California High-Speed Train Project

TECHNICAL MEMORANDUM

Geotechnical Design Guidelines
TM 2.9.10

Prepared by: *Signed document on file*__________ ___30 Jun 10_
James Gingery, PE, GE Date
Geotechnical Engineer

Prepared by: *Signed document on file*__________ ___30 Jun 10_
Brian O'Neill, PE, GE Date
Program Geotechnical Engineer

Checked by: *Signed document on file*__________ ___30 Jun 10_
Bruce Hilton, PG, CEG Date
Program Engineering Geologist

Approved by: *Signed document on file*__________ ___1 Jul 10_
Ken Jong, PE, Engineering Manager Date

Released by: *Signed document on file*__________ ___9 Jul 10_
Anthony Daniels, Program Director Date

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for the California High-Speed Rail Authority

System Level Technical and Integration Reviews

The purpose of the review is to ensure:

- Technical consistency and appropriateness
- Check for integration issues and conflicts

System level reviews are required for all technical memorandums. Technical Leads for each subsystem is responsible for completing the reviews in a timely manner and identifying appropriate senior staff to perform the review. Exemption to the System Level technical and integration review by any Subsystem must be approved by the Engineering Manager.

System Level Technical Reviews by Subsystem:

Systems: NOT REQUIRED________________ ____________
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Infrastructure: *Signed document on file*__________ ___1 Jul 10___
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Operations: NOT REQUIRED________________ ____________
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Maintenance: NOT REQUIRED________________ ____________
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TABLE OF CONTENTS

ABSTRACT

This Technical Memorandum (TM) presents guidelines for geotechnical analysis and design criteria for high-speed train infrastructure facilities. The guidelines, standards, and requirements provided in this TM represent the minimum standard of practice and criteria for analysis and design.

This TM discusses geotechnical characterization and interpretation of subsurface conditions, and the development of engineering parameters for soil and rock materials that will be used for geotechnical analyses and design of features and structures including:

- Foundations for structures such as bridge and aerial viaducts
- Slopes
- Cuts
- Fills and Embankments
- Retaining Walls
- Excavation Bracing Systems
- Culverts
- Drainage and Subdrainage
- Dewatering

Earthquake engineering elements of geotechnical design and analyses are addressed. Limited guidance is provided on ground improvement required for detailed design. The analyses and design for these topics shall be performed following generally accepted geotechnical engineering principles and procedures adapted to the high-speed train project.

The information presented in this TM is based predominantly on documented, well-known methodologies and established reference publications that are considered applicable to the CHSTP design. Where available, existing guidelines are briefly summarized and referenced without duplicating their contents.

1.0 INTRODUCTION

The information presented in this Technical Memorandum is based predominantly on documented well-known methodologies and established reference publications. These references provide generally accepted (standardized) methods of Geotechnical analyses for engineering design purposes. However, the information in this Technical Memorandum extends and, in some cases, modifies these common Geotechnical analytical methods to include additional criteria.

Elements of Geotechnical analyses and design criteria subjected to these guidelines and standards may include; (1) data interpretation, (2) data analysis and modeling, and (3) Geotechnical design calculations. The analyses for these topics shall be performed following generally accepted Geotechnical engineering principles and procedures, as described herein. The values for key parameters and properties to be used in analyses and design shall be selected by Geotechnical staff with appropriate levels of expertise and who are intimately familiar with the types of soil and rock in the region, and intimately knowledgeable about the regional construction procedures that are required for the proper installation of earthworks and foundations in local soil and rock units. Soil properties such as stress-strain relationships and strengths depend on the design or evaluation situation; for example dynamic properties shall be used when considering seismic actions. As such, the response and properties of soil and rock materials used in Geotechnical assessments shall be based on properties, tests and analyses appropriate to the assessment conditions.

Geotechnical analysis shall be consistent with the “performance-based” for engineering design of structures and features/facilities. This TM includes introduction of the geotechnical design basis for performance-based design, including design flow, design life, and varying levels of required performance criteria for the project. Description of the performance requirements are presented in various structural engineering and geotechnical/seismic TMs. The CHSTP makes use of the Load and Resistance Factor Design (LRFD) methodology for engineering design approach in both geotechnical analysis and structural engineering.

1.1 PURPOSE OF TECHNICAL MEMORANDUM

The purpose of this Technical Memorandum for Geotechnical criteria is to provide guidance for the design process, including methodology, analytical procedures, and assumptions; and to establish acceptable standards in terms of expected performance of infrastructure facilities and/or integrity of the final design.

1.1 1.2 STATEMENT OF TECHNICAL ISSUE

This Technical Memorandum presents guidelines for geotechnical analysis and design criteria for high-speed train infrastructure facilities. The guidelines, standards, and requirements provided in this TM represent the minimum standard of practice and criteria for analysis and design. Earthquake engineering elements of geotechnical design and analyses are also addressed. Limited guidance is provided on ground improvement required for detailed design.

1.2 1.3 GENERAL

There is no practical way to cover all the intricate aspects of Geotechnical engineering analyses and design criteria for the project in one guidance document. Even though the material presented generally represents the current state-of-the-practice in California, engineering judgment based on local conditions and knowledge must also be applied. This is true of most engineering disciplines and it is especially true in the area of Geotechnical engineering. It is important that the Geotechnical analyses work and reports that will in turn be used for design and construction of infrastructure facilities be performed by qualified Geotechnical staff with appropriate levels of licensure and expertise in transportation projects in the State of California. This Technical Memorandum has been prepared assuming that the ‘users’ have the appropriate Geotechnical qualifications and experience as deemed required under licensure and registration by the State of California’s Board for Professional Engineers and including Geologists and Geophysicists, under the Department of Consumer Affairs.

In order to provide a consistent and dependable design, Geotechnical practitioners responsible for analyses for the project use state of the practice methodologies, procedures, and terminology in a somewhat standardized manner to maintain consistency in Geotechnical analyses and reporting practices across the entire project. This consistency will also facilitate interface and sharing among technical/designers throughout the design and construction stages of the project. Designers are advised that early submittal of initial Geotechnical information and preliminary recommendations or engineering evaluation of preliminary data may be necessary to establish basic design concepts. This is commonly the case on large projects or projects containing complex or difficult Geotechnical problems where alignment and/or grade adjustments maybe appropriate based on Geotechnical recommendations regarding major site or subsurface constraints.

Each design team will be responsible for performing and documenting an internal and independent peer review of all deliverables.

1.2.1 1.3.1 Definition of Terms

The following technical terms and acronyms used in this document have specific connotations with regard to California High-Speed Train system.

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AREMA	American Railway Engineering and Maintenance of Way Association
ASTM	American Society for Testing and Materials
Caltrans	California Department of Transportation
CEG	Certified Engineering Geologist
CFR	Code of Federal Regulations
CGS	California Geological Survey
Authority	California High-Speed Rail Authority
CHST	California High-Speed Train
CHSTP	California High-Speed Train Project
CPT	Cone Penetrometer Test
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
GBR	Geotechnical Baseline Report
GDR	Geotechnical Data Report
GE	California-registered Geotechnical Engineer
ISRM	International Society for Rock Mechanics
LOTB	Logs of Test Borings
LRFD	Load and Resistance Factor Design method
MPH/mph	Miles per hour
NHI	National Highway Institute
RTRI	Railway Technical Research Institute (Japan)
SPT	Standard Penetration Test
TM	Technical Memorandum
UIC	International Union of Railways
USCS	United Soil Classification System
USGS	United States Geological Survey

1.2.2 1.3.2 Units

The California High-Speed Train Project is based on U.S. Customary Units consistent with guidelines prepared by the California Department of Transportation and defined by the National Institute of Standards and Technology (NIST). U.S. Customary Units are officially used in the United States, and are also known in the US as “English” or “Imperial” units. In order to avoid confusion, all formal references to units of measure should be made in terms of U.S. Customary Units.

1.3 LAWS AND CODES

Initial high-speed train (HST) design criteria will be issued in technical memoranda that provide guidance and procedures to advance the preliminary engineering. When completed, a Design Manual will present design standards and criteria specifically for the design, construction and operation of the CHSTP's high-speed railway.

Criteria for design elements not specific to HST operations will be governed by existing applicable standards, laws and codes. Applicable local building, planning and zoning codes and laws are to be reviewed for the stations, particularly those located within multiple municipal jurisdictions, state rights-of-way, and/or unincorporated jurisdictions.

In the case of differing values, the standard followed shall be that which results in the satisfaction of all applicable requirements. In the case of conflicts, documentation for the conflicting standard is to be prepared and approval is to be secured as required by the affected agency for which an exception is required, whether it be an exception to the CHSTP standards or another agency standards.

2.0 DEFINITION OF TECHNICAL TOPIC

2.1 GENERAL

This Technical Memorandum presents guidelines for geotechnical analysis and design criteria for high-speed train infrastructure facilities. The information presented in this TM is based predominantly on documented, well-known methodologies and established reference publications that are considered applicable to the CHSTP design. Where available, existing guidelines are briefly summarized and referenced without duplicating their contents.

3.0 ASSESSMENT AND ANALYSIS

3.1 GENERAL

Geotechnical criteria is intended to provide guidance for the design, methodology, assumptions and analytical procedures, and to establish acceptable standards in terms of expected performance of infrastructure facilities and integrity of the final design. For structures built on, in or with earthen materials of soil and rocks, the geotechnical engineer needs to know the engineering properties of these materials, in the same way as the designer acquires properties for other man-made materials such as steel and concrete. Due to their non-uniform nature, soils and rocks exhibit more complicated engineering properties as compared to that of steel and concrete.

The engineering team, including geotechnical, civil, and structural disciplines shall identify design and constructability requirements (e.g., support loads from bridge superstructure, and foundation deformation tolerances) and their effect on the geotechnical information and parameters needed. Subsequently, the geotechnical engineering analyses to be performed (e.g., bearing capacity or settlement or global stability) shall be identified, so that engineering parameters and properties required for these analyses can be determined. The values selected for the parameters should be appropriate to the particular performance requirement, including consideration of limit state(s) and its/their correspondent calculation model under consideration. There should be continuous communication between the structural and geotechnical engineers as design issues evolve and change.

Subject to the restrictions imposed by licensing laws in the state of California, recommendations for design parameters shall be made under the responsible charge of a California-licensed geotechnical engineers. Geologic hazards and interpretations will be performed under the responsible charge of professional geologists and certified engineering geologists. Soil mechanics, rock mechanics, and geological concepts must be combined with knowledge of geotechnical engineering or hydrogeology to make a complete application of the soil, rock, and groundwater investigation.

3.1.1 Data Evaluation and Geotechnical Analysis

This TM topic includes Geotechnical characterization and interpretation of subsurface conditions, and the development of engineering parameters for soil and rock materials. Guidance on Geotechnical analysis and design is provided for a variety of structures including: foundations for bridge and aerial viaducts; slopes, cuts, fills and embankments, retaining walls, earthquake engineering, and drainage, subdrainage, infiltration and dewatering. The analyses for these topics shall be performed following generally accepted Geotechnical engineering principles and procedures adapted to CHSTP, as described herein.

Elements of geotechnical analyses and design criteria subjected to these guidelines and standards shall include; (1) data interpretation, (2) data analysis and modeling, and (3) geotechnical design calculations. The analyses for these topics shall be performed following generally accepted geotechnical engineering principles and procedures and any project-specific methods or criteria contained herein. Soil properties such as stress-strain relationships and strengths depend on the design or evaluation scenario such as the dynamic properties shall be used when considering seismic loading. As such, the response and properties of soil and rock materials used in geotechnical assessments shall be based on properties, tests and analyses appropriate to the assessment conditions. The engineering analysis of “natural materials” (soils, rock, and groundwater) is typically more complex than the analysis of other construction materials because soil/rock is not a continuum. Therefore, soil and rock typically do not strictly meet the assumptions of the theories of solid mechanics and strength of materials. The engineering properties of these natural materials can vary over time and space so that their physical properties cannot be assessed at all locations for all conditions. In addition, since each piece of civil or structural infrastructure presents a unique set of design challenges, the designers must determine the appropriate method(s) and level of refinement necessary to analyze and design each structure or portion of civil works. As such, the methods and procedures for geotechnical data evaluation and analysis for design of infrastructure facilities described herein are intended to act as a 'baseline' for the designers.

Geotechnical engineers must exercise judgment in the application of these criteria and, where appropriate, the use of other established industry standards and procedures.

3.1.2 Incorporation of Geohazard Study Results

The Geologic and Seismic Hazard Evaluation Guidelines Technical Memorandum (TM) 2.9.3 document provides guidelines for identifying and evaluating these hazards for input to project design criteria. The designer shall incorporate the findings of the geologic and seismic hazard studies into the geotechnical design documents and address how they have been utilized and/or mitigated. Geologic and seismic hazard evaluation reports shall be prepared in advance of other geotechnical reports in order to provide a geologic framework for future geotechnical studies.

The geologic and seismic hazards need to be identified and evaluated to assess their potential impact on the design, construction, and operation of the high-speed train project. In some instances, these hazards will have significant impact on the design, construction, and/or operation of the CHSTP and therefore will require mitigation measures that may be achieved through avoidance and/or design modifications. It is necessary for the designers to be informed of these hazards at an early stage in the design process to ensure that the CHSTP can be designed, constructed and operated to meet the defined performance requirements and objectives.

For consistency with the ground motion analyses, the results of geologic and seismic hazard evaluations shall be provided to the geotechnical engineer and seismic design engineer for their evaluation at a quantitative level as input to the geotechnical investigation and analysis progresses. In addition, the preparation of geotechnical reports shall utilize the information contained in these geologic and seismic hazard evaluations from a qualitative standpoint and shall address how the hazards have been both quantified and determined to be inconsequential to the high-speed train performance, or the method of project mitigations employed. The geotechnical engineer will evaluate each of the identified geologic or seismic hazards to determine whether they are within the tolerance of the CHSTP components. If these hazards are found to exceed project tolerances, subsequent and more detailed analysis is warranted and shall be performed by the responsible geologist and project geotechnical engineer. This will ensure that geotechnical investigations and analyses performed under separate guidance are consistent with characterized geologic conditions and hazards.

3.1.3 Geotechnical Reports

Preparation of geotechnical reports are required to address both design-related issues (basis for design) and construction issues. The primary purpose of preparing geotechnical reports is to establish single source documents that provide design-level information and recommendations as well as describe the geotechnical conditions anticipated (or to be assumed) to be encountered during subsurface construction.

The requirements for the content and format of Geotechnical Reports described in TM 2.9.2 shall be used by the designer for all geotechnical design documents.

3.1.4 Basis of Guidelines, and Geotechnical Standards and References

The information presented in this Technical Memorandum is based predominantly on documented well-known methodologies and other established reference publications that are considered applicable to the CHSTP design. The geotechnical guidelines referenced include publications issued by AASHTO, FHWA, ASTM, UIC, RTRI, and State of California - Caltrans and CBC code. These references provide generally accepted (standardized) methods of Geotechnical analyses for engineering design purposes. Where available, existing guidelines are briefly summarized and referenced without duplicating their contents. In other instances, such guidelines do not exist. Hence, the information in this TM extends and, in some cases, modifies these common Geotechnical analytical methods to include additional criteria and unique guidelines for CHSTP.

The development or selection of geotechnical analyses methodologies and design criteria requirements was based on a review and assessment of available information and "best practices", including (but not limited to) the references listed in Section 5. Additionally, local building, planning and zoning standards or codes must be met. In the case of differing values, or conflicts in the various requirements for design, conflicts between any of them, or following design guidelines, the standard followed shall be that which results in the highest level of satisfaction for all requirements or that is deemed as the most appropriate by the California High-Speed Rail Authority (Authority). The standard shall be followed as required for securing regulatory approval.

3.2 GEOTECHNICAL CHARACTERIZATION

Characterization of surface and subsurface conditions shall be performed in three dimensions based on plans and profiles depicting subsurface units with unique properties and the associated geotechnical engineering properties. This geotechnical model shall then be refined into a surface/subsurface engineering domain model based on the unique design elements. The following sections describe the guidelines for the development of the engineering model to promote consistency and to meet project-specific requirements.

These guidelines are intended for use by the geotechnical engineers in developing consistent, quality geo-characterization models for the CHST. While references are made to existing guidelines that are an integral part of this guideline, no attempt is made to duplicate or reiterate these other guidelines. In that regard, there are three guidance documents that are fundamental to the development and presentation of this geo-characterization process, including:

- Geotechnical Investigation and Laboratory Testing Guidelines, TM 2.9.1,
- Geotechnical Report Preparation Guidelines, TM 2.9.2, and
- Geologic and Seismic Hazard Evaluations Guidelines, TM 2.9.3

Recommendations for subsurface exploration methods, in-situ testing, and laboratory testing of specimen samples as part of geotechnical investigations will be provided on the basis of these guidelines. In addition to discussion of soil and rock identification, testing, description, and classification, this technical memorandum contains guidelines that present the process and protocol for interpretation of subsurface conditions for use during geotechnical analyses supporting engineering design activities for CHSTP.

Soil shall be characterized and classified using ASTM 2488 guidelines for field classification and ASTM 2487 based on laboratory test results. Rock should be classified using FHWA GEC 3 (FHWA, 2002) guidelines which are largely based on ISRM guidelines. Rock and other formational materials, e.g., very soft rock and intermediate geotechnical materials should also be identified with the name of the geologic formation.

3.2.1 Laboratory Test Requirements and Reports

Standards to be used for laboratory testing of soil and rock for CHSTP are described in TM 2.9.1, Geotechnical Investigation Guidelines.

3.2.2 Development of Geo-Characterization Model

3.2.2.1 Objectives

This section identifies appropriate methods and technical references to be used for soil and rock property assessment, and how to use the soil and rock property data to establish the final soil and rock parameters to be used for geotechnical design.

3.2.2.2 Preliminary Geologic Model

The geologist shall develop a geologic model based on applicable existing data such as geologic maps, aerial photography, published literature, and existing subsurface data. The model shall be refined using field reconnaissance, remote sensing, and mapping methods. The geologic model shall be used to prepare a surface geologic map and a corresponding subsurface profile along the CHST alignment. The map and profile shall be accompanied by cross-sections perpendicular to the alignment where needed to reveal the three dimensional configuration of the subsurface conditions. Maps, profiles, and cross-sections shall also depict the related design elements (structures, embankments, cuts, etc.) of the CHST project. The geologic model shall serve as a fundamental tool to develop the subsurface exploration plan for the CHST, and shall be updated as project-specific information is obtained. Subsurface conditions shall be presented in plan and profile and also accompanied by cross-sections perpendicular to the alignment where needed to fully depict the three dimensional configuration of these units. Subsurface logs, in-situ test results, and laboratory testing shall be used for further refinement of units and groundwater conditions having unique engineering properties as they relate to geotechnical analyses. Units having similar engineering properties but unique geologic description shall only be differentiated if it is beneficial to the interpretation of stratigraphy between data points.

The geotechnical staff should realize that while there may be potential limitations in the use of historical borings, it is necessary to review these borings relative to the CHST design element under consideration. As an example, a historical boring may indicate a thick layer of very soft clay as evidenced by the description "weight of rod/weight of hammer" in the SPT recording box of the log at a large number of test depths. While shear strength and consolidation properties cannot be reliably estimated based on SPT blow count values, the historical boring may provide useful information concerning the depth to a firm stratum. Also, it is likely that different drill rigs with different operators and different energy efficiencies were used in the collection of SPT data on historical boring logs. This factor must also be recognized when an attempt is made to correlate engineering properties to SPT blow count values.

Uncertainties in the development of a subsurface exploration usually indicate the need for additional explorations or testing. Because of the diverse nature of the geologic processes that contribute to soil formation, actual subsurface profiles can be extremely varied both vertically and horizontally, and can differ significantly from interpreted profiles developed from boring logs. Therefore, subsurface profiles developed from boring logs should contain some indication that the delineation between strata do not necessarily suggest that distinct boundaries exist between the strata or that the interpolations of strata thickness between borings are necessarily correct. The main purpose of subsurface profiles is to provide a starting point for design and not necessarily to present an accurate description of subsurface conditions.

3.2.2.3 Geotechnical Model

The geotechnical engineer shall develop a geotechnical model based on the geologic model and subsurface information collected for the CHST project. As field and laboratory test data become available, engineering properties for each of the unique units shall be developed and portrayed on the geotechnical model (map, profile, and cross-sections). These engineering properties must effectively document and support all geotechnical analyses and designs for the CHST project.

The geotechnical model shall represent the geologist and geotechnical engineer's interpretation of all available subsurface data, and shall include (at a minimum) the following:

- Interpreted boundaries of soil and rock,
- Average physical properties of the soil layers (unit weight, shear strength, etc.),
- Visual description of each layer including USCS symbols for soil classification,
- Location of the ground water (see next section), and
- Notations for special items (boulders, artesian pressure, known buried infrastructure, etc.)

Complementary tables shall be developed to accompany the geotechnical model (map, profile, and cross sections), in order to reduce visual clutter and aid the user. As described in TM 2.9.1, CHSTP will make use of electronic records for borings, CPTs, etc. An appropriately developed database and GIS shall be used to great advantage for data management, analyses (in support of engineering design), and construction. In addition to the previously mentioned advantages of having electronic data records compliment paper logs, it is possible to:

1. Catalog borings that were conducted previously,
2. Inventory data regarding specific problematic formations along the HST corridor, and
3. Develop cross-sections that depict subsurface conditions along the CHSTP segments or within a region.

3.2.2.4 Groundwater Conditions

The geologist and geotechnical engineer shall evaluate groundwater conditions and establish water levels/elevations for use in facility design and construction planning. Guidance pertaining to collecting and interpreting hydrogeologic field data is contained in TM 2.9.1. Important factors that shall be considered in groundwater characterization include:

- Hydrostatic or flowing groundwater conditions,
- Whether aquifers are confined or unconfined,
- The upper and lower limits and slope of the aquifer,
- Aquifer characteristics (soil type and permeability, rock discontinuities),
- Presence (and influence) of perched groundwater table conditions,
- Potential for raised or lowered groundwater level during project design-life, and
- Possibility for artesian conditions.

Due to the variability in aquifer storage characteristics and response to rainfall, the groundwater conditions to be used for analysis and geotechnical design shall be based on water levels measured in the field, coupled with hydrograph information describing historic water level trends. For sites where there is no groundwater data available, the “wetting band” approach (FHWA, 2005) should be used to provide an estimate of reasonable groundwater level.

Groundwater conditions are especially relevant for slope design. The water level of a specified return period shall be determined using one of the following approaches:

1. Analysis of piezometric data taken before, during and after rainfall. Various methods are available for estimating water levels from piezometric records, including the statistical correlation of groundwater response with rainfall, groundwater modeling of the aquifer system, and the extrapolation of observed piezometric responses.
2. Solution of the equation describing the formation of a wetting band zone of 100 percent saturation (FHWA, 2005). The geologist and geotechnical engineer shall consider all relevant hydrogeologic aspects for the slope stability analyses, especially:
    - The highest anticipated phreatic (groundwater) surface for an unconfined aquifer and/or piezometric surface for a confined aquifer,
    - The height of the groundwater at the time of failure (for an existing failure),
    - The proximity of the aquifer to the existing or potential failure surface, and
    - The presence and influence of seepage, pore pressure conditions, tension cracks, runoff, and surface drainage patterns.

For purposes of developing additional criteria for use in design and construction of CHSTP facilities, further reference information regarding assessment (and influence) of groundwater conditions and geotechnical and hydrogeologic considerations is contained in the reference documents listed in TM 2.9.1, (and FHWA slope manual 2005). This includes assessment of hydrostatic pressure, positive or negative pore water pressure, flow and seepage, total stress and effective stress, hydraulic gradient and ‘piping’, soil permeability, and impact of sudden drawdown (rapid lowering in the level of groundwater).

3.2.3 Soil and Rock Properties and Parameters

3.2.3.1 Ground Engineering - Properties and Parameters

For structures built on, in or with earthen materials of soil and rocks, the geotechnical engineer needs to know the engineering properties of these materials, in the same way as the designer acquires properties for other man-made materials such as steel and concrete. Due to their non-uniform nature, soils and rocks exhibit more complicated engineering properties as compared to that of steel and concrete. The two most important engineering properties are strength and deformation characteristics.

The detailed measurement and interpretation of soil and rock properties shall be consistent with the guidelines provided in FHWA-IF-02-034, Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5 (GEC5) (FHWA, 2002), except as specifically indicated herein. The process for soil and rock property selection is illustrated graphically in flow-chart format in Figure no. 1 of GEC5, Chapter 2. The GEC5 reference document also provides a summary of geotechnical data needs and testing considerations for various geotechnical applications. Additional information is presented in Section no. 10 (Foundations) of AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Regarding Standard Penetration Test (SPT) tests, the blow-count N-values obtained are dependent on the equipment used and the skill of the operator, and shall be corrected for field procedures to standard N60 values (an efficiency of 60 percent is typical for traditional rope and cathead systems). This blow-count correction is necessary because many of the correlations developed to evaluate soil properties are based on N60-values. In addition, blow-count corrections shall be performed for evaluation of liquefaction triggering. Requirements for these additional corrections are presented in Section 6.10.8 of this TM.

Laboratory testing of soils is grouped broadly into two general classes, including ‘classification’ tests and ‘performance’ (quantitative) tests for estimation of shear strength, compressibility, permeability, etc. Laboratory index property testing is mainly used to classify soils, though in some cases, they can also be used with correlations to estimate specific soil design properties. Classification (index type) tests include soil gradation and plasticity indices, and may be performed on either disturbed or undisturbed samples. Performance type tests to evaluate strength, compressibility, permeability etc. of existing subsurface deposits must be conducted on undisturbed specimens, and the less disturbance the better. See GEC5 for additional requirements regarding these and other types of laboratory performance tests that should be followed.

For soil, shear strength may be evaluated based on either undisturbed specimens of finer grained soil (undisturbed specimens of granular soils are very difficult, if not impossible, to get), or disturbed or remolded specimens of fine or coarse grained soil. There are a variety of shear strength tests that can be conducted, and the specific type of test selected depends on the specific application. See GEC5 for specific guidance on the types of shear strength tests needed for various applications, as well as the sections in this CHSTP TM that cover specific geotechnical design topics.

For rock, the focus is typically on the shear strength of the intact rock, or on the shear strength of discontinuities (i.e., joint/seam) within the rock mass. Rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. Laboratory testing of rock is used primarily for classification of intact rock samples, and if performed properly, serves as useful function in this regard.

With regard to the ‘quality’ of soil and rock laboratory data, if based on review of the data the interpreted laboratory test results are not consistent with expectations, or if results indicate that the sample was disturbed, it will be necessary to make adjustments. Laboratory results at the early stage (preliminary design phase) of CHSTP can be used to plan and initiate a more detailed and focused phase of investigation for use during final design. A phased investigation approach is particularly helpful in cases where there are many unknowns regarding the subsurface conditions prior to conducting the proposed site investigation program.

Correlations for soil properties as provided in GEC5 may be used if the correlation is well established and if the accuracy of the correlation is considered regarding its influence if the estimate obtained from the correlation in the selection of the property value used for design. Local geologic formation-specific correlations may also be used if well established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question. Correlations shall not be used as a substitute for an adequate subsurface investigation program, but rather to complement and verify specific project-related information.

3.2.4 Rock Properties

With regard to the engineering properties of rock, geotechnical evaluations for design shall consider that these properties are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock shall account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations shall be employed to evaluate the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

Rock properties are divided into two categories: intact rock properties and rock mass properties. Intact rock properties are evaluated from laboratory tests on small samples typically obtained from coring, outcrops or exposures along existing cuts. Engineering properties typically obtained from laboratory tests include specific gravity, unit weight, ultrasonic velocity, compressive strength, tensile strength, and shear strength. Rock mass properties are evaluated by visual examination of discontinuities within the rock mass, and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction.

The methodology and related considerations provided by GEC5 shall be used to assess the design properties for the intact rock and the rock mass as a whole. However, the portion of GEC5 that addresses the evaluation of fractured rock mass shear strength parameters (Hoek and Brown, 1988) is outdated. The original work by Hoek and Brown has been updated and is described in Hoek, et. al. (2002). Therefore the Hoek, et. al. (2002) method shall be used for fractured rock mass shear strength evaluation. This method is only to be used for highly fractured rock masses in which the stability of the rock slope is not structurally controlled.

3.2.4.1 Geotechnical Engineering Parameters

The geotechnical engineer shall evaluate the validity and reliability of the data and its usefulness in selecting final design parameters. After a review of data reliability, a review of the variability of the selected parameters shall be carried out. Variability is typically introduced in two ways: 1) natural heterogeneity within the unit, and 2) test method selection or execution.

Inconsistencies in data shall be evaluated and the need for mitigation procedures may be warranted to correct or exclude any questionable data. The geotechnical engineer shall comply with GEC 5, which provides guidance for analyzing data and resolving inconsistencies. The geotechnical engineer shall also use GEC 5 to assess variability for a given engineering property in a particular geologic unit, and how that variability should influence the selection of the final design values.

Evaluations of geotechnical engineering parameters shall consider how the parameters could change over the design life of the structure. Changes may occur as a result of weathering, groundwater level changes, increase in stress due to fill or foundation loads, decrease in stress due to excavation, or other factors.

Geotechnical evaluations for design shall keep in mind that resistance factors have been developed assuming mean values for soil properties. However, design values that are more conservative than the mean may still be appropriate, especially if there is an unusually level of uncertainty associated with the design property. Depending on the availability and variability of ground conditions, it may not be possible to reliably estimate an average value for design. In this case, the geotechnical engineer shall select a more conservative value. For those resistance factors that were evaluated based on calibration by “fitting” to allowable stress design, property selection shall be based on the considerations discussed previously.

3.3 AERIAL TRACKWAY STRUCTURES AND BRIDGE FOUNDATIONS

3.3.1 Design Process

This section describes geotechnical engineering and design requirements for aerial structure and bridge foundations. The following sections discuss data required for foundation design, foundation type selection, loading evaluations, tolerable settlements and displacements, limit states to consider, load and resistance factors, and analysis procedures for foundations and abutments.

3.3.2 Data Requirement for Foundation Design

Geotechnical, geologic and seismic hazard data shall characterize the subsurface geologic and geotechnical conditions adequately so that foundation analysis, design and constructability can be evaluated. Guidelines on geologic and geotechnical site characterization are provided in Section 6.2.

Structure type and loads shall be in accordance with TM 2.3.2, Structure Design Loads.

3.3.3 Foundation Design and Construction Considerations

The design shall indicate the proposed structure type and function and proposed locations of foundation elements, including foundation loads. Structure type and loads shall comply with TM 2.3.2. Special performance requirements, such as unique or unusual displacement limitations, shall be considered in the design. Geotechnical site characterization shall be adequately advanced to support the design, and geologic and seismic hazards that affect the proposed structures shall have been identified.

Construction limitations that could affect foundation design shall be identified. These include local availability of equipment, equipment access limitations, staging restrictions, right-of-way restrictions, permit requirements, proximity to sensitive structures, and proximity to sensitive utilities.

3.3.4 Foundation Type Selection

Foundation selection shall consider the following:

- The ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states, given the soil or rock conditions encountered,
- Consideration of flooding and scour, where applicable,
- Consideration of frost depth, where applicable,
- The constructability of the foundation type,
- The impact of the foundation installation (in terms of time and space required) on existing facilities and right-of-way,
- The environmental impact of the foundation construction,
- Physical constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities), and
- The impact of the foundation on the performance of adjacent foundations, structures, or utilities, considering both the design of the adjacent foundations, structures, or utilities, and the performance impact the installation of the new foundation will have on these adjacent facilities; and the cost of the foundation, considering all of the issues listed above

Shallow spread footings shall be used for foundation support where competent soil or rock is present within relatively shallow depths. Shallow footings may also be appropriate where ground improvement is performed to poor soils to improve their strength and stiffness characteristics, provided that performance requirements are met. Shallow footings are typically not appropriate for soils that are soft, loose, expansive, prone to hydro-collapse, liquefiable, or prone to excessive scour.

Where spread footings are not feasible or cost effective, deep foundations shall be used. Two general types of deep foundations are typically considered: pile foundations, and drilled shaft (or cast-in-drilled-hole, CIDH) foundations. Shaft foundations can be advantageous where pile driving may be precluded by the presence of obstructions such as dense layers, boulders, or fill with debris. Shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam or shoring is required to construct the pile cap. Shafts may not be desirable where contaminated soils are present, because of the associated handling and disposal requirements. Shafts shall be considered in lieu of piles where pile driving vibrations could cause damage or unacceptable disturbance or disruption to existing adjacent facilities. Piles may be more cost effective than shafts where pile cap construction is relatively easy, or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The stability of soils during shaft construction and the need for casing shall also be considered when choosing between driven piles and drilled shafts.

3.3.5 LRFD Overview for Foundations

The CHSTP utilizes the Load and Resistance Factor Design (LRFD) method, which is based on statistical reliability. In the LRFD methodology, loads and resistances (that is, a capacity to resist load such as foundation bearing capacity) are factored (weighted) in order to achieve a desired level of reliability. In this case, reliability can be defined as a probability of a load not exceeding the capacity for the entire design life of the foundation.

The basic equation for LRFD states that the loads multiplied by factors to account for uncertainty, ductility, importance, and redundancy must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the resistance per the AASHTO LRFD Bridge Design Specifications. Foundations must be designed to satisfy the LRFD limit state equation:

$$\Sigma\ \eta_i\ \gamma_i\ Q_i \leq \Phi R_n = R_r$$

Where:

γ_i = load factor applied to force effects
Φ = resistance factor applied to minimal resistance
η_i = load modifier relating to ductility, redundancy and importance (>1.05).
Q_i = force effect
R_n = nominal resistance
R_r = factored resistance, $\Phi\ R_n$

Except where noted herein, foundation design shall be performed in accordance with the AASHTO LRFD Bridge Design Specifications with California (Caltrans) Amendments, Customary U.S. Units, latest edition, as adapted and modified by this and other Technical Memoranda. Three general Limit States are considered for foundation design in the AASHTO LRFD methodology:

- Strength Limit State – Evaluation of strength under various loading conditions,
- Extreme Event Limit State – Evaluation of strength and performance under extreme loading conditions that result from rare events such as earthquakes, collision, and extreme storms, and
- Service Limit State – Evaluation of performance (i.e., settlements and displacements) under normal service loads

3.3.6 LRFD Loads, Load Groups and Limit States

LRFD loads, load groups and limit states for aerial viaduct and bridge structure design are defined in TM 2.3.2. Earth loads are listed below and shall be calculated by the geotechnical engineer in accordance with Section 3.11 of AASHTO LRFD.

Table 3.3.6-1 Summary of Earth Loads

CHST Load Abbreviation	AASHTO LRFD Load Abbreviation and (Section)	Load Type Description
EV	EV (3.5.1)	Vertical earth pressure from dead load of fill
EHAR	EH (3.11.5.2)	Horizontal earth pressure load for at-rest condition
EHAC	EH (3.11.5.3)	Horizontal earth pressure load for active condition
ESET	DD (3.11.8)	Earth settlement effects
EHS	ES (3.11.6.2 and 3.11.6.3)	Earth surcharge due to live loads

Service, Strength, Buoyancy, and Extreme Event Limit States used for design of foundation for aerial viaduct and bridge structures shall be as defined in TM 2.3.2 Structure Design Loads.

At a minimum, foundation shall be designed and proportioned for the following Limit States and mechanisms:

<u>Service Limit State:</u>

- Settlement,
- Lateral deflection,
- Overall stability (including slope stability), and
- Scour at the design flood

<u>Strength Limit State:</u>

Spread Footings:

- Nominal bearing resistance,
- Overturning or excess loss of contact,
- Sliding at the base of the footing, and
- Constructability

Driven Piles:

- Axial compression resistance for single piles,
- Pile group compression resistance,
- Uplift resistance for single piles,
- Uplift resistance of pile groups,
- Pile punching failure into a weaker stratum below the bearing stratum (where applicable),
- Single pile and pile group lateral resistance, and
- Constructability (including pile drivability)

Drilled Shafts:

- Axial compression resistance for single drilled shafts,
- Shaft group compression resistance,
- Uplift resistance for single shafts,

- Uplift resistance of shaft groups,
- Single shaft and shaft group lateral resistance,
- Shaft punching failure into a weaker stratum below the bearing stratum (where applicable), and
- Construability (including methods of shaft construction)

Micropiles:

- Axial compression resistance for single micropile,
- Micropile group compression resistance,
- Uplift resistance for single micropiles,
- Uplift resistance of micropile groups,
- Micropile group punching failure into a weaker stratum below the bearing stratum, and single micropile punching failure where tip resistance is considered,
- Single and group micropile lateral resistance, and
- Constructability (including methods of micropile construction)

Extreme Event Limit State:

For the Extreme Event Limit State, foundations shall be designed for the cases indicated above for Strength Limits State Analyses (as applicable) but with appropriate Extreme Event load and resistance factors. In addition, where applicable, foundations shall be designed to withstand earth loading due to lateral spreading or seismically-induced slope displacements. Refer to Section 6.10 of this TM for further requirements, including assessment of earth loading due to lateral spreading or seismically-induced slope displacements.

3.3.7 Tolerable Foundation Settlement and Displacements

Requirements for tolerable foundation settlements and displacements presented herein shall supersede criteria indicated in AASHTO LRFD Bridge Design Specifications and the California Amendments. For deep foundations, tolerable settlements or displacements are measured at the top of the foundation: the pile cap, pile head, or the ground surface for drilled shaft pier-extensions. Limiting values for allowable deformations that are based on tolerable movements for the proposed bridges and tracks are in development. The following table presents preliminary tolerable settlement or displacement criteria. These criteria are subject to change.

TM 2.1.5 indicates that the tolerance of fasteners for the track can accommodate no more than 3 inches of vertical displacement based on the ability to adjust the fasteners spaced at intervals of 24 to 30 inches apart. Further performance requirements for allowable deformations are prescribed in the TM 2.10.10.

Table 3.3.7-1 Tolerable Foundation Vertical Settlement / Displacement Criteria

Limit State	Structure Type	Tolerable Settlement / Displacement	Comment
Service	Abutments	≤ 0.75 inch Settlement ≤ 0.375 inch Horizontal ≤ 0.0006 radians Angular Distortion	
Service	Bents/Piers	≤ 0.75 inch Settlement ≤ 0.375 inch Horizontal ≤ 0.0006 radians Angular Distortion	
Strength	All	Not applicable	Settlements and displacements need not be evaluated for the Strength Limit State
Extreme Event	Abutments	OPL1: ≤ ¼ inch Settlement ≤ ¼ inch Horizontal ≤ 0.0004 radians Angular Distortion SPL2: ≤ 1 inch Settlement ≤ ½ inch Horizontal ≤ 0.0008 radians Angular Distortion NCL3: ≤ 3 inches Settlement ≤ 3 inches Horizontal ≤ 0.0015 radians Angular Distortion	Extreme Event displacements defined in this table are permanent displacements following the cessation of ground shaking.
Extreme Event	Bents/Piers	OPL1: ≤ ¼ inch Settlement ≤ ¼ inch Horizontal ≤ 0.0004 radians Angular Distortion SPL2: ≤ 1 inch Settlement ≤ ½ inch Horizontal ≤ 0.0008 radians Angular Distortion NCL3: ≤ 3 inches Settlement ≤ 3 inches Horizontal ≤ 0.0015 radians Angular Distortion	Extreme Event displacements defined in this table are permanent displacements following the cessation of ground shaking.

Notes:

1. OPL = Operability Performance Level
2. SPL = Safety Performance Level
3. NCL = No Collapse Performance Level
4. Refer to TM 2.10.4 Interim Seismic Design Criteria regarding seismic design philosophy and requirements for the various performance levels.

The settlements and displacements noted in the table above are considered minimum performance criteria. Designers may elect to use more stringent criteria. Structural designers may require that foundations be designed to more stringent criteria for certain structures depending upon specific performance requirements, especially for the NCL performance level.

3.3.8 Resistance Factors for Foundation Design

Resistance factors for foundation design shall be consistent with those defined in the most current version of the AASHTO LRFD Bridge Design Specifications with California Amendments, Section 10.5.

3.3.9 Shallow Foundations

Geotechnical engineering analyses as well as structural designs for spread footing foundations shall be performed in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Section 10.6. Shallow foundation guidelines that shall be considered for geotechnical design are summarized in FHWA-SA-02-054 “Geotechnical Engineering Circular No. 6 - Shallow Foundations” dated September 2002, and FHWA-NHI-05-094 “LRFD for Highway Bridge Substructures and Earth Retaining Structures" dated January 2007.

3.3.10 Driven Piles and Drilled Shafts

Geotechnical engineering analyses as well as structural designs for driven piles and drilled shafts (deep foundations) shall be in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10.7, 10.8 and 10.9. Deep foundation guidelines that shall be considered for geotechnical design are summarized in FHWA-NHI-05-042/043 “Design and Construction of Driven Pile Foundations – Volumes I and II” dated April 2006, FHWA-IF-99-025 “Drilled Shafts - Construction Procedures and Design Methods” dated August 1999, and FHWA-NHI-05-094 “LRFD for Highway Bridge Substructures and Earth Retaining Structures" dated January 2007.

3.3.11 Proprietary Foundation Systems

Proprietary foundation systems typically require specialized analysis and design techniques that are not explicitly covered by the AASHTO LRFD Bridge Design Specifications with California Amendments. Examples of such systems include shallow or deep foundations bearing upon improved ground, screw-in helical foundation elements, or other systems. Proprietary foundation systems shall be permitted only if all of the following conditions are met:

- Established analytical methodologies with bases in widely accepted geotechnical literature are available to evaluate all relevant resistances and limit states,
- Resistance factors have been developed based on substantial statistical data combined with calibration, or substantial successful experience justifying the values can be demonstrated. Where resistance factors are developed through statistical analysis, they shall be based on reliability indices (β) and associated probabilities of failure indicated in Section C10.5.5.2.1 of AASHTO LRFD Bridge Design Specifications. Additional background on resistant factor development for geotechnical applications can be found in Paikowsky et. al. (2004) and Allen (2005), and
- Prior to use of the proprietary foundation system in design for the CHST project, the analytical methodologies and resistance factors noted above must be presented to and approved by the Authority or its agent.

This section to be prepared for use during final design.

3.3.12 Abutments and Abutment Foundations

Bridge abutments have components of both foundation design and retaining wall design. The retaining wall aspects of abutments shall be designed in accordance with Section 6.7 of this TM, and also Section 11 of the AASHTO LRFD Bridge Design Specifications. Foundations for abutments shall be designed in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10 and 11. Abutment foundation guidelines that shall be considered for geotechnical design are summarized in FHWA-NHI-05-094 "LRFD for Highway Bridge Substructures and Earth Retaining Structures" dated January 2007.

3.3.13 Seismic Analysis and Design for Foundations and Abutments

Foundations and abutments shall be designed for the Extreme Event I seismic case. Seismic design procedures for foundations and abutments are addressed in Section 6.10 of this TM.

This section will be prepared for 30% design.

3.4 Foundations for Buildings and other At-Grade Structures

This section will be prepared for 30% design.

3.5 Tunnels and Other Underground Structures

This section will be prepared for 30% design.

3.6 Track Bed Embankments and Embankment Foundations

This section will be prepared for 30% design.

3.7 Retaining Walls, Fill Walls, and Reinforced Earth Systems

3.7.1 Definitions and Wall Types Including Acceptable and Unacceptable Walls

Engineered earth retention systems may retain soil permanently, or (in the case of construction) temporarily. Similar to the function of retaining walls, the function of reinforced soil slopes (RSS) is to strengthen the mass of earth material such that a steep (generally up to about 1H:2V) slope can be formed. Steep RSSs generally do not require a structural facing, whereas retaining walls typically use structural facing. RSSs often use a permanent erosion control matting with low vegetation as a slope cover to prevent erosion.

Walls shall be classified as either a "fill wall" or a "cut wall." Examples of fill walls include standard cantilever walls, Mechanically Stabilized Earth (MSE) walls, and modular gravity walls (gabions, bin walls, and crib walls). Cut walls include soil nail walls, cantilever soldier-pile walls, and ground anchored walls (other than nail walls).

Walls shall be further classified as gravity, semi-gravity, non-gravity cantilever, anchored, or in-situ reinforced. For geotechnical design, the various wall classifications, definitions and additional detail are provided in Section 11 of AASHTO LRFD-BDS, and FHWA's Earth Retaining Structures Reference Manual (FHWA 2008). For CHSTP, each of these wall categories will be considered as "generally acceptable" walls provided that the combined earth/structural system meets all of the design and performance criteria. Wall types considered to be "unacceptable" include mortar rubble gravity walls, timber or metal bin walls, and "rockery" walls.

3.7.2 Design Considerations

Retaining wall and slope designs shall be coordinated with other project design elements that might interfere with or impact the design or construction of the wall or slope. This includes coordination with the Structures and Civil Design Discipline, Systems Discipline, and Hydrology and Hydraulics Disciplines to select the most appropriate earth retention system for a given setting based on design constraints, geotechnical subsurface investigations, and surface and groundwater issues. Consideration must be given to presence of (and potential conflicts with) drainage features; buried and overhead utilities; lighting or sign structures; adjacent retaining walls or bridges; concrete traffic barriers and/or fences; and guardrails. These design elements shall be located in a manner that will minimize the impacts to the retaining wall or reinforced slope elements. The potential effect that site constraints might have on the constructability of the specific wall/slope shall be considered. Additional constraints to be considered include but are not limited to site geometry, access, time required to construct the wall, environmental issues, and impact on traffic flow and other construction activities.

The structural elements of the wall or slope and the soil below, behind, and/or within the structure shall be designed together as a system. The wall or slope system shall be designed for overall external stability as well as internal stability. Overall external stability includes stability of the slope the wall/reinforced slope is a part of and the local external stability (overturning, sliding, and bearing capacity). Internal stability includes resistance of the structural members to load and, in the case of MSE walls and reinforced slopes, pullout capacity of the structural members or soil reinforcement from the soil.

Geotechnical Investigation - all retaining wall and RSSs require subsurface data representative of the underlying soil/rock that supports the structure. The stability and support characteristics of the underlying soils, their potential to settle under the imposed loads, the usability of any existing excavated soils for wall/reinforced slope backfill, and the location of the groundwater table shall be evaluated through the geotechnical investigation.

For wall and/or RSS type selection, factors that must be considered include the intended application; the soil/rock conditions in terms of settlement; need for deep foundations; constructability; impacts to traffic; and the overall geometry in terms of wall/slope height and length, location of adjacent structures and utilities, aesthetics, and cost.

Other considerations that wall/slope selection is dependent upon include:

- Whether the wall/slope will be located primarily in a cut or fill,
- How much excavation/shoring will be required to construct the wall or slope,
- If located in a cut, the type of soil/rock present,
- The need for space between the right of way line and the wall/slope or easement,
- The amount of settlement expected,
- The potential for deep failure surfaces to be present,
- The structural capacity of the wall/slope in terms of maximum allowable height,.
- The nature of the wall/slope application,
- Whether or not structures or utilities will be located on or above the wall,
- Architectural requirements, and
- Overall economy

For “type selection” purposes, geotechnical design shall consider the summary of various wall/slope options available (including their advantages, disadvantages, and limitations) provided in FHWA-NHI-07-071. Specific wall types shown in the exhibits of FHWA-NHI-07-071 may represent multiple wall systems, some or all of which will be proprietary. There are a number of factors that control wall type selection and design considerations, including:

- Magnitude and direction of loading,
- Depth to suitable bearing materials (foundation support),
- Potential for earthquake loading and liquefaction,
- Proximity of physical constraints,
- Tolerable total and differential settlement,
- Facing durability and aesthetics,
- Ease and cost of construction,
- Potential for undermining or scour, swelling potential (clay soil, and frost depth), and
- Cross sectional wall/slope geometry

Wall/slope geometry is developed considering the following:

- Geometry of the transportation facility itself,
- Design Clear Zone requirements,
- Right of way constraints,
- Existing ground contours,
- Existing and future utility locations,
- Impact to adjacent structures,
- Impact to environmentally sensitive areas, and
- Consider the foundation embedment and type anticipated, which requires coordination between the various design groups involved.

Feasible retaining wall heights to be considered for geotechnical design are affected by issues such as the capacity of the wall structural elements, past experience with a particular wall, current practice, seismic factors, long-term durability, and aesthetics. Wall facing selection considerations are dependent on the aesthetic and structural needs of the wall system. Wall settlement may also affect the feasibility of the facing options. More than one wall facing may be available for a given system. The available facing options shall be considered when selecting a particular wall. Wall type selection and facing options are summarized in FHWA-NHI-07-071, Chapter 10.

In brief summary, the design of a retaining wall or RSS consists of the following principal activities:

- Develop wall/slope geometry,
- Provide adequate subsurface investigation,
- Evaluate loads and pressures that will act on the structure,
- Design the structure to withstand the loads and pressures,
- Design the structure to meet aesthetic requirements,
- Ensure wall/slope constructability, and
- Coordinate with other design elements

The structure and adjacent soil mass need to be stable as a system, and the anticipated wall settlement needs to be within acceptable limits.

3.7.3 Limit States and Resistance Factors

Geotechnical designs for retaining walls shall be performed in accordance with AASHTO LRFD Bridge Design Specifications. The LRFD process and example calculations for individual wall types are provided in FHWA-NHI-07-071. Section 11 of the AASHTO (2007) LRFD Specification provides information on LRFD for earth retaining structures including conventional retaining walls, nongravity cantilevered walls, anchored walls, mechanically stabilized earth (MSE) walls, and prefabricated modular walls. Publication number FHWA-NHI-05-094 "LRFD for Highway Bridge Substructures and Earth Retaining Structures" dated January 2007 contains comprehensive guidance on LRFD for retaining wall systems and abutments and shall be considered by the geotechnical engineer.

AASHTO LRFD load combinations for earth retaining systems and bridge substructures are provided in Tables 3.4.1-1 of AASHTO (2007). The load factors for permanent loads used for earth retaining systems are provided in Table 3.4.1-2 of AASHTO (2007). In general, minimum load factors shall be used if permanent loads increase stability and maximum load factors shall be used if permanent loads reduce stability. See AASHTO (2007) Section 3.3 for complete definition of loads. For reference purposes, the resistance factors for design of earth retaining walls are presented in Table 11.5.6-1 of AASHTO for LRFD, and so are not reprinted here.

3.7.4 External Loads and Stability Analysis

AASHTO LRFD shall be used for evaluation of stability for retaining walls and abutments. Retaining walls and abutments shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads. For wall evaluation and design, earth pressure shall be considered as a function of the following:

- Type and unit weight of the earth,
- Water content,
- Soil creep characteristics,
- Degree of compaction,
- Location of 'design' groundwater table,
- Earth-structure interaction,
- Amount of surcharge load,
- Earthquake effects,
- Back slope angle, and
- Wall inclination

Calculation methods for analysis of earth pressure and water/hydrostatic pressures, including consideration of the various factors listed above, are provided in Section 3, Loads and Load Factors, of current AASHTO LRFD BDS. Earth pressures used in design of walls and abutments shall be selected consistent with the requirement that the abutment movement shall not exceed tolerable displacement and settlement limits described in Section 6.7.7 of this TM. Analyses methods for application of these various pressures in retaining wall design and stability evaluation of wall and abutment structures are provided in Section 11, Abutments Piers and Walls, of current AASHTO LRFD BDS.

The provisions of AASHTO LRFD BDS Section 11, including methods of analyses/calculations for various wall types, shall be used for evaluation of stability for retaining walls and abutments. This includes analyses for overturning, bearing resistance, external stability (soil failure) and internal stability (safety against structural failure or combined soil-structure failure), sliding, seismic-load case, etc. Overall stability shall be evaluated using limit equilibrium methods of analysis. For global stability analysis of walls on steep slopes consider the initial stability of the slope and the impact (or lack of) that the proposed construction has on the slope.

3.7.5 Groundwater, Seepage, and Drainage Design

Adequate drainage behind all retaining walls and engineered slopes shall be included in the design and implemented during construction. Designs shall provide positive drainage at periodic intervals to prevent entrapment of water. Native soil may be used for retaining wall and reinforced slope backfill provided that it meets the requirements for the particular wall/slope system, and will satisfy long term deformation requirements particularly upon wetting.

Backfills behind retaining walls and abutments shall be drained, and drainage systems shall be designed to completely drain the entire retained soil volume behind the retaining wall face. If drainage cannot be provided due to site constraints, the abutment or wall shall be designed for loads due to earth pressure, plus full hydrostatic pressure due to water in the backfill.

For MSE walls and RSSs, internal drainage measures shall be considered for all structures to prevent saturation of the reinforced backfill and to intercept any surface flows containing corrosive elements. MSE walls in cut areas and side-hill fills with established groundwater levels shall be constructed with drainage blankets in back of, and beneath, the reinforced zone. In cut and side-hill fill areas, if prefabricated modular wall units are used then the structure shall be designed with a continuous subsurface drain placed at, or near, the footing grade and outletted as required. In cut and side-hill fill areas with established or potential groundwater levels above the footing grade, a continuous drainage blanket shall be provided and connected to the longitudinal drain system. For systems with open front faces, a surface drainage system shall be provided above the top of the wall.

At locations where retaining walls or reinforced slopes can be in contact with water (such as a culvert outfall, ditch, wetland, lake, river, or floodplain), there is a potential risk of scour at the toe. This risk must be analyzed and mitigated for design and construction.

Where thin drainage panels are used behind walls and saturated or moist soil behind the panels may be subjected to expansion due to freezing, either insulation shall be provided on the walls to prevent freezing of the soil, or the wall shall be designed for the pressures exerted on the wall by frozen soil.

3.7.6 Seismic Analysis for Retaining Walls and Reinforced Earth Systems

Section 6.10 of this TM presents procedures for developing dynamic soil pressures for seismic analysis and design of retaining walls.

This section to be expanded for 30% design.

3.7.7 Settlement and Horizontal Deformation / Movement Tolerances

Settlement issues, especially differential settlement, are of primary concern in the selection of walls. Some wall types are inherently flexible and tolerate more settlement without poor structural performance. Other wall types are inherently rigid and cannot tolerate much settlement. The total and differential vertical deformation of a retaining wall shall be small for rigid gravity and semigravity retaining walls and shall meet structural and track tolerance performance requirements.

Retaining wall and abutment structures shall be investigated for excessive vertical and lateral displacement, and overall stability, at the service limit state. Tolerable vertical and lateral deformation limits for retaining walls and abutments shall be developed from the structural engineering design and performance criteria based on the function and type of wall, design service life (100 years), and consequences of unacceptable movements to the wall and any potentially affected nearby structures, i.e., both structural and aesthetic.

Vertical wall movements are primarily the result of soil settlement beneath the wall foundation. The provisions of AASHTO (Section 10) shall apply for analytical methods to estimate vertical wall movements. For gravity and semi-gravity walls, lateral movement estimates shall be assessed resulting from a combination of differential vertical settlement between the heel and the toe of the wall, and the rotation necessary to develop active earth pressure conditions. Tolerable total and differential vertical deformations for a particular retaining wall are dependent on the ability of the wall to deflect without causing damage to the wall elements or adjacent structures, or without exhibiting deformations that are unsightly and/or affect wall performance. Regarding impact to the wall itself, differential settlement along the length of the wall and to some extent from front to back of wall is the best indicator of the potential for retaining wall structural damage or overstress. Wall facing stiffness and ability to adjust incrementally to movement affect the ability of a given wall system to tolerate differential movements, and shall be evaluated by the geotechnical engineer.

For MSE walls, deflections shall be estimated in accordance with the provisions of AASHTO Section 11. MSE walls have the greatest flexibility and tolerance to total and differential vertical settlement, followed by prefabricated modular gravity walls. Reinforced soil slopes RSSs are also inherently flexible. For MSE walls, the facing type used can affect the ability of the wall to tolerate settlement, and shall be evaluated by the geotechnical engineer. Other factors to be considered include MSE wall configuration and timing of facing construction.

Semigravity (cantilever) walls and rigid gravity walls have the least tolerance to settlement. In general, total settlement for these types of walls shall be limited to approximately ___ inch or less (subject to confirmation). Therefore, semigravity cantilever walls, and rigid gravity walls shall not be used in settlement prone areas. If very weak soils are present that will not support the wall and are too deep to be overexcavated, or if a deep failure surface is present that results in inadequate slope stability, a wall type shall be selected that is capable of using deep foundation support and/or anchors. In general, MSE walls, prefabricated modular gravity walls, and some rigid gravity walls are not appropriate for these situations. Walls that can be pile-supported, such as concrete semigravity cantilever walls, nongravity cantilever walls, and anchored walls, are more appropriate for these situations. For anchored walls, downward movement can cause significant stress relaxation of the anchors and shall be considered for design. Anchored wall deflections shall be estimated in accordance with the provisions of AASHTO Section 11.

In evaluating settlement of retaining walls whose backfill supports train tracks, consideration shall be given to the time rate of settlement. To avoid excessive deflections in the track, track structures shall not be constructed until the majority of expected retaining wall settlement has already occurred, and been monitored and documented. In some cases, this may necessitate the use of added construction measures to expedite settlement such as surcharging or wick drains.

3.7.8 Design of Reinforced Soil Slopes (RSS) and Mechanically Stabilized Earth (MSE) Structures

Definitions for Reinforced Soil Slope (RSS) embankments and Mechanically Stabilized Earth (MSE) structures, as well as step-by-step design methodology and analyses that shall be used for MSE and RSS systems are provided in the LRFD version of FHWA's manual FHWA-NHI-10-024/25 "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes", Volumes I and II, dated November 2009. The RSS and MSE manuals also provide instructions for computer-aided analysis that shall be used for design. Numerous geosynthetic reinforcements and facing systems are available. The embankment fill may be either granular or cohesive material, however granular fill materials are preferable and may be necessary in order to meet the various performance requirements.

Advantages of using MSE and RSS systems are that embankments and slopes can be constructed at an angle steeper than could otherwise be safely constructed with the same soil (with the existence of a firm foundation). This results in savings of materials and right-of-way. Right-of-way savings can be a substantial benefit, especially for CHSTP construction in urban areas where acquiring new right-of-way is expensive or, in some cases, unobtainable.

The following general limitations may be associated with MSE and RSS systems, and should be accounted for in design and construction:

- Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of geosynthetic elements due to exposure to ultra violet rays, chemical attack, heat and other potentially degrading elements in the ground. See FHWA reference manual FHWA-NHI-00-044 “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes”, dated September 2000.
- Since certain systems require select granular fill, they may become uneconomical if granular borrow sources are not readily available.
- Maintenance of vegetation (e.g., grass mowing) on steep side slopes may require special equipment.

Reinforcement placed at the edges of a compacted slope can provide lateral resistance during compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope. The effects of compaction on the performance of MSE systems is described in FHWA 132036A – Earth Retaining Structures.

The CHSTP may include non-standard proprietary wall systems (such as MSE) and non-standard non-proprietary wall systems (such as soil nail walls, anchored walls, reinforced slopes, etc.). From development of wall designs to the final wall product, all preliminary designs by the engineering team and final designs/construction submittals by the D-B Contractor for walls (both proprietary and non-proprietary types) shall be reviewed and approved.

Standard walls may not be the most cost effective option. Proprietary walls provide more options in terms of cost-effectiveness and aesthetics. Non-standard walls that may involve elements such as soil nail and anchored wall systems are acceptable, provided that requirements are met. Reinforced slopes are similar to non-standard / non-proprietary walls in terms of their design process.

For preliminary design of these wall or slope systems, required information to be provided is as follows:

- The allowable bearing capacity and foundation embedment criteria for the wall,
- Backfill and foundation soil properties (assume that gravel borrow or structural backfill material will be used for the walls when assessing soil parameters),
- A general wall and/or slope plan; a profile showing neat line top and bottom of the wall; profiles showing the existing and a final ground line in front of and in back of the wall; site data and a typical cross-section,
- Location of right-of-way lines and other constraints to wall/slope construction,
- Location of adjacent existing and/or proposed structures, utilities, and obstructions,
- Generic details for the desired appurtenances and drainage requirements, and load or other design acceptance requirements for these appurtenances,
- Location of catch basins, grate inlets, signal foundations, and the like (it is best to locate these outside the reinforced MSE wall backfill zone to avoid interference with the soil reinforcement),

- In cases where conflict with these reinforcement obstructions cannot be avoided, indicate the location(s) and dimensions of the reinforcement obstruction(s) relative to the wall on the plans, and
- Wall/slope facing alternatives to meet the CHSTP aesthetic and performance requirements

For non-proprietary RSSs, anchored walls, walls containing geo-synthetics, and soil nail walls, the designer initiates the design effort and develops wall/slope profiles, preliminary engineering plans, cross sections, quantities, special provisions, cost estimates etc., for the proposed wall/slope and subsequently a complete and detailed wall/slope design and construction is coordinated and carried out during final design.

Additional geotechnical guidance will be prepared for use during final design.

3.7.9 Wall Foundation Improvement Needs using Ground Improvement Methods

At locations where ‘poor’ ground conditions are present that could result retaining walls or abutment features to not meet performance requirements, due to settlement or stability problems, advanced mitigation measures such as ground improvement shall be considered for geotechnical design. Ground improvement measures may also be necessary to mitigate potential seismic hazards, such as liquefaction or seismic stability.

Ground improvement has one or more than one of the following main functions, including to:

- Increase bearing capacity, shear or frictional strength,
- Increase density,
- Control or reduce deformations,
- Accelerate consolidation,
- Decrease imposed loads,
- Provide lateral stability,
- Form seepage cutoffs or fill voids,
- Increase resistance to liquefaction, and
- Transfer embankment loads to more competent layers

The selection of candidate ground improvement methods for any specific project shall follow the process described in detail in FHWA’s Ground Improvement Reference Manuals Volumes I and II, FHWA-NHI-06-019/020 dated 2006. A brief summary list of the sequential selection process (derived from the FHWA manual) is provided as follows:

1. Identify potential poor ground conditions, their extent and type of negative impact. Poor ground conditions are typically characterized by potentially compressible foundation soils which under load would cause unacceptable settlement or instability.
2. Identify and establish performance requirements. Performance requirements generally consist of deformation limits (horizontal and vertical), as well as some minimum factors of safety for stability. The available time for construction is also a performance requirement.
3. Identify and assess any space or environmental constraints. Space constraints typically refer to accessibility for construction equipment to operate safely and environmental constraints may include the disposal of spoil (hazardous or otherwise) and the effect of construction vibrations or noise.
4. Assessment of subsurface conditions. The type, depth and extent of the poor soils must be considered as well as the location of the ground-water table. It is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils.
5. Preliminary Selection. Preliminary selection of potentially applicable method(s) is generally made on a qualitative basis taking into consideration the performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints and the level of improvement that is required.
6. Preliminary Design. A preliminary design is developed for each method identified under Preliminary Selection and a cost estimate prepared on the basis of available data. The

guidance in developing preliminary designs is contained within technical summary sections of the FHWA manual.

7. Comparison and Selection. The selected methods are then compared and a selection made by considering performance.

3.7.10 Lateral Support of Temporary Excavation Systems

This section will be prepared for use during final design phase.

3.8 CUT SLOPES AND NATURAL SLOPES

This section will be prepared for use during 30% design phase.

3.9 DRAINAGE, SUBDRAINAGE, INFILTRATION FACILITIES AND DEWATERING

This section will be prepared for use during 30% design phase.

3.10 GEOTECHNICAL EARTHQUAKE ENGINEERING

3.10.1 Seismic Analysis and Design Requirements

This section presents analysis and design requirements for geotechnical earthquake engineering aspects of the CHSTP. Topics covered in this section include design ground motions, liquefaction triggering and consequences, lateral spreading, seismic slope stability, seismic earth pressures for retaining walls, seismic foundation design, and seismic compaction.

Some aspects of geotechnical earthquake engineering may overlap with geologic hazards and seismic design issues that are addressed by other CHSTP TMs.

3.10.2 Seismic Design Criteria

Seismic design criteria for geotechnical earthquake engineering have been established in terms of three levels of project performance criteria and associated ground motion levels in TM 2.10.4.

Geotechnical seismic design shall be consistent with the philosophy for structure design for all three performance levels. The performance objective shall be achieved at a seismic risk level that is consistent with the seismic risk level required for that seismic event. Slope instability and other seismic hazards such as liquefaction, lateral spread, post-liquefaction pile downdrag, and seismic settlement may require mitigation to ensure that acceptable performance is obtained during a design seismic event. The geotechnical designer shall evaluate the potential for differential settlement between mitigated and non mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions. The foundations shall also be designed to address liquefaction, lateral spread, and other seismic effects to prevent collapse. All earth retaining structures shall be evaluated and designed for seismic stability internally and externally (i.e., sliding and overturning). Cut slopes in soil and rock, fill slopes, and embankments, especially those which could have significant impact on the operations of high speed trains should be evaluated for instability due to design seismic events and associated geologic hazards.

3.10.3 Design Ground Motions

Methods to develop design ground motions for this project which are applicable to geotechnical earthquake engineering are presented in TM 2.9.6 for 30% design. Methods to develop design ground motions for final design have not been prepared at this time.

3.10.4 Site Response and Ground Amplification

Methods to perform site-specific site response analysis, where needed, are presented in TM 2.9.6 for the 30% design.

3.10.5 Limits on Site Response Analyses

If site-specific ground motions in terms of design response spectra are obtained using site response analysis methods per TM 2.9.6 for 30% design, the resulting response spectra must be limited to the limits of ASCE 7-05 Chapter 21. The geotechnical engineer shall refer to TM 2.9.6 for additional details.

3.10.6 Seismic Soil-Structure Interaction Analysis

Requirements for soil-structure interaction pertaining to soil-structure-interaction (SSI) analyses are pending.

3.10.7 Evaluation of Liquefaction Triggering and Consequences

Evaluation of soil liquefaction triggering potential shall be performed in two steps. The first step involves evaluating whether the soil meets the compositional criteria necessary for liquefaction. For soils meeting the compositional criteria, the next step is to evaluate whether the design level ground shaking is sufficient to trigger liquefaction given the soil's in-situ density. If it is determined that liquefaction will be triggered, the engineering consequences of liquefaction shall be evaluated. In addition to Factor of Safety-based criteria for liquefaction, the designer shall also consider the allowable deformation values described in Section 6.3.5 and the long-term, post construction performance requirements for earth and fill conditions.

3.10.7.1 Criteria for Liquefaction Susceptibility of Silts and Clays

Evaluation of whether silty and clayey soils meet the criteria for liquefaction susceptibility shall be performed using the criteria developed by Bray and Sancio (2006), and compared to results by analysis using the methods presented in Idriss and Boulanger (2008). Results of these two methods of analyses shall be interpreted and applied to design using engineering judgment.

Considering the range of criteria currently available in the literature, geotechnical engineers shall consider performing cyclic triaxial or simple shear laboratory tests on undisturbed soil samples to assess liquefaction susceptibility for critical cases. For fine grained soils that do not meet the above criteria for liquefaction, cyclic softening resulting from seismic shaking shall be considered.

No specific guidance regarding susceptibility of gravels to liquefaction is currently available. The primary reason why gravels may not liquefy is that their high permeability frequently precludes the development of undrained conditions during and after earthquake loading. When bounded by lower permeability layers, however, gravels shall be considered potentially susceptible to liquefaction and their liquefaction susceptibility evaluated. A gravel layer that contains sufficient sand to reduce its permeability to a level near that of the sand, even if not bounded by lower permeability layers, shall also be considered susceptible to liquefaction and its liquefaction potential evaluated as such.

3.10.8 Liquefaction Triggering Evaluations

Liquefaction triggering analyses should be performed for sites that meet the following criteria:

- The estimated maximum groundwater elevation at the site is determined to be within 75 ft of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile is characterized in the upper 75 ft as having soils that meet the compositional criteria for liquefaction with a measured SPT resistance, corrected for overburden pressure and hammer energy $(N_1)_{60}$, less than 30 blows/ft, or a cone tip resistance q_{c1N} of less than 180, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

Liquefaction triggering analyses shall be limited to the upper 75 feet. If the site meets the conditions described above, a detailed assessment of liquefaction potential shall be conducted.

Liquefaction analysis involves estimating factor of safety (FS) against liquefaction. Factor of safety against liquefaction is defined as the ratio between Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR). The most common method of assessing liquefaction involves the use of empirical methods (i.e., Simplified Procedures) to estimate CSR and CRR. These methods provide an estimate of liquefaction potential based on Standard Penetration Test (SPT) blowcounts, Cone Penetration Test (CPT) tip resistance, Becker Hammer Penetration Test (BPT) blowcounts, or shear wave velocity. SPT and CPT test methods are most common and generally considered to be more reliable for liquefaction analyses than BPT and Vs tests. Vs and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods, such as gravelly soils. This type of analysis shall be conducted as a baseline evaluation, even when more rigorous methods are used. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modeled well by the simplified methods.

3.10.8.1 Simplified Procedures

The two updated simplified methods by Seed et. al. (2003) and Idriss and Boulanger (2008) shall be used for liquefaction triggering analysis. Results of these analyses shall be interpreted and applied to design using engineering judgment.

As an alternative to the simplified methods, to improve the assessment of liquefaction potential, especially at greater depths, if soft or loose soils are present, equivalent linear or nonlinear site specific, one dimensional site response analyses may be conducted to determine the maximum earthquake induced shear stresses at depth in the Simplified Method. For example, the linear total stress computer programs ProShake (EduPro Civil Systems, 1999) or Shake2000 (Ordoñez, 2000) may be used for this purpose. Consideration should be given to the consistency of site specific analyses with the procedures used to develop the liquefaction resistance curves. A minimum of seven spectrally matched time histories should be used to conduct these analyses. More specifics about site response analysis are presented in TM 2.9.6.

3.10.8.2 Nonlinear Effective Stress Methods

An alternative to the simplified procedures for evaluating liquefaction susceptibility is to complete a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation, such as D-MOD2000 (Matasović, et. al., 2007). This is a more rigorous analysis that requires additional parameters to describe the stress-strain behavior and pore pressure generation characteristics of the soil.

It should be recognized that the results of nonlinear effective stress analyses can be quite sensitive to soil parameters that are often not as well established as those used in equivalent linear analyses. Therefore, it is incumbent upon the user to calibrate the model and evaluate the sensitivity of its results to any uncertain parameters or modeling assumptions. Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, approval by the EMT Geotechnical Engineer is required to use nonlinear effective stress methods for liquefaction evaluation.

3.10.8.3 Minimum Factor of Safety against Liquefaction

The potential consequences of liquefaction and (if necessary) liquefaction hazard mitigation measures shall be evaluated if the factor of safety against liquefaction is less than 1.2.

3.10.8.4 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Methods to estimate settlement of unsaturated granular deposits are presented in a section 6.10.14. Liquefaction induced ground settlement of saturated granular deposits shall be estimated using the procedures by Tokimatsu and Seed (1987), or Ishihara and Yoshimine (1992). The corrected SPT blow counts for the Tokimatsu and Seed (1987) method shall include all corrections, including the corrections for fines. However, the corrections for fines for settlement calculations are different than the corrections for liquefaction analyses. In addition, the CSR values shall also be corrected for magnitude before estimating settlements. If a laboratory-based analysis of liquefaction induced settlement is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the liquefaction induced vertical settlement in lieu of empirical SPT or CPT based criteria. Even when laboratory-based volumetric strain test results are obtained and used for design, the empirical methods shall be used to qualitatively check the reasonableness of the laboratory test results.

The designer shall compare the estimated settlement values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The designer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

3.10.8.5 Liquefied Residual Strength Parameters

Lower-third value of the range of values proposed by Seed and Harder (1990) curve shall be used to estimate residual strength of liquefied soil unless soil specific laboratory performance tests are conducted as described below. Results of laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the residual strength in lieu of empirical SPT or CPT based criteria. Even when laboratory-based test results are obtained and used for design, the Seed and Harder (1990) curve shall be used to qualitatively check the reasonableness of the laboratory test results. It shall be noted that SPT N fines content corrections for residual strength calculations are different than corrections for liquefaction triggering and settlement.

3.10.8.6 Surface Manifestations

The assessment of whether surface manifestation of liquefaction (such as sand boils, ground fissures etc.) will occur during earthquake shaking at a level-ground site shall be made using the method outlined by Ishihara (1985) and shall be compared against results by the method presented in Youd and Garris (1994 and 1995). It is emphasized that settlement may occur, even with the absence of surface manifestation. The 1985 Ishihara method is based on the thickness of the potentially liquefiable layer (H_2) and the thickness of the non-liquefiable crust (H_1) at a given site. In the case of a site with stratified soils containing both potentially liquefiable and non-liquefiable soils, the thickness of a potentially liquefiable layer (H_2) shall be estimated using the method proposed by Ishihara (1985) and Martin et. al., (1991). If the site contains potential for surface manifestation, then use of mitigation methods shall be evaluated.

3.10.9 Evaluation of Lateral Spreading and Consequences

Lateral spreading is a term commonly used to describe permanent, predominantly lateral deformation of sloping ground or level ground near a “free face”, such as a river bank, that occurs during earthquake shaking as a result of soil liquefaction. Its effects on structures can be devastating because its occurrence has been observed in loose, medium-dense, and even dense soils. Deformations can range from millimeters to several meters, with the greatest displacements usually occurring near free-faces. Therefore, facilities and structures adjacent to bodies of water (e.g. ports/harbors, lakes, and rivers) are usually at the greatest risk of experiencing damage due to lateral spreading. The result of lateral spreading is typically horizontal movement of non-liquefied soils located above liquefied soils, in addition to the liquefied soils themselves.

Lateral spreading shall be evaluated for a site if liquefaction is expected to trigger within 50 feet of the ground surface and a slope gradient of 0.1% or more exists within the liquefiable layer. Historic and paleoseismic evidence of lateral spreading is valuable information that shall also be reviewed and addressed. Such evidence may include sand boils, soil shear zones, and topographic geometry indicating a spread has occurred in the past.

3.10.9.1 Methodologies for Predicting Lateral Spreading

In order to predict the permanent deformations resulting from the occurrence of lateral spreading during earthquake loading, several methods of analyses are available. These different methods of analyses can be categorized into two general types: *Empirical Methods* and *Analytical Methods*.

<u>Empirical Methods</u>

The most common empirical methods to estimate lateral displacements are Youd et. al. (2002), Bardet et. al. (1999), Zhang et. al. (2004), Faris et. al. (2006). Analysts shall be aware of the applicability and limitations of each method. Lateral displacements shall be evaluated using the Youd et. al. (2002) method, and one of the other methods described above.

Empirical methods shall be used as the primary means to estimate deformations due to lateral spreading. Multiple models shall be considered and the range of results shall be reported.

Analytical Methods

For cases where slope geometry, structural reinforcement or other site-specific features are not compatible with the assumptions of the empirical methods, Newmark sliding block analyses shall be considered. Newmark analyses shall be conducted similar to that described in the seismic slope stability section, except that estimation of the yield acceleration shall consider strength degradation due to liquefaction.

The designer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The designer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

3.10.10 Analysis for Conceptual Design of Liquefaction Mitigation Methods

Liquefaction mitigation and performance criteria vary according to the acceptable level of risk and required levels of performance for each structure type. Implementation of mitigation measures shall be designed to either eliminate all liquefaction potential or to allow partial improvement of the soils, provided that acceptable performance (i.e., stability and deformation levels) can be achieved.

During the liquefaction evaluation, the engineer shall determine the extent of liquefaction and potential consequences such as bearing failure, slope stability, and/or vertical and/or horizontal deformations. Similarly, the engineer will determine the liquefaction hazard in terms of depth and lateral extent affecting the structure in question. The lateral extent affecting the structure will depend on whether there is potential for large lateral spreads toward or away from the structure and the influence of liquefied ground surrounding mitigated soils within the perimeter of the structure.

Large lateral spread or flow failure hazards may be mitigated by the implementation of containment structures, removal or treatment of liquefiable soils, modification of site geometry, structural resistance, or drainage to lower the groundwater table.

Where liquefiable clean sands are present, geotechnical evaluations for design shall consider an area of softening due to seepage flow occurring laterally beyond the limit of improved ground a distance of two-thirds of the liquefiable layer thickness, as described in studies by Lai (1988). To calculate the liquefiable thickness, similar criteria shall be used as that employed to evaluate the issue of surface manifestation by the Ishihara (1985) method. For level ground conditions where lateral spread is not a concern or the site is not a water front, this buffer zone shall not be less than 15 feet and it is likely not to exceed 35 feet when the depth of liquefaction is considered as 50 feet and the entire soil profile consists of liquefiable sand.

The performance criteria for liquefaction mitigation, established during the initial investigation, shall be in the form of a minimum, or average, penetration resistance value associated with a soil type (fines content, clay fraction, USCS classification, CPT soil behavior type index Ic, normalized CPT friction ratio), or a tolerable liquefaction settlement as calculated by procedures discussed earlier. The choice of mitigation methods will depend on the extent of liquefaction and the related consequences. Also, the cost of mitigation must be considered in light of an acceptable level of risk. In general, options for mitigations are divided into two categories: ground improvement options and structural options.

3.10.10.1 Ground Improvement Options

The five general methods of ground improvement to be considered for soil liquefaction mitigation are:

- Densification
- Drainage
- Reinforcement
- Mixing/Solidification, and
- Replacement

The implementation of these techniques may be designed to fully, or partially, eliminate the liquefaction potential, depending on the performance requirements of the engineered facility under consideration. With regards to drainage techniques for liquefaction mitigation, only permanent dewatering works satisfactorily. The use of gravel or prefabricated drains, installed without soil densification, is unlikely to provide pore pressure relief during strong earthquakes and may not prevent excessive settlement.

Densification Techniques

The most widely used techniques for in-situ densification of liquefiable soils are:

- Vibrocompaction
- Vibro-replacement (also known as vibro-stone columns)
- Deep dynamic compaction, and
- Compaction (pressure) grouting (Hayden and Baez, 1994)

Further details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

Mixing/Solidification Techniques

Mixing and/or solidification techniques seek to reduce the void space in the liquefiable soil by introducing grout materials either through permeation, mixing mechanically, or jetting. The most widely used hardening techniques are:

- Permeation grouting
- Deep soil mixing, and
- Jet grouting

Further details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

3.10.10.2 Structural Options

Structural mitigation involves designing the structure to withstand the forces and displacements that result from liquefaction. In some cases, structural mitigation for liquefaction effects may be more economical than soil improvement mitigation methods. However, structural mitigation may have little or no effect on the soil itself and may not reduce the potential for liquefaction. With structural mitigation, liquefaction and related ground deformations will still occur. The structural mitigation shall be designed to protect the structure from liquefaction-induced deformations, recognizing that the structural solution may have little or no improvement on the soil conditions that cause liquefaction. The appropriate means of structural mitigation may depend on the magnitude and type of liquefaction-induced soil deformation or load. If liquefaction-induced flow slides or significant lateral spreading is expected, structural mitigation may not be practical or feasible in many cases. If the soil deformation is expected to be primarily vertical settlement, structural mitigation may be economically and technically feasible and shall be considered for design.

Depending on the type of structure and amount and extent of liquefaction, common structural options to be considered are:

- Post-tensioned slab foundation
- Continuous spread footings having isolated footings interconnected with grade beams
- Mat foundation, and
- Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils

Details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

3.10.11 Seismic Considerations for Lateral Design of Piles in Liquefiable Soils

Seismic considerations for lateral design of pile/shaft design soils include the effects of liquefaction on the lateral response of piles/shafts and designing for the additional loads due to lateral spread and/or slope failures. Effects of liquefiable soils shall be included in the lateral analysis of piles/shafts by using appropriate p-y curves to represent liquefiable soils. Computer programs such as LPILE include p-y curves for liquefiable soils. The p-y curves available within the program have limited application and may give unconservative results. Furthermore, in fully liquefied sand, there appears to be virtually no lateral soil resistance for the first 1 to 2 inches of lateral movement (Rollins et. al., 2005). Available static p-y curve models reduced adequately to account for the loss of strength caused by liquefaction, such as a p-multiplier approach, could provide an approximate prediction of the measured p-y response. Liquefied soil p-y curves shall be estimated using the static API sand model reduced by a p-multiplier using the method of Brandenberg, et. al. (2007b) and Boulanger, et. al. (2003).

In general, there are two different approaches to estimate the lateral spread/slope failure induced load on deep foundations systems – a displacement based method and a force based method.

3.10.11.1 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading loads on deep foundation systems is presented in Boulanger, et. al. (2003) and Brandenberg, et. al. (2007a and b). Deep foundations in liquefied, lateral spreading ground shall be designed to resist lateral forces imposed on the pile by the lateral spreading ground. LPILE or similar computer programs can be used to perform this analysis. The design steps that consider the kinematic loading from the liquefaction-induced lateral spreading ground are presented in Boulanger et. al. (2007a and b).

The designer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The designer shall also consider the requirements in the Track Design Guidelines TM 2.1.5.

3.10.11.2 Force Based Approaches

A force based approach to assess lateral spreading induced loads on deep foundations is based on back-calculations from pile foundation failures caused by lateral spreading. The pressures on pile foundations shall be evaluated for design as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress), and
- Non-liquefied crustal layers exert full passive pressure on the foundation system

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that this is an adequate design method (Finn and Fujita, 2004). The force-based approach is appropriate where larger displacements occur that can mobilize the full passive pressure against the foundation. Where smaller displacements occur, the displacement-based approach shall be considered and may be more appropriate.

Another force-based approach to estimate lateral spreading induced foundation loads is to use a limit equilibrium slope stability program to determine the load the foundation must resist to achieve a target safety factor of 1.1. This force is distributed over the foundation in the liquefiable zone as a uniform stress. This approach may be utilized to estimate the forces that foundation elements must withstand if they are to act as shear elements stabilizing the slope.

3.10.12 Evaluation of P-Y and T-Z Springs for Seismic Analysis

Geotechnical and structural engineering guidance for seismic analysis using P-Y and T-Z ‘springs’ will be prepared for use during the Final Design.

3.10.13 Evaluation of Foundation Dynamic Stiffness and Damping

Geotechnical and structural engineering guidance for seismic analysis considering foundation dynamic stiffness and damping will be prepared for Final Design.

3.10.14 Dynamic Soil Pressures on Earth Retaining Structures

All retaining walls, abutment walls, and basement walls shall be evaluated and designed for seismic stability internally and externally (i.e. sliding and overturning). With regard to overall seismic slope stability (often referred to as global stability) involving a retaining wall, with or without liquefaction, the geotechnical designer shall evaluate the potential for failure and its impacts on performance. If unacceptable performance of the wall is likely during the design seismic event, the stability of the wall shall be improved such that the life safety during the design seismic event is preserved.

For retaining walls that are not restrained from rotation at the top and contain cohesionless materials as backfill, seismic pressures shall be estimated using the Mononobe-Okabe (M-O) method. Horizontal seismic coefficient (k_h) shall be taken as ½ of the peak ground acceleration value (PGA). For 15% design, the PGA value shall be estimated for the MCE level event as presented in TM 2.10.4. For the 30% design phase and also final design, PGA values associated with three performance levels shall be used. The earth pressures shall be separated into the incremental seismic pressures and the active earth pressures in the following manner:

$$\Delta K_{AE} = K_{AE} - K_A$$

where

ΔK_{AE} = Incremental seismic pressure coefficient

K_{AE} = Total seismic pressure coefficient

K_A = Active pressure coefficient

The incremental seismic earth pressure shall be taken as inverted triangle with the resultant acting at 0.65H from the bottom. This pressure shall be added to the active earth pressure for the design. It should be noted that seismic pressures increase significantly with slight increase in slope of the backfill. For higher angles of sloping back fills, the M-O solution will not converge. For those cases, methods presented in Chapter 7 of the NCHRP Report 611 shall be utilized. For backfill materials consisting of cohesive or cohesive and frictional (c-φ) material, methods presented in Chapter 7 of the NCHRP Report 611 shall be used.

For basement walls (or walls restrained against rotation) in locations where PGA values are less than or equal to 0.25g, walls shall be designed for only at-rest pressures and additional seismic loads shall not be considered. For higher PGA values, the higher of the at-rest pressures or the active plus M-O pressures shall be used for the design. Seismic coefficient value of ½ of the PGA shall be used in calculations.

3.10.15 Seismic Settlement of Unsaturated Soils

Seismically induced settlement of unsaturated granular soils (dry sands) shall be estimated using procedures provided by Tokimatsu and Seed (1987). Estimated values in terms of total and differential settlements shall be reported.

The designer shall compare the estimated settlement values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The designer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

3.10.16 Seismic Slope Stability and Deformation Analyses

Instability of slopes during seismic loading could be due to liquefaction or due to inertial loading or a combination of both. In this section instability of both the natural existing slopes and embankment slopes is addressed.

The designer shall compare the estimated deformation values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The designer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

3.10.16.1 Slope Instability Due to Liquefaction

Slopes could fail or experience deformations due to liquefaction either in the form of lateral spreading or flow failures. Liquefaction induced lateral spreading has been addressed in Section 6.10.8.

3.10.16.2 Liquefaction Induced Flow Failure

Liquefaction leading to catastrophic flow failures driven by static shearing stresses that result in large deformation or flow shall also be addressed by designers. These flow failures may occur near the end of strong shaking or shortly after shaking and shall be evaluated using conventional limit equilibrium static slope stability analyses. The analysis shall use residual undrained shear strength parameters for the liquefied soil assuming seismic coefficient to be zero (i.e., performed with k_h and k_v equal to zero). The residual strength parameters estimated using the method presented in Section 10.7 shall be used. In addition, 20-percent reduced strength of the normally consolidated clayey layers shall be used, ands strength reductions shall be considered for saturated sandy layers where excess pore water pressure is generated but full liquefaction does not occur. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium factor of safety, FS, is less than 1.0, flow failure shall be considered likely. Liquefaction flow failure deformation is usually too large to be acceptable for design of structures, and some form of mitigation will likely be needed. However, structural mitigation may be acceptable if the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads.

If the factor of safety for this decoupled analysis is greater than 1.0 for liquefied conditions, yield acceleration (k_y) values shall be estimated using pseudo-static slope stability analysis. The same strength parameters as used during the flow failure analysis shall be used. A new critical failure plane shall be searched assuming both circular and non-circular failure surfaces. Yield acceleration is defined as the minimum horizontal acceleration in a pseudo-static analysis for which FS is 1.0. Using the estimated k_y values, deformations shall be estimated using simplified methods such as Makdisi and Seed (1978) and Bray and Travasarou (2007). These simplified methods are not directly applicable to slopes with liquefiable layers, however, they provide a good estimate of the range of deformations expected during the seismic event. Other methods such as Newmark time history method or more advanced methods involving numerical analysis may also be used. If advanced methods are used, the results shall be checked against the simplified methods.

For pseudo-static analyses to estimate k_y values, residual strengths for the liquefied layers and reduced strengths for normally consolidated clayey and saturated sandy layers with excess pore water pressure generation (as described earlier) shall be used. This is generally a conservative approach but is appropriate for preliminary engineering design. For final design, more advanced methods involving numerical analyses may be used to better characterize the initiation of liquefaction and pore pressure generation and subsequent reduction in strength.

3.10.16.3 Slope Instability Due to Inertial Effects

Pseudo-static slope stability analyses shall be used to evaluate the seismic stability of slopes and embankments due to inertial effects. The pseudo-static analysis consists of conventional limit equilibrium slope stability analysis with horizontal (k_h) that act upon the critical failure mass. A horizontal seismic coefficient, k_h, of ½ PGA and a vertical seismic coefficient, k_v, equal to zero shall be used for the evaluation of seismic slope stability. For these conditions, the minimum required factor of safety is 1.1. Alternately, pseudo-static analyses may be performed to estimate k_y values. There is a debate in literature whether the slope failure plane during the pseudo-static analysis should be fixed based on the results of static analyses or a new failure plane is searched. A new failure plane shall be searched for the pseudo-static analysis. The analysis shall look for both circular and non-circular failure surfaces.

3.10.16.4 Deformations

Deformation analyses shall be performed where an estimate of the magnitude of seismically induced slope deformation is required, and the pseudo-static slope stability factor of safety is less than 1.0. Acceptable methods of estimating the magnitude of seismically induced slope deformation include Newmark sliding block (time history) analysis, simplified displacement charts and equations based on Newmark-type analyses (Makdisi and Seed, 1978; Saygili and Rathje, 2008; and Rathje and Saygili, 2008; Bray and Travasarou, 2007), or dynamic stress-deformation models. These methods shall not be employed to estimate displacements if the post earthquake static slope stability factor of safety using residual strengths is less than 1.0, since the slope will be unstable against static gravity loading and large displacements would be expected.

Seismically induced slope deformations can be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, FLAC, and OpenSees. The accuracy of these models is highly dependent upon the quality of the input parameters and the level of model validation performed by the user for similar applications. As the quality of the constitutive models used in dynamic stress-deformation models improves, the accuracy of these methods will improve. A key benefit of these models is their ability to illustrate mechanisms of deformation, which can provide useful insight into the proper input for simplified analyses. In general, dynamic stress deformation models shall not be used for routine design due to their complexity, and due to the sensitivity of deformation estimates to the constitutive model selected and the accuracy of the input parameters. If dynamic stress deformation models are used, they should be validated for the particular application. Use of dynamic stress-deformation models for design shall be approved by the EMT Geotechnical Engineer.

3.10.17 Downdrag Loading (Dragload) on Structures Due to Seismic Settlement

This sub-section to be prepared for a future version of TM.

3.11 GROUND IMPROVEMENT

This section to be prepared for use during 30% design phase.

3.12 OTHER GEOTECHNICAL TOPICS

This section to be prepared for use during 30% design phase.

4.0 SUMMARY AND RECOMMENDATIONS

4.1 GENERAL

Geotechnical guidance to be used for the 30% level design of CHSTP features is presented in Section 6 of this technical memorandum.

5.0 SOURCE INFORMATION AND REFERENCES

The development of the Geotechnical analysis requirements was based on a review and assessment of available reference documents, including the following:

1. American Association of State Highway and Transportation Officials (AASHTO) LFRD Bridge Design Specifications, 4th Edition, 2007
2. AASHTO LFRD Bridge Design Specifications with California (Caltrans) Amendments
   - Section 3 “Loads and Load Factors
   - Section 11 “Abutments Piers and Walls
3. American Society for Testing Materials
4. Federal Highway Administration GEC 3
   - FHWA-IF-02-034, Evaluation of Soil and Rock Properties
   - FHWA-SA-02-054 (GEC 6), Shallow Foundations, Sept. 2002
   - FHWA-NHI-05-042/043 “Design and Construction of Driven Pile Foundations – Volumes I and II,” Apr. 2006
   - FHWA-NHI-05-094 “LRFD for Highway Bridge Substructures and Earth Retaining Structures,” Jan. 2007
   - FHWA-IF-99-025 “Drilled Shafts – Construction Procedures and design Methods,” Aug. 1999
   - FHWA’s Earth Retaining Structures Reference manual (FHWA, 2008)
   - FHWA-NHI-07-071
   - FHWA-NHI-10-024/25 “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,” Volumes I and II, Nov. 2009
   - FHWA-NHI-00-044 “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes ,” Sept. 2009
   - FHWA 132036A – Earth Retaining Structures
   - FHWA’s Ground Improvement Reference manuals Volumes I and II
   - FHWA-NHI-06-019/020, 2006
5. Geotechnical Engineering Circular No. 5 (GEC 5)
6. CHSTP TM 2.9.1 Geotechnical Investigation and Laboratory Testing Guidelines, R)
7. CHSTP TM 2.9.2 Geotechnical Report Preparation Guidelines, R0
8. CHSTP TM 2.9.3 Geotechnical and Seismic Hazard Evaluations Guidelines, R0
9. CHSTP TM 2.3.2 Structure Design Loads, R1
10. CHSTP TM 2.1.5 Track Design, R0
11. CHSTP TM 2.10.10 Track-Structure Interaction, R0
12. CHSTP TM 2.10.4 Interim Sesimic Design Criteria,R0
13. CHSTP TM 2.9.6 Interim Ground Motions, R0
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21. Matasovic (et al., 2007) Validation of Generic Municipal Solid Waste Material Properties for Seismic Design of Landfills
22. Tokimatsu, K. and Seed, H.B. (1987), "Evaluation of Settlements in Sand Due to Earthquake Shaking," Journal of Geotechnical Engineering, ASCE. Vol. 113, No. 8, pp. 861-878.
23. Ishihara, K. and Yoshimine, M., "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes," Soils and Foundations, JSSMFE, Vol. 32, No. 1, March, 1992
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27. Martin et al., (1991)
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41. Bray, J. D., and Travasarou, T., 2007, Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 4, pp. 381-392.

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43. NCHRP Report 611, Chapter 7

6.0 DESIGN MANUAL CRITERIA

Guidance for geotechnical criteria and analysis in support of preliminary engineering during the preliminary design phase follows. The analyses for these topics shall be performed following generally accepted geotechnical engineering principles and procedures adapted to the CHSTP as described herein.

6.1 GENERAL

Geotechnical criteria prescribed herein address the design, methodology, assumptions, and analytical procedures, as well as any acceptable standards in terms of expected performance of infrastructure facilities and integrity of the final design. Subject to the restrictions imposed by licensing laws in the state of California, designs shall be completed only by California-licensed geotechnical engineers, geologists, and engineering geologists.

6.1.1 Data Evaluation and Geotechnical Analysis

Elements of geotechnical analyses and design criteria subjected to these guidelines and standards shall include; (1) data interpretation, (2) data analysis and modeling, and (3) geotechnical design calculations.

6.1.2 Geohazard Studies

TM 2.9.3 provides guidelines to identify and evaluate geologic and seismic hazards for input to project design criteria. The geotechnical engineer shall incorporate the findings of the geologic and seismic hazard studies into the geotechnical design.

6.1.3 Geotechnical Report Requirements

The requirements for the content and format of Geotechnical Reports described in TM 2.9.2 shall be used by the geotechnical engineer for all geotechnical design documents.

6.2 GEOTECHNICAL CHARACTERIZATION

6.2.1 General

Characterization of surface and subsurface conditions shall be performed in three dimensions based on plans and profiles depicting subsurface units with unique properties and the associated geotechnical engineering properties. This geotechnical model shall then be refined into a surface/subsurface engineering domain model based on the unique design elements. The following sections describe the guidelines for the development of the engineering model to promote consistency and to meet project-specific requirements.

6.2.2 Geotechnical Investigation

Geotechnical investigations shall be performed in accordance with TM 2.9.1. Recommendations for subsurface exploration methods, in-situ testing, and laboratory testing of specimen samples as part of geotechnical investigations shall be provided on the basis of these guidelines. In addition to discussion of soil and rock identification, testing, description, and classification, this technical memorandum contains guidelines that present the process and protocol for interpretation of subsurface conditions for use during geotechnical analyses supporting engineering design activities.

6.2.3 Soil and Rock Classification

Soil shall be characterized and classified using ASTM D 2488 guidelines for field classification and ASTM D 2487 based on laboratory test results. Rock shall be classified using GEC 3 (FHWA, 2002) which is largely based on ISRM guidelines. Rock and other formational materials, e.g., very soft rock and intermediate geotechnical materials shall also be identified with the name of the geologic formation.

6.2.4 Laboratory Test Requirements

Standards for laboratory testing of soil and rock are described in TM 2.9.1.

6.2.5 Geotechnical Characterization Model

This section identifies appropriate methods and technical references to be used for soil and rock property assessment, and how to use the soil and rock property data to establish the final soil and rock parameters to be used for geotechnical design.

6.2.5.1 Geologic Model

The geologist shall develop a geologic model based on applicable existing data such as geologic maps, aerial photography, published literature, and existing subsurface data. The model shall be refined using field reconnaissance, remote sensing, and mapping methods. The geologic model shall be used to prepare a surface geologic map and a corresponding subsurface profile along the HST alignment. The map and profile shall be accompanied by cross-sections perpendicular to the alignment where needed to reveal the three dimensional configuration of the subsurface conditions. Maps, profiles, and cross-sections shall also depict the related design elements (structures, embankments, cuts, etc.) of the CHST project. The geologic model shall serve as a fundamental tool to develop the subsurface exploration plan for the CHST, and shall be updated as project-specific information is obtained. Subsurface conditions shall be presented in plan and profile and also accompanied by cross-sections perpendicular to the alignment where needed to fully depict the three dimensional configuration of these units. Subsurface logs, in-situ test results, and laboratory testing shall be used for further refinement of units and groundwater conditions having unique engineering properties as they relate to geotechnical analyses. Units having similar engineering properties but unique geologic description shall only be differentiated if it is beneficial to the interpretation of stratigraphy between data points.

Uncertainties in the development of subsurface condition profiles indicate the need for additional explorations or testing. Because of the diverse nature of the geologic processes that contribute to soil formation, actual subsurface profiles can be extremely varied both vertically and horizontally, and can differ significantly from interpreted profiles developed from boring logs. Therefore, subsurface profiles developed from boring logs shall contain some indication that the delineation between strata do not necessarily suggest that distinct boundaries exist between the strata or that the interpolations of strata thickness between borings are necessarily correct. The main purpose of subsurface profiles is to provide a starting point for design and not necessarily to present an accurate description of subsurface conditions.

6.2.5.2 Geotechnical Model

The geotechnical engineer shall develop a geotechnical model based on the geologic model and subsurface information collected for the project. As field and laboratory test data become available, engineering properties for each of the unique units shall be developed and portrayed on the geotechnical model (map, profile, and cross-sections). These engineering properties must effectively document and support all geotechnical analyses and designs.

The geotechnical model shall represent the geologist and geotechnical engineer's interpretation of all available subsurface data, and shall include (at a minimum) the following:

- Interpreted boundaries of soil and rock
- Average physical properties of the soil layers (unit weight, shear strength, etc.)
- Visual description of each layer including USCS symbols for soil classification
- Location of the ground water (see next section), and
- Notations for special items (boulders, artesian pressure, known buried infrastructure, etc.)

Complementary tables shall be developed to accompany the geotechnical model (map, profile, and cross sections), in order to reduce visual clutter and aid the user. As described in TM 2.9.1, CHSTP will make use of electronic records for borings, CPTs, etc. An appropriately developed database and GIS shall be used to great advantage for data management, analyses (in support of engineering design), and construction.

6.2.5.3 Groundwater Conditions

The geologist and geotechnical engineer shall evaluate groundwater conditions and establish water levels/elevations for use in facility design and construction planning. Guidance pertaining to collecting and interpreting hydrogeologic field data is contained in TM 2.9.1. Important factors that shall be considered in groundwater characterization include:

- Hydrostatic or flowing groundwater conditions
- Whether aquifers are confined or unconfined
- The upper and lower limits and slope of the aquifer
- Aquifer characteristics (soil type and permeability, rock discontinuities)
- Presence (and influence) of perched groundwater table conditions
- Potential for raised or lowered groundwater level during project design-life, and
- Possibility for artesian conditions

Due to the variability in aquifer storage characteristics and response to rainfall, the groundwater conditions to be used for analysis and geotechnical design shall be based on water levels measured in the field, coupled with hydrograph information describing historic water level trends. For sites where there is no groundwater data available, the “wetting band” approach (FHWA, 2005) should be used to provide an estimate of reasonable groundwater level.

Groundwater conditions are especially relevant for slope design. The water level of a specified return period shall be determined using one of the following approaches:

1. Analysis of piezometric data taken before, during and after rainfall. Various methods are available for estimating water levels from piezometric records, including the statistical correlation of groundwater response with rainfall, groundwater modeling of the aquifer system, and the extrapolation of observed piezometric responses.
2. Solution of the equation describing the formation of a wetting band zone of 100 percent saturation (FHWA, 2005). The geologist and geotechnical engineer shall consider all relevant hydrogeologic aspects for the slope stability analyses, especially:
   - The highest anticipated phreatic (groundwater) surface for an unconfined aquifer and/or piezometric surface for a confined aquifer,
   - The height of the groundwater at the time of failure (for an existing failure),
   - The proximity of the aquifer to the existing or potential failure surface, and
   - The presence and influence of seepage, pore pressure conditions, tension cracks, runoff, and surface drainage patterns.

6.2.6 Soil and Rock Properties and Parameters

6.2.6.1 General

Soil and rock properties shall be measured and interpreted using the guidelines provided in GEC 5 (FHWA, 2002), except as specifically indicated herein. The process for soil and rock property selection is illustrated graphically in flow-chart format in Figure 1, Chapter 2 of GEC 5. Additional guidelines that shall be considered are summarized in Section 10 of AASHTO LRFD (2007).

Correlations for soil properties as provided in GEC5 may be used if the correlation is well established and if the accuracy of the correlation is considered regarding its influence on the estimate obtained from the correlation in the selection of the property value used for design. Local geologic formation-specific correlations may also be used if well established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question. Correlations shall not be used as a substitute for an adequate subsurface investigation program, but rather to complement and verify specific project-related information.

6.2.6.2 Rock Properties

The engineering properties for rock shall account for the properties of the intact rock and for the discontinuities within the rock mass. A combination of laboratory testing, empirical analysis, and field observations shall be employed to evaluate the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

GEC 5 shall be used to assess the design properties for the intact rock and the rock mass as a whole. However, GEC 5 shall **not** be used to develop fractured rock mass shear strength parameters. Instead, the geologist and geotechnical engineer shall use the updated procedures proposed by Hoek, et. al. (2002). This method is only to be used for highly fractured rock masses in which the stability of the rock slope is not structurally controlled.

6.2.6.3 Geotechnical Engineering Parameters

The geotechnical engineer shall evaluate the validity and reliability of the data and its usefulness in selecting final design parameters. After a review of data reliability, a review of the variability of the selected parameters shall be carried out. Variability is typically introduced in two ways: 1) natural heterogeneity within the unit, and 2) test method selection or execution.

Inconsistencies in data shall be evaluated and the need for mitigation procedures may be warranted to correct or exclude any questionable data. The geotechnical engineer shall comply with GEC 5, which provides guidance for analyzing data and resolving inconsistencies. The geotechnical engineer shall also use GEC 5 to assess variability for a given engineering property in a particular geologic unit, and how that variability should influence the selection of the final design values.

Evaluations of geotechnical engineering parameters shall consider how the parameters could change over the design life of the structure. Changes may occur as a result of weathering, groundwater level changes, increase in stress due to fill or foundation loads, decrease in stress due to excavation, or other factors.

Geotechnical evaluations for design shall keep in mind that resistance factors have been developed assuming mean values for soil properties. However, design values that are more conservative than the mean may still be appropriate, especially if there is an unusually level of uncertainty associated with the design property. Depending on the availability and variability of ground conditions, it may not be possible to reliably estimate an average value for design. In this case, the geotechnical engineer shall select a more conservative value. For those resistance factors that were evaluated based on calibration by “fitting” to allowable stress design, property selection shall be based on the considerations discussed previously.

6.3 AERIAL TRACKWAY STRUCTURE AND BRIDGE FOUNDATIONS

6.3.1 General

The design shall indicate the proposed structure type and function and proposed locations of foundation elements, including foundation loads. Structure type and loads shall comply with TM 2.3.2. Special performance requirements, such as unique or unusual displacement limitations, shall be considered in the design. Geotechnical site characterization shall be adequately advanced to support the design, and geologic and seismic hazards that affect the proposed structures shall have been identified.

Construction limitations that could affect foundation design shall be identified. These include local availability of equipment, equipment access limitations, staging restrictions, right-of-way restrictions, permit requirements, proximity to sensitive structures, and proximity to sensitive utilities.

6.3.2 Foundation Type Selection

Foundation selection shall consider the following:

- The ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states, given the soil or rock conditions encountered,
- Consideration of flooding and scour, where applicable,
- Consideration of frost depth, where applicable,
- The constructability of the foundation type,

- The impact of the foundation installation (in terms of time and space required) on existing facilities and right-of-way,
- The environmental impact of the foundation construction,
- Physical constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities), and
- The impact of the foundation on the performance of adjacent foundations, structures, or utilities, considering both the design of the adjacent foundations, structures, or utilities, and the performance impact the installation of the new foundation will have on these adjacent facilities; and the cost of the foundation, considering all of the issues listed above.

Shallow spread footings shall be used for foundation support where competent soil or rock is present within relatively shallow depths. Shallow footings may also be appropriate where ground improvement is performed to poor soils to improve their strength and stiffness characteristics, provided that performance requirements are met. Shallow footings are typically not appropriate for soils that are soft, loose, expansive, prone to hydro-collapse, liquefiable, or prone to excessive scour.

Where spread footings are not feasible or cost effective, deep foundations shall be used. Two general types of deep foundations are typically considered: pile foundations, and drilled shaft (or cast-in-drilled-hole, CIDH) foundations. Shaft foundations can be advantageous where pile driving may be precluded by the presence of obstructions such as dense layers, boulders, or fill with debris. Shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam or shoring is required to construct the pile cap. Shafts may not be desirable where contaminated soils are present, because of the associated handling and disposal requirements. Shafts shall be considered in lieu of piles where pile driving vibrations could cause damage or unacceptable disturbance or disruption to existing adjacent facilities. Piles may be more cost effective than shafts where pile cap construction is relatively easy, or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The stability of soils during shaft construction and the need for casing shall also be considered when choosing between driven piles and drilled shafts.

6.3.3 LRFD Overview for Foundations

Except where noted herein, foundation design shall be performed in accordance with the AASHTO LRFD Bridge Design Specifications with California (Caltrans) Amendments, Customary U.S. Units, latest edition, as adapted and modified by this and other technical memoranda.

6.3.4 LRFD Loads, Load Groups and Limit States

LRFD loads, load groups and limit states for aerial viaduct and bridge structure design are defined in TM 2.3.2. Earth loads are listed below and shall be calculated by the geotechnical engineer in accordance with Section 3.11 of AASHTO LRFD.

Table 6.3.6-1 Summary of Earth Loads

CHST Load Abbreviation	AASHTO LRFD Load Abbreviation and (Section)	Load Type Description
EV	EV (3.5.1)	Vertical earth pressure from dead load of fill
EHAR	EH (3.11.5.2)	Horizontal earth pressure load for at-rest condition
EHAC	EH (3.11.5.3)	Horizontal earth pressure load for active condition
ESET	DD (3.11.8)	Earth settlement effects
EHS	ES (3.11.6.2 and 3.11.6.3)	Earth surcharge due to live loads

Service, Strength, Buoyancy, and Extreme Event Limit States used for design of foundation for aerial viaduct and bridge structures shall be as defined in TM 2.3.2.

At a minimum, foundations shall be designed and proportioned for the following Limit States and mechanisms:

Service Limit State:

- Settlement,
- Lateral deflection,
- Stability (including slope stability), and
- Scour at the design flood

Strength Limit State:

Spread Footings:

- Nominal bearing resistance
- Overturning or excess loss of contact
- Sliding at the base of the footing, and
- Constructability

Driven Piles:

- Axial compression resistance for single piles
- Pile group compression resistance
- Uplift resistance for single piles
- Uplift resistance of pile groups
- Pile punching failure into a weaker stratum below the bearing stratum (where applicable)
- Single pile and pile group lateral resistance, and
- Constructability (including pile drivability)

Drilled Shafts:

- Axial compression resistance for single drilled shafts
- Shaft group compression resistance
- Uplift resistance for single shafts
- Uplift resistance of shaft groups
- Single shaft and shaft group lateral resistance
- Shaft punching failure into a weaker stratum below the bearing stratum (where applicable), and
- Construability (including methods of shaft construction)

Micropiles:

- Axial compression resistance for single micropile
- Micropile group compression resistance
- Uplift resistance for single micropiles
- Uplift resistance of micropile groups
- Micropile group punching failure into a weaker stratum below the bearing stratum, and single Micropile punching failure where tip resistance is considered
- Single and group micropile lateral resistance, and
- Constructability (including methods of micropile construction)

Extreme Event Limit State:

For the Extreme Event Limit State, foundations shall be designed for the cases indicated above for Strength Limits State Analyses (as applicable) but with appropriate Extreme Event load and resistance factors. In addition, where applicable, foundations shall be designed to withstand earth loading due to lateral spreading or seismically-induced slope displacements. Refer to Section 6.10 for further requirements, including assessment of earth loading due to lateral spreading or seismically-induced slope displacements.

6.3.5 Tolerable Foundation Settlement and Displacements

Requirements for tolerable foundation settlements and displacements presented herein shall supersede criteria indicated in AASHTO LRFD Bridge Design Specifications and the California Amendments. For deep foundations, tolerable settlements or displacements are measured at the top of the foundation: the pile cap, pile head, or the ground surface for drilled shaft pier-extensions. Limiting values for allowable deformations that are based on tolerable movements for the proposed bridges and tracks are in development. Table 6.3.5-1 presents preliminary tolerable settlement or displacement criteria. These criteria are subject to change.

TM 2.1.5 indicates that the tolerance of fasteners for the track can accommodate no more than 3 inches of vertical displacement based on the ability to adjust the fasteners spaced at intervals of 24 to 30 inches apart. Further performance requirements for allowable deformations are prescribed in TM 2.10.10.

Table 6.3.5-1 Tolerable Foundation Vertical Settlement / Displacement Criteria

Limit State	Structure Type	Tolerable Settlement / Displacement	Comment
Service	Abutments	≤ 0.75 inch Settlement ≤ 0.375 inch Horizontal ≤ 0.0006 radians Angular Distortion	
Service	Bents/Piers	≤ 0.75 inch Settlement ≤ 0.375 inch Horizontal ≤ 0.0006 radians Angular Distortion	
Strength	All	Not applicable	Settlements and displacements need not be evaluated for the Strength Limit State
Extreme Event	Abutments	OPL[1]: ≤ ¼ inch Settlement ≤ ¼ inch Horizontal ≤ 0.0004 radians Angular Distortion SPL[2]: ≤ 1 inch Settlement ≤ ½ inch Horizontal ≤ 0.0008 radians Angular Distortion NCL[3]: ≤ 3 inches Settlement ≤ 3 inches Horizontal ≤ 0.0015 radians Angular Distortion	Extreme Event displacements defined in this table are permanent displacements following the cessation of ground shaking.
Extreme Event	Bents/Piers	OPL[1]: ≤ ¼ inch Settlement ≤ ¼ inch Horizontal ≤ 0.0004 radians Angular Distortion SPL[2]: ≤ 1 inch Settlement ≤ ½ inch Horizontal ≤ 0.0008 radians Angular Distortion NCL[3]: ≤ 3 inches Settlement ≤ 3 inches Horizontal ≤ 0.0015 radians Angular Distortion	Extreme Event displacements defined in this table are permanent displacements following the cessation of ground shaking.

Notes:
1. OPL = Operability Performance Level
2. SPL = Safety Performance Level
3. NCL = No Collapse Performance Level
4. Refer to TM 2.10.4 Interim Seismic Design Criteria regarding seismic design philosophy and requirements for the performance levels.

The settlements and displacements noted in the table above are considered minimum performance criteria. Designers may elect to use more stringent criteria. Structural designers may require that foundations be designed to more stringent criteria for certain structures depending upon specific performance requirements, especially for the NCL performance level.

6.3.6 Resistance Factors for Foundation Design

Resistance factors for foundation design shall be consistent with those defined in the most current version of the AASHTO LRFD Bridge Design Specifications with California Amendments, Section 10.5.

6.3.7 Shallow Foundations

Geotechnical engineering analyses as well as structural designs for shallow foundations shall be performed in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Section 10.6. Shallow foundation guidelines that shall be considered for geotechnical design are summarized in GEC 6 (FHWA, 2002), and "LRFD for Highway Bridge Substructures and Earth Retaining Structures," (FHWA, 2007).

6.3.8 Driven Piles and Drilled Shafts

Geotechnical engineering analyses and structural designs for driven piles and drilled shafts shall be in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10.7, 10.8 and 10.9. Deep foundation guidelines that shall be considered for geotechnical design are summarized in "Design and Construction of Driven Pile Foundations – Volumes I and II" (FHWA, 2006), "Drilled Shafts - Construction Procedures and Design Methods" (FHWA, 1999), and "LRFD for Highway Bridge Substructures and Earth Retaining Structures" (FHWA, 2007).

6.3.9 Proprietary Foundation Systems

This section is to be prepared for final design.

6.3.10 Abutments and Abutment Foundations

Bridge abutments have components of both foundation design and retaining wall design. The retaining wall aspects of abutments shall be designed in accordance with Section 6.7 of this TM, and also Section 11 of the AASHTO LRFD Bridge Design Specifications. Foundations for abutments shall be designed in accordance with AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10 and 11. Abutment foundation guidelines that shall be considered for geotechnical design are summarized in "LRFD for Highway Bridge Substructures and Earth Retaining Structures" (FHWA, 2007).

6.3.11 Seismic Analysis and Design

Foundations and abutments shall be designed for the Extreme Event I seismic case. Seismic design procedures for foundations and abutments are addressed in Section 6.10.

This section is to be prepared for 30% design.

6.4 FOUNDATIONS FOR BUILDINGS AND OTHER AT-GRADE STRUCTURES

This section is to be prepared for 30% design.

6.5 TUNNELS AND OTHER UNDERGROUND STRUCTURES

This section is to be prepared for 30% design.

6.6 TRACK BED EMBANKMENTS AND EMBANKMENT FOUNDATIONS

This section is to be prepared for 30% design.

6.7 RETAINING WALLS, FILL WALLS, AND REINFORCED EARTH SYSTEMS

6.7.1 Definitions and Wall Types Including Acceptable and Unacceptable Walls

Walls shall be classified as either a "fill wall" or a "cut wall." Examples of fill walls include standard cantilever walls, Mechanically Stabilized Earth (MSE) walls, and modular gravity walls (gabions, bin walls, and crib walls). Cut walls include soil nail walls, cantilever soldier-pile walls, and ground anchored walls (other than nail walls).

Walls shall be further classified as gravity, semi-gravity, non-gravity cantilever, anchored, or in-situ reinforced. For geotechnical design, the various wall classifications, definitions and additional detail are provided in Section 11 of AASHTO LRFD-BDS, and FHWA's Earth Retaining Structures Reference Manual (FHWA 2008). For CHSTP, each of these wall categories will be considered as "generally acceptable" walls provided that the combined earth/structural system meets all of the design and performance criteria. Wall types considered to be "unacceptable" include mortar rubble gravity walls, timber or metal bin walls, and "rockery" walls.

6.7.2 Design Considerations

Retaining wall and slope designs shall be coordinated with other project design elements that might interfere with or impact the design or construction of the wall or slope. This includes coordination with the Structures and Civil Design Discipline, Systems Discipline, and Hydrology and Hydraulics Disciplines to select the most appropriate earth retention system for a given setting based on design constraints, geotechnical subsurface investigations, and surface and groundwater issues. Consideration must be given to the presence of (and potential conflicts with) drainage features; buried and overhead utilities; lighting or sign structures; adjacent retaining walls or bridges; concrete traffic barriers and/or fences; and guardrails. These design elements shall be located in a manner that will minimize the impacts to the retaining wall or reinforced slope elements. The potential effect that site constraints might have on the constructability of the specific wall/slope shall be considered. Additional constraints to be considered include but are not limited to site geometry, access, time required to construct the wall, environmental issues, and impact on traffic flow and other construction activities.

The structural elements of the wall or slope and the soil below, behind, and/or within the structure shall be designed together as a system. The wall or slope system shall be designed for overall external stability as well as internal stability. Overall external stability includes stability of the slope the wall/reinforced slope is a part of and the local external stability (overturning, sliding, and bearing capacity). Internal stability includes resistance of the structural members to load and, in the case of MSE walls and reinforced slopes, pullout capacity of the structural members or soil reinforcement from the soil.

Geotechnical Investigation - Retaining wall and RSSs require subsurface data representative of the underlying soil/rock that supports the structure. The stability and support characteristics of the underlying soils, their potential to settle under the imposed loads, the usability of any existing excavated soils for wall/reinforced slope backfill, and the location of the groundwater table shall be evaluated through the geotechnical investigation.

For wall and/or RSS type selection, factors that must be considered include the intended application; the soil/rock conditions in terms of settlement; need for deep foundations; constructability; impacts to traffic; and the overall geometry in terms of wall/slope height and length, location of adjacent structures and utilities, aesthetics, and cost.

Other considerations that wall/slope selection is dependent upon shall include:

- Wall/slope will be located primarily in a cut or fill
- Excavation/shoring will be required to construct the wall or slope
- Type of soil/rock present
- Need for space between the right of way line and the wall/slope or easement
- Amount of settlement expected
- Potential for deep failure surfaces to be present
- Structural capacity of the wall/slope in terms of maximum allowable height
- Nature of the wall/slope application
- Structures or utilities will be located on or above the wall
- Architectural requirements, and
- Overall economy

For “type selection” purposes, geotechnical design shall consider the summary of various wall/slope options available (including their advantages, disadvantages, and limitations) provided in FHWA-NHI-07-071. Specific wall types shown in the exhibits of FHWA-NHI-07-071 may represent multiple wall systems, some or all of which will be proprietary. There are a number of factors that control wall type selection and design considerations, including:

- Magnitude and direction of loading
- Depth to suitable bearing materials (foundation support)
- Potential for earthquake loading and liquefaction
- Proximity of physical constraints
- Tolerable total and differential settlement
- Facing durability and aesthetics
- Ease and cost of construction
- Potential for undermining or scour, swelling potential (clay soil, and frost depth), and
- Cross sectional wall/slope geometry

Wall/slope geometry is developed considering the following:

- Geometry of the transportation facility
- Design Clear Zone requirements
- Right of way constraints
- Existing ground contours
- Existing and future utility locations
- Impact to adjacent structures
- Impact to environmentally sensitive areas, and
- Also consider the foundation embedment and type anticipated.

Feasible retaining wall heights to be considered for geotechnical design are affected by issues such as the capacity of the wall structural elements, past experience with a particular wall, current practice, seismic factors, long-term durability, and aesthetics. Wall facing selection considerations are dependent on the aesthetic and structural needs of the wall system. Wall settlement may also affect the feasibility of the facing options. More than one wall facing may be available for a given system. The available facing options shall be considered when selecting a particular wall. Wall type selection and facing options are summarized in FHWA-NHI-07-071, Chapter 10.

The structure and adjacent soil mass must be stable as a system, and the anticipated wall settlement needs to be within acceptable limits.

6.7.3 Limit States and Resistance Factors

Geotechnical designs for retaining walls shall be performed in accordance with AASHTO LRFD Bridge Design Specifications. The LRFD process and example calculations for individual wall types are provided in FHWA-NHI-07-071. Section 11 of the AASHTO (2007) LRFD Specification provides information on LRFD for earth retaining structures including conventional retaining walls, nongravity cantilevered walls, anchored walls, mechanically stabilized earth (MSE) walls, and prefabricated modular walls. Publication number FHWA-NHI-05-094 “LRFD for Highway Bridge Substructures and Earth Retaining Structures" dated January 2007 contains comprehensive guidance on LRFD for retaining wall systems and abutments and shall be considered by the geotechnical engineer.

AASHTO LRFD load combinations for earth retaining systems and bridge substructures are provided in Tables 3.4.1-1 of AASHTO (2007). The load factors for permanent loads used for earth retaining systems are provided in Table 3.4.1-2 of AASHTO (2007). In general, minimum load factors shall be used if permanent loads increase stability and maximum load factors shall be used if permanent loads reduce stability. See AASHTO (2007) Section 3.3 for complete definition of loads. For reference purposes, the resistance factors for design of earth retaining walls are presented in Table 11.5.6-1 of AASHTO for LRFD, and so are not reprinted here.

6.7.4 External Loads and Stability Analysis

AASHTO LRFD shall be used for evaluation of stability for retaining walls and abutments. Retaining walls and abutments shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads. For wall evaluation and design, earth pressure shall be considered as a function of the following:

- Type and unit weight of the earth
- Water content
- Soil creep characteristics
- Degree of compaction
- Location of 'design' groundwater table
- Earth-structure interaction
- Amount of surcharge load
- Earthquake effects
- Back slope angle, and
- Wall inclination

Calculation methods for analysis of earth pressure and water/hydrostatic pressures, including consideration of the various factors listed above, are provided in Section 3, Loads and Load Factors, of current AASHTO LRFD BDS. Earth pressures used in design of walls and abutments shall be selected consistent with the requirement that the abutment movement shall not exceed tolerable displacement and settlement limits described in Section 6.7.7 of this technical memorandum. Analyses methods for application of these various pressures in retaining wall design and stability evaluation of wall and abutment structures are provided in Section 11, Abutments Piers and Walls, of current AASHTO LRFD BDS.

The provisions of AASHTO LRFD BDS Section 11, including methods of analyses/calculations for various wall types, shall be used for evaluation of stability for retaining walls and abutments. This includes analyses for overturning, bearing resistance, external stability (soil failure) and internal stability (safety against structural failure or combined soil-structure failure), sliding, seismic-load case, etc. Overall stability shall be evaluated using limit equilibrium methods of analysis. For global stability analysis of walls on steep slopes geotechnical design shall consider the initial stability of the slope and the impact (or lack of) that the proposed construction has on the slope.

6.7.5 Groundwater, Seepage, and Drainage Design

Adequate drainage behind all retaining walls and engineered slopes shall be included in the design and implemented during construction. Designs shall provide positive drainage at periodic intervals to prevent entrapment of water. Native soil may be used for retaining wall and reinforced slope backfill provided that it meets the requirements for the particular wall/slope system, and will satisfy long term deformation requirements particularly upon wetting.

Backfills behind retaining walls and abutments shall be drained, and drainage systems shall be designed to completely drain the entire retained soil volume behind the retaining wall face. If drainage cannot be provided due to site constraints, the abutment or wall shall be designed for loads due to earth pressure, plus full hydrostatic pressure due to water in the backfill.

For MSE walls and RSSs, internal drainage measures shall be considered for all structures to prevent saturation of the reinforced backfill and to intercept any surface flows containing corrosive elements. MSE walls in cut areas and side-hill fills with established groundwater levels shall be constructed with drainage blankets in back of, and beneath, the reinforced zone. In cut and side-hill fill areas, if prefabricated modular wall units are used, then the structure shall be designed with a continuous subsurface drain placed at, or near, the footing grade and outletted as required. In cut and side-hill fill areas with established or potential groundwater levels above the footing grade, a continuous drainage blanket shall be provided and connected to the longitudinal drain system. For systems with open front faces, a surface drainage system shall be provided above the top of the wall.

At locations where retaining walls or reinforced slopes can be in contact with water (such as a culvert outfall, ditch, wetland, lake, river, or floodplain), there is a potential risk of scour at the toe. This risk must be analyzed and mitigated for design and construction.

Where thin drainage panels are used behind walls and saturated or moist soil behind the panels may be subjected to expansion due to freezing, either insulation shall be provided on the walls to prevent freezing of the soil, or the wall shall be designed for the pressures exerted on the wall by frozen soil.

6.7.6 Seismic Analysis for Retaining Walls and Reinforced Earth Systems

Section 6.10 presents procedures for developing dynamic soil pressures for seismic analysis and design of retaining walls.

This section will be expanded for 30% design.

6.7.7 Settlement and Horizontal Deformation Tolerances

Settlement issues, especially differential settlement, are of primary concern in the selection of walls. Some wall types are inherently flexible and tolerate more settlement without poor structural performance. Other wall types are inherently rigid and cannot tolerate much settlement. The total and differential vertical deformation of a retaining wall shall be small for rigid gravity and semigravity retaining walls and shall meet structural and track tolerance performance requirements.

Retaining wall and abutment structures shall be investigated for excessive vertical and lateral displacement, and overall stability, at the service limit state. Tolerable vertical and lateral deformation limits for retaining walls and abutments shall be developed from the structural engineering design and performance criteria based on the function and type of wall, design service life (100 years), and consequences of unacceptable movements to the wall and any potentially affected nearby structures, i.e., both structural and aesthetic.

Vertical wall movements are primarily the result of soil settlement beneath the wall foundation. The provisions of AASHTO (Section 10) shall apply for analytical methods to estimate vertical wall movements. For gravity and semi-gravity walls, lateral movement estimates shall be assessed resulting from a combination of differential vertical settlement between the heel and the toe of the wall, and the rotation necessary to develop active earth pressure conditions. Tolerable total and differential vertical deformations for a particular retaining wall are dependent on the ability of the wall to deflect without causing damage to the wall elements or adjacent structures, or without exhibiting deformations that are unsightly and/or affect wall performance. Regarding impact to the wall itself, differential settlement along the length of the wall and to some extent from front to back of wall is the best indicator of the potential for retaining wall structural damage or overstress. Wall facing stiffness and ability to adjust incrementally to movement affect the ability of a given wall system to tolerate differential movements, and shall be evaluated by the geotechnical engineer.

For MSE walls, deflections shall be estimated in accordance with the provisions of AASHTO Section 11. MSE walls have the greatest flexibility and tolerance to total and differential vertical settlement, followed by prefabricated modular gravity walls. Reinforced soil slopes RSSs are also inherently flexible. For MSE walls, the facing type used can affect the ability of the wall to tolerate settlement, and shall be evaluated by the geotechnical engineer. Other factors to be considered include MSE wall configuration and timing of facing construction.

Semigravity (cantilever) walls and rigid gravity walls have the least tolerance to settlement. Therefore, semigravity cantilever walls, and rigid gravity walls shall not be used in settlement prone areas. If very weak soils are present that will not support the wall and are too deep to be overexcavated, or if a deep failure surface is present that results in inadequate slope stability, a wall type shall be selected that is capable of using deep foundation support and/or anchors. In general, MSE walls, prefabricated modular gravity walls, and some rigid gravity walls are not appropriate for these situations. Walls that can be pile-supported, such as concrete semigravity cantilever walls, nongravity cantilever walls, and anchored walls, are more appropriate for these situations. For anchored walls, downward movement can cause significant stress relaxation of the anchors and shall be considered for design. Anchored wall deflections shall be estimated in accordance with the provisions of AASHTO Section 11.

In evaluating settlement of retaining walls whose backfill supports train tracks, consideration shall be given to the time rate of settlement. To avoid excessive deflections in the track, track structures shall not be constructed until the majority of expected retaining wall settlement has already occurred, and been monitored and documented. In some cases, this may necessitate the use of added construction measures to expedite settlement such as surcharging or wick drains.

6.7.8 Design of Reinforced Soil Slopes (RSS) and Mechanically Stabilized Earth (MSE) Structures

Definitions for Reinforced Soil Slope (RSS) embankments and Mechanically Stabilized Earth (MSE) structures, as well as step-by-step design methodology and analyses that shall be used for MSE and RSS systems are provided in the LRFD version of FHWA's manual FHWA-NHI-10-024/25 "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes", Volumes I and II, dated November 2009. The RSS and MSE manuals also provide instructions for computer-aided analysis that shall be used for design. Numerous geosynthetic reinforcements and facing systems are available. The embankment fill may be either granular or cohesive material, however, granular fill materials are preferable and may be necessary in order to meet the various performance requirements.

The CHSTP may include non-standard proprietary wall systems (such as MSE) and non-standard non-proprietary wall systems (such as soil nail walls, anchored walls, reinforced slopes, etc.). From development of wall designs to the final wall product, all preliminary designs by the engineering team and final designs/construction submittals by the D-B Contractor for walls (both proprietary and non-proprietary types) shall be reviewed and approved.

Standard walls may not be the most cost effective option. Proprietary walls provide more options in terms of cost-effectiveness and aesthetics. Non-standard walls that may involve elements such as soil nail and anchored wall systems are acceptable, provided that requirements are met. Reinforced slopes are similar to non-standard / non-proprietary walls in terms of their design process.

For preliminary design of these wall or slope systems, required information to be provided is as follows:

- The allowable bearing capacity and foundation embedment criteria for the wall
- Backfill and foundation soil properties (assume that gravel borrow or structural backfill material will be used for the walls when assessing soil parameters)
- General wall and/or slope plan; profile showing neat line top and bottom of wall; profiles showing the existing and final ground line in front of and in back of wall; site data and typical cross-section
- Location of right-of-way lines and other constraints to wall/slope construction
- Location of adjacent existing and/or proposed structures, utilities, and obstructions
- Generic details for the desired appurtenances and drainage requirements, and load or other design acceptance requirements for these appurtenances
- Location of catch basins, grate inlets, signal foundations, and the like (it is best to locate these outside the reinforced MSE wall backfill zone to avoid interference with the soil reinforcement)
- In cases where conflict with these reinforcement obstructions cannot be avoided, indicate the location(s) and dimensions of the reinforcement obstruction(s) relative to the wall on the plans, and
- Wall/slope facing alternatives to meet the aesthetic and performance requirements

For non-proprietary RSSs, anchored walls, walls containing geo-synthetics, and soil nail walls, the designer initiates the design effort and develops wall/slope profiles, preliminary engineering plans, cross sections, quantities, special provisions, cost estimates etc., for the proposed wall/slope and subsequently a complete and detailed wall/slope design and construction is coordinated and carried out during final design.

Additional geotechnical guidance will be prepared for final design.

6.7.9 Wall Foundation Improvement using Ground Improvement Methods

At locations where ‘poor’ ground conditions are present that could result in retaining walls or abutment features not meeting performance requirements, due to settlement or stability problems, advanced mitigation measures such as ground improvement shall be considered for geotechnical design. Ground improvement measures may also be necessary to mitigate potential seismic hazards, such as liquefaction or seismic stability. The selection of candidate ground improvement methods for any specific project shall follow the process described in detail in FHWA’s Ground Improvement Reference Manuals Volumes I and II, FHWA-NHI-06-019/020 dated 2006.

6.7.10 Lateral Support of Temporary Excavations Systems

This section will be prepared for final design.

6.8 Cut Slopes and Natural Slopes

This section will be prepared for 30% design.

6.9 Drainage, Subdrainage, Infiltration Facilities and Dewatering

This section will be prepared for 30% design.

6.10 Geotechnical Earthquake Engineering

6.10.1 Seismic Design Criteria

Seismic design criteria for geotechnical earthquake engineering have been established in terms of three levels of project performance criteria and associated ground motion levels in TM 2.10.4.

Geotechnical seismic design shall be consistent with the philosophy for structural design for all three performance levels. The performance objective shall be achieved at a seismic risk level that is consistent with the seismic risk level required for that seismic event. Slope instability and other seismic hazards such as liquefaction, lateral spread, post-liquefaction pile downdrag, and seismic settlement may require mitigation to ensure that acceptable performance is obtained during a design seismic event. The geotechnical designer shall evaluate the potential for differential settlement between mitigated and non mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions. The foundations shall also be designed to address liquefaction, lateral spread, and other seismic effects to prevent collapse. All earth retaining structures shall be evaluated and designed for seismic stability internally and externally. Cut slopes in soil and rock, fill slopes, and embankments, especially those which could have significant impact on the operations of high speed trains shall be evaluated for instability due to design seismic events and associated geologic hazards.

6.10.2 Design Ground Motions

Methods to develop design ground motions for this project which are applicable to geotechnical earthquake engineering are presented in TM 2.9.6 for 30% design.

6.10.3 Site Response and Ground Amplification

Methods to perform site-specific site response analysis, where needed, are presented in TM 2.9.6 for the 30% design.

6.10.4 Limits on Site Response Analyses

If site-specific ground motions in terms of design response spectra are obtained using site response analysis methods per TM 2.9.6 for 30% design, the resulting response spectra must be limited to the limits of ASCE 7-05 Chapter 21. The geotechnical engineer should refer to TM 2.9.6 for additional details.

6.10.5 Seismic Soil-Structure Interaction Analysis

Requirements pertaining to soil-structure-interaction (SSI) analyses are pending.

6.10.6 Liquefaction Triggering and Consequences

Evaluation of soil liquefaction triggering potential shall be performed in two steps. The first step involves evaluating whether the soil meets the compositional criteria necessary for liquefaction. For soils meeting the compositional criteria, the next step is to evaluate whether the design level ground shaking is sufficient to trigger liquefaction given the soil's in-situ density. If it is determined that liquefaction will be triggered, the engineering consequences of liquefaction shall be evaluated. In addition to Factor of Safety-based criteria for liquefaction, the geotechnical engineer shall also consider the allowable deformation values described in Section 6.3.5 and the long-term, post construction performance requirements for earth and fill conditions.

6.10.6.1 Criteria for Liquefaction Susceptibility of Silts and Clays

Evaluation of whether silty and clayey soils meet the criteria for liquefaction susceptibility shall be performed using the criteria developed by Bray and Sancio (2006), and compared to results by analysis using the methods presented in Idriss and Boulanger (2008). Results of these two methods of analyses shall be interpreted and applied to design using engineering judgment.

Considering the range of criteria currently available in the literature, geotechnical engineers shall consider performing cyclic triaxial or simple shear laboratory tests on undisturbed soil samples to assess liquefaction susceptibility for critical cases. For fine grained soils that do not meet the above criteria for liquefaction, cyclic softening resulting from seismic shaking shall be considered.

No specific guidance regarding susceptibility of gravels to liquefaction is currently available. The primary reason why gravels may not liquefy is that their high permeability frequently precludes the development of undrained conditions during and after earthquake loading. When bounded by lower permeability layers, however, gravels shall be considered potentially susceptible to liquefaction and their liquefaction susceptibility evaluated. A gravel layer that contains sufficient sand to reduce its permeability to a level near that of the sand, even if not bounded by lower permeability layers, shall also be considered susceptible to liquefaction and its liquefaction potential evaluated as such.

6.10.7 Liquefaction Triggering Evaluations

Liquefaction triggering analyses shall be performed for sites that meet the following criteria:

- The estimated maximum groundwater elevation at the site is determined to be within 75 ft of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile is characterized in the upper 75 ft as having soils that meet the compositional criteria for liquefaction with a measured SPT resistance, corrected for overburden pressure and hammer energy (N1)60, less than 30 blows/ft, or a cone tip resistance qc1N of less than 180, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

Liquefaction triggering analyses shall be limited to the upper 75 feet. If the site meets the conditions described above, a detailed assessment of liquefaction potential shall be conducted.

Liquefaction analysis involves estimating factor of safety (FS) against liquefaction. Factor of safety against liquefaction is defined as the ratio between Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR). The most common method of assessing liquefaction involves the use of empirical methods (i.e., Simplified Procedures) to estimate CSR and CRR. These methods provide an estimate of liquefaction potential based on Standard Penetration Test (SPT) blowcounts, Cone Penetration Test (CPT) tip resistance, Becker Hammer Penetration Test (BPT) blowcounts, or shear wave velocity. SPT and CPT test methods are most common and generally considered to be more reliable for liquefaction analyses than BPT and Vs tests. Vs and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods, such as gravelly soils. This type of analysis shall be conducted as a baseline evaluation, even when more rigorous methods are used. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modeled well by the simplified methods.

6.10.7.1 Simplified Procedures

The two updated simplified methods by Seed et. al. (2003) and Idriss and Boulanger (2008) shall be used for liquefaction triggering analysis. Results of these analyses shall be interpreted and applied to design using engineering judgment.

6.10.7.2 Minimum Factor of Safety against Liquefaction

The potential consequences of liquefaction and (if necessary) liquefaction hazard mitigation measures shall be evaluated if the factor of safety against liquefaction is less than 1.2.

6.10.7.3 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Methods to estimate settlement of unsaturated granular deposits are presented in a section 6.10.14. Liquefaction induced ground settlement of saturated granular deposits shall be estimated using the procedures by Tokimatsu and Seed (1987), or Ishihara and Yoshimine (1992). The corrected SPT blow counts for the Tokimatsu and Seed (1987) method shall include all corrections, including the corrections for fines. However, the corrections for fines for settlement calculations are different than the corrections for liquefaction analyses. In addition, the CSR values shall also be corrected for magnitude before estimating settlements. If a laboratory-based analysis of liquefaction induced settlement is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the liquefaction induced vertical settlement in lieu of empirical SPT or CPT based criteria. Even when laboratory-based volumetric strain test results are obtained and used for design, the empirical methods shall be used to qualitatively check the reasonableness of the laboratory test results.

The geotechnical engineer shall compare the estimated settlement values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The geotechnical engineer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

6.10.7.4 Liquefied Residual Strength Parameters

Lower-third value of the range of values proposed by Seed and Harder (1990) curve shall be used to estimate residual strength of liquefied soil unless soil specific laboratory performance tests are conducted as described below. Results of laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the residual strength in lieu of empirical SPT or CPT based criteria. Even when laboratory-based test results are obtained and used for design, the Seed and Harder (1990) curve shall be used to qualitatively check the reasonableness of the laboratory test results. It shall be noted that SPT N fines content corrections for residual strength calculations are different than corrections for liquefaction triggering and settlement.

6.10.7.5 Surface Manifestations

The assessment of whether surface manifestation of liquefaction (such as sand boils, ground fissures etc.) will occur during earthquake shaking at a level-ground site shall be made using the method outlined by Ishihara (1985) and shall be compared against results by the method presented in Youd and Garris (1994 and 1995). It is emphasized that settlement may occur, even with the absence of surface manifestation. The 1985 Ishihara method is based on the thickness of the potentially liquefiable layer (H_2) and the thickness of the non-liquefiable crust (H_1) at a given site. In the case of a site with stratified soils containing both potentially liquefiable and non-liquefiable soils, the thickness of a potentially liquefiable layer (H_2) shall be estimated using the method proposed by Ishihara (1985) and Martin et. al., (1991). If the site contains potential for surface manifestation, then use of mitigation methods shall be evaluated.

6.10.8 Evaluation of Lateral Spreading and Consequences

Lateral spreading shall be evaluated for a site if liquefaction is expected to trigger within 50 feet of the ground surface, and either a ground surface slope gradient of 0.1% exists or a free face conditions (such as an adjacent river bank) exists. Historic and paleoseismic evidence of lateral spreading is valuable information that shall also be reviewed and addressed. Such evidence may include sand boils, soil shear zones, and topographic geometry indicating a spread has occurred in the past.

6.10.8.1 Methodologies for Predicting Lateral Spreading

In order to predict the permanent deformations resulting from the occurrence of lateral spreading during earthquake loading, several methods of analyses are available. These different methods of analyses can be categorized into two general types: Empirical Methods and Analytical Methods.

Empirical Methods

The most common empirical methods to estimate lateral displacements are Youd et. al. (2002), Bardet et. al. (1999), Zhang et. al. (2004), Faris et. al. (2006). Analysts shall be aware of the applicability and limitations of each method. Lateral displacements shall be evaluated using the Youd et. al. (2002) method, and one of the other methods described above.

Empirical methods shall be used as the primary means to estimate deformations due to lateral spreading. Multiple models shall be considered and the range of results shall be reported.

Analytical Methods

For cases where slope geometry, structural reinforcement or other site-specific features are not compatible with the assumptions of the empirical methods, Newmark sliding block analyses shall be considered. Newmark analyses shall be conducted similar to that described in the seismic slope stability section, except that estimation of the yield acceleration shall consider strength degradation due to liquefaction.

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The geotechnical engineer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

6.10.9 Analysis for Conceptual Design of Liquefaction Mitigation Methods

Liquefaction mitigation and performance criteria vary according to the acceptable level of risk and required levels of performance for each structure type. Implementation of mitigation measures shall be designed to either eliminate all liquefaction potential or to allow partial improvement of the soils, provided that acceptable performance (i.e., stability and deformation levels) can be achieved.

During the liquefaction evaluation, the engineer shall determine the extent of liquefaction and potential consequences such as bearing failure, slope stability, and/or vertical and/or horizontal deformations. Similarly, the engineer will evaluate the liquefaction hazard in terms of depth and lateral extent affecting the structure in question. The lateral extent affecting the structure will depend on whether there is potential for large lateral spreads toward or away from the structure and the influence of liquefied ground surrounding mitigated soils within the perimeter of the structure.

Large lateral spread or flow failure hazards may be mitigated by the implementation of containment structures, removal or treatment of liquefiable soils, modification of site geometry, structural resistance, or drainage to lower the groundwater table.

Where liquefiable clean sands are present, geotechnical evaluations for design shall consider an area of softening due to seepage flow occurring laterally beyond the limit of improved ground a distance of two-thirds of the liquefiable layer thickness, as described in studies by Lai (1988). To calculate the liquefiable thickness, similar criteria shall be used as that employed to evaluate the issue of surface manifestation by the Ishihara (1985) method. For level ground conditions where lateral spread is not a concern or the site is not a water front, this buffer zone shall not be less than 15 feet and it is likely not to exceed 35 feet when the depth of liquefaction is considered as 50 feet and the entire soil profile consists of liquefiable sand.

The performance criteria for liquefaction mitigation, established during the initial investigation, shall be in the form of a minimum, or average, penetration resistance value associated with a soil type (fines content, clay fraction, USCS classification, CPT soil behavior type index Ic, normalized CPT friction ratio), or a tolerable liquefaction settlement as calculated by procedures discussed earlier. The choice of mitigation methods will depend on the extent of liquefaction and the related consequences. Also, the cost of mitigation must be considered in light of an acceptable level of risk. In general, options for mitigations are divided into two categories: ground improvement options and structural options.

6.10.9.1 Ground Improvement Options

The five general methods of ground improvement to be considered for soil liquefaction mitigation are:

- Densification
- Drainage
- Reinforcement
- Mixing/Solidification, and
- Replacement

The implementation of these techniques shall be designed to fully, or partially, eliminate the liquefaction potential, depending on the performance requirements of the engineered facility under consideration. With regards to drainage techniques for liquefaction mitigation, only permanent dewatering works satisfactorily. The use of gravel or prefabricated drains, installed without soil densification, is unlikely to provide pore pressure relief during strong earthquakes and may not prevent excessive settlement.

Densification Techniques

The most widely used techniques for in-situ densification of liquefiable soils are:

- Vibrocompaction,
- Vibro-replacement (also known as vibro-stone columns),
- Deep dynamic compaction, and
- Compaction (pressure) grouting (Hayden and Baez, 1994)

Further details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

Mixing/Solidification Techniques

Mixing and/or solidification techniques seek to reduce the void space in the liquefiable soil by introducing grout materials either through permeation, mixing mechanically, or jetting. The most widely used hardening techniques are:

- Permeation grouting
- Deep soil mixing, and
- Jet grouting

Further details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

6.10.9.2 Structural Options

Structural mitigation involves designing the structure to withstand the forces and displacements that result from liquefaction. In some cases, structural mitigation for liquefaction effects may be more economical than soil improvement mitigation methods. However, structural mitigation may have little or no effect on the soil itself and may not reduce the potential for liquefaction. With structural mitigation, liquefaction and related ground deformations will still occur. The structural mitigation shall be designed to protect the structure from liquefaction-induced deformations, recognizing that the structural solution may have little or no improvement on the soil conditions that cause liquefaction. The appropriate means of structural mitigation may depend on the magnitude and type of liquefaction-induced soil deformation or load. If liquefaction-induced flow slides or significant lateral spreading is expected, structural mitigation may not be practical or feasible in many cases. If the soil deformation is expected to be primarily vertical settlement, structural mitigation may be economically and technically feasible and shall be considered for design.

Depending on the type of structure and amount and extent of liquefaction, common structural options to be considered are:

- Post-tensioned slab foundation,
- Continuous spread footings having isolated footings interconnected with grade beams,
- Mat foundation, and
- Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils

Details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

6.10.10 Seismic Considerations for Lateral Design of Piles in Liquefiable Soils

Seismic considerations for lateral design of pile/shaft design soils include the effects of liquefaction on the lateral response of piles/shafts and designing for the additional loads due to lateral spread and/or slope failures. Effects of liquefiable soils shall be included in the lateral analysis of piles/shafts by using appropriate p-y curves to represent liquefiable soils. Computer programs such as LPILE include p-y curves for liquefiable soils. The p-y curves available within the program have limited application and may give unconservative results. Furthermore, in fully liquefied sand, there appears to be virtually no lateral soil resistance for the first 1 to 2 inches of lateral movement (Rollins et. al., 2005). Available static p-y curve models reduced adequately to account for the loss of strength caused by liquefaction, such as a p-multiplier approach, could provide an approximate prediction of the measured p-y response. Liquefied soil p-y curves shall be estimated using the static API sand model reduced by a p-multiplier using the method of Brandenberg, et. al. (2007b) and Boulanger, et. al. (2003).

In general, there are two different approaches to estimate the lateral spread/slope failure induced load on deep foundations systems – a displacement based method and a force based method.

6.10.10.1 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading loads on deep foundation systems is presented in Boulanger, et. al. (2003) and Brandenberg, et. al. (2007a and b). Deep foundations in liquefied, lateral spreading ground shall be designed to resist lateral forces imposed on the pile by the lateral spreading ground. LPILE or similar computer programs shall be used to perform this analysis. The design steps that consider the kinematic loading from the liquefaction-induced lateral spreading ground are presented in Boulanger et. al. (2007a and b).

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The geotechnical engineer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

6.10.10.2 Force Based Approaches

A force based approach to assess lateral spreading induced loads on deep foundations is based on back-calculations from pile foundation failures caused by lateral spreading. The pressures on pile foundations shall be evaluated for design as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).
- Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that this is an adequate design method (Finn and Fujita, 2004). The force-based approach is appropriate where larger displacements occur that can mobilize the full passive pressure against the foundation. Where smaller displacements occur, the displacement-based approach shall be considered and may be more appropriate.

Another force-based approach to estimate lateral spreading induced foundation loads is to use a limit equilibrium slope stability program to determine the load the foundation must resist to achieve a target safety factor of 1.1. This force is distributed over the foundation in the liquefiable zone as a uniform stress. This approach may be utilized to estimate the forces that foundation elements must withstand if they are to act as shear elements stabilizing the slope.

6.10.11 Evaluation of P-Y and T-Z Springs for Seismic Analysis

Geotechnical and structural engineering guidance for seismic analysis using P-Y and T-Z 'springs' will be prepared for Final Design.

6.10.12 Evaluation of Foundation Dynamic Stiffness and Damping

Geotechnical and structural engineering guidance for seismic analysis considering foundation dynamic stiffness and damping will be prepared for Final Design.

6.10.13 Dynamic Soil Pressures on Earth Retaining Structures

All retaining walls, abutment walls, and basement walls shall be evaluated and designed for seismic stability internally and externally (i.e. sliding and overturning). With regard to overall seismic slope stability (often referred to as global stability) involving a retaining wall, with or without liquefaction, the geotechnical designer shall evaluate the potential for failure and its impacts on performance. If unacceptable performance of the wall is likely during the design seismic event, the stability of the wall shall be improved such that performance criteria are met.

For retaining walls that are not restrained from rotation at the top and contain cohesionless materials as backfill, seismic pressures shall be estimated using the Mononobe-Okabe (M-O) method. Horizontal seismic coefficient (k_h) shall be taken as ½ of the peak ground acceleration value (PGA). For 15% design, the PGA value shall be estimated for the MCE level event as presented in TM 2.10.4. For the 30% design phase and final design, PGA values associated with three performance levels shall be used. The earth pressures shall be separated into the incremental seismic pressures and the active earth pressures in the following manner:

$$\Delta K_{AE} = K_{AE} - K_A$$

where

ΔK_{AE} = Incremental seismic pressure coefficient

K_{AE} = Total seismic pressure coefficient

K_A = Active pressure coefficient

The incremental seismic earth pressure shall be taken as inverted triangle with the resultant acting at 0.65H from the bottom. This pressure shall be added to the active earth pressure for the design. It shall be noted that seismic pressures increase significantly with slight increase in slope of the backfill. For higher angles of sloping back fills, the M-O solution will not converge. For those cases, methods presented in Chapter 7 of the NCHRP Report 611 shall be utilized. For backfill materials consisting of cohesive or cohesive and frictional (c-φ) material, methods presented in Chapter 7 of the NCHRP Report 611 shall be used.

For basement walls (or walls restrained against rotation) in locations where PGA values are less than or equal to 0.25g, walls shall be designed for only at-rest pressures and additional seismic loads shall not be considered. For higher PGA values, the higher of the at-rest pressures or the active plus M-O pressures shall be used for the design. Seismic coefficient value of ½ of the PGA shall be used in calculations.

6.10.14 Seismic Settlement of Unsaturated Soils

Seismically induced settlement of unsaturated granular soils (dry sands) shall be estimated using procedures provided by Tokimatsu and Seed (1987). Estimated values in terms of total and differential settlements shall be reported.

The geotechnical engineer shall compare the estimated settlement values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section

6.10.9, if necessary. The geotechnical engineer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

6.10.15 Seismic Slope Stability and Deformation Analyses

Instability of slopes during seismic loading could be due to liquefaction or due to inertial loading or a combination of both. In this section instability of both the natural existing slopes and embankment slopes is addressed.

The geotechnical engineer shall compare the estimated deformation values with the allowable deformation values described in Section 6.3.5 and develop mitigation plans described in Section 6.10.9, if necessary. The geotechnical engineer shall also consider the long-term, post construction performance requirements for earth and fill conditions.

6.10.15.1 Slope Instability Due to Liquefaction

Slopes could fail or experience deformations due to liquefaction either in the form of lateral spreading or flow failures. Liquefaction induced lateral spreading is addressed in Section 6.10.8.

6.10.15.2 Liquefaction Induced Flow Failure

Liquefaction leading to catastrophic flow failures driven by static shearing stresses that result in large deformation or flow shall also be addressed by geotechnical engineers. These flow failures may occur near the end of strong shaking or shortly after shaking and shall be evaluated using conventional limit equilibrium static slope stability analyses. The analysis shall use residual undrained shear strength parameters for the liquefied soil assuming seismic coefficient to be zero (i.e., performed with k_h and k_v equal to zero). The residual strength parameters estimated using the method presented in Section 10.7 shall be used. In addition, 20-percent reduced strength of the normally consolidated clayey layers shall be used, ands strength reductions shall be considered for saturated sandy layers where excess pore water pressure is generated but full liquefaction does not occur. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium factor of safety, FS, is less than 1.0, flow failure shall be considered likely. Liquefaction flow failure deformation is usually too large to be acceptable for design of structures, and some form of mitigation will likely be needed. However, structural mitigation may be acceptable if the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads.

If the factor of safety for this decoupled analysis is greater than 1.0 for liquefied conditions, yield acceleration (k_y) values shall be estimated using pseudo-static slope stability analysis. The same strength parameters as used during the flow failure analysis shall be used. A new critical failure plane shall be searched assuming both circular and non-circular failure surfaces. Yield acceleration is defined as the minimum horizontal acceleration in a pseudo-static analysis for which FS is 1.0. Using the estimated k_y values, deformations shall be estimated using simplified methods such as Makdisi and Seed (1978) and Bray and Travasarou (2007). These simplified methods are not directly applicable to slopes with liquefiable layers; however, they provide a good estimate of the range of deformations expected during the seismic event. Other methods such as Newmark time history method or more advanced methods involving numerical analysis may also be used. If advanced methods are used, the results shall be checked against the simplified methods.

For pseudo-static analyses to estimate k_y values, residual strengths for the liquefied layers and reduced strengths for normally consolidated clayey and saturated sandy layers with excess pore water pressure generation (as described earlier) shall be used. This is generally a conservative approach but is appropriate for preliminary engineering design. For final design more advanced methods involving numerical analyses may be used to better characterize the initiation of liquefaction and pore pressure generation and subsequent reduction in strength.

6.10.15.3 Slope Instability Due to Inertial Effects

Pseudo-static slope stability analyses shall be used to evaluate the seismic stability of slopes and embankments due to inertial effects. The pseudo-static analysis consists of conventional limit equilibrium slope stability analysis with horizontal (k_h) that act upon the critical failure mass. A horizontal seismic coefficient, k_h, of ½ PGA and a vertical seismic coefficient, k_v, equal to zero shall be used for the evaluation of seismic slope stability. For these conditions, the minimum required factor of safety is 1.1. Alternately, pseudo-static analyses may be performed to estimate k_y values. There is a debate in literature whether the slope failure plane during the pseudo-static analysis shall be fixed based on the results of static analyses or a new failure plane is searched. A new failure plane shall be searched for the pseudo-static analysis. The analysis shall look for both circular and non-circular failure surfaces.

6.10.15.4 Deformations

Deformation analyses shall be performed where an estimate of the magnitude of seismically induced slope deformation is required, and the pseudo-static slope stability factor of safety is less than 1.0. Acceptable methods of estimating the magnitude of seismically induced slope deformation include Newmark sliding block (time history) analysis, simplified displacement charts and equations based on Newmark-type analyses (Makdisi and Seed, 1978; Saygili and Rathje, 2008; and Rathje and Saygili, 2008; Bray and Travasarou, 2007), or dynamic stress-deformation models. These methods shall not be employed to estimate displacements if the post earthquake static slope stability factor of safety using residual strengths is less than 1.0, since the slope will be unstable against static gravity loading and large displacements would be expected.

6.10.16 Downdrag Loading (Dragload) on Structures Due to Seismic Settlement

6.11 GROUND IMPROVEMENT

This section is to be prepared for 30% design.

6.12 OTHER GEOTECHNICAL TOPICS

This section is to be prepared for 30% design.