DEVELOPMENT AND IMPLEMENTATION OF GEOTECHNICAL SEISMIC DESIGN POLICY AT WSDOT

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Abstract

In 2004, the Washington State Department of Transportation (WSDOT) began the development of a Geotechnical Design Manual (GDM) to consolidate its geotechnical design practice, and to provide direction to its consultants and design-build contractors. Seismic design is one of the least developed geotechnical design disciplines. While the development of new national (i.e., AASHTO) specifications for seismic design are underway, even once completed they will be incomplete regarding geotechnical design, as the main focus of those design specifications will be structural. This paper describes the development and scope of the geotechnical seismic design policy chapter of the WSDOT GDM. In addition, specific policy issues and their implementation for WSDOT design are discussed. Areas in seismic design practice that are inadequately defined and that need additional research are also identified.

Introduction

In 2004, the Washington State Department of Transportation (WSDOT) began the development of a Geotechnical Design Manual (GDM) to consolidate its geotechnical design practice, and to provide direction to its consultants and design-build contractors. Previous to the creation of this manual, WSDOT geotechnical design practice consisted of the AASHTO Bridge Design Specifications, the NAVFAC manual, national guidance documents published by the Federal Highway Administration, text books, and unpublished internal documents. While the pre-GDM system of manuals, text books, and guidance documents provided some degree of consistency in design practice among WSDOT staff, the consistency was limited, especially where conflicts existed between the various manuals and documents. Furthermore, the pre-GDM system was unwieldy for use in design-build contract documents for the purpose of defining the desired design policy. Furthermore, there was no home for deployment of new, but well accepted design procedures, derived from research.

Seismic design is one of the least developed geotechnical design disciplines in consideration of all aspects of geotechnical design. The seismic provisions of the AASHTO LRFD Bridge Design Specifications are based on 1983 seismic design technology, and additionally are focused on structural design. Furthermore, the geotechnical aspects of bridge foundation and wall seismic design in the AASHTO design specifications are incomplete. Some aspects of geotechnical seismic design are also not well developed technically, and need additional research to be fully defined. While the development of new national (i.e., AASHTO) specifications for seismic design

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are underway, even once completed they will be incomplete regarding geotechnical design, as the focus of those design specifications will be structural. In the interim, geotechnical design policies needed to be created to guide designers on how geotechnical seismic design issues should be handled, to insure that those issues are handled in a consistent manner. Therefore, the creation of a geotechnical seismic design policy document was especially important.

This paper describes the development and scope of the geotechnical seismic design policy chapter of the WSDOT GDM. In addition, specific policy issues and their implementation for WSDOT design and project development practice are discussed. Areas in seismic design practice that are inadequately defined and that need additional research are also identified.

Seismic Policy Development

As a first step in the development of a geotechnical seismic design policy for WSDOT, key information sources from which to build the design policy must be identified and compiled. The AASHTO LRFD Bridge Design Specifications (AASHTO 2004) provided the general framework and a starting point for the development of geotechnical seismic design policy. Other authoritative seismic design procedure and information sources used included International Code Council (2002), Kavazanjian, et al. (1997), Kramer (1996), ATC-MCEER Joint Venture (2001, 2002), the United States Geological Survey (USGS) website (http://eqhazmaps.usgs.gov/), as well as a number of technical papers.

The next step taken was to establish a framework that interfaces well with the needs of the structural engineers who will use geotechnical seismic design recommendations that are based on these design policies. The seismic design policy must be set up to provide the key geotechnical input parameters needed for seismic structural modeling. Therefore, the WSDOT geotechnical seismic design policy was set up to be compliant with and to support the AASHTO design procedures (AASHTO 2004).

Seismic hazard characterization and characterization of the response of the ground to the hazard can either be design specification based or can be determined analytically for the specific site. Therefore, the framework of the design policy must be set up to handle both avenues of design, and must provide guidance as to when a routine specification based design versus a more complex site specific analysis should be selected.

Since the AASHTO design procedures are by no means complete with regard to geotechnical seismic design, the other sources identified above were used to fill in the gaps. For example, site response factors to account for amplification of bedrock motion as it propagates through the soil column are combined with the “standard” structure response spectra in the AASHTO specifications. These standard response spectra are focused on bridges with a single degree of freedom and 5 percent damping and are not applicable to geotechnical seismic design for walls, liquefaction, and general slope stability. Therefore, ground motion amplification factors for typical soil columns were obtained from the literature (Stewart et al., 2003). In some cases, there is neither consensus nor guidance in the literature regarding design approach or criteria. An example of this is the maximum considered depth for liquefaction. It is in these areas that the seismic design policy provides crucial guidance that can be found nowhere else. Some of these gaps are technical in nature, and the gaps can be filled using engineering judgment, where as other gaps may require a political decision to fill them.

A summary of the subjects covered in the WSDOT GDM (WSDOT, 2005) are as follows:

- Responsibility of the geotechnical designer regarding seismic design
- Seismic performance objectives
- Liquefaction policies
- Governing design specifications
- Selection of an analysis approach (i.e., specification based design versus site specific analysis)
- Geotechnical site characterization requirements for seismic design
- Methods to obtain seismic design geotechnical properties (in-situ testing, laboratory testing, correlations with in-situ test results)
- Design specification/code based seismic hazard and site response
    - Determination of seismic hazard level
    - Determination of site response
    - Bedrock versus ground surface acceleration
    - Earthquake magnitude
- Input for structural design
    - Shallow and deep foundation springs
    - Seismic lateral earth pressures
    - Downdrag due to liquefaction
    - Lateral spread/slope failure loads on structure foundations and their mitigation
- Seismic geologic hazard characterization
    - Fault rupture
    - Liquefaction
    - Slope instability
    - Settlement

- Site specific seismic hazard and site response

The WSDOT GDM (WSDOT, 2005) can be viewed at the following website:

http://www.wsdot.wa.gov/fasc/EngineeringPublications/Manuals/2005GDM/GDM.htm

Regarding the technical subjects identified above, key gaps in the seismic design procedures that required policy level decisions to make them adequately complete are as follows:

- Seismic design objectives and performance criteria as applied to geotechnical issues
- Liquefaction assessment and maximum depth of liquefaction
- Determination of residual shear strength and stiffness of liquefied soils
- Prediction of lateral spreading or flow failure and the forces applied to foundation elements due to this hazard
- Combining seismic loading with reduced shear strength due to liquefaction
- Site specific seismic hazard and site response determination

These issues were identified as gaps because specification based design procedures to address these issues were not available, and no information/guidance was available in the literature, or if some guidance was available, the guidance was either not consistent or not practical.

Seismic Design Objectives and Performance Criteria

In US design practice (AASHTO 2004), the seismic design objective is to prevent collapse of structures during the design seismic event. The structure may still suffer major damage and may need to be replaced after the design seismic event, but loss of life or serious injury due to the damage to the structure is minimized. Current US design specifications (AASHTO 2004) do not provide much guidance on how to specifically apply this design objective to geotechnical seismic design. One of the key geotechnical issues that can cause structure collapse is the occurrence of liquefaction and associated geological hazards (e.g., slope failure, lateral spreading). For bridges, the greatest concern regarding liquefaction effects is a liquefaction induced abutment fill slope failure, or an intermediate pier foundation failure due to lateral spreading or flow failure induced by liquefaction, and in severe cases, foundation settlement or failure due to liquefaction induced downdrag. Similarly, liquefaction can cause retaining walls to collapse and slopes to fail. Common US practice is to protect bridge foundations and abutments from collapse due to liquefaction effects through ground modification or through designing the foundations to resist the potential loads. However, regarding protection of retaining walls and slopes from liquefaction effects, there is not a consistent practice nationwide. Therefore, the WSDOT GDM provides liquefaction policies regarding:

- Protection of bridge abutments

- Protection of bridge intermediate piers
- Protection of retaining walls and slopes

One of the key considerations to develop a policy on these issues is the cost of liquefaction mitigation. Liquefaction mitigation can add millions of dollars to the cost of a single structure. Since transportation agencies generally have limited funds to make bridges and other structures seismically safe, an executive level decision must be made at the program level as to how much liquefaction stabilization will be conducted. This may depend on the size and nature of the structure, and the nature of the facility the structure will support, and the potential to quickly reopen a facility should a collapse due to liquefaction occur. For WSDOT, this resulted in a policy to make sure that all bridge abutment and pier designs include liquefaction mitigation, if liquefaction mitigation is needed. The WSDOT policy also requires that the impact of retaining wall failure due to liquefaction be evaluated, and in general, retaining walls that directly support the traveled way, or walls adjacent to the traveled way and that are 10 ft in height or more, must be designed to resist failure due to liquefaction. Failure of smaller walls adjacent to the traveled roadway, or failure of walls that do not support the traveled roadway, were deemed to not present a severe enough risk to the traveling public to warrant the expense of liquefaction mitigation. Design of slopes not supporting structures and that do not have the potential to impact structures located downslope also do not require measures be taken to mitigate for instability caused by liquefaction. All of these limitations on where liquefaction mitigation is conducted are used to prioritize funding so that the most important and highest risk public safety needs are addressed.

Situations where a bridge needs to be widened for traffic capacity improvements present a special challenge to implementation of liquefaction mitigation policy. From an engineering standpoint, it makes little sense to fully stabilize the new portion of the bridge when the existing portion of the bridge could collapse due to liquefaction, possibly even dragging the new widening with it as it collapses. However, at least in some situations, it may be very difficult, if not impossible, to stabilize the existing portion of the bridge for liquefaction. Furthermore, determination of the presence of liquefiable soils and the effect that may have on the stability of the existing bridge may not be known at the time project budgets are set during the planning phase of a project. The need to stabilize both the widening and the existing bridge may present too much of a funding burden to accomplish the needed stabilization. Therefore, the policy regarding liquefaction mitigation for bridge widening situations was written to require mitigation for both the widening and the existing bridge, but that the mitigation for the existing bridge could be deferred by executive level management to a later time when adequate funding is available. This provision was made to allow WSDOT the flexibility it needed to use the available funds to provide the greatest benefit to the traveling public regarding safety of the transportation system.

Liquefaction Assessment

While the basics of liquefaction assessment are in general covered by the AASHTO LRFD Bridge Design Specifications (AASHTO 2004), many important details are missing and must be addressed through other published sources of information and even engineering judgment. For example, the minimum factor of safety required to consider the soil to not liquefy under the design seismic event (defined in Equation 1) is not specified in available design specifications.

$$FS_{liq} = CRR/CSR \tag{1}$$

where, CRR is the cyclic resistance ratio of a soil layer (i.e., the cyclic shear stress required to cause liquefaction), and CSR to the earthquake induced cyclic shear stress ratio. Guidance in the literature regarding how big FS_{liq} needs to be to be confident that liquefaction will not occur is not specific, and considerable judgment needs to be applied. Therefore, the specified minimum FS_{liq} of 1.2 in the WSDOT GDM had to be based on local practice and engineering judgment, yet had to be consistent with the range of FS_{liq} discussed in the literature (Kramer, 1996; ATC-MCEER Joint Venture, 2001).

Other details that had to be addressed in the WSDOT GDM that are not addressed in current design specifications include liquefaction of silts and gravels. The Modified Chinese Criteria (Finn, et al., 1994) that has been in use in the past has been found to be unconservative based on laboratory and field observations Bray and Sancio (2006). Therefore, the new criteria proposed by Bray and Sancio (2006) is recommended. At present, there is not enough information to develop a complete policy on liquefaction of gravels. Therefore, the WSDOT GDM only provides general guidance on this issue.

Maximum Depth of liquefaction

The maximum depth to which liquefaction can occur has been debated for many years by both researchers and practitioners. The problems that have contributed to this controversy include lack of verifiable evidence that liquefaction has occurred at depths of greater than 50 to 60 ft (15 to 18 m), the extrapolation of the Simplified Method (Seed and Idriss 1971) well beyond its empirical basis, and a lack of understanding regarding what limits the ability of the soil to liquefy at great depths (e.g., what role does overburden stress really play in limiting the ability of various soils to liquefy?) and accounting for that in theoretical design models. The impracticality and cost of mitigating for liquefaction at large depths must also be considered.

ATC-MCEER Joint Venture (2001) recommended that liquefaction be considered to depths of 80 ft (25 m), though they did not specifically state that liquefaction could not occur at greater depths. WSDOT adopted a similar philosophy regarding maximum depth of liquefaction. However, for critical WSDOT structures, if theoretical analyses indicate that liquefaction is likely at greater depths, a reduced soil shear strength below 80 ft (25 m), representing a partially liquefied condition, has been considered in design.

Residual Shear Strength and Stiffness of Liquefied Soils

Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice (Kramer, 1996). This problem also affects the ability to predict lateral stiffness of liquefied deposits, and the ability to predict downdrag forces on deep foundations due to liquefaction settlement.

A variety of methods are available to estimate the residual strength of liquefied soils; however, arguably the most widely accepted procedure, and the procedure recommended in the WSDOT GDM, is that proposed by Seed and Harder (1990). The Seed and Harder procedure for estimating the residual strength of a liquefied soil deposit is based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blow counts. This relationship is based on back-calculation of the apparent shear strengths from case histories of flow slides. However, there is a very wide range of possible residual shear strength values for a given SPT blow count in the Seed and Harder relationship, which can cause the cost of liquefaction mitigation to also vary widely. Hence, there is a need for better residual strength prediction accuracy.

Regarding the reduction of P-y soil strength and stiffness parameters to account for liquefaction, past design practice has been to treat fully liquefied soil as a soft clay, using residual strength parameters from Seed and Harder (1990), assuming the strain required to mobilize 50 percent of the residual strength to be equal to 0.02, or alternatively, the soil is treated as a very loose sand. However, recent research indicates that both the sand and the clay P-y models, which utilize a strain softening response, inaccurately model the response of liquefied soil to deep foundation lateral loading (Ashour and Norris, 1999 and 2003; Ashour, et al., 2002). Use of the soft clay model to simulate the as-liquefied soil may result in an overly stiff lateral load response, whereas the use of a very weak sand P-y model to simulate liquefied soil can result in a response to lateral load that is too soft and weak. Instead, this research indicates that a strain hardening response is more correct, with initial low stiffness, but at higher deformations, a rapidly increasing stiffness (Ashour and Norris, 1999 and 2003; Ashour, et al., 2002). Strain Wedge Theory uses a liquefied soil model that is consistent with the strain hardening response observed through recent research (Ashour and Norris, 1999 and 2003; Ashour, et al., 2002) and is the WSDOT preferred approach to predicting lateral deformation of deep foundations in liquefied soil. Regardless of the method selected, good engineering judgment will be necessary.

For downdrag loads, the liquefied shear strength directly affects the shear force applied to the sides of the foundation elements. Therefore, prediction of downdrag loads on foundations due to liquefaction is affected by the uncertainty in the prediction of residual strength. The AASHTO LRFD Bridge Design Specifications (AASHTO 2004) recommend the use of residual shear strength for predicting downdrag loads within the liquefied soil zone, and the static shear strength for layers above or within the liquefied soil zone that do not liquefy. The WSDOT GDM is consistent with that recommendation.

Another key issue related to the prediction of residual shear strength for liquefied soil is the prediction of lateral spreading and flow failure, and the forces applied to foundations caused by lateral spreading and flow failure. Uncertainty in the magnitude of the residual shear strength creates uncertainty in the prediction of whether or not lateral spreading or flow failure will occur. In addition, the forces that a liquefied soil mass can place on the foundation elements is also uncertain. For example, can the liquefied soil flow around the foundation or should the full passive soil forces be applied to the foundation? Furthermore, the force applied to the foundation depends on the ability of the foundation element to displace as well as on the amount of deformation the soil experiences, and is in effect a soil-structure interaction problem. Due to the complex nature of this phenomenon, only general guidance on this issue is provided in the WSDOT GDM. Both displacement based and force based methods are allowed in the WSDOT GDM. Force based approaches include both limit equilibrium (i.e., slope stability) methods and the Japanese Force Method (Finn and Fujita, 2004), both of which are recommended in the WSDOT GDM.

Related to this is the timing of peak ground motion, and the inertial forces that result, relative to the liquefaction of the soil, the development of residual strength, and the development of large ground movements due to slope instability or flow failure. Peak vibration response is likely to occur in advance of maximum ground displacement, and displacement induced maximum shear and moments will generally occur at deeper depths than those from inertial loading. For most cases, the WSDOT GDM recommends that the inertial seismic forces be decoupled from the reduction of the soil strength to its liquefied strength condition and the soil movement that results from this. However, for very long duration large magnitude earthquakes, some acceleration should be considered, and the peak ground vibration may overlap with the movement of the soil due to liquefaction. For example, for slope stability analysis, the WSDOT GDM recommends that the horizontal acceleration k_h be set equal to 33 percent of the peak ground acceleration. Alternatively, more advanced analytical techniques may be used to assess this overlap.

Site Specific Seismic Hazard and Site Response Determination

While available design specifications mention that site specific analysis techniques for determination of the seismic hazard level or the site response may be considered, little guidance is provided regarding acceptable approaches and techniques to accomplish this. Therefore, a design policy is required to define acceptable approaches and techniques to accomplish this. An appendix was added to the WSDOT GDM to accomplish this. The focus of this appendix is to provide some minimum standards, including source zones that should be considered (focused on the state of Washington), use of probabilistic versus deterministic analyses, recommended techniques to develop site specific response spectra, and development of the geotechnical input parameters needed for these analyses.

Gaps that Need to be Addressed through Future Research

As discussed in the previous section, a number of gaps in geotechnical seismic design exist. While for the state of Washington the GDM provides guidance to help fill some of those gaps, research is needed to fill the remaining gaps, or to provide improvement in the accuracy of the currently recommended design procedures. Therefore, for continued development of geotechnical seismic design, the following research areas should be addressed:

- Improved prediction of the range of soils that can liquefy (e.g., silts, gravels), and the depth to which soils can liquefy. How does overburden stress affect the ability of the soil to liquefy? Are depth effects affected by soil gradation? How is the prediction of the occurrence of liquefaction related to the effects of liquefaction in terms of ground deformation and soil properties?

- Improved estimation of residual strength and stiffness of liquefied soils, and its effect on stability of slopes, walls, and structure foundations. Prediction of residual strength and stiffness of soils is crude at best, and improved prediction accuracy can have large effects on mitigation costs for structure foundations and walls. Soil-structure interaction with regard to liquefied or partially liquefied soils, including design properties to be used, design models to predict the loads applied to foundations by liquefied soil, and the timing of the development of residual strength relative to the occurrence of strong shaking and the inertial forces that result need to be better defined. Well documented case histories where liquefaction induced flow slides or lateral spreading occurred are needed.
- Improved prediction of lateral earth pressures induced on walls due to seismic shaking. Current design specifications recommend the use of the Mononobe-Okabe method to estimate seismic earth pressure for wall design. While this method has been in use for many years, its limitations hinder accurate design of walls for seismic forces. Furthermore, for some combinations of seismic acceleration and surcharge conditions, this method is overly conservative to the point of being impractical to use.
- Application of reliability based load and resistance factor design (LRFD) to geotechnical seismic design. Procedures for seismic design currently available in design specifications and in the literature are not based on reliability concepts. For example, recent WSDOT-supported research at the University of Washington has shown that consistent application of conventional procedures for liquefaction evaluation provides inconsistent actual risks of the occurrence of liquefaction. Furthermore, statistical characterization of seismic loads, and the ability to combine loads with different recurrence intervals, is needed so that all aspects of geotechnical seismic design can be calibrated using reliability theory to a consistent level of safety.

Additionally, each time a major earthquake occurs an opportunity is provided to learn more about soil and structure response to ground shaking. To take full advantage of such opportunities, long-term monitoring programs to obtain site specific measurements of ground acceleration, loads applied to structures, and deformations should be developed. Furthermore, efforts to create a database such as the Geotechnical Virtual Data Center (GVDC) into which such data could be stored and presented in a way that everyone can share and use (see https://geodata.cosmos-data.org/index.asp) should be continued.

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