STATUS REPORT: NCHRP PROJECTS TO UPDATE GEOTECHNICAL PROVISIONS IN THE SEISMIC SECTIONS OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

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Abstract

Many of the seismic design provisions in the *AASHTO LRFD Bridge Design Specifications* were developed in the early to mid 1980s. Some of the geotechnical provisions, such as determination of ground motions and design provisions for foundations and abutments need to be updated. Two NCHRP Projects, 12-49 and 12-70, have addressed some of these update needs. This paper provides a summary of the work performed on each project.

Introduction

This paper reports on the status of two projects involving development of updates to the geotechnical provisions within the seismic sections of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2004). The AASHTO Specifications document the methods used for the design of bridges within most of the United States. Geotechnical provisions within seismic sections of the *AASHTO LRFD Design Specifications* cover seismic ground motions to use, as well as appropriate design procedures for foundations and retaining walls.

Although the AASHTO Specifications form the basis of current seismic design of bridges in most of the United States, there are some significant limitations to the current Specifications. Specifically, procedures for determining ground motions are outdated, and guidance on the seismic design of foundations and retaining walls is relatively limited. This situation has led to concerns about the ability of the current seismic design methods to provide adequate safety in some parts of the United States, and the lack of guidance in the geotechnical area has resulted in a variety of methods being used to design foundations and retaining walls for seismic loading, and these methods have not always resulted in safe or economic designs.

In an effort to address current limitations within the *AASHTO LRFD Bridge Design Specifications*, two projects have been recently sponsored by the National Cooperative Highway Research Program (NCHRP) with specific goals of updating the provisions within the seismic sections of the LRFD.

- **NCHRP 12-49 Project – Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (Part I: Specifications and Part II: Commentary and Appendices):** This project began in 1998 and involved an update of all seismic provisions within the then-current version of the *AASHTO LRFD Bridge Design Specifications*. The project was administered through a joint venture of the

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Multidisciplinary Center for Earthquake Engineering Research (MCEER) and the Applied Technology Council (ATC). Mr. Ian Friedland served as the Principal Investigator for the Project; Dr. Ronald Mayes was the Technical Director. A project team comprised of structural and geotechnical engineers worked on the seismic provisions within the LRFD sections – resulting in updates to sections covering ground motions, structural analysis, concrete and steel design, and foundations and abutments. This project was completed in 2003 and is summarized in a three-volume report (ATC/MCEER, 2003a, b, c).

- **NCHRP 12-70 Project – Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments:** This project is in its second year of a 3-year program. The prime contractor for this work is CH2M HILL; principal investigators for this work include Dr. Geoffrey Martin of the University of Southern California, Mr. Ignatius (Po) Lam of Earth Mechanics Inc., and Dr. Joe Wang of Parsons Brinckerhoff. Dr. Donald Anderson of CH2M HILL is serving as the project manager for the work. This project is developing seismic design guidelines for free-standing retaining structures, slopes and embankments, and buried structures. One of the products from the NCHRP 12-70 project will be proposed draft LRFD Design Specifications for each of these topics.

The objective of this paper is to provide a summary of these two NCHRP projects – from a geotechnical perspective. This paper is organized to give an overview of the (1) limitations of the current *AASHTO LRFD Bridge Design Specifications* relative to geotechnical engineering, (2) the geotechnical work done or being done in the two NCHRP projects to address current limitations in the *AASHTO LRFD Bridge Design Specifications*, and (3) a geotechnical perspective on the seismic design needs for bridges in seismically active areas. Finally, conclusions are made regarding the relevance of the two NCHRP projects to current seismic design needs for bridges.

Limitations of Current AASHTO LRFD Bridge Design Specifications

As noted in the introduction to this paper, the seismic design of most bridges in the United States follows provisions given in the *AASHTO LRFD Bridge Design Specifications*. The geotechnical requirements are primarily found in Sections 3, 10, and 11 of these Specifications. Section 3 covers methods for determining seismic ground motions, Section 10 involves the design of shallow and deep foundations, and Section 11 deals with the design of abutments, piers, and walls. While these methods are followed by most states, there are exceptions. The California Department of Transportation (Caltrans) has adopted seismic design guidelines that cover geotechnical issues, such as ATC-32 (ATC, 1996). More recently, the Washington State Department of Transportation (WSDOT) documented their approach to seismic design in their Geotechnical Design Manual (WSDOT, 2004).

For those states following design requirements in the *AASHTO LRFD Bridge Design Specifications*, seismic design is based on an acceleration coefficient having a 10 percent probability of exceedance in a 50-year period, which gives an earthquake return period of 475 years. The resulting acceleration coefficient is defined in the AASHTO Specifications as the motion occurring on rock or stiff soils where the soil depth is less than 60 meters. For this case the soil above rock is specified as comprising stable deposits of sands, gravels, or stiff clays. The seismic coefficient is modified according to the soil profile type if these conditions are not met.

One of the primary limitations within the 2004 AASHTO Specifications is that the Specifications use ground motion maps developed by the United States Geological Survey (USGS) in 1988. These maps have been superseded several times, as the USGS have updated their maps to account for changes in understanding of seismic source mechanisms and factors affecting ground motion propagation in the United States. The most recent USGS maps date from 2002 and can be found on the USGS website http://eqhazmaps.usgs.gov/. Some states have adopted the new USGS maps for use in determining seismic ground motions; however, others still rely on the existing maps in the AASHTO Specifications. As will be noted in the discussion of the NCHRP 12-49 project, there is also a realization that the use of a design basis having a 10 percent probability of exceedance in 50 years may be inadequate in the central and eastern parts of the United States, resulting in a level of earthquake risk that is not acceptable.

Another geotechnical limitation in the current *AASHTO LRFD Bridge Design Specifications* is the method used to adjust rock motions for soil amplifications effects. The current AASHTO Specifications include site coefficients that range from 1.0 to 2.0 depending on the soil profile type. The method of determining the appropriate factor is relatively subjective, often leading to uncertainty on the part of the designer when deciding which of the soil profile types to select. The selection of the soil profile type affects the determination of the design response spectrum and resulting seismic forces directly, and therefore, current ambiguity has a direct impact on bridge design. The use of the AASHTO factors can also underestimate spectral accelerations at short periods as shown by Takasumi et al. (2004) for cases where the depth to rock is relative small. This condition can result in serious underestimation of design forces for stiff bridges. As a final note, the site coefficients in the AASHTO Specifications also do not account for increasing degrees of soil nonlinearity at higher acceleration levels, and this effect can result in significant reduction of soil amplification where soft soil conditions exist.

Some state departments of transportation (DOTs) such as Caltrans, South Carolina Department of Transportation (SCDOT), and WSDOT have adopted methods of determining site factors that are based on an explicit determination of soil properties in the upper 30 meters of soil profile. This methodology follows either procedures given in the current International Building Code (2003) or recently recommended methods by Stewart et al. (2003), such as given in WSDOT's Geotechnical Design Manual (WSDOT, 2004). These methods account for the variations in the amplification of seismic ground motions that occur with changes in soil stiffness and bedrock acceleration level. For low levels of peak rock acceleration (e.g., 0.1g) the amplification factors can range from 1.7 to 2.4 at a soft site; for high levels of rock accelerations the amplification factors can be less that 1 for a soft site. These effects are not captured in the existing site coefficients within the AASHTO Specifications.

In the areas of foundation and retaining wall design, the *AASHTO LRFD Bridge Design Specifications* provide limited guidance. Appendix A to Section 10 covers simplified and analytical methods for estimating liquefaction potential. This information is largely unchanged from what was published a decade ago, as evidence by the use of liquefaction resistance plots that are now outdated. Again, some DOTs have updated their design procedures to reflect current methodologies; however, other DOTs still use information in the *AASHTO LRFD Bridge Design Specifications* as a basis for design. Appendix A to Section 10 also provides some recommendations for seismic design of foundations. These include comments on soil property degradation from the effects of repeated cycles of loading, consideration of soil shearing strain amplitude on foundation compliance, lateral loading of piles, and an approach to soil-pile interaction. These discussions are also very limited and largely unchanged from first publication in 1981.

The guidance for the seismic design of retaining walls is also very limited in the *AASHTO LRFD Bridge Design Specifications*. Appendix A to Section 11 *Abutments, Pier, and Walls* summarizes procedures for design of free-standing abutments, which are able to yield laterally during an earthquake. This approach involves use of the well-established Mononobe-Okabe pseudo static equations. While this method is simple to use and has been shown during research studies to give reasonable comparisons to results of small-to-intermediate size experiments, there are a variety of limitations associated with this approach. These limitations include difficulties with estimating reasonable earth pressures when ground shaking levels are high, when steep back slope occur above the retaining wall, and when mixed soil conditions exist. For cases where the retaining structure is restrained either by use of piles to support the wall or by tieback anchors, higher forces must be considered. The *AASHTO LRFD Bridge Design Specifications* suggest using the peak ground acceleration in conjunction with a factor of 1.5 for these cases.

Proposed Updates from NCHRP 12-49 Project

The NCHRP 12-49 project was initiated in 1998 to update seismic provisions in the *AASHTO LRFD Bridge Design Specifications*. A number of significant updates resulted from this work, including

- Changes to the design earthquake and performance objectives. These changes included adoption of the current USGS hazard maps for defining firm-ground motions, use of a two-level approach to design (specifically, a 3 percent probability of exceedance in 75 years for the upper level and a 50 percent in 75 years for the lower level), a change to the shape of the descending (long-period) portion of the response spectrum, and new soil classification factors.

- Various revisions to the design requirements in the AASHTO Specifications. For example, paragraphs were added or revised to cover permissible earthquake resisting systems and elements (ERS & ERE), the “no analysis” design condition, capacity spectrum design methods, displacement capacity verification, foundation and abutment design provisions, liquefaction considerations, steel and concrete design, bearing design and seismic isolation, and design incentives.

The results of the NCHRP 12-49 project are documented in a series of reports that summarize recommended revisions to the *AASHTO LRFD Bridge Design Specifications*. These reports include (1) Part I: Specifications and Part II:

Commentaries and Appendices covering many of the proposed revisions (ATC/MCEER, 2003a), (2) a Liquefaction Study showing application of the proposed methodologies at liquefiable sites in western and central United States (ATC/MCEER, 2003b), and (3)m Design Examples (ATC/MCEER, 2003c). In the following discussions, some of the more significant geotechnical developments and recommendations are summarized.

Ground Motions

These changes involved updating maps used to estimate the seismic coefficient currently being used in the AASHTO Specifications to the most recent USGS maps. However, with this revision there was another very significant decision. Rather than using a 475-year return period (i.e., 10 percent probability of exceedance over a 50-year period), the NCHRP 12-49 project team recommended an earthquake return period of 2475 years. For the 75-year design life now used for bridges, this return period represents approximately a 3 percent probability of exceedance. The consequence of this change in return period from 475 to 2475 years was that the peak ground accelerations on rock increased by a factor of 1.5 to 2 in Western United States (WUS) and by a factor of 3 or more in Central and Eastern United States (CEUS).

There were various reasons for the proposed change in design return period, and the decision was not made without considerable discussions within the project team, with the project's advisory panel, and with various staff at DOTs. The primary reason for the increase was the general recognition that ground motions in the CEUS for the 475-year earthquake would not capture the rare events that have previously occurred in this area, such as the 1811-1812 New Madrid earthquakes. From a risk standpoint this would potentially leave a large inventory of bridges in the CEUS vulnerable to a large, rare event. By increasing the design ground motions to the 2475-year event, the vulnerability of future bridges would be reduced. A second reason for adopting the 2475-year return period was that use of this return period would make the hazard level consistent with the 2475-year return period adopted by the International Building Code (e.g., IBC, 2003). However, in contrast to the IBC approach, the NCHRP 12-49 project team opted not to include a 2/3rds design factor that was used by the IBC to compute the design response spectrum. The 2/3rds factor in IBC is intended to recognize a reserve capacity that exists within most building structures. There had been some controversy in the original decision to use the 2/3rds factor when the IBC provisions were first developed, and there was some doubt whether future versions of the IBC would include this factor. Realizing this, the NCHRP 12-49 project team decided to use the 2475-year return period for establishing the ground motion hazard; any reserve capacity was dealt with more directly through the use of ductility factors and other design provisions. Note that while later studies would show that the combination of ductility factors and 2475-year ground motion resulted in limited changes in bridge design, it did have a significant impact on geotechnical design. Evaluations of liquefaction, slope stability, and lateral earth pressures did not incorporate "ductility adjustments" and, as a result, these hazards would often increase significantly.

Another change that was made to the existing *AASHTO LRFD Bridge Design Specifications* was to adopt the IBC method of defining site classification. The IBC approach defines the site classification on the basis of measured soil properties, such as shear wave velocity or Standard Penetration Test (SPT) blowcounts, in the upper 30 meters of soil profile. Two factors are used to account for site effects, a short-period factor (F_a) and a long-period factor (F_v). These factors vary with soil stiffness and level of peak ground acceleration on rock – with the definition of rock being that the shear wave velocity ranges from 760 meters per second (m/sec) to 1500 m/sec. The NEHRP factors were developed in the early 1990s through an effort of university, government, and practicing engineers and scientists (Martin et al., 1994); these factors were first published in early versions of the NEHRP report *Recommended Provisions for the Seismic Regulations for New Buildings and Other Structures* (BSSC, 2004). While this method was more complicated than the four factors within the *AASHTO LRFD Bridge Design Specifications,* it eliminated much of the subjectivity with the current AASHTO factors by providing a methodology that account for different stiffnesses within the soil profile and soil nonlinearity during higher levels of ground shaking.

As a final revision, the shape of the response spectrum used for design was modified such that the descending portion of the spectrum decreased at 1/T rather than $1/T^{2/3}$. This change recognizes that the $1/T^{2/3}$ decrease was too conservative and not justified by the current understanding of seismic ground motions, and it made the requirements consistent with approach given in the IBC and recommended in the NEHRP documents.

Foundation Design Requirements

The foundation design requirements summarized in Section 10 of the *AASHTO LRFD Bridge Design Specifications* were re-written by the NCHRP 12-49 project team to provide specific guidance relative to seismic design in the areas of foundation investigations, spread footing design, driven pile design, and drilled shaft design. Design was determined by the Seismic Design Requirement (SDR) category. The SDR category was defined in terms of the seismic hazard level (i.e., F_aS_s or F_vS_1) and differed depending on whether the design satisfied life safety or operational requirements. SDRs ranged from requiring very little or no analysis in SDR 1 and 2, to more detailed seismic analyses for SDR 3 through 6.

From a geotechnical perspective the NCHRP 12-49 project team provided specific guidance on appropriate foundation design requirements in SDR 3 through 6. This guidance covers design methods for the three primary types of foundations used for bridges:

- **Spread Footings:** The proposed specifications update identifies methods for dealing with nonlinear soil properties and geometric nonlinearities when developing spring constants for spread footing; and procedures for dealing with moment and shear capacity, including guidance on the stress distribution and the amount of lift-off. Provisions are also provided for handling liquefaction with and without lateral spreading and flow failures. Generally, the guidance indicates that spread footings should not be located where liquefaction is predicted unless the foundation is located below the liquefiable layer, analyses show that the effects of liquefaction are acceptable, or the ground is improved.

- **Driven Piles:** The proposed specifications cover methods for developing axial and rotational springs for pile groups, the maximum load on the leading pile in a group, gapping of the trailing row of piles within a group, and procedures for accounting for additional stiffness from the movement of the pile cap under lateral response. Procedures for handling liquefaction with and without flow or spreading are also provided. The minimum acceptable penetration of piles below liquefied soil layers is also defined.

- **Drilled Shafts:** Procedures generally follow those for driven piles. Guidance is also provided on the minimum stable length of shafts in cohesive and cohesionless soils. The need to consider diameter adjustments for piles greater than 600 mm in diameter and base shear during lateral analysis is also noted.

Abutments and Wingwalls

Similar updates were developed for the seismic design of abutments and wingwalls. For integral abutments this guidance includes use of a passive resisting force based on a best-estimate passive earth pressure coefficient. The passive pressure is computed with wall friction. Guidance is also given on the use of presumptive passive pressures for cohesive and cohesionless backfill within the "passive pressure zone." This passive pressure zone is defined as extending horizontally over a width equal to the wall height before extending to the ground surface at an angle of 45 degrees. The discussions point out that this zone is much wider than the normal active pressure zone, and often encounters soil layers with much higher strength properties than the drainage material immediately behind the wall. Failure to account for this higher strength material in the passive failure zone can seriously underestimate the passive resistance of the abutment.

A number of other topics are also covered within the discussions of abutment and wingwall design. For example, procedures are provided for estimating the stiffness of integral and diaphragm abutments. This procedure assumes an elastic-plastic spring with the amount of displacement required to mobilize the capacity equal to 2 percent of the abutment wall height. For active pressure design, the provisions indicate that Mononobe Okabe equations should be used with the peak acceleration defined as at least 50 percent of the site peak ground acceleration to limit wall displacements.

Other Topics

The provisions, commentary, and appendices to the NCHRP 12-49 report include a number of other useful provisions and discussions. These cover the following areas:

- **Liquefaction Assessments**: Guidance is provided on evaluations to be performed for and consequences of liquefaction within near-surface soils. Appendix D in Part II covers current methods for screening for liquefaction. These methods basically follow the Youd et al. (2001) methodology – with the evaluation of the effects of liquefaction drawn from the SCEC (1999) document *Recommended Procedures for Implementation of DMG Special Technical Publication 117 "Guidelines for*

Analyzing and Mitigating Liquefaction in California." In the specifications (Part I) guidance is also provided on when to conduct a liquefaction assessment. This guidance involves a combination of mean earthquake magnitude from earthquake deaggregation information that can be obtained on the USGS website and SPT blowcounts. For example, if the mean magnitude under the 2475-year earthquake is less than 6.0, a liquefaction analysis is not required. A graded approach involving SPT blowcounts, mean magnitude, and short-period spectral acceleration is used for higher magnitudes.

- **Provisions for Site Characterization**: Appendix C to the NCHRP 12-49 report provides discussions on site characterization considerations, with a specific focus on characterization for seismic studies. Discussions in this appendix cover requirements for liquefaction investigations, such as the depth of exploration, methods of exploration (e.g., SPT, cone penetrometer [CPT], and shear wave velocity methods), and special considerations with these methods. Reference is also made to laboratory testing required for dynamic property determination.

- **Dynamic Site Response Analysis**: Guidance on conducting dynamic site response analyses is provided in Appendix C to Part II. This guidance includes methods of modeling the soil profile, selecting the input rock motions, and conducting the site response analysis. The use of both equivalent linear and nonlinear site response methods is discussed. Nonlinear methods, such as in the computer programs DESRA and SUMDES, are recommended when the soil response is highly nonlinear.

- **Liquefaction Studies**: One of the volumes (ATC/MCEER, 2003b) summarizes results of liquefaction studies that were carried out for hypothetical sites in the WUS and CEUS. This study shows the results of simplified and nonlinear effective stress methods for estimating the response of bridge abutments where liquefaction develops in the supporting soil. Of particular relevance is the method used to account for the potential pinning effects of the foundation system on the response evaluations.

Status of NCHRP Recommendation

These recommended updates to the *AASHTO LRFD Bridge Design Specifications* addressed a number of needed improvements in the areas of geotechnical and bridge design; however, when the recommended provisions were reviewed by each state and a vote was held by the AASHTO representatives, the recommended provisions were not approved. As a result, neither the bridge nor geotechnical recommendations have been adopted in the 2004 AASHTO Bridge Specifications.

There were several apparent reasons for not adopting the recommendations as written. Many of the DOTs thought that the provisions were too complex and would lead to unwarranted design work. The change in earthquake return period from 475 to 2475 years was a particularly controversial issue. The initial reaction was that design forces would double to quadruple, depending on the state. While the NCHRP 12-49 project team provided comparisons to show that with changes in ductility factors and other considerations, the cost of analysis and construction were generally less than

16 percent, the state DOTs felt that their design and construction costs would be significantly impacted. The method was also judged to be too difficult for DOT staff and consultants to apply. For example, there was a concern about the ability of the DOTs and consultants in some areas to conduct liquefaction analyses, when they had not been required to do this in the past. This was somewhat surprising response as almost every geotechnical engineer graduating from a university in the United States over the last one to two decades, has been trained to perform liquefaction analyses, and the methods are based on simple field methods (i.e., SPTs) with analysis procedures that are easily carried out on an Excel spreadsheet.

Work is currently underway on another NCHRP project, NCHRP 20-07/Task 193, to address the limitations to the NCHRP 12-49 reports identified by AASHTO. This work is being conducted by Dr. Roy Imbsen of Roy Imbsen of Imbsen & Associates. These revisions include use of a shorter return period, most likely 1000 years – though the AASHTO Highways Subcommittee on Bridges and Structures has yet to formally adopt the proposed return period. The overall method will be made simpler by eliminating the two levels of design, by considering only life safety, and by increasing the areas involving no seismic design. It is understood that many of the geotechnical provisions given in the NCHRP 12-49 reports will be included in the NCHRP 20-07 revision but with changes made to account for the shorter return period (i.e., 1000 years versus 2475 years) and to reduce the perceived complexity. A draft of the revised document is planned for later this year or early in 2006.

Future Recommendations from NCHRP 12-70 Project

One of the limitations identified at the start of the NCHRP 12-49 project was that it only treated the bridge component of the seismic provisions in the *AASHTO LRFD Bridge Design Specifications*. Several areas important to the roadway and bridge design were not covered, namely free-standing retaining walls, slopes and embankments, and buried structures. The buried structure involves drainage culverts and pipes, box culverts, and pedestrian tunnels as covered by Section 12 *Buried Structures and Tunnel Linings* of the *AASHTO LRFD Bridge Design Specifications.* In recognition of this need, the NCHRP 12-70 project was initiated in April of 2004. The objectives of the NCHRP 12-70 project are to (1) develop methods of analysis for seismic design of retaining walls, slopes and embankments, and buried structures, and (2) prepare draft LRFD specifications addressing the seismic design of retaining walls, slopes and embankments, and buried structures.

Limitations and Knowledge Gaps

The first task within the NCHRP 12-70 project was to identify limitations and knowledge gaps relative to the seismic design of retaining walls, slopes and embankments, and buried structures. This information was developed through literature reviews, discussions with representatives of DOTs, and review of current design methods given in the AASHTO Specifications and in the FHWA design guidelines. A number of these limitations and knowledge gaps were evident from this effort, including:

- Difficulties with retaining wall designs for seismic loading. Specifically, the Mononobe Okabe method is unable to handle steep back slopes or high peak ground accelerations. There is also uncertainty or debate on the determination of seismic coefficients and the methods to use for the design of soldier pile, tieback, nail, and mechanically stabilized earth (MSE) walls.

- Lack of guidance for evaluating seismic slope stability of both constructed embankments and cut slopes. Questions include the use of pseudo static versus deformational approaches for assessment of stability, the appropriate seismic coefficient, how to handle ground motion amplification, and treatment of liquefaction effects.

- Lack of guidance in Section 12 of the *AASHTO LRFD Bridge Design Specifications* on seismic design of buried structures. Issues include design for transient versus permanent ground displacements; liquefaction effects including flotation, lateral spreading, and settlement; and the possibility of different design requirements for flexible versus rigid pipes.

Ongoing and Planned Developments

Work on the NCHRP 12-70 project is focusing on the development of methods to address the current limitations. Four primary work tasks have been identified – development of ground motions, new retaining wall design methods, procedures for addressing the performance of slopes and embankments, and methods of design for buried structures. Preliminary plans and results from this ongoing work are summarized in the following paragraphs.

The ground motion study is focusing on development of correlations between peak ground velocity (PGV) and peak spectral acceleration at 1 second (S_1) for central and eastern United States (CEUS) and for western United States (WUS). The reason for using S_1 is that measures of ground shaking at an intermediate period (i.e., PGV) have been observed to be a better indicator of displacement demand than peak ground acceleration (PGA). The most convenient measure of ground shaking is the S_1 value that can be obtained from the USGS maps. The correlation between PGV and S_1 will differ between CEUS and WUS because of the different seismological conditions in each area. This correlation is being developed by conducting regression analyses of a large earthquake record database that has been developed through United States Nuclear Regulatory Commission (USNRC) work involving the seismic hazard to existing and planned nuclear power plants. While the focus of the ground motion studies is on relationships between PGV and S_1, some geotechnical evaluations, such as liquefaction, currently depend on PGA rather than PGV or S_1. For this reason the ground motion studies include development of methods for obtaining PGA from S_1 for CEUS and WUS locations. These ground motion studies will make the resulting determination compatible with recommendations that are being developed for the seismic design of bridges in the NCHRP 20-07 project. The resulting PGV- S_1 correlation will serve as a basis for the performance-based approach being developed for the NCHRP 12-70 project.

The approach being taken for the seismic design of retaining walls involves a departure from the conventional Mononobe Okabe procedure currently summarized in the *AASHTO LRFD Bridge Design Specifications* and used by most engineers for estimating seismic earth pressures. The method involves implementing a limit-equilibrium approach through the use of slope stability computer methods. It was concluded during preliminary phases of the NCHRP 12-70 project that the limit-equilibrium approach would be a practical way of addressing current limitations with the Mononobe Okabe equations – namely inability to handle high ground accelerations, steep back slopes, and mixed backfill conditions. The limit equilibrium approach can be coupled with Newmark displacement analyses to determine pressures against walls for different amounts of ground displacement. For rigid retaining walls this approach is consistent with current AASHTO recommendations that allow 50 percent of the peak ground acceleration for a deformation of roughly 100 mm. The development becomes more complex for walls that are restrained by tiebacks or pile foundations systems. In this case compatibility needs to exist between predicted displacement for the soil-structure system and acceptable structural performance – likely requiring an iterative approach. To simplify the overall design methodology, situations that are amenable to the use of Mononobe Okabe equations will be identified, such as level ground conditions with a homogeneous backfill and moderate ground accelerations. As part of this effort, wave scattering studies are being conducted to develop seismic coefficient adjustment factors that are wall-height dependent to use in the displacement analyses.

The work for slopes and embankments is focusing on two issues: development of Newmark-type displacement charts that are applicable to CEUS and WUS and wave scattering studies. The use of displacement charts to investigate performance of slopes and embankments is not new. This method has been used to estimate performance of earth dams since the 1960s. It requires using conventional slope stability methods to estimate the acceleration causing yield of the slope, referred to as the yield acceleration (k). Displacements are related to the ratio of yield acceleration to maximum ground acceleration (k_{max}). Martin and Qiu (1994) show that the form of the resulting curves can be separated into two cases, one set for low ground velocities (e.g., $V = 30\ k_{max}$ where V is in inches per second and k_{max} is the peak ground acceleration) and the second set for high ground velocity (e.g., $V = 60\ k_{max}$). Curves developed by Martin and Qiu are being re-evaluated based on the USNRC ground motion database described above. As also noted, wave scattering studies are also being conducted. Results of these analyses will indicate how the peak ground acceleration should be modified as the ratio of the length of the critical slope failure surface changes relative to the wave length of the propagating seismic wave. For very long slopes, reductions in ground motion occur due to the incoherence of the ground motion.

In the fourth area of study seismic design methods are being developed for buried structures. This development has been separated into two basic tasks. One will address the transient ground displacements (TGD); the other will address permanent ground displacements (PGD). The former category considers the longitudinal, ovalling, and racking modes of response, while PGD considers the displacements from fault ruptures, landsliding, and liquefaction-induced ground deformations.

Methodologies developed for the seismic design of tunnels are being reviewed for use during the TGD evaluation. This review is evaluating the effects of diameter and embedment on equations and relationships currently used for tunnel design – in view of the small diameter and shallow depth of burial for most culverts and drainage structures used on highway projects. The PGD component of the work involves development of methodologies for the design of buried structures rather than design equations or charts. As part of this development, seismic screening methods are being developed. These screening methods will show when more detailed evaluations of the buried structure are required.

Status of NCHRP 12-70 Project

The NCHRP 12-70 project is in its second year of work in a 39-month effort. The NCHRP 12-70 project team plans to have results of their methodology development, including trial example applications, completed by early 2006. An interim report will be prepared for review by an NCHRP 12-70 technical oversight panel. Interim results will also be presented at a meeting of the AASHTO Highway Subcommittee in 2006.

Seismic Design Needs

Work that has been done either as part of the NCHRP 12-49 project or as part of the ongoing work on the NCHRP 12-70 project addresses important design needs for those involved in the geotechnical phases of the seismic design of bridges. The implementation of this or similar information into standard design methods is not, however, simple. Throughout the work on the NCHRP 12-49 project, there was a constant reminder that the level of analysis had to be minimized and that the ability of designers to adopt new methods was limited. This atmosphere placed severe restrictions on the progress that could be made.

Regardless of these implementation difficulties, there are a number of geotechnical issues associated with seismic design of bridge foundations that need to be carefully considered by geotechnical and bridge engineers during their work:

- The ability of simplified ground response analyses to adequately capture the response of the soil-foundation-bridge system. Good communications is required between the bridge engineer and the geotechnical engineer on expected performance from both a bridge and geotechnical engineering perspective. This communication cannot be achieved simply by reading a geotechnical report but must include person-to-person discussions with the report providing support and documentation for the resulting recommendations.

- Methods for evaluating the response of a pile-supported structure when liquefied soil occurs. While geotechnical engineers can make reasonable estimates of the occurrence of liquefaction, the effects of liquefaction on a soil-structure system are more difficult to understand. This is particularly the case at bridge abutments where liquefaction can lead to slope movement that in turn results in loading to a pile-support system which then provides restraint to the moving soil. A recent trend seems to be to use relatively sophisticated two-dimensional computer methods to model these conditions. However, many times simplified limit equilibrium methods can provide a more understandable result – though again the simple methods work best when there is good communication between the geotechnical engineer and the bridge engineer.

- Procedures used to estimate seismic earth pressures on retaining structures for certain wall types, soil conditions, and ground motion levels. The Mononobe Okabe earth pressure equations have served designers very well for past 40 (plus) years. But the limitations of these methods are not widely appreciated, often resulting in incorrect usage. More effort has to be made in the documentation of when the Mononobe Okabe approach is acceptable, when it is not, and what alternate methods exist. As performance-based design methods become the norm, alternate procedures will have to be developed and used.

- The value of more extensive field explorations, laboratory testing, and geotechnical evaluation. Seismic response in most areas will be determined by the details of the soil profile and the ability to adequately characterize this profile in conducting seismic response studies. This is particularly the case with liquefaction where thin layers can serve as sliding surfaces. Careful planning, conduct, and interpretation of field and laboratory work are essential.

- The need for performance data to validate approaches used in the geotechnical design of retaining walls and foundations for seismic loading. Recent centrifuge tests have provided some of this information. However, there is also a need for full-scale testing of bridge foundation systems, as well as performance evaluations following seismic events. These programs are often very expensive, but they are required to understand the likely success of current design methods during seismic events.

<u>Conclusions and Recommendations</u>

The NCHRP 12-49 and 12-70 projects summarized above represent important attempts to update the current version of the *AASHTO LRFD Bridge Design Specifications*. In the absence of an update there is considerable uncertainty on how to deal with certain geotechnical issues during seismic design. This has led some states, such as California and Washington, to develop their own state-specific seismic design guidelines. However, a number of other states must rely on the *AASHTO LRFD Bridge Design Specifications*. Unfortunately, some of the guidance in the current AASHTO Specifications is either outdated or is relatively limited in nature.

In recognition of the current limitations and the difficulties in achieving a consensus on the requirements for an update, it is important that some form of interim provisions be identified to allow geotechnical and bridge engineers to move forward on a common basis for evaluating the current alternatives for seismic design. This approach is similar to what was done with the current seismic provisions in the *AASHTO LRFD Bridge Design Specifications*, which started as ATC-6 *Seismic Design Guidelines for Highway Bridge* in 1981 (ATC, 1981) and served as an interim provision until being adopted by AASHTO in 1991. Information in the NCHRP 12-49 reports provides a good starting point for developing interim provisions.

Acknowledgements

Work on the NCHRP 12-49 and NCHRP 12-70 projects was provided through the National Academies of Sciences and funded by the AASHTO member states. Mr. David Beal served as the program officer for both projects. His guidance, as well as the support of AASHTO, was appreciated. Likewise, the recommendations from the two projects resulted from a collaborative effort of many project participants with advice from reviewers within FHWA, academia, and industry. The ideas created from these efforts have been very worthwhile and will contribute to the success of these projects and the improvement seismic design procedures.

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