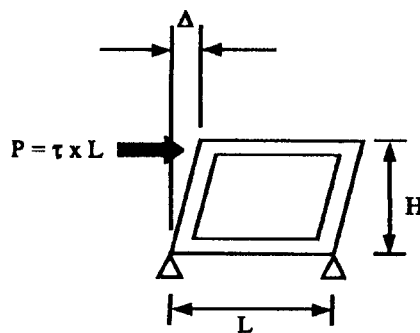$$F = \frac{G}{24}\left(\frac{H^2 L}{EI_h} + \frac{HL^2}{EI_L}\right) \tag{72}$$

Where E = plane strain elastic modulus of frame
G = shear modulus of soil
I_L, I_H = moments inertia per unit width for slabs and walls, respectively

126
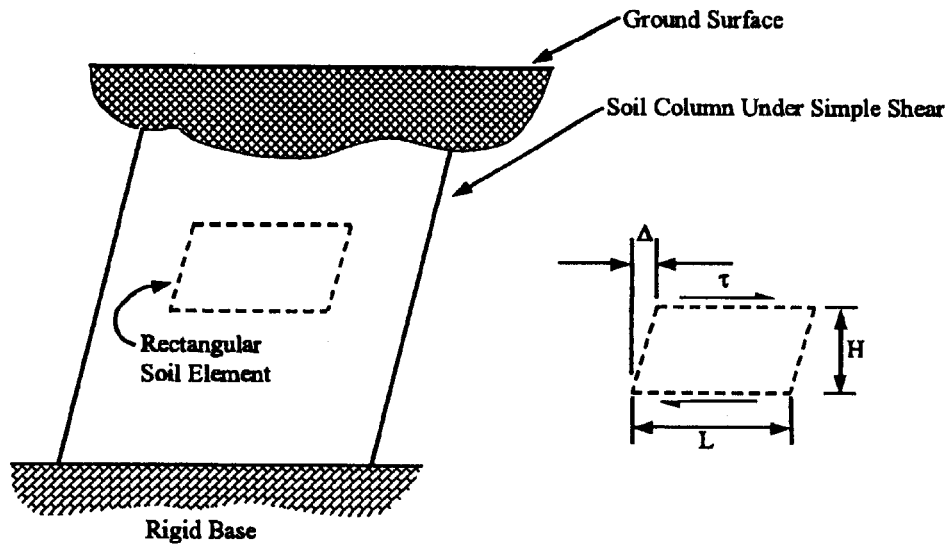
Figure 49. Relative stiffness between soil and a rectangular frame [13]. (a) Flexural shear distortion of free-field soil medium, (b) Flexural racking distortion of a rectangular frame.

Special Case 2: The flexibility ratio derived for a one-barrel frame with roof slab moment inertia, I_R, invert slab moment inertia, I_I, and sidewall moment inertia, I_W, is expressed as:

$$F = \frac{G}{12}\left(\frac{HL^2}{EI_r} \bullet \Psi\right) \tag{73}$$

$$\Psi = \frac{(1+a_2)(a_1+3a_2)^2 + (a_1+a_2)(3a_2+1)^2}{(1+a_1+6a_2)^2} \tag{74}$$

$$a_1 = \left(\frac{I_R}{I_I}\right) \quad \text{and} \quad a_2 = \left(\frac{I_R}{I_W}\right) \bullet \frac{H}{L} \tag{75,76}$$

127

Structural racking and racking coefficient

A racking coefficient, R, defined as the normalized structure racking distortion with respect to the free-field ground distortion is given as:

$$R = \frac{\gamma_S}{\gamma_{free-field}} = \frac{\left(\dfrac{\Delta_S}{H}\right)}{\left(\dfrac{\Delta_{free-field}}{H}\right)} = \frac{\Delta_S}{\Delta_{free-field}}$$ (77)

where
 γ_S = angular distortion of the structure
 Δ_S = lateral racking deformation of the structure
 $\gamma_{free-field}$ = shear distortion/strain of the free-field
 $\Delta_{free-field}$ = lateral shear deformation of the free-field

As expected, the finite element results show distortion of the structure due to racking deformation is mainly dependent on the relative stiffness between the soil and the structure (i.e. flexibility ratio) as described below [13]:

- $F \rightarrow 0.0$ The structure is rigid, it will not rack regardless of the distortion of the ground (i.e. the structure must take the entire load)
- $F < 1.0$ The structure is considered stiff relative to the medium and will therefore deform less
- $F = 1.0$ The structure and the medium have equal stiffness, so the structure will undergo approximately free-field distortion
- $F > 1.0$ The racking distortion of the structure is amplified relative to the free field, though not because of dynamic amplification. Instead, the distortion is amplified because the medium now has a cavity, providing lower shear stiffness than non-perforated ground in the free field
- $F \rightarrow \infty$ The structure has no stiffness, so it will undergo deformations identical to the perforated ground.

Flexibility

1. A flexibility ratio of 1.0 implies equal shear stiffness of the structure and the ground. Thus, the structure should theoretically distort the same magnitude as estimated for the ground in the free field.
2. For flexibility ratios less than 1.0, the structure is considered stiff relative to the free field and should distort less.
3. An infinitely large flexibility ratio represents a perfectly flexible structure. Basically a cavity at this state, the deformed shape of the structure should be identical to that of a perforated ground.

Discussion of flexibility

Analyses have also shown that for a given flexibility ratio, the normalized distortion of a rectangular tunnel is approximately 10 percent less than that of a circular tunnel as shown in Figure 50. This allows the response of a circular tunnel to be used as an upper bound for a rectangular structure with a similar flexibility ratio, and shows that conventional design practice (i.e. structures conform to the free-field deformations) for rectangular tunnels is too conservative for cases involving stiff structures in soft soil (*F*<1.0). Conversely, designing a rectangular tunnel according to the free-field deformation method leads to an underestimation of the tunnel response when the flexibility ratio is greater than one. From a structural standpoint, this may not be of major concern because such flexibility ratios imply very stiff media and therefore, small free-field deformations. This condition may also imply a very flexible structure that can absorb greater distortions without distress [13]. The racking deformations can be applied to an underground structure using the equivalent static load method such as those shown in Figure 51. For deeply buried rectangular structures, most of the racking is generally attributable to the shear forces developed at the exterior surface of the roof. The loading may then be simplified as concentrated force acting at the roof-wall connection as shown in Figure 51 (a). For shallow rectangular tunnels, the shear force developed at the soil roof interface decreases with decreasing overburden. The predominant external force that causes structure racking may gradually shift from the shear force at the soil roof interface to normal earth pressures developed along the side walls, so a triangular pressure distribution is applied to the model as shown in Figure 51(b). Generally, the triangular pressure distribution model provides a more critical value of the moment capacity of rectangular structures at bottom joints, while the concentrated force method gives a more critical moment response at the roof-wall joints [13].

OVALING DEFORMATION OF CIRCULAR TUNNELS

Ovaling of a circular tunnel lining is primarily caused by seismic waves propagating in planes perpendicular to the tunnel axis. Usually, it is the vertically propagating shear waves that produce the most critical ovaling distortion of the lining. The results are cycles of additional stress concentrations with alternating compressive and tensile stresses in the tunnel lining. Several critical modes may result:

1. Compressive dynamic stresses added to the compressive static stresses may exceed the compressive capacity of the lining locally.
2. Tensile dynamic stresses subtracted from the compressive static stresses reduce the lining's moment capacity, and sometimes the resulting stresses may be tensile.
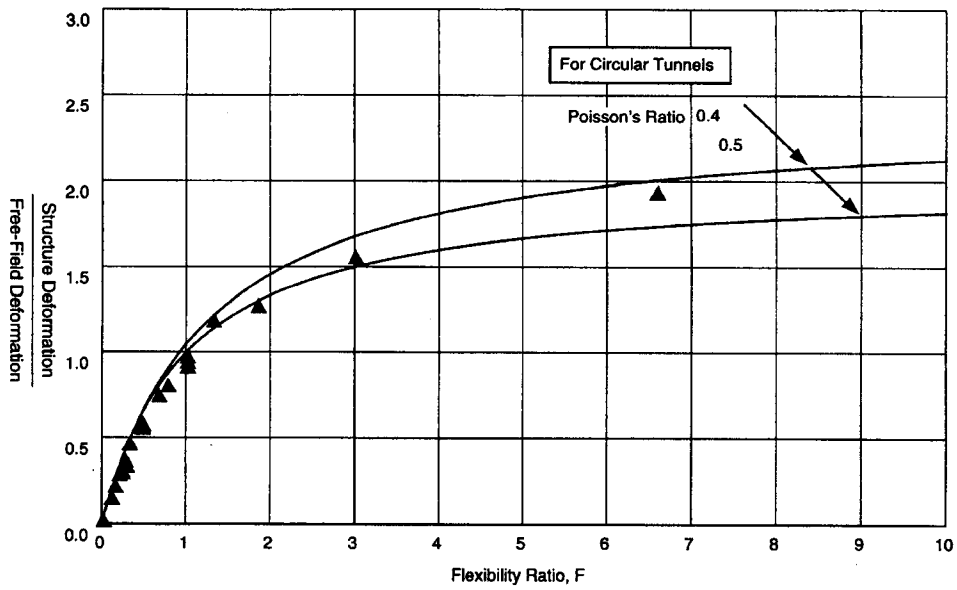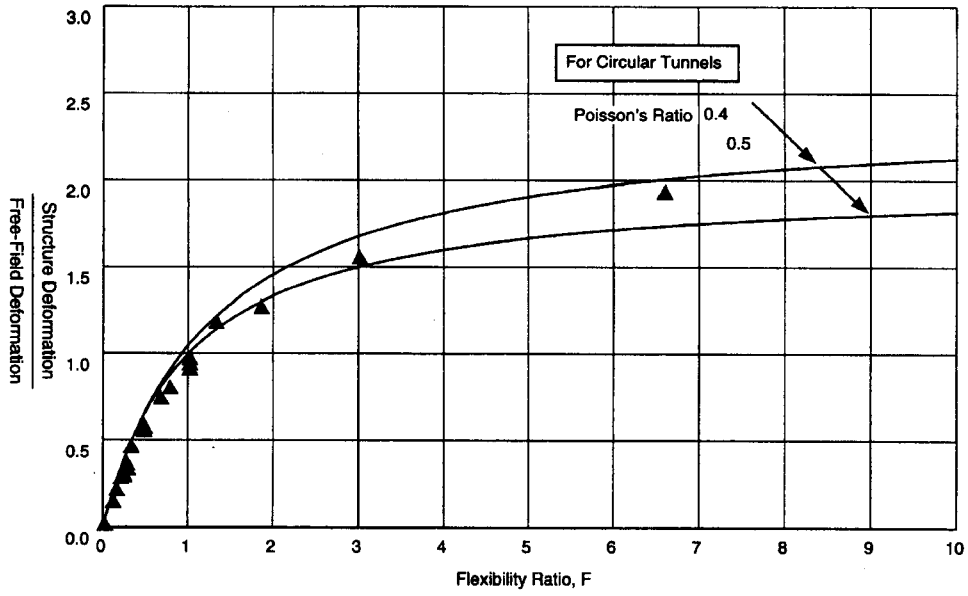
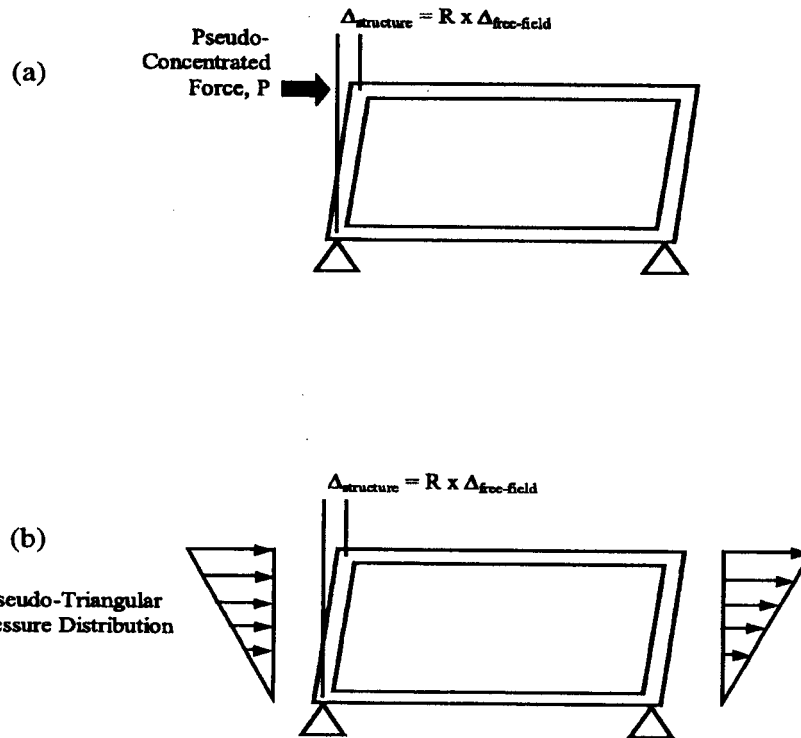Figure 50. Normalized structure deflections.

Figure 51. Simplified frame analysis models [13]. (a) Pseudo-concentrated force for deep tunnels; (b) pseudo-triangular pressure distribution for shallow tunnels.

Calculation of Ovaling Considering Free Field Deformation Approach

Assumptions are made to simplify the site geology of a horizontally layered system and to derive a solution using one-dimensional propagation theory. The resulting free-field shear distortion of the ground from this type of analysis can be expressed as a shear strain distribution or shear deformation profile versus depth. For a deep tunnel located in a relatively homogeneous soil or rock, the simplified procedure by Newmark (see Table 14) may also provide a reasonable estimate. Here, the maximum free-field shear strain, γ_{max}, can be expressed as:

$$\gamma_{max} = \frac{V_s}{C_s} \tag{78}$$

where V_s = peak particle velocity
$\quad\quad C_s$ = apparent (effective) shear wave propagation velocity

The values of C_S can be estimated from in-situ and laboratory tests. The equation relating the effective propagation velocity of shear waves to effective shear modulus, G_m, is:

$$C_s = \sqrt{\frac{G_m}{\rho}} \qquad (79)$$

Where ρ= mass density of the ground

The propagation velocity and the shear modulus to be used should be compatible with the level of shear stain that may develop in the ground under the design earthquake loading. This is particularly critical for soil sites due to the highly non-linear behavior of soils.

Perforated or Not-Perforated

When a circular lining is assumed to oval in accordance with the deformation imposed by the surrounding ground (e.g., shear), the lining's transverse sectional stiffness is completely ignored. This assumption is probably reasonable for most circular tunnels in rock and stiff soils, because the lining stiffness against distortion is low compared to that of the surrounding medium. Shear distortion of the surrounding ground, for this discussion, can be defined in two ways. If the non-perforated ground in the free field is used to derive the shear distortion surrounding the tunnel lining, the lining is to be designed to conform to the maximum diameter change ΔD, shown in Figure 52. The diametric strain of the lining for this case can be derived as:

$$\frac{\Delta D}{D} = \pm \frac{\gamma_{max}}{2} \qquad (80)$$

Where D = the diameter of the tunnel
γ_{max}= the maximum free-field shear strain

On the other hand, if the ground deformation is derived by assuming the presence of a cavity due to the tunnel excavation (see Figure 52, for perforated and non-perforated ground), then the lining should be designed according to the diametric strain expressed as:

$$\frac{\Delta D}{D} = \pm 2\gamma_{max}(1 - \upsilon_m) \qquad (81)$$

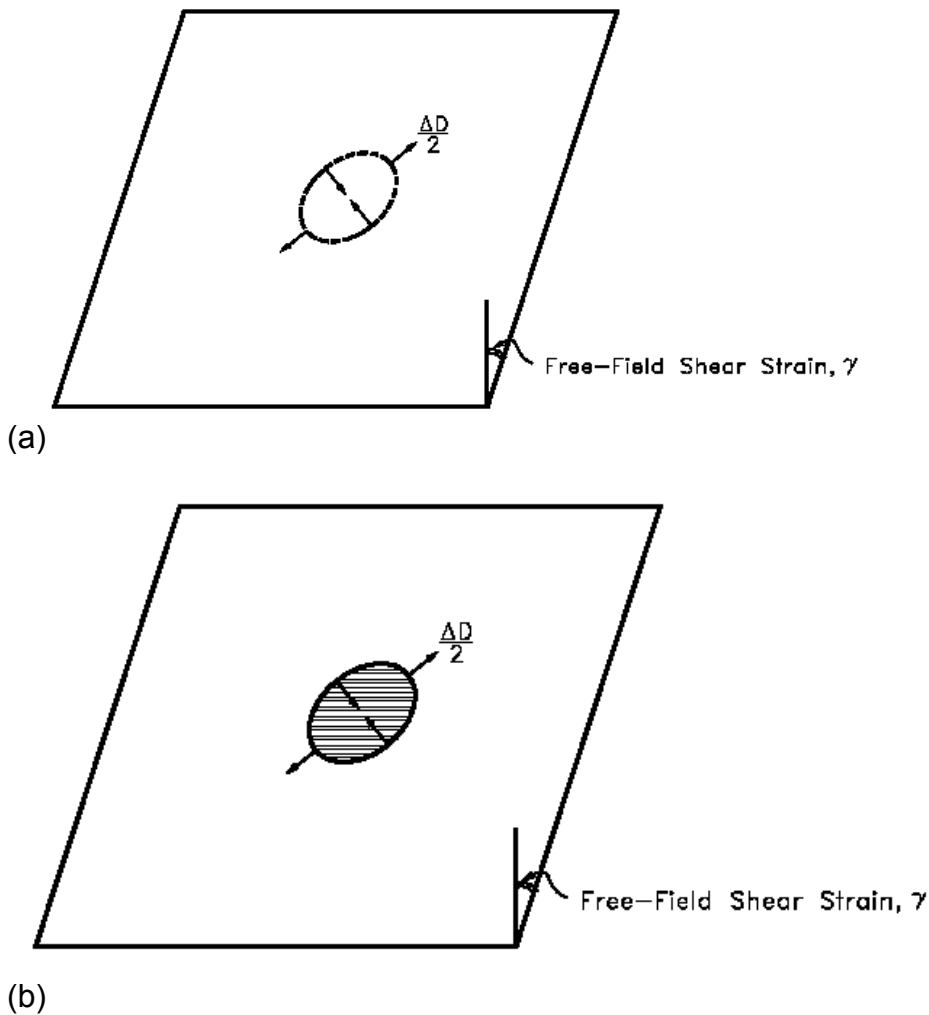Where ν_m= the Poisson's ratio of the medium

(a)

(b)

Figure 52. Free-field shear distortion for, (a) non-perforated ground, (b) perforated ground.

Equations (80) and (81) both assume the absence of the lining. In other words, the tunnel-ground interaction is ignored. Comparison between eqs. (80) and (81) shows that the perforated ground deformation would yield a much greater distortion than the non-perforated, free field ground deformation. For a typical ground medium, solutions provided by eqs. (80) and (81) may differ by a ratio ranging from 2 to about 3. It can be concluded that:

- Eq. (81) for the perforated ground deformation, should serve well for a lining   that has little stiffness (against distortion) in comparison to that of the medium.
- Eq. (80) on the other hand, should provide a reasonable distortion criterion for a lining with the distortional stiffness equal to the surrounding medium.

It is logically acceptable to speculate that a lining with a greater distortion stiffness than the surrounding medium should experience a lining distortion even

less than that calculated by eq. (80). This case may occur when a tunnel is built in soft to very soft soils. To understand the importance of the lining stiffness and how it is quantified relative to the ground, two ratios designated as the compressibility ratio, C, and the flexibility ratio, F are defined by the following equations:

Compressibility Ratio, $C = \dfrac{E_m(1-\upsilon_1^2)R}{E_l t(1+\upsilon_m)(1-2\upsilon_m)}$ (82)

Flexibility Ratio, $F = \dfrac{E_m(1-\upsilon_1^2)R^3}{6E_l I(1+\upsilon_m)}$ (83)

Where

E_m = modulus of elasticity of the medium
υ_m= Poisson's Ratio of the medium
E_l = the modulus of elasticity of the tunnel lining
υ_1 =Poisson's Ratio of the tunnel lining
R = radius of the tunnel lining
t = thickness of the tunnel lining
I = moment of inertia of the tunnel lining (per unit width)

The Compressibility Ratio is a measure of the relative stiffness of the tunnel-ground system under a uniform or symmetric loading condition (horizontal ground stress equal to the vertical ground stress) in the free field, i.e., it reflects the circumferential stiffness of the system. Whereas, the Flexibility Ratio is a measure of the relative stiffness of the tunnel-ground system under an anti-symmetric loading condition (horizontal ground stress equal to, but of the opposite sign of the vertical ground stress in the free-filed), i.e., it reflects the flexural stiffness of the system. Of these two ratios, it is often suggested that the flexibility ratio is more important because it is related to the ability of the lining to resist distortion imposed by the ground.

Ovaling Considering The Lining-Ground Interaction Approach [17]

There are two possibilities of interaction between a structure and soil: (1) Full-slip, and (2) No-slip. According to these cases the thrust force, bending moment and diametric strain can be calculated as follows:

For Full-Slip:

$$T_{max} = \pm\frac{1}{6}K_1\frac{E_m}{(1+\upsilon_m)}R\gamma_{max}$$ (84)

$$M_{max} = \pm \frac{1}{6} K_1 \frac{E_m}{(1+\upsilon_m)} R^2 \gamma_{max} \qquad (85)$$

$$\frac{\Delta D}{D} = \pm \frac{1}{3} K_1 F \gamma_{max} \qquad (86)$$

$$K_1 = \frac{12(1-\upsilon_m)}{2F+5-6\upsilon_m} \qquad (87)$$

Where E_m, ν_m = modulus of elasticity and Poisson's Ratio of the medium
      R = radius of the tunnel lining
      γ_{max} = maximum free-field shear strain
      F = flexibility ratio

For the case of No-Slip:

$$T_{max} = \pm K_2 \tau_{max} R = \pm k_2 \frac{E_m}{2(1+\upsilon_m)} R \upsilon_{max} \qquad (88)$$

$$K_2 = 1 + \frac{F\left[(1-2\upsilon_m)-(1-2\upsilon_m)C\right] - \frac{1}{2}(1-2\upsilon_m)^2 + 2}{F\left[(3-2\upsilon_m)+(1-2\upsilon_m)C\right] + C\left[\frac{5}{2} - 8\upsilon_m + 6\upsilon_{m2}\right] + 6 - 8\upsilon_m} \qquad (89)$$

F = flexibility ratio as defined in Eq. (81)
C = Compressibility ratio as defined in Eq. (80)
E_m, ν_m = modulus of elasticity and Poisson's Ratio of the medium
R = radius of the tunnel lining
τ_{max} = maximum free-field shear stress
γ_{max} = maximum free-field shear strain

To simplify the use of the formula, the graphs in Figure 53 can be used instead. The response coefficient graphs are functions of flexibility and Poisson's ratios.

Full-slip Versus No-slip

According to previous research only during sever earthquakes or in soft soils, slip is possible. For the most cases, the conditions are between the full slip and no slip. So, it is rational to compute both cases and apply the one that is more critical. In a full-slip condition, because of the consideration of the slip, T_{max} is underestimated. It is recommended to the use non-slip formula to find it.

To simplify the use of the formula, Figure 54 graphs can be used instead. The response coefficient graphs are functions of the flexibility and Poisson's ratios.

A finite difference reference solution shows that M_{max} and ΔD_{lining} calculated considering the full-slip, and T_{max} considering the no slip, give the most accurate result. So, it is recommended to calculate M_{max}, ΔD_{lining} and T_{max} using eqs (85), (86) and (88). Another useful formula that describes the relationship between the deformations of lining and free field:

$$\frac{\Delta D_{lining}}{\Delta D_{free-field}} = \frac{2}{3} K_1 F \tag{90}$$

To simplify the use of the formula, Figure 55 can be used instead.

According to these formulas and graphs, the lining tends to resist and deforms less than the free field when F is less than 1. This happens when a stiff lining is built in a soft or very soft soil. As F increases, the lining deflection increases.

ESTIMATING VELOCITY AND DISPLACEMENT

Attenuation relationships are generally available for estimating peak ground surface accelerations, but are also available for estimating peak velocities and displacements. Tables 16 and 17 can be used to relate the known peak ground acceleration to estimates of peak ground velocity and displacement, respectively, in the absence of site-specific data.

$$\Delta_{Structure} = R \times \Delta_{free\ field} \tag{91}$$

Wave Propagation and Site-Specific Response Analysis

Research has shown that transverse shear waves transmit the greatest proportion of the earthquake's energy, and amplitudes in the vertical plan have been typically estimated to be a half to two-third as great as those in the horizontal plane. Ample strong ground motion data are generally not available at the depths of concern for underground structures, so the development of design ground motions needs to incorporate depth-dependent attenuation effects.

There are methods to find the ground motion at depth but in a case of lack or absence of data, Table 18 can be used to find a relationship between the ground motion at depth and at the surface.
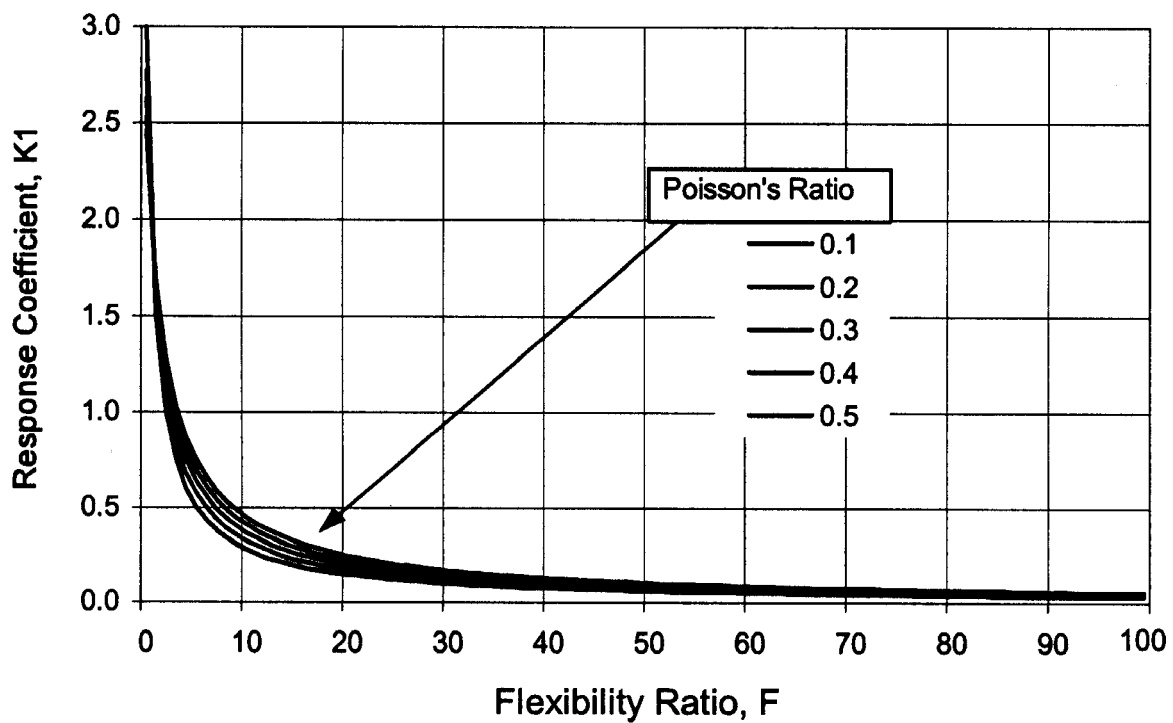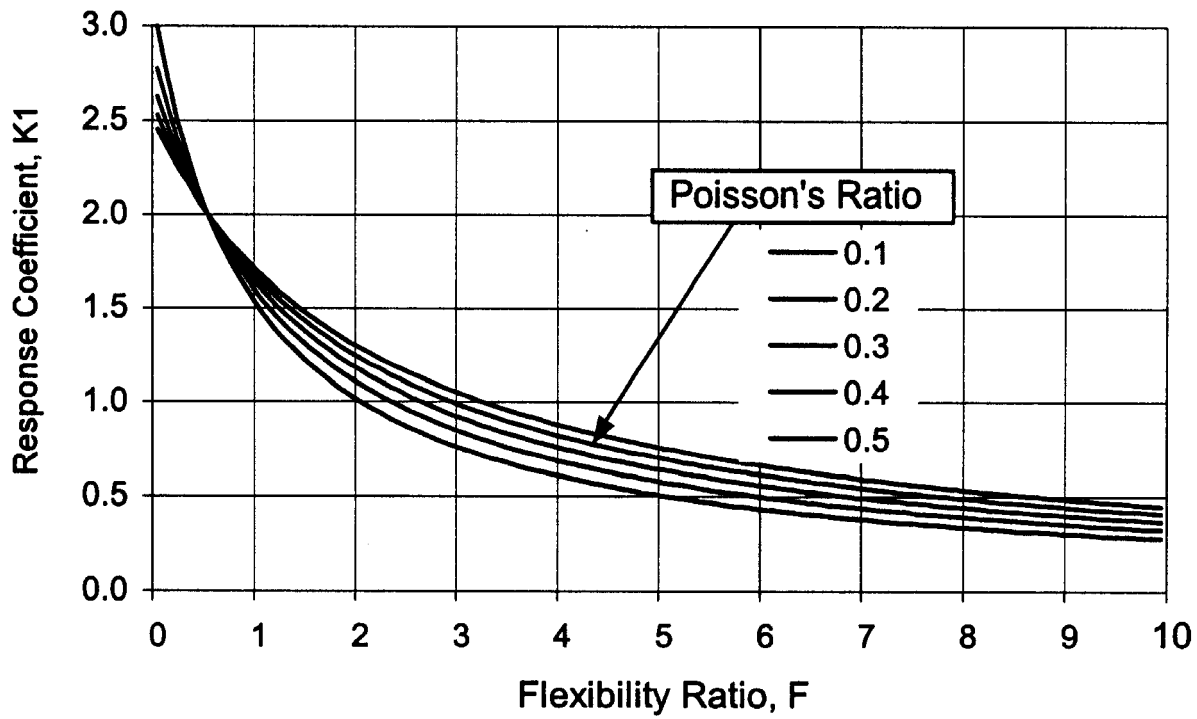
Figure 53. Lining response coefficient vs. flexibility ratio, full-slip interface, and circular tunnel [13].
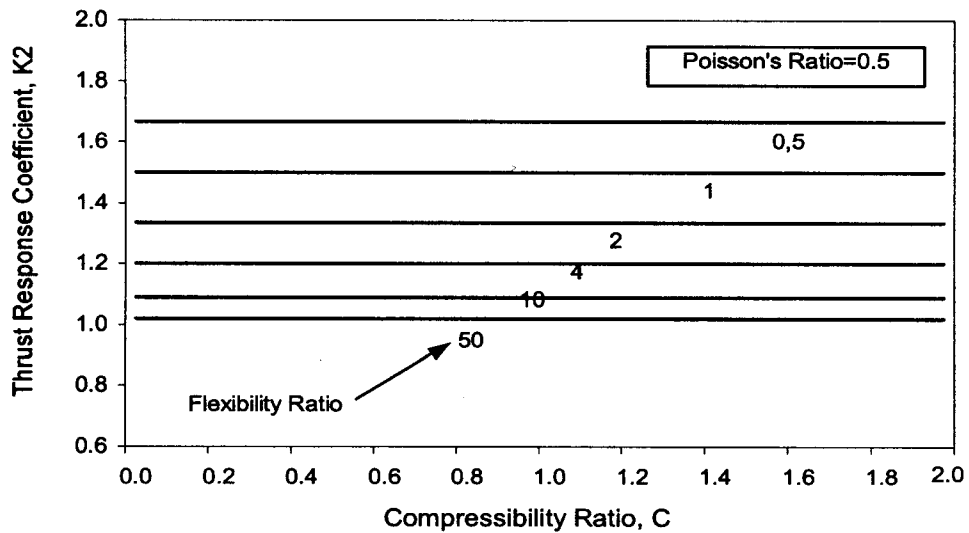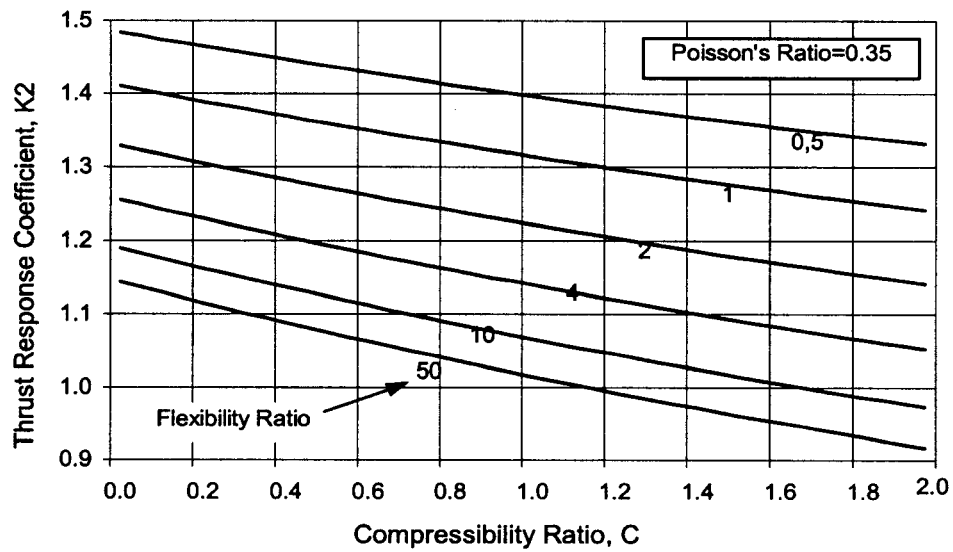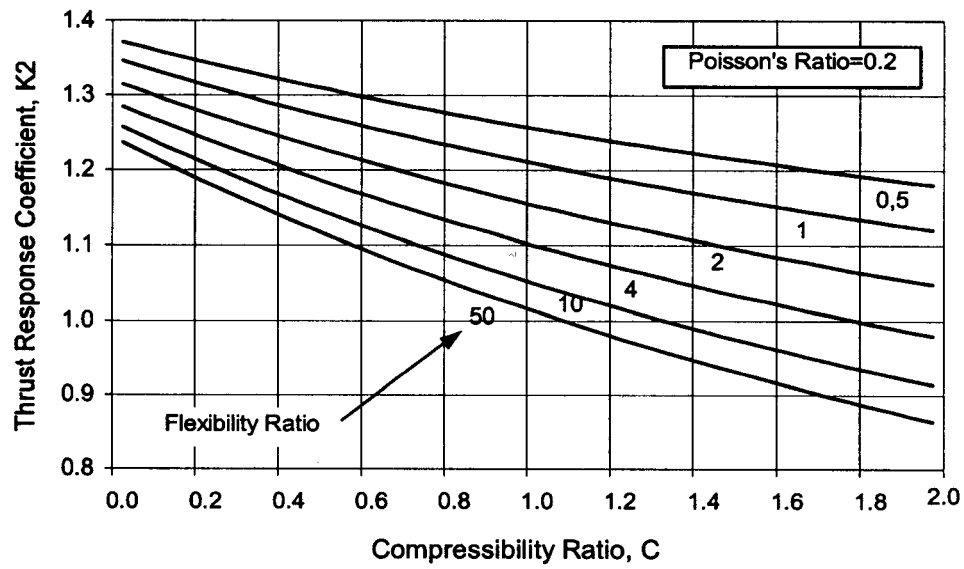
Figure 54. Lining thrust response coefficient vs. compressibility ratio,
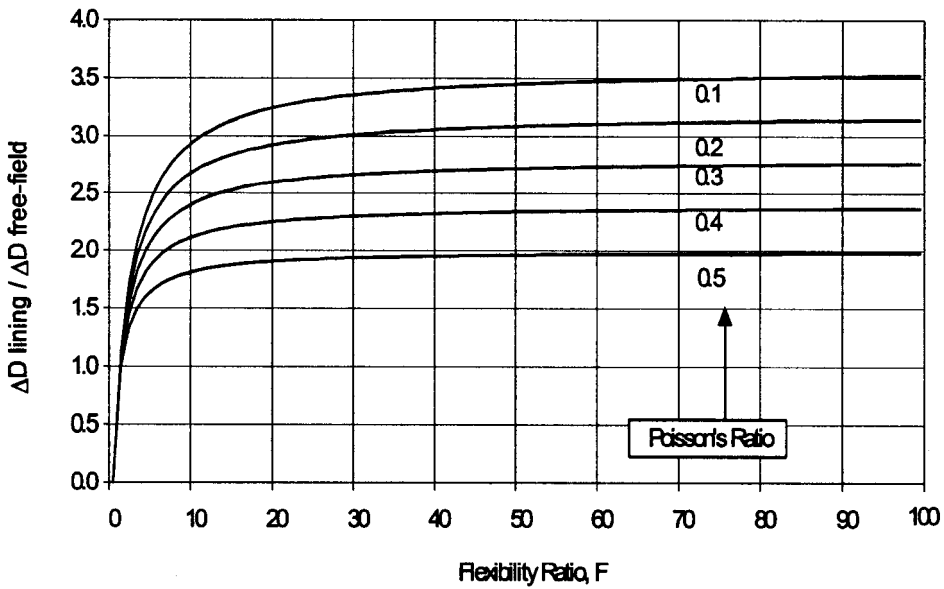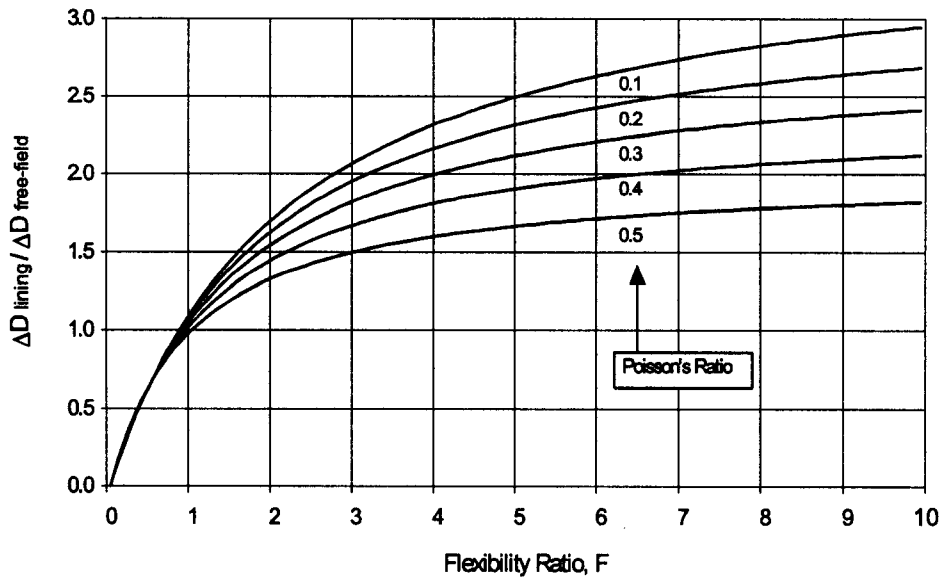No-slip interface, and circular tunnel [13]

138

Figure 55. Normalized lining deflection vs. flexibility ratio, full slip interface, and circular lining [13].

139

Table 16 - Ratios of peak ground velocity to peak ground acceleration at the surface in rock and soil (Power et al., 1996).

| Moment Magnitude (M_w) | Ratio of peak ground velocity (cm/s) to peak ground acceleration (g) | | |
| | Source-to-site distance (km) | | |
	0-20	20-50	50-100
Rock[a]			
6.5	66	76	86
7.5	97	109	97
8.5	127	140	152
Stiff Soil[a]			
6.5	94	102	109
7.5	140	127	155
8.5	180	188	193
Soft Soil[a]			
6.5	140	132	142
7.5	208	165	201
8.5	269	244	251

[a]In this table, the sediment types represent the following shear wave velocity ranges: rock $\geq$ 750 m/s; stiff soil 200-750 m/s; and soft soil <200 m/s. the relationship between peak ground velocity and peak ground acceleration is less certain in soft soils.

Table 17 - Ratios of peak ground displacement to peak ground acceleration at surface in rock and soil (Power et al., 1996)

| Moment Magnitude (M_w) | Ratio of peak ground displacement (cm) to peak ground acceleration (g) | | |
| | Source-to-site distance (km) | | |
	0-20	20-50	50-100
Rock[a]			
6.5	18	23	30
7.5	43	56	69
8.5	81	99	119
Stiff Soil[a]			
6.5	35	41	48
7.5	89	99	112
8.5	165	178	191
Soft Soil[a]			
6.5	71	74	76
7.5	178	178	178
8.5	330	320	305

Table 18- Ratios of ground motion at depth to motion at ground surface
(Power et al., 1996)

Tunnel depth (m)	Ratio of ground motion at tunnel depth to motion at ground surface
≤ 6	1.0
6-15	0.9
15-30	0.8
>30	0.7

DESIGN EXAMPLES

Design Example 1- Linear Tunnel in Soft Ground

In this example a cast-in-place circular concrete lining is assumed to be built in a soft soil site. Geotechnical, structural and earthquake parameters are listed below:

Geotechnical Parameters
- Apparent velocity of S-waves propagation, C_s=110 m/s
- Soil unit weight, γ_t=17.0 kN/m^3
- Soil Poisson's ratio, ν_m=0.5 (saturated soft clay)
- Soil deposit thickness over rigid bedrock, h=30.0 m

Structural Parameters
- Lining thickness, t=0.30 m
- Lining diameter, d=6.0 m$\rightarrow$ r=3.0 m
- Length of tunnel, L_t=125 m
- Moment of inertia of the tunnel section,

$$I_c = \frac{\pi(3.15^4 - 2.85^4)}{4}(0.5) = 12.76 m^4$$ (only half of the full section

  moment of inertia to account to account for concrete cracking and non-linearity during the MDE)
- Lining cross section area, A_c=5.65 m^2
- Concrete Young's Modulus, E_l=24840 Mpa
- Concrete yield strength, f_c'=30 MPa
- Allowable concrete compression strain under combined axial and bending compression, ε_{allow}=0.003 (during the MDE)

Earthquake Parameter
- Peak ground particle acceleration in soil, a_s=0.6 g
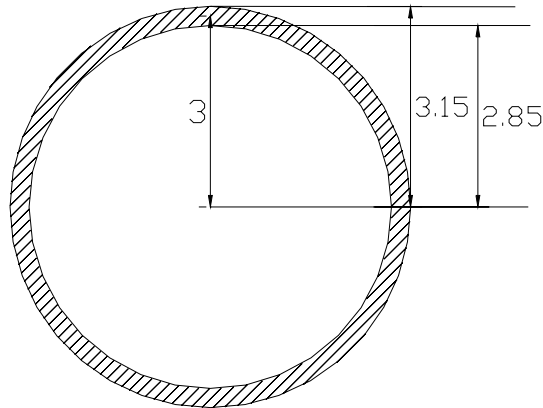- Peak ground particle velocity in soil, V_s=1.0 m/s

Figure 56. Tunnel cross section (Example 1)

Using simplified equation

ϕ=45 gives the maximum value of ε so it is calculated as below:

$$\varepsilon^{ab} = \pm \frac{V_s}{C_s} \sin\phi\cos\phi \pm \frac{a_s r}{C_s^2} \cos^3\phi = \pm\frac{1.0}{(110)} \sin(40)\cos(40) \pm \frac{(0.6)(9.81)(3.0)}{(110)^2} \cos^3(40) = \pm 0.0051$$

The calculated maximum compression strain exceeds the allowable compression strain of concrete: $\varepsilon^{ab} > \varepsilon_{allow} = 0.003$ Negative

Using tunnel-ground interaction procedure

1. Estimate the predominant natural period of the soil deposit (Dobry et al., 1976): $T = \dfrac{4h}{C_s} = \dfrac{(4)(30.0)}{110} = 1.09s$

2. Estimate the idealized wavelength:
   $L = TC_s = 4h = 4(30.0) = 120m$

3. Estimate the shear modulus of soil:
   $G_m = \rho_m C_s^2 = \dfrac{17.0}{9.81}(110)^2 = 20968kPa$

4. Drive the equivalent spring coefficients of the soil:

$$K_a = K_t = \frac{16\pi G_m(1-\nu_m)}{(3-4\nu_m)}\frac{d}{L} = \frac{(16\pi)(20968)(1-0.5)}{(3-(4)(0.5))}\left(\frac{6.0}{120}\right) = 26349kN/m$$

5. Derive the ground displacement amplitude, D:

The ground displacement amplitude is a function of wavelength, L. reasonable estimate of the displacement amplitude must consider the site-specific subsurface conditions as well as the characteristics of the input ground motion. In this design example, however, the ground displacement amplitudes are calculated in such a manner that the ground strains as a result of these displacement amplitudes are comparable to the ground strains used in the calculations based on the simplified free-field equations. The purpose of this assumption is to allow a direct and clear evaluation of the effect of tunnel ground interaction. Thus, by assuming a sinusoidal wave with a displacement amplitude A and a wavelength L, we can obtain:

For free-field axial strain:

$$\frac{2\pi A}{L} = \frac{V_s}{C_s}\sin\varphi\cos\varphi \Rightarrow A = \frac{(120)(1.0)}{2\pi(110)}\sin 45\cos 45 = 0.085m$$

Let $A_a = A = 0.085m$

For free-field bending curvature:

$$\frac{a_s}{C_s^2}\cos^3\varphi = \frac{4\pi^2 A}{L^2} \Rightarrow A = \frac{(120^2)(0.6)(9.81)}{4\pi^2(110^2)}\cos^3\varphi = 0.080m$$

Let $A_b = A = 0.080m$

6.   Calculate the maximum axial strain and the corresponding axial force of the tunnel lining [16]:

$$\varepsilon^a{}_{max} = \frac{\left(\dfrac{2\pi}{L}\right)}{2+\left(\dfrac{E_l A_c}{K_a}\right)\left(\dfrac{2\pi}{L}\right)^2}A_a = \frac{\left(\dfrac{2\pi}{120}\right)}{2+\left(\dfrac{(24{,}840{,}000)(5.65)}{26{,}349}\right)\left(\dfrac{2\pi}{120}\right)^2}(0.085) = 0.00027\text{ The}$$

axial force is limited by the maximum frictional force between the lining and the surrounding soils.

Estimate the maximum frictional force:

$$Q_{max} = (Q_{max})_f = \frac{fL}{4} = E_l A_c \varepsilon^a_{max} = (24840000)(5.65)(0.00027) = 37893 \text{ kN}$$

7.   Calculate the maximum bending strain and the corresponding bending moment of the tunnel lining [17]:

$$\varepsilon_{\max}^{b} = \frac{\left(\dfrac{2\pi}{L}\right)^{2} A_{b}}{1 + \dfrac{E_{l}I_{c}}{K_{t}}\left(\dfrac{2\pi}{L}\right)^{4}}(3.0) = 0.00060$$

$$M_{\max} = \frac{E_{l}I_{c}\varepsilon_{\max}^{b}}{r} = \frac{(24,840,000)(12.76)(0.00060)}{3.0} = 63392kN - m$$

8. Compare the combined axial and bending compression strains to the allowable:

$$\varepsilon^{ab} = \varepsilon_{\max}^{a} + \varepsilon_{\max}^{b} = 0.00027 + 0.00060 = 0.00087 < \varepsilon_{allow} = 0.003 \quad \text{O.K.}$$

9. Calculate the maximum shear force due to the bending curvature:

$$V_{\max} = M_{\max}\left(\frac{2\pi}{L}\right) = (63,391)\left(\frac{2\pi}{120}\right) = 3319kN$$

10. Calculate the allowable shear strength of concrete during the MDE:

$$\phi V_{c} = \frac{0.85\left(\sqrt{f_{c}'}A_{shear}\right)}{6} = \frac{(0.85)\sqrt{30}}{6}\left(\frac{5.65}{2}\right)(1000) = 2192kN$$

Where ϕ=shear strength reduction factor (0.85), f_{c}' =yield strength of concrete (30 MPa), and A_{shear} =effective shear area= A_{c}/2. Note: using ϕ=0.85 for earthquake design may be very conservative.
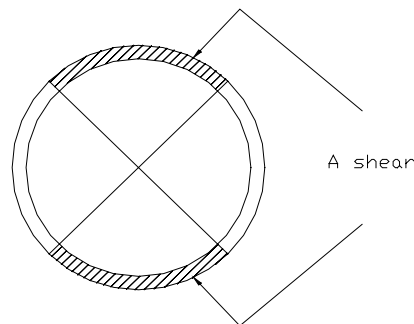

Figure 57. Tunnel shear area (Example 1).

11. Compare induced maximum shear force to the allowable shear resistance:

$$V_{max} = 3319 kN > \phi V_c = 2192 kN$$

Although the calculations indicate that the induced maximum shear force exceeds the available shear resistance provided by the plain concrete, this problem may not be of major concern in actual design because:

1. The nominal reinforcement, generally required for other purposes, may provide additional shear resistance during earthquakes.

2. The ground displacement amplitudes, *A*, used in this example are very conservative. Generally, the spatial variations of ground displacements along a horizontal axis are much smaller than those used in this example, provided that there is no abrupt change in subsurface profiles.

Wang's method to calculate the strains

According to this method, the moment is calculated first afterwards, strain is measured. Also to calculate axial strain, first maximum axial force is calculated:

$$M_{max} = \frac{K_t \left(\dfrac{L}{2\pi}\right)^2}{1 + \left(\dfrac{K_t}{E_l I_c}\right)\left(\dfrac{L}{2\pi}\right)^4} A_b = \frac{26349 \left(\dfrac{120}{2\pi}\right)^2}{1 + \left(\dfrac{26349}{24,840,000 \times 12.76}\right)\left(\dfrac{120}{2\pi}\right)^4} \times 0.08 = 63752.64$$

$$\varepsilon_{bending} = \frac{M_{max} R}{E_l I_c} = \frac{63752.64 \times 3}{24,840,000 \times 12.76} = 0.0006$$

Finding axial strain:

$$Q_{max} = \frac{\dfrac{K_a L}{2\pi}}{1 + 2\left(\dfrac{K_a}{E_l A_c}\right)\left(\dfrac{L}{2\pi}\right)^2} A_a = \frac{\dfrac{26349 \times 120}{2\pi}}{1 + 2\left(\dfrac{26349}{24,840,000 \times 12.76}\right)\left(\dfrac{120}{2\pi}\right)^2} \times 0.085 = 40,329$$

$$\varepsilon_{axial} = \frac{Q_{max}}{E_l A_c} = \frac{40329}{24840000 \times 5.65} = 0.000287$$

$$\varepsilon_{max} = \varepsilon_{axial} + \varepsilon_{bending} = 0.000287 + 0.0006 = 0.00088 < \varepsilon_{allow} = 0.003$$

ε_{max} calculated by Wang's and Hashash's method are respectively 0.00088 and 0.00087 that seems to be pretty good the same.

Design Example 2- Axial And Curvature Deformation Due To S-Waves, Beam- On-Elastic Foundation Analysis Method [16]

Geotechnical Parameters
- Peak ground particle acceleration at the surface, a_{max}=0.5 g
- Apparent velocity of S-wave propagation in soil due to a presence of the underlying rock, $C_{s(R)}$=2 km/s
- Predominant natural period of shear waves, T=2s
- Apparent velocity of S-wave propagation in soil only, $C_{s(s)}$=250 m/s
- Soil density, ρ_m=1920 kg/m³, stiff soil
- Soil Poisson's ratio, ν_m=0.3

Structural Parameters
- Circular reinforced concrete tunnel
- $d = 6m \rightarrow r = 3.0m, t = 0.3m,$ depth (below ground surface)=35 m
- $E_l \, 24.8 \times 10^6 \, kPa, v_l = 0.2, A_c \, 5.65m^2, I_c = \dfrac{\pi(3.15^4 - 2.85^4)}{4} = 25.4m^4$

Determine the longitudinal and transverse soil spring constants:

$$L = C_{s(R)}T = (2000)(2) = 4000m$$

$$G_m = \rho_m C_m^2 = 119800kPa$$

$$K_a = K_t = \frac{16\pi G_m (1 - \upsilon_m)}{(3 - 4\upsilon_m)}\frac{d}{L} = \frac{16\pi(119800)(1 - 0.3)}{3 - (4)(0.3)}\frac{6}{4000} = 3510kPa$$

Determine the maximum axial strain due to S-waves:

Estimate the ground motion at the depth of the tunnel. (Data are not adequate so Tables 15, 16,and 17 should be used)

$a_s = 0.7a_{max} = (0.7)(0.5g) = 0.35g$ Table 17
$A = (35cm/g)(0.35g) = 12.2cm = 0.12m$ Table 16

$$\varepsilon^a_{\ max} = \frac{\left(\dfrac{2\pi}{L}\right)}{2 + \left(\dfrac{E_l A_c}{K_a}\right)\left(\dfrac{2\pi}{L}\right)^2}A = \frac{\left(\dfrac{2\pi}{4000}\right)(0.12)}{2 + \left(\dfrac{(24,800,000)(5.65)}{3510}\right)\left(\dfrac{2\pi}{4000}\right)^2} = 0.00009$$

146

Determine the maximum bending strain due to S-waves:

$$\varepsilon_{\max}^{b} = \frac{\left(\dfrac{2\pi}{L}\right)^{2} A}{1 + \dfrac{E_{l}I_{c}}{K_{t}}\left(\dfrac{2\pi}{L}\right)^{4}} r = \frac{\left(\dfrac{2\pi}{4000}\right)^{2}(0.12)}{1 + \dfrac{(24,800,000)(25.4)}{3510}\left(\dfrac{2\pi}{4000}\right)^{4}}(3) = 0.0000003$$

Determine the combined strain:

$$\varepsilon^{ab} = \varepsilon_{\max}^{a} + \varepsilon_{\max}^{b} = 0.00009 + 0.0000003 \approx 0.00009 < \varepsilon_{allow} = 0.003$$

If the calculated stress from the beam-on-elastic foundation solution is larger than from the free-field solution, the stress from the free-field solution should be used in the design.

Design Example 3- Raking Deformation Of A Rectangular Tunnel [16]

Earthquake and soil parameters:

- M_w=7.5, source-to-site distance =10 km
- Peak ground particle acceleration at surface, a_{max}=0.5 g
- Apparent velocity of S-wave propagation in soil, C_m=180 m/s
- Soft soil, soil density, ρ_m=1920 kg/m³

Tunnel Parameters (rectangular reinforced concrete tunnel):

- Tunnel width (W)=10m.height of tunnel (H)=4m, depth to top=5 m

Determine the free-field shear deformation $\Delta_{free\text{-}field}$:

Estimate the ground motion at depth of tunnel.

$a_{s} = 1.0a_{\max} = (1.0)(0.5g) = 0.5g$ Table 17

Assume soft soil,

$V_{s} = (280m/s/g))(0.5g) = 104cm/s = 1.0m/s$ Table 15

$\gamma_{\max} = \dfrac{V_{s}}{C_{m}} = \dfrac{1.0}{180} = 0.0056$

$\Delta_{free-field} = \gamma_{\max} H = (0.0056)(4) = 0.022m$

Determine the flexibility ratio F:

$$G_m = \rho_m C_m^2 = \left(\frac{1920}{1000}\right)(180^2) = 62000 kPa$$

$$F = \frac{G_m W}{S_1 H}$$

Through structural analysis, the force required to cause a unit racking deflection (1 m) for a unit length (1 m) of the cross-section was determined to be 310 000 kPa. Note that for the flexibility ratio F to be dimensionless, the units of S must be in force per area.

$$F = \frac{(62000)(10)}{(310000)(4)} = 0.5$$

For F=0.5, the racking coefficient R is equal to 0.5.

Determine the racking deformation of the structure $\Delta_{structure}$:

$$\Delta_{structure} = R\Delta_{free-field} = (0.5)(0.022) = 0.011m$$

Determine the stresses in the liner by performing a structural analysis with an applied racking deformation of 0.011 m. Both the point load and triangularly distributed load pseudo-lateral force models should be applied to identify the maximum forces in each location of the liner.

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