California High-Speed Train Project

TECHNICAL MEMORANDUM

Geotechnical Analysis and Design Guidelines
TM 2.9.10

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Released by:	*Signed document on file* Hans Van Winkle Program Director	18 Jul 11 Date

Revision	Date	Description
0	30 Jun 10	Initial Release
1	22 May 11	Addresses TAP Comments

Note: Signatures apply for the latest technical memorandum revision as noted above.

Prepared by PB 100 YEARS

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 Rick Schmedes	DD Month YY Date
Infrastructure:	Signed document on file Robert Valenti	09 June 11 Date
Operations:	NOT REQUIRED Joseph Metzler	DD Month YY Date
Maintenance:	NOT REQUIRED Joseph Metzler	DD Month YY Date
Rolling Stock:	NOT REQUIRED Frank Banko	DD Month YY Date

Note: Signatures apply for the technical memorandum revision corresponding to revision number in header and as noted on cover.

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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 as applied to the 30% design stage of development.

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
- Tunnels and underground structures
- Cuts
- Fills and embankments
- Retaining walls
- Culverts
- Drainage and subdrainage

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 California High-Speed Train Project design. Where available, existing guidelines are briefly summarized and referenced without duplicating their contents.

1.0 INTRODUCTION

The information presented in this technical memorandum (TM) 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 TM 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 who are 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 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 California High-Speed Train Project (CHSTP) makes use of the load and resistance factor design (LRFD) methodology per American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications (BDS, most current version) with State of California Department of Transportation (Caltrans) Amendments for engineering design approach in both geotechnical analysis and structural engineering. Some aspects of applying LRFD procedures to the geotechnical discipline and, in particular, geotechnical earthquake engineering, have not been fully vetted or calibrated. If LRFD-based input from geotechnical engineering leads to unusual or unconventional designs (including either overly conservative or unconservative), then designers should compare with results of conventional geotechnical engineering design procedures including pre-LRFD AASHTO. In order to submit any proposed deviations as variances, the designers shall follow the "Design Variance Request" process to seek review and approval as described in TM 1.1.18 guidelines.

1.1 PURPOSE OF TECHNICAL MEMORANDUM

The purpose of this TM 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.

1.2 STATEMENT OF TECHNICAL ISSUE

This TM presents guidelines for geotechnical analysis and design criteria for high-speed train (HST) 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.

1.3 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 TM 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 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.

1.3.1 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
BPT	Becker Hammer Penetration Test
Caltrans	California Department of Transportation
CBR	California Bearing Ratio (standard test methods ASTM D 1883 / D 4429)
CBC	California Building Code
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 Penetration Test
CPTu	Cone Penetration Test with pore water pressure measurement
FHWA	Federal Highway Administration
FOS	Factor of Safety
FRA	Federal Railroad Administration
GBR	Geotechnical Baseline Report
GDR	Geotechnical Data Report
GE	California Registered Geotechnical Engineer
HST	High-Speed Train
ISRM	International Society for Rock Mechanics
LOTB	Logs of Test Borings
LRFD	Load and Resistance Factor Design method
MCE	Maximum Considered Earthquake
MPH/mph	Miles per hour
MSE	Mechanically Stabilized Earth
NHI	National Highway Institute

OBE	Operating Basis Earthquake
PGA	Peak Ground Acceleration
RMR	Rock Mass Rating
RSS	Reinforced Soil Slopes
SAT	Soil Abrasion Test
SPT	Standard Penetration Test
TBM	Tunnel Boring Machine
TM	Technical Memorandum
UIC	International Union of Railways
USCS	United Soil Classification System
USGS	United States Geological Survey
WSD	Working Stress Design

1.3.2 Units

The CHSTP 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 U.S. 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.

1.4 LAWS AND CODES

Initial 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 TM presents guidelines for geotechnical analysis and design criteria for HST 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 are 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., the bearing capacity, or the settlement or global stability) shall be identified so that engineering parameters and properties required for these analyses can be evaluated. The values selected for the parameters should be appropriate to the particular performance requirement, including consideration of limit states and 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 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 the 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 and features 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 that 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 methods 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

TM 2.9.3, Geologic and Seismic Hazard Evaluation Guidelines 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 HST 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. The preparation of geotechnical reports shall utilize the information contained in 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 HST performance, or the method of project mitigations employed. The geotechnical engineer shall evaluate each of the identified geologic or seismic hazards to evaluate 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 is 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).

The requirements for the content and format of geotechnical reports described in TM 2.9.2, Geotechnical Reports Preparation Guidelines, 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 TM 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, Federal Highway Administration (FHWA), American Society of Testing and Materials International (ASTM), International Union of Railways (UIC), and Caltrans and California Building Code (CBC). 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 of this TM. 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 among 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 HST. 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:

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

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 TM contains guidelines that present the process and protocol for interpretation of subsurface conditions for use during geotechnical analyses supporting engineering design activities for the CHSTP.

Soil shall be characterized and classified using ASTM D 2488 guidelines for field classification and ASTM D 2487 based on laboratory test results. Rock should be classified using FHWA GEC5 (FHWA, 2002) guidelines which are largely based on International Society for Rock Mechanics (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. Dynamic properties of soil and rock shall be assessed for consideration of seismic actions and design.

3.2.1 Laboratory Test Requirements and Reports

Standards to be used for laboratory testing of soil and rock for the 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 in collaboration with the geotechnical engineer 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 CHSTP 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 CHSTP. The geologic model shall serve as a fundamental tool to develop the subsurface exploration plan for the CHSTP, 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.

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 CHSTP 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 standard penetration test (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 does 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 CHSTP. 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 CHSTP.

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 United States Classification System (USCS) symbols for soil classification
- Location of the ground water (see next section)
- 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, Geotechnical Investigation Guidelines, CHSTP will make use of electronic records for borings, cone penetration tests (CPTs), etc. An appropriately developed database and geographic information system (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:

- Catalog borings that were conducted previously
- Inventory data regarding specific problematic formations along the HST corridor
- 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:

- Historically high groundwater levels
- 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
- Potential for groundwater level rise resulting from anticipated rise of sea level due to climate change
- 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) for zone of 100 percent saturation 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 evaluated using one of the following approaches:

1. Analyze 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. Solve 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
    - 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. The properties resulting from LRFD-based evaluations shall be consistent with those obtained with general geotechnical practice and shall not be overly conservative or unconservative.

Regarding SPTs, 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 N_{60} 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 N_{60}-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 also can 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 a 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 is used for design. Additional guidelines that shall be considered for correlations are presented in Manual on Estimating Soil Properties for Foundation Design by Electric Power Research Institute (EPRI) Report EL-6800, (EPRI, 1990). 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 controlled by regular, systematic discontinuities in the rock mass.

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 GEC5, which provides guidance for analyzing data and resolving inconsistencies. The geotechnical engineer shall also use GEC5 to assess variability for a given engineering property in a particular geologic unit and how that variability influences the selection of the final design values.

Development of the geotechnical model outlined in Section 3.2.2.3 of this TM shall include an estimate of the scatter surrounding average physical properties of soil and rock units. The geotechnical engineer shall provide upper and lower reasonable estimates of key engineering properties to describe the uncertainty associated with estimates of the median properties. The upper and lower reasonable estimates are not upper and lower bounds, but instead represent approximately 84th and 16th percentile values, respectively.

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 statistical 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 high level of uncertainty associated with the design property. Since strict application of statistics may not provide an appropriate approach for developing best estimates of geotechnical properties, then engineering judgment shall also be applied. 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 consistent with engineering judgment. 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 of this TM.

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 the 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.

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 rights-of-way
- The environmental impact of the foundation construction
- Physical constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities)
- 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
- Cost and schedule

Shallow foundations shall be used for foundation support where competent soil or rock is present within relatively shallow depths. Shallow foundations may consist of spread footings or mat foundations. Shallow foundations 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 foundations are typically not appropriate for soils that are soft, loose, expansive, prone to hydro-collapse, liquefiable, or prone to excessive scour.

Where shallow foundations are not feasible (i.e., they cannot meet the required bearing capacity or settlement criteria) 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, 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. Where larger bridge spans and higher foundation loads are required, caissons, barrette, or diaphragm wall foundations may be considered.

Scour – The selection of foundation types and design of foundations shall consider the effects of scour on the capacity requirements and size (dimensions, embedment, and length) of foundations. The capacity of deep foundations shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes shall be evaluated by analysis methods per TM 2.6.5, Hydraulics and Hydrology Design Guidelines.

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. It should be noted that Caltrans Amendments require that abutment foundations be designed using Service 1 limit state and working stress design (WSD) per Caltrans 2000 Bridge Design Specifications dated November 2003.

Three general limit states are considered for foundation design in the AASHTO LRFD methodology:

1. Strength Limit State – Evaluation of strength under various loading conditions

2. Extreme Event Limit State – Evaluation of strength and performance under extreme loading conditions resulting from rare events such as earthquakes, collision, and extreme storms
3. 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

CHSTP 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

Notes: 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, foundation shall be designed and proportioned for the following limit states and mechanisms:

Service Limit State:

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

Strength Limit State:

Spread footings and mats

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

Driven Piles:

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

Drilled Shafts:

- o Axial compression resistance for single drilled shafts

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

Micropiles:

- o Axial compression resistance for single micropile
- o Micropile group compression resistance
- o Uplift resistance for single micropiles
- o Uplift resistance of micropile groups
- o Micropile group punching failure into a weaker stratum below the bearing stratum, and single micropile punching failure where tip resistance is considered
- o Single and group micropile lateral resistance
- o 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 Allowable 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, allowable 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. For structure foundations, settlements calculated from the Service 1 load combination plus any settlements resulting from the OBE load combination (such as those resulting from post-liquefaction downdrag, seismic compaction, etc.) shall not exceed the settlement limits denoted in Table 3.3.7-1. For approach embankments, the Service 1 settlement limits are applicable to settlements that occur after the placement of track.

Further performance requirements for allowable deformations are prescribed in the TM 2.10.10, Track-Structure Interaction.

Table 3.3.7-1 Settlement Limits[4] for Service 1 and OBE Load Cases

Settlement Criteria	Non-Ballasted Track	Ballasted Track
Differential settlement between adjacent supports [1]	<= L/1500 and 3/4”, where L = smaller span	N/A [3]
Differential settlement between the abutment and approach embankment [2]	<= 3/8” over 62 feet	<= 3/4" over 62 feet
Differential settlement between the abutment and tunnel portal	<= 3/8” over 62 feet	N/A [3]
Uniform settlement at piers and abutments	<= 3/4”	N/A [3]

Notes:

1. The additional forces imposed on the structural system by differential settlements shall be calculated and considered as part of dead load in the design.

2. Prior to placement of tracks, the approach embankment shall be instrumented and monitored for a period of at least 6 to 12 months to ensure the embankment is in compliance with the settlement requirements set forth in the table above.
3. Not applicable based on the assumption that ballasted track will not be used for bridges, aerial structures, or tunnels.
4. The settlements are calculated from the Service 1 load combination plus any settlements resulting from the operating basis earthquake (OBE) load combination (such as those resulting from post-liquefaction downdrag, seismic compaction, etc.).

No specific settlement or displacement limits are required for the extreme event maximum considered earthquake (MCE) loading case, only that the structure shall not collapse. For deep foundations, the maximum relative horizontal displacement between the bottom (i.e., toe of pile) and top (i.e., pile cap) of the foundation resulting from OBE loading shall not be more than 1.75 inches.

The settlements and displacements noted in the table above are considered minimum performance criteria. Designers may elect to use more stringent criteria. The structural design may require that foundations be designed to more stringent criteria for certain structures depending on specific performance requirements.

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.

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.

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 7bases 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).
- Prior to use of the proprietary foundation system in design for the CHSTP, the analytical methodologies and resistance factors noted above must be presented to and approved by the Authority or its agent.

3.3.12 Abutments and Abutment Foundations

Bridge abutments have components of both foundation design and retaining wall design. It should be noted that Caltrans Amendments require that abutment foundations be designed using Service 1 limit state and Working Stress Design (WSD) per Caltrans 2000 Bridge Design Specifications dated November 2003. The retaining wall aspects of abutments shall be designed in accordance with Section 6.7 of this TM, and AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10 and 11.

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.

3.4 Foundations for Buildings and other At-Grade Structures

This section describes the methods that shall be applied in foundation analysis and design of buildings, and other at-grade structures such as signals, signs, and noise barriers.

3.4.1 Buildings

Foundations and retaining walls for buildings shall be designed in accordance with the provisions of the 2010 CBC, California Code of Regulations, Title 24, Part 2, California Building Standards Commission (2010 CBC), and TM 2.5.1, Structural Design of Surface Facilities and Buildings. In absence of site-specific data, presumptive values provided in Chapter 18 of the 2010 CBC for allowable foundation bearing pressure, lateral earth pressures, and sliding coefficients shall be used. Seismic issues related to foundation design such as seismic earth pressures, downdrag and lateral spread due to liquefaction shall conform to the limits provided in Table 3.3.7-1.

3.4.2 Noise Barriers

Foundation design for noise barrier shall be conducted in accordance with Caltrans Memo To Designers 22-1, Soundwall Design Criteria. Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed per Section 6.10.

3.4.3 Signs and Signals

Cantilever signs and signals shall be supported on drilled shaft foundations. Design for cantilever signals and cantilever signs shall be performed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO, 2001). The foundation design will require ultimate and allowable downward and uplift capacities. In addition, lateral capacities of shafts shall be provided. Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed per Section 3.10.

3.5 Tunnels and Other Underground Structures

This section describes the methods that shall be applied in geotechnical and geological analysis for design of tunnels and other underground structures. Tunnels and other underground structures include bored tunnels (i.e., in rock and/or soft ground), mined tunnels, underground chambers, cut-and-cover tunnels, portals, shafts, and tunnel crossovers. U-shaped structures are addressed in Section 6.7 of this TM.

3.5.1 Guideline Resource Materials

Published resource information is cited in Section 5 of TM 2.9.1. Key resource materials providing guidance useful for tunnel design and construction issues include:

- AASHTO, Manual on Subsurface Investigations, MSI-1, 1988
- CalOSHA, California Code of Regulations (CCR), Title 8, Chapter 4, Subchapter 20, Tunnel Safety Orders
- FHWA Technical Manual for Design and Construction of Road Tunnels - Civil Elements, FHWA-NHI-10-034, Dec. 2009
- United States Army Corps of Engineers, Engineering and Design, Tunnels and Shafts in Rock, Manual No. 1110-2-2901, 1997

3.5.2 Site Investigation

Site investigations shall be planned and conducted in sufficient detail to evaluate the subsurface conditions of the rock and/or soil medium, and groundwater regime within which tunnels and/or underground structures will be constructed. Site investigations shall be planned and executed in accordance with TM 2.9.1 and Chapter 3, FHWA-NHI-10-034 – Technical Manual for Design and Construction of Road Tunnels-Civil Elements (FHWA 2009). The site investigations shall be sufficient in scope to explore and characterize the site conditions at the specific location and elevation of the proposed underground excavation.

Subsurface investigation for underground excavations will not only evaluate the general subsurface conditions but shall specifically target subsurface conditions that may affect design assumptions, final designs, or construction methods. Such conditions may include but not be limited to geologic hazards, and tunnel/underground design and construction issues such as fault zones, soft ground, landslides, rock falls, ground settlement, liquefaction effects, groundwater flow, groundwater head, thermal water, high in-situ stress, raveling ground, running ground, flowing ground, abrasive ground, boulders in soft ground, sheared ground, gassy ground (e.g., methane), contaminated ground and water, etc. Sufficient data shall be developed to characterize the geotechnical design and construction issues sufficiently for analysis and appropriate mitigation. Such data shall be presented in the Geotechnical Data Report (GDR) and the Geotechnical Baseline Report (GBR) for all CAHST tunnel segments as outlined in TM 2.9.2 and FHWA (2009) Chapters 4.2 and 4.4.

3.5.2.1 Geologic Mapping

Geologic mapping shall be carried out in accordance with TM 2.9.1 for documenting surficial soil and rock units and measurements of rock discontinuities. Geologic data are necessary for planning field exploration programs, but also for anticipating and confirming subsurface structural geology. As part of the structural data mapping, the measurements of rock discontinuities shall focus on collecting data along tunnel alignments and at portal and shaft locations. The data shall be collected under the guidelines set forth in Chapter 3.4 of FHWA (2009) and TM 2.9.1. The acquired data shall be analyzed to identify common trends in discontinuities using accepted statistical methods of analysis as summarized in Section 6.5.6 of this TM. The resulting data shall be used for analysis of geometric orientation to the proposed underground excavations for use in design and construction planning.

3.5.2.2 Subsurface Investigations

Geotechnical field investigations shall include subsurface investigations to develop three-dimensional geological models (refer to Section 6.2.5.1 of this TM) including hydrogeological and geotechnical (refer to Section 6.2.5.2 of this TM) features, for use in evaluating ground conditions for evaluating constructability, for design of tunnels and other underground structures, for interim and final ground support systems, and for developing groundwater control systems. Geotechnical investigations shall be conducted in accordance with TM 2.9.1 and FHWA (2009). Groundwater and hydrogeology investigations shall be completed in accordance with Chapter 3.5.6 of FHWA (2009) and guidelines for in-situ testing in TM 2.9.1.

Inclined borings and horizontal borings shall be used for tunnels and portal sites to economize on drilling footage when conventional vertical borings cannot provide the quality of geotechnical information for developing the geological and geotechnical models of underground conditions (refer to Chapter 3.5.2 of FHWA 2009).

Where appropriate, the geotechnical investigations shall use in-situ testing methods for evaluating soil and rock properties. Soil testing shall include CPTs, pressuremeter tests, flat-plate dilatometer tests, and field vane shear tests and shall be conducted in accordance with TM 2.9.1 and Chapter 3 of FHWA (2009). In-situ rock testing shall include hydraulic fracturing and overcoring to measure the in-situ stress ratio, where high in-situ stresses are suspected relative to that predicted by elastic theory (refer to Chapter 3 of FHWA [2009]).

In-situ geophysical testing shall be applied in the subsurface to evaluate depth to bedrock, rippability of rock, geologic structures and orientations, and recognition and correlations of lithologic and stratigraphic units in accordance with TM 2.9.1 and Chapter 3 of FHWA (2009).

3.5.2.3 Laboratory Testing

Soil samples shall be described and classified using ASTM D 2488 guidelines for field classification and ASTM D 2487 based on laboratory test results. Rock (both hard and soft) shall be classified using ASTM D 5878 and be in conformance with GEC 5, Evaluation of Soil and Rock Properties (FHWA 2002), which is based on the ISRM guidelines. Laboratory testing shall be performed in accordance with Section 6 of TM 2.9.1. Sufficient laboratory testing shall be performed to represent in-situ rock and soil conditions of the project.

In addition to laboratory testing identified in TM 2.9.1, specialized testing of rock for underground excavations mined by tunnel boring machine (TBM) shall also include drilling rate index, bit wear index, and cutter life index, as described in Chapter 3 of FHWA (2009). Additional testing may include cherchar abrasion index, and punch penetration test for use by TBM designers. The abrasion characteristics of soils (Abrasion Value for Soils AVS) shall be evaluated by applying the Norwegian Institute of Technology (ibid NTNU) soil abrasion test (SAT).

Petrographic analysis shall be conducted on representative rock samples. The petrographic analyses shall be conducted on rock thin sections prepared for analysis under a polarizing microscope to identify principal mineral constituents (especially quartz content and presence of asbestos), textural relationships, alteration/metamorphism, percentages, and other unusual properties that may affect TBM performance.

3.5.3 Characterization

3.5.3.1 Soil Classification

Descriptions of soils shall also be in accordance with TM 2.9.1 and Chapters 3.5 and 7.2 of FHWA (2009). For tunnels and other underground excavation, special attention shall be given to documenting soil grain-size characteristics and stratification features, both of which strongly influence ground behavior for excavations. Terzaghi (1950) classified soils in the Tunnelman’s ground classification according to the anticipated soft ground behavior based on soil identification (grain size) and whether the excavation is above or below groundwater. Emphasis shall focus on cohesionless soils (i.e., composition, gradation, and density) and on cohesive soils (i.e., consistency and strength) with respect to the proposed excavation and with respect to groundwater conditions (e.g., perched and confined conditions, permeability, and evidence of artesian conditions or groundwater barriers, i.e., faults).

3.5.3.2 Rock Mass Classification

Rock mass classifications shall be evaluated from the geotechnical and geological data collected during the field investigations to describe the rock mass conditions that shall be predominant within the proposed tunnels and underground excavations in rock in accordance with FHWA (2009). Terzaghi (1946) proposed a qualitative description of rock mass classes and successful applications of various tunnel support systems that prevent rock masses from dropping from the tunnel roof. If used for the CHSTP, Terzaghi’s rock mass classifications shall be implemented only for preliminary estimates of tunnel support requirements based on professional judgment. The more recently proposed numerical classifications of rock shall be used for the CHSTP design recommendations. The numerical rock quality designation (RQD) proposed by Deere and Deere (1989), the tunneling quality index (Q) proposed by Barton et al. (1974) of the Norwegian Geotechnical Institute, and the Rock Mass Rating (RMR) by Z.T. Bieniawski (1989) shall be developed for site-specific application to rock tunnel support and lining of the CHSTP. The numerical rock mass classifications shall be used for evaluating and demonstrating the design of proposed rock support systems for the tunnel excavations.

3.5.3.3 Geologic Structure

Analysis of geologic structure shall be performed for all proposed excavations in rock. Rock discontinuities, which typically control the behavior of a rock mass with respect to slope stability and underground stability, shall be analyzed in outcrops by geologic mapping, in rock-core logging as outlined in TM 2.9.1, and in using in-situ methods of logging as outlined in AASHTO MSI-1, Section 6.1.2. All structural mapping and logging methods are to document in-situ geologic structural trends for use in structural analyses of the rock mass and for design of tunnel and underground excavation interim and final supports.

Compiled discontinuity orientations defined by strike, dip, and dip direction shall be analyzed using Rocscience's software program Dips V6.0 (Rocscience, 2010), or other equivalent analysis software. The resulting stereonet plots of data shall be used in estimating appropriate orientation adjustments (R_A) for calculating the RMR of rock. The discontinuity data shall also be used for estimating structurally controlled roof and wall failures in excavations (e.g., wedge failures) and for qualitatively estimating the slope stability at rock portals, including basal slip, wedge, and toppling failures as outlined in Section 8.5.2 of this TM.

3.5.3.4 Hydrogeology

The hydrogeology of both soil and rock sites shall be evaluated for tunnels and other underground structures and presented in accordance with FHWA (2009), including groundwater elevations of static and perched water zones derived from published and research sources, geotechnical field investigations, and groundwater investigations of proposed excavations. The site characterization shall include the checklist for GBRs included in Table 4-3 of FHWA (2009). The hydrogeologic data shall be used to define static groundwater elevations, seasonal fluctuations, flow directions, hydraulic conductivity, perched and confined aquifer conditions (artesian), porous medium or fractured medium, pH, temperature, and water chemistry.

Hydraulic conductivities shall be calculated based on data collected in accordance with TM 2.9.1, including pumping and slug tests, packer tests, open borehole seepage tests, and infiltration tests. For groundwater monitoring, monitoring wells and piezometers shall be installed and monitored for at least one year but preferred for multiple years (i.e., wet and dry years). Procedures for calculating hydraulic conductivities from pumping test data and effective hydraulic conductivities from borehole packer tests and falling head tests shall be implemented in accordance with FHWA (2009) Chapter 3.5.6 and FHWA (2002) - Subsurface Investigations – Geotechnical Site Characterization – Reference Manual.

Where groundwater inflow or dewatering is a concern, hydraulic conductivity testing shall be conducted by the designer as part of the subsurface investigations in accordance with TM 2.9.1 and shall include permeability tests, pumping tests, slug tests, packer tests, open borehole seepage tests, and/or infiltration tests. These tests are further described in ASTM D 4043 and shall form the basis of understanding groundwater occurrence, hydraulic pressures, and groundwater flow characteristics.

The designer shall identify conditions that could result in design changes, construction delays, and unanticipated construction costs due to unanticipated groundwater conditions. Unexpected groundwater conditions can include but are not limited to instantaneous inflows and sustained flows higher than estimated, groundwater barriers, flowing saturated soils, geothermal waters, and gas-bearing water. The designer shall investigate and quantify all potential groundwater conditions that could result in changed conditions for the project and fully develop and explain the variability of potential hydrogeology conditions in the GDR and the GBR for each project structure according to TM 2.9.2.

Water level measurements and/or hydraulic pressures shall be measured for predicting uplift pressures and hydraulic pressures on tunnel lining systems. Measurements and pressures shall be taken at least one diameter above and below the tunnel structure. Groundwater characterization shall account for potential variations resulting from seasonal changes, rainfall, irrigation, and other factors.

3.5.4 Design Issues

3.5.4.1 Groundwater Management

Influences of dewatering on existing structures (e.g., settlement) and on project excavations shall be included in design of below ground excavations and for calculating uplift pressures for slab design. The designer shall provide calculations for estimates of inflows including initial flows (e.g., flush flows) and sustained flows (long term), which are dependent on the occurrence of groundwater (static, artesian, fracture systems, etc.), hydraulic conductivity, hydraulic head, volume of water reservoir source, and groundwater barriers such as aquitards, aquicludes, and faults.

3.5.4.2 Seepage Control

Undrained and drained tunnel designs shall be considered. Tunnels (i.e., bored and mined) may be designed as undrained (i.e., with waterproofing) with the objective of eliminating impacts to groundwater and surface water resources due to groundwater drawdown. Drained tunnels may be viable for specific conditions such as short tunnels or where pre-construction grouting of the rock mass is applied to minimize long-term inflows. Conditions for design of undrained and drained tunnels are outlined in TM 2.4.5, for static loads resulting from groundwater pressures. For guidance on watertightness and drainage for tunnel structures refer to TM 2.4.5, Section 6.2.5. Cut-and-cover tunnels and U-wall trackway structures shall be designed as undrained to accommodate groundwater conditions (including seasonal changes) at the site and the designer's proposed excavation support and lining design.

3.5.4.3 Induced Ground Settlement (Movement)

The potential for induced settlement shall be evaluated using FHWA (2009) guidelines outlined in Section 7.5, which provide methods for calculating movement either due to a groundwater depression during dewatering or due to ground loss during tunnel excavation. The designer shall calculate the settlement trough depth, width, and shape to estimate the potential surface settlement and effects on surface structures. The settlement calculation shall include both single-bore and multiple-bore tunnels, where proposed.

Potential damage to structures due to ground settlement shall be evaluated using the guidance provided in Section 7.6 of the FHWA (2009). The relationships presented in FHWA shall be used for initial estimates of structural damage as part of tunnel lining and TBM design and planning for ground support to mitigate construction impacts. Mitigation methods of ground settlement shall be included in the design process as outlined in Section 7.6.

3.5.4.4 Gassy Ground Hazard

Tunnels and underground structures shall be designed to protect against potential hazardous conditions due to the presence of explosive, corrosive, or poisonous gasses (e.g. methane, petroleum-derived gases, hydrogen sulfide) entering tunnels. Identification and evaluation of potential gassy ground shall be part of the GDR and GBR for the tunnel. Special attention shall be given to areas of petroleum-bearing geologic materials, especially within or near known active or abandoned oil fields. The California Division of Oil and Gas and Geothermal Resources (DOGGR) maintains records of active and abandon oil fields and wells. Research of oil wells and petroleum-bearing areas at DOGGR shall be part of the source information to be reviewed as outlined in TM 2.9.1 and shall be investigated and evaluated as a potential geologic hazard with reference to TM 2.9.3. Mitigation of gassy conditions shall be included in the design of the tunnel lining system, ventilation system, and electrical and mechanical components for use in the tunnel. The designer shall include protections including gas-resistant waterproofing, impermeable membrane, full-time ventilation system, and gas-detection system monitoring both the crown and invert spaces, where gases could collect. Concrete and steel shall be protected against corrosive gases or gas-saturate groundwater. Designs shall conform to all local fire department requirements for confined space and fire safety.

3.5.4.5 Seismic Loads

Tunnels and underground structures shall be designed to resist the effects of ground shaking (i.e., ovaling/racking, longitudinal curvature, and axial straining) and permanent ground deformations (i.e., seismic slope instability, liquefaction, and lateral spreading) that result from the design earthquakes. These analyses shall be performed in accordance with TM 2.10.4 – Interim Seismic Design Criteria and TM 2.3.2, Structural Design Loads. Additional guidance can be found in FHWA (2009) Chapter 13. Seismic ground shaking parameters shall be developed in accordance with TM 2.9.6, Interim Ground Motion Guidelines, and ground failure potential shall be evaluated in accordance with Section 6.10 of this TM.

3.5.4.6 Static Loads

Earth loads shall be addressed in accordance with TM 2.3.2, TM 2.4.5, the FHWA (2009) Chapters 6 and 7, and the following provisions. The designer shall account for earth loads that include rock, soil, and groundwater.

Rock loads are influenced by gravity, rock structure orientation, discontinuity spacing, in-situ stresses, locked-in tectonic stresses, groundwater, and conditions such as underground excavation geometry, including tunnel intersections, chambers, pillars between parallel tunnels, and crossover tunnels. Usually rock loads are evaluated as roof loads, side loads, and eccentric loads. Other conditions in rock that shall be accounted for in designs include squeezing ground in which weak ground deforms under in-situ stresses, and swelling ground in which slaking or absorption of water causes an increase in volume. Tunnel designs including initial and final linings shall accommodate all rock loads.

Soft ground (soil) loads are influenced by plasticity, fines content, grain-size distribution, consolidation history, strength, construction methodology and sequencing, and groundwater. Tunnel support systems and shoring support for open excavations shall accommodate all combinations of soft ground loads.

Groundwater loads for design shall represent the full hydrostatic head or height of the column of water above the excavation (refer to TM 2.4.5). The tunnel lining shall support the full hydrostatic head along the length of any particular tunnel.

3.5.5 Excavation Issues

3.5.5.1 Rock Tunnels and Chambers

In accordance with Chapter 6 of FHWA (2009), design and construction of tunnels and other underground excavations in rock shall consider all potential rock stresses, failure modes, and difficult ground including but not restricted to wedge failures, rock burst, stress-induced failures, tunnel face or roof instability, squeezing ground, swelling ground, mixed-face conditions, high horizontal stresses, ground displacements, groundwater, and any combination of conditions that can adversely impact tunnel construction and performance. The designer shall evaluate and address all site conditions and material properties that can influence the behavior of rock in which an excavation is planned, including intact rock strengths, discontinuities, rock mass classifications, deformation modulus, abrasiveness, in-situ stresses, fault zones, gassy ground, flowing and running ground, water inflows, water pressures, geothermal conditions, and water chemistry.

Ground support shall be designed for initial and final support to resist all induced rock stresses and shall be considered with all applicable options of support, including rock bolts, ribs and lagging, shotcrete, lattice girder, spiles and forepoles, and precast segmented lining systems.

The designer shall apply methods of groundwater flow control in accordance with Chapter 6.7 of FHWA (2009). Groundwater controls may include pre- and post-construction grouting, ground freezing, and the use of pressurized-face TBMs operated in closed mode. Inflow of groundwater will be limited to not more than 100 gallons per minute for sustained flows. However, allowable inflow rates may be set by negotiation with local jurisdictions.

3.5.5.2 Portals and Shafts

Portal and shaft excavations shall be designed to be stable (i.e., static stability) during excavation and upon being put into service in accordance with Section 8 of this TM. All cut slopes and excavations shall be designed for any possible mode of failure to meet minimum FOS that are 1.3 or greater for short-term construction (temporary slopes or construction slopes) and 1.5 or greater for permanent or final design slopes. These FOSs shall apply to both soil and rock slopes for all portal and shaft excavations.

The overburden materials, weathered rock, rock, and pre-existing landslides shall be investigated and evaluated for physical conditions (lithology and geologic structure) and strength properties (friction angle – φ; and cohesion-c) in accordance with TM 2.9.1 and FHWA (2009) for use in calculating FOSs for excavated slopes and openings. The applicable modes of failure shall be evaluated by site-specific field investigations in accordance with TM 2.9.1, TM 2.9.3, Goelogic and Seismic Hazards Analysis Guidelines, and FHWA (2009). The designer shall evaluate all ground conditions affecting stability of the portal or shaft that may include overburden excavation, weathered rock and unweathered rock. The modes of failure shall be modeled using two-dimensional geologic cross sections and/or three-dimensional modeling for use in conducting slope stability analysis using limit equilibrium methods outlined in Section 6.8 of this TM to evaluate the FOSs of each slope and slope-support method.

Initial ground support for shaft excavations will depend on the site conditions, whether the excavation is above or below groundwater, and construction preferences of the construction contractor, and, therefore are not detailed in this document. However, support systems shall consider the following methods:

- Soldier piles and lagging in soils without water
- Ring beams and lagging or liner plate
- Precast concrete segmental shaft lining
- Steel sheet pile walls in soils with or without water
- Diaphragm walls cast in slurry trenches, which can minimize settlement and dewatering effects
- Secant pile walls or soil-mix walls instead of diaphragm walls

3.5.5.3 Cut-and-Cover Structures

Cut-and-cover tunnels shall be analyzed and designed in accordance with AASHTO LRFD Bridge Design Specifications with Caltrans Amendments, and Chapter 5 of the FHWA (2009) Technical Memorandum for Design and Construction of Road Tunnels – Civil Elements. Site characterization for cut-and-cover tunnels shall be in accordance with TM 2.9.1 and Chapter 3 of FHWA (2009), which can apply to soil or rock excavations; however, most cut-and-cover applications are expected at soil sites.

3.6 TRACK BED EMBANKMENTS AND EMBANKMENT FOUNDATIONS

3.6.1 Design Overview for Embankments

The geotechnical analyses and design guidelines included in this section supersede the technical guidelines provided in TM 2.6.7, Earthwork and Trackbed Design. Additional guidance is provided in TM 2.1.5, Track Design, and TM 2.10.10, Track-Structure Interaction. Engineering and geotechnical designers shall use these guidelines along with the CHSTP Specification standards for earthwork. This section is not intended to be a specification for materials and/or a construction document.

For trackway type selection purposes during the design phase, the feasibility of selecting ground-supported trackwork (on fill embankment or in cut) shall depend on the ability to meet the project performance criteria and shall consider cost and construction schedule. Other track guideway types for consideration and comparison against embankment-supported track include viaduct or retaining wall supported track. Embankment/fill design considerations shall also be linked to the earthwork material availability and handling strategy on a regional basis for CHSTP, including proportioning of cut and fill with the goal to balance quantities, where feasible.

3.6.2 Design Considerations

The embankments and foundations for trackways shall be designed considering the durability and longevity over the 100-year design life and the ability to meet applicable levels of required criteria that may vary depending on track segment, train speed and loading, frequency of train traffic, and track use such as mainline, stations, sidings, yards, etc., as described in TM 2.1.5. Geotechnical designs shall consider that embankment track substructure must meet geometric accuracy for maintaining the overlying track surface geometry (profile and alignment) and satisfy stability. This includes resistance to static load and dynamic load (passing trains), as well as extreme action/loading events resulting from seismic shaking, heavy precipitation, frost action, etc.

Design guidelines for the track bed (layers, dimensioning, and materials) overlying soil embankments or trackways in cut are provided in TM 2.1.5. Track designers shall provide minimum subgrade stiffness criteria as well as anticipated loading and required bearing capacity information to the geotechnical engineers regarding the required subgrade surface (in cut segments) and embankment fill surface minimum stiffness values (subgrade modulus and modulus of deformation) for evaluation during site exploration and design.

Designs for trackway on embankment or in cut sections shall be coordinated with other project features that might interfere with or impact the design or construction of CHSTP elements. This includes coordination with other design disciplines (including structural and civil, hydrology and hydraulics, and systems) to select appropriate earthworks for a given setting based on design constraints and potential conflicts, geotechnical subsurface investigations, and surface and groundwater issues. General earthwork terms with a typical track cross section are depicted on Figure No. 3-1.

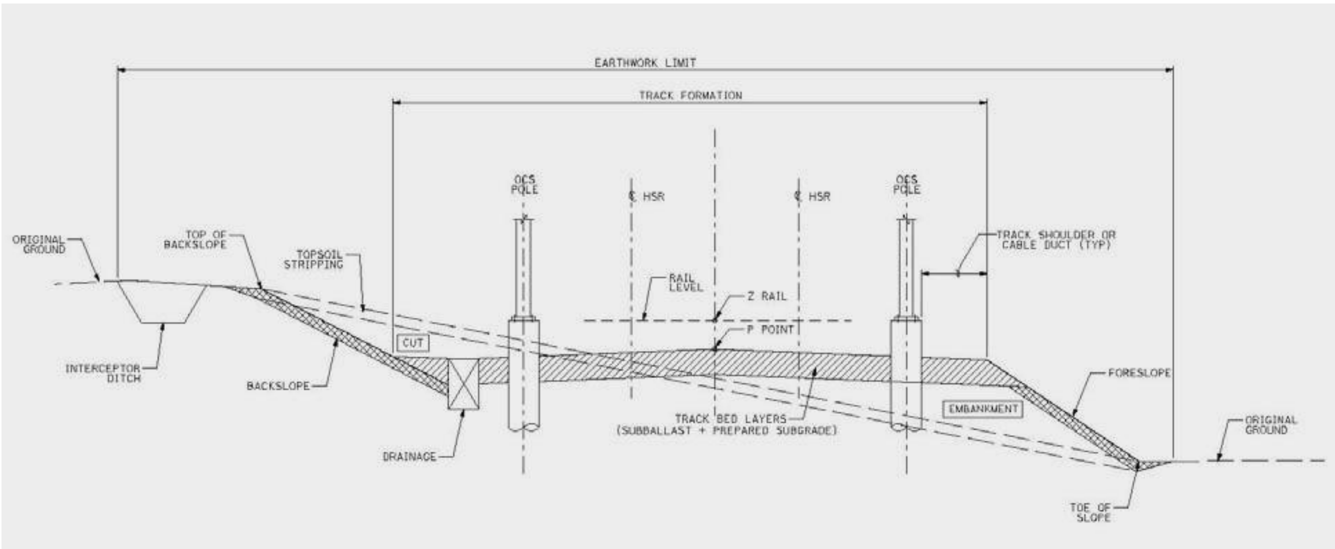

Figure 3-1: General Earthwork Cross Section and Terminology

Additional constraints that shall be considered include but are not limited to site geometry, access, time required to construct the fill and allow settlement, environmental issues, impact on traffic flow, and other construction activities. Analyses and design for embankment and cut sections during preliminary engineering require advance geotechnical investigations to obtain subsurface data representative of the underlying soil/rock that supports the trackway.

3.6.3 Soil Material Suitability for Use as Engineered Fill in Embankments

For design purposes, evaluation of soil suitability for re-use within the body of track embankments shall be based on the following guidelines which supersede the definitions for "Soil Quality Classes" presented in UIC 719R and Section 6.3.2 (Table 6-4) of TM 2.6.7.

Table 3.6.3-1 Soil Material Suitability for Engineered Fill in Embankments (per ASTM D 3282 / AASHTO subgrade soil group system)

Acceptable[1]	Unacceptable[2]
A-1-a	A-4 (CBR <10)
A-1-b	A-2-7
A-2-4	A-5
A-2-5	A-6
A-2-6	A-7-5
A-3	A-7-6
A-4 (CBR >10)	*

Notes:

* Rockfill is not acceptable for track embankment material.

1. In addition to the AASHTO criteria, the maximum soil particle size is limited to 3 inches.
2. Potential embankment fill source materials from groups A-2-7, A-5, A-6, and A-4 (with California bearing ration [CBR] <10) that can be shown by analysis and testing to meet all performance requirements (including strength, stability, settlement/deformation, long-term durability, etc.) shall be submitted for consideration of acceptability on a case-by-case basis. This includes marginal soil types from these groups that can be modified using soil amendments or additives such as cement, lime, hydraulic binders, etc., to be rendered suitable for use provided they meet all performance requirements (described above) as demonstrated by analysis and testing programs, including laboratory trial batching and field test sections.

Soil suitability evaluations shall also consider potentially detrimental properties as follows:

- Frost susceptibility – In order to reduce the potential to cause unacceptable disturbances to track geometry upon freeze/thaw cycles, soil types susceptible to frost, such as silt or clay, shall not be used for embankments in regions where cold conditions can occur.
- Corrosivity – Soil suitability shall also consider corrosion potential. Corrosive soils that are potentially detrimental to buried metal and/or concrete features (such as overhead contact system [OCS] poles, pipes/culverts, geogrid reinforcement, etc.) shall not be used.

3.6.4 Embankment Fill Design

Embankment foreslope inclinations shall be limited to 2 Horizontal:1 Vertical (H:V) or flatter for preliminary engineering design.

For embankments with dimensions exceeding 30 feet in height (measured from original ground to top of slope), designs shall include mid-slope benches for purposes of drainage and facilitating future access for maintenance reasons. Slope benches shall be 6 feet wide minimum with 6% gradient toward the low end of the fill slope, and shall include a lined gutter channel at the drainage surface. For tall embankments, slope benches shall be laid out on average of every 30 feet in height (allowance from 26- to 32-foot range is considered acceptable) and shall be connected to the surrounding ground surface for access.

At the top surface of fill embankment or cut subgrade level (immediately underlying track roadbed section), the design section shall have a transverse cross-slope drainage gradient of at least 4%, preferably sloped toward the outer edges of the embankment foreslopes. The 4% minimum cross-slope at subgrade surface must be met even after long-term settlement. A general track cross section depicting the required cross-slope drainage at the subgrade surface is shown below in Figure 3-2.

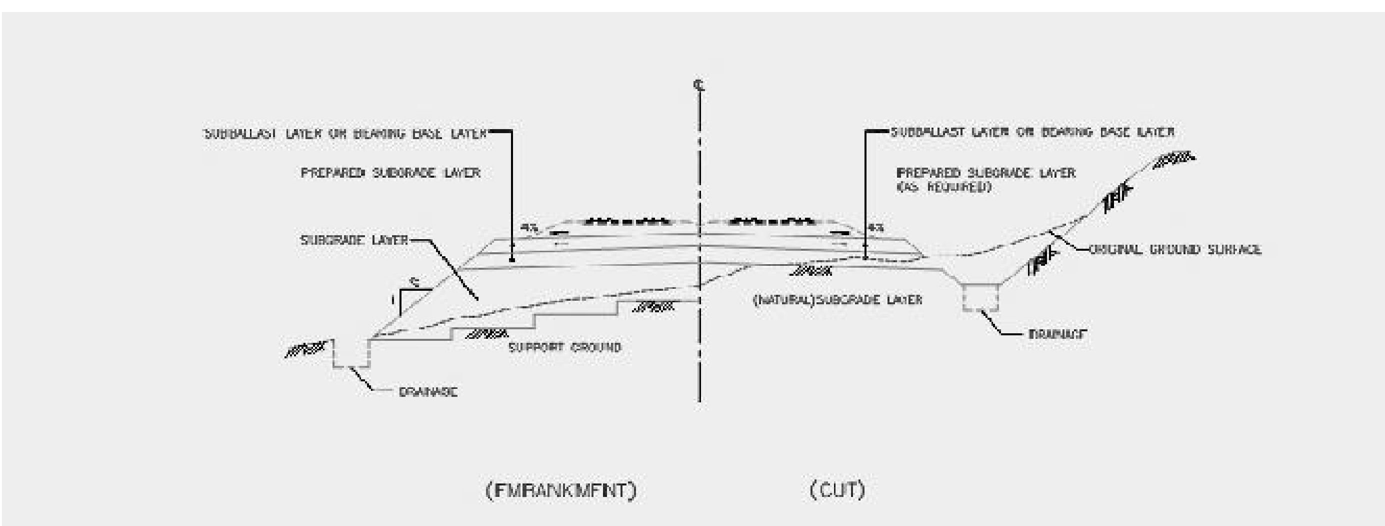

Figure 3-2: Track Cross Section with Sloped Subgrade Surface (4% Gradient)

The proposed geometry and dimensions for track embankment sections shall be confirmed by geotechnical analyses (short-term, and long-term conditions) for stability and settlement/movement in accordance with CHSTP TM guidelines. Embankment engineering guidelines that shall be considered for geotechnical design are summarized in FHWA Soil Slope and Embankment Design Manual FHWA/NHI-05-123, 2005.

Design shall consider potential problems associated with movements, including internal deformations (compression of fill materials) within the embankments, as well as external deformation in underlying foundation / native soils below the embankment. Internal deformation within embankments shall be controlled by design and use of fill materials (in accordance with CHSTP TM guidelines and referenced specifications) that have the ability to resist the expected loads. Properly designed and well-constructed soil embankments are not expected to excessively deform internally if adequate quality control is exercised with regard to material and compaction requirements. Deformation considerations for the embankment shall consider both vertical as well as lateral deformation movements. Vertical deformation movements are referred to as settlements. Lateral deformation movements can result in rotation of embankment earth structures at abutments, commonly referred to as tilting. Various design solutions that shall be considered for deformation problems are provided in Sections 7.6 and 7.7 of FHWA Soils and Foundations Reference Manual FHWA/NHI-06-088 Volume I dated 2006, UIC 719R 3rd edition dated 2008, and FHWA Soil Slope and Embankment Design Manual FHWA/NHI-05-123 dated 2005.

Since overstressing the embankment or foundation soil may result in failures that can occur when embankments are built on low-strength foundation soils without special foundation treatment, the track substructure (foundation/embankment system) shall be analysed for stability by the geotechnical designer. For assessment of load due to the earth embankment structure, designers shall assume fill soil compacted to at least 95% of maximum density per ASTM D1557 for estimation of soil unit weight. Foundation soils and embankments generally provide adequate support for transportation infrastructure, provided that the additional stress from geo-structures and added loads (including passing trains) does not exceed the shear strength of the embankment soils or underlying strata (NRC/TRB Ariema and Butler, 1990). Potential failure modes that shall be considered include bearing capacity, displacement failure, translatory failure, rotational sliding failure (extending through the foundation), and lateral squeezing. Analytical procedures for use to assess stress distribution in soil foundations underlying embankment fills is given in Section 7.3 of FHWA/NHI-06-088, Volume I, dated 2006. Guidelines for additional stability assessments for both fill slopes and natural or cut slopes are provided in Section 6.8 of this TM.

For the FOS against bearing capacity failure, the level of stress in the subgrade material (directly underlying track structure) due to loading of track structure components and ballast, if present, plus loading from trains shall not exceed an allowable bearing pressure that includes a minimum FOS of 2.5.

For stability and settlement analysis, consideration shall be given to additional actions/loading due to dynamic load from passing trains and also extreme events (seismic shaking, liquefaction and related strength loss due to seismic load, etc.). Seismic design guidelines for embankments and earth structures are provided in Section 6.10 of this TM. The seismic case evaluations and associated analyses should be displacement based leading to estimates of potential lateral deformations of embankments/slopes and ground settlement.

Embankment section designs shall avoid having trackways straddle the cut/fill line on side-hill sections of mainline segments, where feasible. Where embankments are to be located and constructed on slopes or where a new fill is to be placed against an existing embankment, the slopes of the original hillside or existing embankment shall be benched in order to provide a notched interface between the new fill and the existing ground. Bench widths are expected to be variable depending on the slope angle; however, bench heights shall be limited to 4 feet. A keyway shall be excavated to provide support for the toe of new fill slopes constructed against slopes.

Design recommendations shall be provided to control subsurface drainage since it is integral to the performance and stability of earth structures as well as track segments in cuttings (cut ground). Standard design guidelines for longitudinal (sideline) subdrainage features at track shoulder subgrade zones are provided in UIC 719R – Section 2.8.

For reaches of earth embankment at transition zones (immediately adjacent to bridge and viaduct abutments, tunnels, cut-and-cover structures, and cut sections with an abrupt topographic change) the approach embankment shall be designed to minimize the potential for differential settlement and to provide a smooth transition in the structural stiffness between different infrastructure features.

3.6.5 Embankment Strengthening and Stabilization using Geogrids

For mainline track segments, embankment design for fill sections greater than 5 feet in height and with slope inclinations steeper than 2.5H:1V shall include horizontal layers of geogrid-reinforcing material extending a minimum of 8 feet from the outer edge surface of fill foreslopes inward toward the center of the embankment body. The geogrid reinforcing is required in order to improve strength/resistance of the foreslopes of the fills and to enhance overall durability of the earth structures for HST mainline track segments. The maximum vertical spacing between consecutive layers of geogrid reinforcement shall be 1.5 feet. A generalized embankment slope section detail is shown below in Figure 3-3.

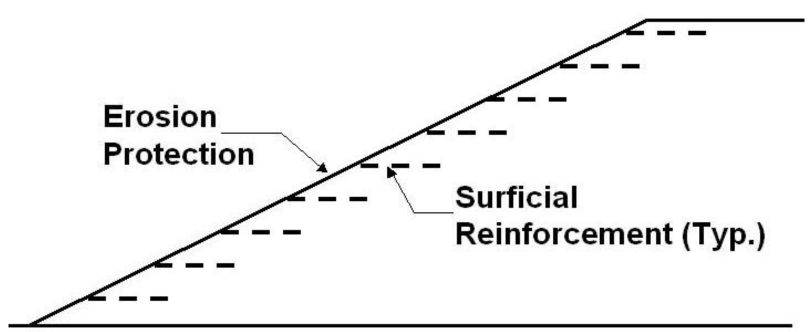

Figure 3-3: Geogrid-Reinforced Embankment Foreslope

Embankment designs shall consider the need for additional layers of continuous horizontal geogrid reinforcing across the full width of embankments to strengthen the body of fills and control deformation for segments where only poorer quality fill types (per Table 3.3.5-1, second column) are available, and/or where there are areas of weak foundation conditions (based on site exploration and geotechnical analysis). Geotechnical evaluation methods for use in the design for geogrid-reinforced embankments and control of embankment deformation are presented in the following technical guidance reference documents:

- Geosynthetic Design and Construction Guidelines, Chapter 7, FHWA-HI-95-038, 1998
- Soils and Foundations Reference Manual, Chapters 6 and 7, FHWA-NHI-06-088 Volume I, 2006
- Soil Slope and Embankment Design Manual, Chapter 8, FHWA-NHI-05-123, 2005

The designer shall select a geogrid material with adequate tensile strength for the proposed use and shall give consideration to other important aspects and properties (such as durability, degradation resistance, creep behavior, high modulus, protective polymer coatings, other mechanical properties that are time dependent, etc.) to meet CHSTP performance requirements. Metallic reinforcing elements shall not be used, since they are potentially susceptible to stray current corrosion that causes significant loss of section over the life of infrastructure supporting track.

3.6.6 Special Requirements for Embankments at Floodplains, and at Fault Crossings

For design of embankment segments located in floodplains, the level of highest water shall be established by the hydrology and hydraulics studies based on the levels of known floods. The embankment fill section shall be designed to protect foreslopes located within the highest water level zone with a surface layer of drainage material and riprap protection as well as the use of an underlying "drainage layer," as shown in Figure 3-4. The drainage layer shall extend upward along the foreslopes to the estimated high flood water level plus 1.5 feet. The granular drainage material shall contain less than 5% fine-grained material (passing the No. 200 sieve) and comply with Terzaghi's filter criteria, as summarized by Cedegren (1989):

$$\frac{D_{15}(filter)}{D_{85}(soil)} < 5 < \frac{D_{15}(filter)}{D_{15}(soil)}$$

and

$$\frac{D_{50}(filter)}{D_{50}(soil)} \leq 25$$

Where D_{15}, D_{50}, and D_{85} are the particle sizes at which 15, 50 and 85%, respectively, of the material by weight is smaller. D_{15}(filter) denotes the D_{15} value for the filter material, D_{15}(soil) denotes the D_{15} value for the fill or subgrade material in contact with the filter, and so forth. Additionally, the filter material should not be gap or broadly graded. The design process for riprap shall follow the approach provided in NCHRP Report 568, Riprap Design Criteria, Recommended Specifications, and Quality Control, NCHRP, 2006.

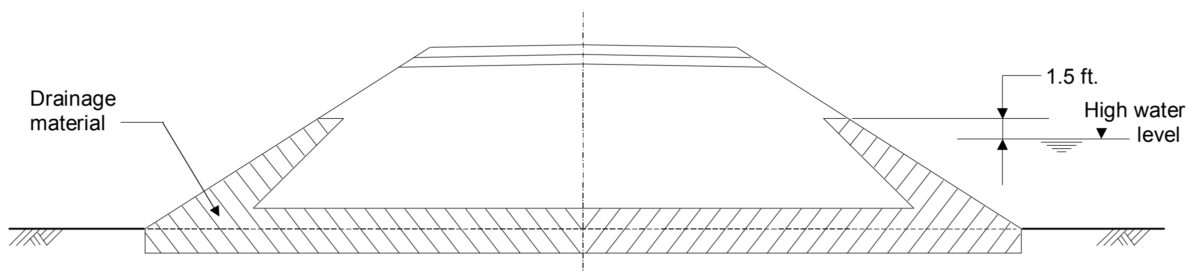

Figure 3-4: Drainage Layer under Embankment in Floodplain

For embankments located in track segments in wet areas where the water table is permanently or periodically at ground level, the embankment shall be constructed on a layer of drainage material as shown in Figure 3-5. This material shall not deteriorate or swell when immersed in water. It shall be well graded with no more than 10% passing the No. 200 sieve. The grading of the drainage material shall comply with Terzaghi's filter criteria against the subgrade original ground material as described above. The thickness of this drainage layer shall be related to the topography of the wet zone, but no less than 1.5 feet. In flat ground areas, the thickness of the layer shall be such that, after consolidation settlement of the bearing subgrade soil, the height of the drainage layer shall be at least 9 inches above the natural ground.

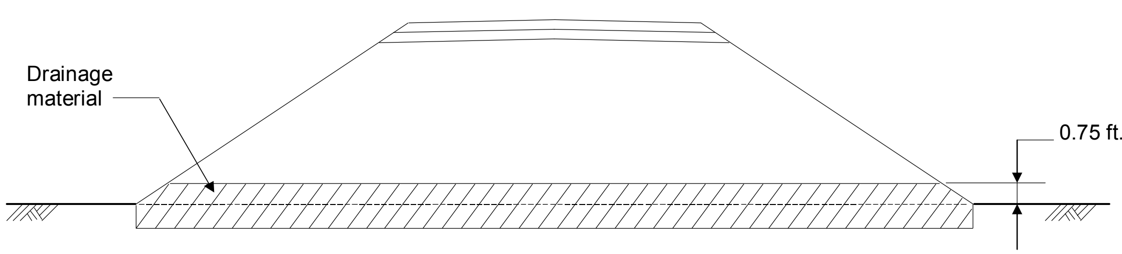

Figure 3-5: Drainage Layer under Embankments in Wet Locations

For locations where transverse box culvert drainage structures or pipes will be constructed within the body of trackway embankments, the embankments shall be designed to minimize the potential for differential settlement and to provide a smooth transition in the fill stiffness between these different infrastructure features at transition zones.

Earth structure designs at these locations shall include “approach embankments,” and the design details shall specifically take into account the geometrical, geological, and geotechnical conditions of the site and the anticipated construction sequence. Design examples for transitions from earthworks to culverts or other railway infrastructure features are given in UIC 719R, and Innovative Track Systems INNOTRACK Guideline for Subgrade Reinforcement with Geosynthetics, Section 9.4 of Report D2.2.6 Project No. TIP5-CT-2006-031415, dated 2009.

Earthquake fault crossings at locations where track segments on embankment or in cut cross earthquake faults classified as hazardous (subject to ground movement/displacement due to potential fault rupture) as defined in TM 2.10.6 Fault Rupture Analysis and Mitigation, the earthworks supporting trackway infrastructure shall be designed according to guidelines in TM 2.10.6.

3.6.7 Embankment Settlement (Magnitude and Rate) and Tolerable Deformations/Movements

The vertical deformation settlement of embankments (which also affects overlying track bed structure) is a combination of the settlement movement of the foundation on which it is resting plus settlement of the embankment fill, as shown in Figure 3-6. Conventional settlement analyses shall consider ‘immediate’, ‘consolidation’ and ‘secondary’ components of settlement against the requirements of CHSTP. For analysis of embankments, calculation procedures that shall be used to assess soil settlement are given in the following references:

- Soil Slope and Embankment Design Manual, chapters 4 and 8, FHWA-NHI-05-123, 2005
- Soils and Foundations Reference Manual, chapter 7, FHWA-NHI- 06-088 Volume I, 2006

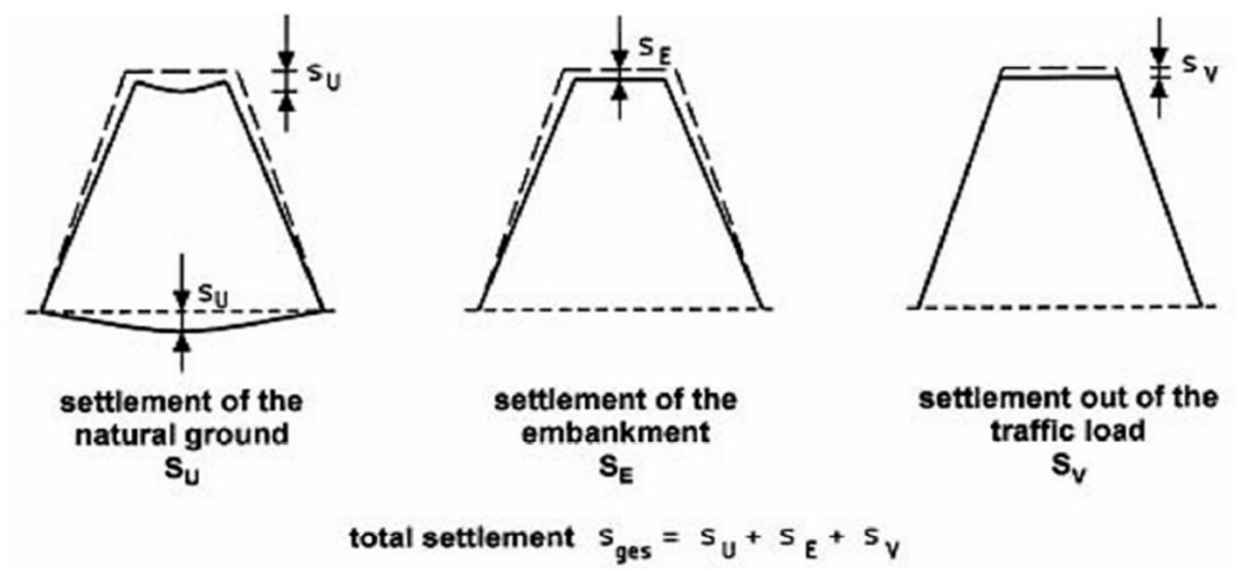

Figure 3-6: Settlements of Embankments

Reference: Figure No. 21 of UIC-719R (2008)

Geotechnical evaluations for embankments and their foundations shall also include the settlement contribution from surcharge/track load, and additional loading and/or ground deformation due to earthquakes.

In general, the settlement of the foundation is more difficult to evaluate than that of the embankment fill, and quite often it is much larger. This is particularly true when embankments are resting on soft compressible soils. As a general guideline for embankments, based on data from earth dams constructed by the United States Bureau of Reclamation, the estimated long-term settlement of a well-compacted earth fill embankment ranges between 0.2 and 0.4% of the embankment height.

Once the embankments are designed based on safe allowable bearing pressures and satisfying stability, the residual settlement (following track installation) estimates and differential displacements between locations along the length of the embankments shall be evaluated to assess potential serviceability problems for the track bed.

Settlement occurring after construction of the permanent way tracks shall be limited along general track segments as follows:

Table 3.6.7-1 Settlement Criteria - Residual Settlement After Placement of Tracks

Residual Settlement [1]	Non-Ballasted Track	Ballasted Track
Differential Settlement [2]	≤ 3/8 inch over 62 feet	≤ 3/4 inch over 62 feet
Uniform Settlement	≤ 5/8 inch	≤ 1-1/8 inch
Rate of Settlement (per year)	≤ 3/16 inch	≤ 3/4 inch

Notes:

1. Prior to placement of tracks, embankment sections shall be instrumented and monitored for a period of at least 6 to 12 months to ensure compliance with these requirements for residual settlement.
2. Differential settlement along track segments is measured along the track (surface profile uniformity) in the vertical plane of each rail at the mid-point of a 62-foot-long chord.

If the predicted differential displacements are excessive and exceed track profile tolerances, then embankment designs shall require further modification, and/or ground improvement may be needed for the foundation systems. Where predicted settlement movements and their duration are excessive, change the design from an embankment to a viaduct or other structure shall be considered.

Settlement estimates shall show not only how fast construction should proceed (appropriate timeframe for when installation of overlying permanent way track structure can begin) but also shall demonstrate that any ongoing settlements, which occur after the rail line is opened, can be rectified economically by shimming and/or adjusting of track fasteners (vertical adjustment capacity approximately 0.25 inch), or performing other routine track maintenance throughout the long-term design life of the earth structure; if not, advance mitigation alternatives shall be considered. For the purpose of this section, “long-term” shall be defined as 100 years. Clearances over rail tracks and roadways shall include an allowance for anticipated short-term and long-term movements of earth structures.

Considering that settlement of earth structures is time-dependent and will vary by segment, the geotechnical engineers shall evaluate and establish the time duration waiting (leaving) period following initial fill embankment placement before releveling the subgrade and subsequent construction of the overlying track bed permanent way is allowed to take place. An illustration of various settlement parts related to time is shown in Figure 3-7. Based on international experience for other HST systems, the waiting period duration is typically 6 to 12 months, or more. To meet CHSTP design and performance requirements, a periodic settlement survey program shall be developed by the geotechnical engineer and then implemented during and after the construction phase to monitor settlement at the acceptance check timeframe after laying track, and then long term residual settlement as part of the track maintenance program.

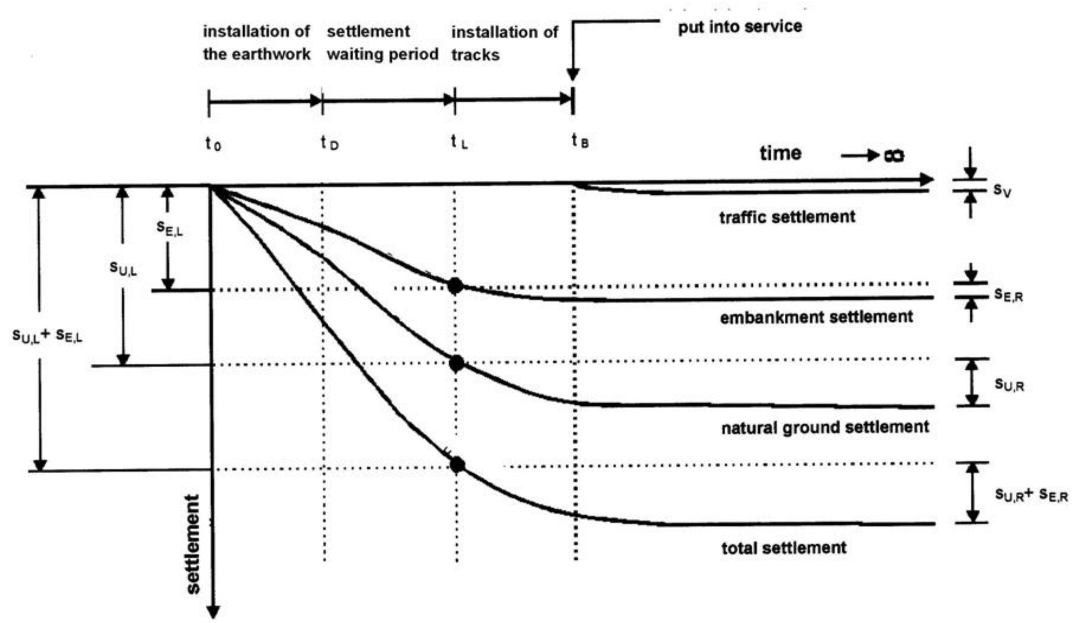

Figure 3-7: Different Settlement Parts by Time
Reference: Figure no. 22 of UIC-719R (2008)

Per UIC 719R Section 2.10.2.2, elastic vertical displacement of earthworks under load is usually not a design criterion, as resistance of continuous supporting structure generally implies very low vertical displacement (typically 0.1 to 0.2 mm on top of supporting structure). However design criteria may exist to limit elastic deformation to a percentage of deformation of track components to manage the global track stiffness.

3.6.8 Embankment Foundation Settlement Mitigation and Foundation Modification Using Ground Improvement Methods

For track embankment segments or at-grade trackway features that do not meet settlement criteria or indicate stability problems, advanced mitigation measures such as pre-loading, over-excavation and replacement, or other ground improvement methods shall be considered for geotechnical design.

Ground improvement measures may also be necessary for advance mitigation of potential seismic hazards (such as liquefaction or seismic stability) or other geologic hazards such as collapsible soils, potential hydro-consolidation, regional subsidence, etc. The selection of mitigation methods or candidate ground improvement options for preliminary design shall follow the process described in detail in the FHWA Ground Improvement Reference Manuals, Volumes I and, FHWA-NHI-06-019/020 dated 2006.

A settlement monitoring program shall be developed and implemented by the geotechnical engineer during the construction phase for any mitigation method selected. Interferometric Synthetic Aperture Radar (InSAR) techniques shall be considered as possible methods for large scale regional monitoring in addition to traditional surveying and the use of geotechnical instrumentation during and after construction.

For track segments located in relatively large-scale geographic areas where deep-seated regional subsidence is an ongoing problem with expected duration to continue over some or all of the design life, typical ground improvement measures may not be economically feasible. The geotechnical engineer shall identify the approximate regional boundary limits for these segments and shall provide information to the track and civil designers regarding expected range in total magnitude and estimated rate (inches per year) of future regional subsidence movements. .

3.6.9 Evaluation of Earthwork-Related Factors for Shrink/Swell (Shrinkage and Bulking) Estimation

The geotechnical engineers shall provide shrinkage/swell factors for the anticipated cut and embankment fill soils for purposes of earthwork quantity computations. Available reference sources in common use for approximate factors (earthwork shrink/swell) are listed as follows:

- Shrink/Swell Factors for Common Materials - Exhibit 4.6-F, FHWA Geotechnical Technical Guidance Manual (draft) 2007
- Geotechnical Design Manual M46-03 - State of Washington Department of Transportation, Chapter 10 Soil Cut Design, Table 10-1 Approximate Shrink/Swell Factors, WADOT Manual dated September 2005

Earthwork quantity estimation shall also consider embankment overbuild (higher elevation than design profile) that may be necessary on a segment-by-segment basis to allow for short-term and long-term settlement movement of the embankment and/or underlying foundation soils supporting trackway embankments.

3.6.10 Erosion Control for Embankment Features

Geotechnical studies for design shall provide recommendations to the engineering designers for erosion control needs. Evaluations shall be based on characterization of embankment materials, potential water sources, railway geometrics and slope design. Design recommendations shall be provided to control surface drainage when integral to the design or performance of the earth structures, such as surface drainage ditches on slopes, interceptor ditches, and drainage channels. Geotechnical evaluation to support selection and preliminary design for erosion control shall follow the processes described in the reference document titled Design and Implementation of Erosion and Sediment Control – Reference Manual, FHWA NHI-05-013, 2006.

The design details or requirements shall be incorporated in the geotechnical report and construction plans. Geotechnical discipline shall coordinate with the hydrology and hydraulics and civil design disciplines for erosion control since they provide project-wide drainage design for the control of surface drainage. If long-term erosion control measures will include establishing vegetation on slopes, then consideration shall be given to the use of erosion mats or other stabilization methods for slope inclinations steeper than 3H:1V.

Geotechnical design recommendations shall also include evaluation of temporary construction erosion control requirements on cut-and-fill slopes when integral to geotechnical design or performance. For example, the requirement to provide bench drainage during construction of slopes may be required to ensure construction-phase stability.

3.7 Retaining Walls, Fill Walls, and Reinforced Soil 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 U-walls, 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, California (Caltrans) Amendments, and the FHWA Earth Retaining Structures Reference Manual (FHWA, 2008). 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, OCS poles, 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.

Retaining walls 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 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 on include:

- Wall/slope to be located primarily in a cut or fill
- Excavation/shoring to 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 to be located on or above the wall
- Impact of the wall/slope on the performance of adjacent foundations, structures, or utilities, considering both the requirements of the adjacent features and the performance impact the installation of the new wall/slope will have on them for the various issues listed above
- Architectural requirements
- Overall cost and 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)
- 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 or facilities
- Impact to environmentally sensitive areas
- Consideration for the foundation embedment and type anticipated, which requires coordination among 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.

3.7.3 Limit States and Resistance Factors

Geotechnical designs for retaining walls shall be performed in accordance with AASHTO LRFD Bridge Design Specifications with California (Caltrans) Amendments. However, the Amendments confirm that abutment foundations are not subject to LRFD design approach, and so conventional WSD shall be used. 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, non-gravity cantilevered walls, anchored walls, MSE walls, and prefabricated modular walls.

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, as well as in the California Amendments, and so they are not reprinted here.

3.7.4 External Loads and Stability Analysis

AASHTO LRFD with California Amendments shall be used for evaluation of stability for retaining walls and abutments. Those provisions include calculation methods for various wall types and shall include 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.

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
- Wall inclination

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.

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. An exception to this requirement is for U-walls (retaining walls with continuous base slab between them) that are used where the top of trackway subgrade is below the groundwater table/flood level. No permanent dewatering shall be assumed for design of U-wall sections that are undrained structures subject to hydrostatic pressures, both laterally and vertically (buoyancy).

Retaining wall drainage 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 satisfies 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 that intersect 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, 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 may 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 Soil Systems

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

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 movement tolerance 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, tracks, 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. 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 1 inch or less. 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 has 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 RSS embankments and 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 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 facing systems and geosynthetic reinforcements are available; however, embedded metallic-strip reinforcing elements shall not be used since they are potentially susceptible to stray current corrosion that causes significant loss of section over the life of infrastructure-supporting track. 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 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 provides 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 to 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.).

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
- 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 shall initiate the design effort and develop 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 to be coordinated and carried out 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 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 of the following main functions, including:

- Increasing bearing capacity, shear or frictional strength
- Increasing density
- Controlling or reduce deformations
- Accelerating consolidation
- Decreasing imposed loads
- Providing lateral stability
- Forming seepage cutoffs or filling voids
- Increasing resistance to liquefaction
- Transferring 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 the FHWA 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 FOS 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 spoils (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 groundwater 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 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. A preliminary design is developed for each method identified under Preliminary Selection, and a cost estimate is prepared on the basis of available data. The guidance in developing preliminary designs is contained within technical summary sections of the FHWA manual.
7. The selected methods are then compared, and a selection is made by considering performance.

3.7.10 Lateral Support of Temporary Excavation Systems

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

3.8 Slopes

3.8.1 Overview

This section addresses the analysis and design of slopes, including cut slopes, fill slopes (embankment slopes), landslides, and natural slopes. In addition to the provisions herein, sloped excavations shall be designed and constructed in accordance with any and all local, state, and federal regulations, including but not limited to Occupational Safety and Health Administration (OSHA) and Cal/OSHA requirements.

The provisions contained herein supersede any slope analysis and design provisions contained in TM 2.6.7.

3.8.2 Qualifications

The analysis and design of existing slopes, landslides, and permanent cut slopes shall be performed in a joint effort by a California Certified Engineering Geologist (CEG) and Registered Geotechnical Engineer (GE). The CEG shall oversee the development of surficial and subsurface geologic structure, stratigraphy, and hydrogeologic conditions. The GE shall oversee the development of the soil and rock engineering parameters and shall oversee the stability analyses. The CEG and GE shall work cooperatively to assure consistency and continuity in the evaluations.

Analysis and design of new fill slopes and embankments on relatively level ground do not require the involvement of a CEG, but the geotechnical reports must meet signature requirements of TM 2.9.2.

3.8.3 Slopes Requiring Evaluation

Slope stability shall be evaluated where any of the following conditions are present:

- Where slope stability related geologic hazards are present as defined in TM 2.9.3.
- Soil and sedimentary rock slopes steeper than 5H:1V and igneous and metamorphic rock slopes steeper than 3H:1V, and any flatter slope where the following adverse conditions could exist:
    - Where there is a potential that adversely oriented and kinematically unstable geologic joints, bedding, or slip surfaces are potentially weak
    - Where evidence of prior landsliding is present (see TM 2.9.3 for evaluation guidelines)
    - Where quick or sensitive clay conditions are present

  - o Where liquefaction-related lateral spreading conditions are present, they shall be evaluated using the methods described in Section 6.10
  - o Any other conditions that the GE or CEG feel warrants slope stability evaluations

With respect to the location of a slope relative to the project right-of-way, the following criteria shall be used to assess when slope stability evaluations are required:

- Where the movement of a potential slide mass either directly or indirectly could affect HST facility operations or integrity, for example, where a slope failure slide mass could intersect or envelop tracks, stations, or appurtenant facilities
- Where instability of slopes away from the HST facilities could impact operations or integrity, for example, landslides, debris flows or rockfalls that may originate away from HST facilities, but that could be deposited within SHT facilities

3.8.4 Design and Analysis

The following sections provide requirements for design and analysis of soil and rock slopes. Static slope stability analysis shall be considered in the Service 1 load case. Slope performance is expected to be such that normal operation of the HST facilities is maintained. Requirements for seismic analysis and design of slopes are provided in Section 6.10 of this TM.

3.8.4.1 Loads, Unit Weight, and Surcharges

Where structures, trains or other non-earth materials are present within a slope area, their corresponding surcharge loads shall be included in slope stability analysis. Loading shall be established in accordance with TM 2.3.2. Soil loads shall be taken at their nominal values; no load factors are applied to soil in slope stability analyses.

3.8.4.2 Slope Stability Analysis Methods

Analysis methods for soil and rock are presented below. In some cases, geomaterials may exhibit behavior that is intermediate between soil and rock. Such materials are sometimes referred to as intermediate geomaterials (IGM). Since slope stability methods specific to IGMs are not generally available, both soil and rock methodologies shall be applied for IGMs, and the most conservative result shall govern design.

3.8.4.2.1 Soil

Static soil slope stability shall be evaluated by calculating a FOS using limit equilibrium procedures with the method of slices. Permissible methods of limit equilibrium analysis include Spencer's (1967) for any slip surface shape, and Taylor's (1937) or modified Bishop's (1955) for circular slip surfaces. Both circular and non-circular potential failures surfaces shall be considered in the analyses. Search routines shall be used to find the slip surface with the lowest FOS. The slip surface with the lowest FOS is considered the "critical slip surface." Slip surfaces with FOS greater than the critical slip surface shall also be reviewed and considered in the design. Widely used and well-validated computer programs for slope stability analysis shall be used such as Slide by Rocscience, SLOPE/W by GEO-SLOPE International, XSTABL, UTEXAS3 by the University of Texas, or other equivalent software.

The FOSs may also be calculated using finite element or finite difference methods that employ the φ-c (strength) reduction method (Dawson et al., 1999). However, conventional method of slices limit equilibrium methods must be run in conjunction with φ-c reduction analyses. The φ-c reduction method does not require an assumed slip surface (as does the method of slices), which can be advantageous when subsurface stratigraphy or other factors could lead to slip surfaces with irregular geometry.

Infinite slope stability analyses shall be performed for soil slopes where shallow (6 feet or less) downslope seepage parallel to the slope face could develop. Such a hydrogeologic condition could result where a shallow layer is underlain by a less permeable layer (i.e., residually weathered soil over bedrock), and rainfall potential or temporary submergence is sufficient to saturate the less permeable layer. Infinite slope stability analyses shall be conducted in accordance with the analytical methods presented in Section 5.5 of FHWA (2005).

3.8.4.2.2 Rock

For the purpose of slope stability analysis, rock slopes are characterized herein by groups based on the anisotropic or isotropic characteristics of the rock mass. Therefore, the first step in analysis of the rock slope is to establish if the rock mass is anisotropic or isotropic.

The first group assumes the rock mass consists of heterogeneous rock masses with structural anisotropic systems of relatively regular discontinuities in the form of joint sets, bedding, fissures, or foliation. The strength and slope stability of these types of rock masses is typically controlled by the discontinuities, and analytical techniques for slope stability assessment shall consider the kinematic stability of blocks or groups of blocks sliding upon the discontinuities, or toppling. Limit equilibrium methods that calculate a FOS shall be used. These analyses shall consider blocks that are kinematically permissible as evaluated by the Markland (1972) method, block theory (Goodman and Shi 1985) or rock slope engineering techniques described by Hoek and Bray (1981) and Wyllie and Mah (2004). If computer software is used for rock slope stability analyses, it shall be well validated and widely accepted.

The second group assumes the rock mass consists of homogeneous and isotropic rock masses with irregular and/or closely spaced discontinuities that do not have well defined systematic planes of weakness. The evaluation of the stability of these types of slopes shall be based on the non-circular limit equilibrium techniques described above for soil, except that a suitable rock strength model shall be used such as General Hoek-Brown criterion (Hoek et al., 2002; Wyllie and Mah, 2004; Hoek, 2010).

Where rock slopes existing upslope of HST facilities and have the potential to shed rock pieces over time, an evaluation of the rock fall hazard shall be performed in accordance with the procedures outlined in the FHWA and Oregon DOT (2001) Rockfall Catchment Area Design Guide. Computer programs that model rockfall physics such as the Colorado Rockfall Simulation Program (CRSP III) or RocFall (by RocScience), or other equivalent software, may be used in conjunction with the FWHA procedures. Rockfall catchment basin width and inclination shall be designed to retain 99% of fallen rocks. If right-of-way is not available to size catchment basins to achieve 99% rockfall retention, additional mitigation measures such as rockfall protection walls, wire mesh, cable drape, or catchment fences shall be used in the design. In areas where rock fall is a critical problem, a railway slide fence with electronic warning system shall be installed in conjunction with an appropriate catchment ditch and rock fall retention system described above. Other warning systems for rockfall events that may be considered are as follows:

1. Acoustic sensing
2. Electromagnetic sensing
3. Seismic sensing
4. Visual sensing, using cameras

3.8.4.2.3 Input Data and Parameters

Input data and parameters used in slope stability analyses for both soil and rock shall take into consideration geology, groundwater and rainfall, and proposed geometry/topography. Soil engineering parameters shall be developed for use in slope stability analyses in accordance with Section 6.2 of this TM.

When available, empirical or historical data and direct observation within the geologic unit on the past performance of similar slopes shall be considered in slope stability evaluations. In particular, when assessing existing landslides, shear strength parameters back-calculated from previous failures shall be considered.

Drained or undrained shear strength parameters shall be selected, depending on the rate of loading and the permeability characteristics of the soil or rock. In general, undrained strengths should be used for relatively short-term loads and end-of-construction cases. Long-term stability should generally use drained strengths. In the analysis of existing landslides, residual shear strengths shall be used for existing landslide slip planes. FHWA (2005) Section 4 should be consulted for additional guidance on the selection of shear strength parameters.

3.8.4.2.4 Minimum Factors of Safety

The FOSs calculated with the methods above for soil and rock shall meet the following minimum requirements. For the Service 1 static slope stability case, the FOS is simply the inverse of the resistance factor (φ); FOS=1/φ.

Slopes shall be designed so that the minimum FOS of any slip surface (or φ–c reduction scenario) is 1.3 or greater for short-term construction cases and 1.5 or greater for long-term static cases. If a slope must be designed to meet the standards of another agency, such as a city, county, or right-of-way holder that has more stringent requirements, then the more stringent requirement shall govern.

Short-term construction cases could include:

- Sidewalls of sloped temporary excavations
- Temporary surcharge fill slopes
- Temporary back cuts
- The end of fill placement (sometimes referred to as the end of construction case) for fills over fine-grained foundation soils that will behave undrained or partially undrained during construction, and will gain strength over time
- Construction stages prior to the end of construction if the staging is such that an intermediate stage could be critical. Such conditions could arise when significant consolidation of fined-grained soils is allowed to occur between stages

3.8.5 Requirements for New Slopes

New fill slopes in soil shall be no steeper than 2H:1V. If 2H:1V slopes cannot be achieved because of geometric and/or right-of-way restrictions, the design will likely have to incorporate retaining walls.

Limitations on the inclination of rock slopes and cut slopes in soil are not imposed. Maximum inclinations shall be established based on achieving the minimum FOS and rockfall protection requirements described earlier.

For soil cut slope configurations over 30 feet in height, designs shall include mid-slope benches for purposes of drainage and of facilitating future access for maintenance reasons. Slope benches are typically 6 feet wide with 6% gradient toward the low end of the cut slope, and include a lined gutter channel at the drainage surface. For deep cut slopes in soil, slope benches shall be laid out on average of every 30 feet in height (allowance from 26- to 32-foot range is considered acceptable) and shall be connected to the surrounding ground surface for access.

3.8.6 Landslides

Evaluations of the stability of landslides require special attention and specialized techniques. Geomorphic evidence of landsliding shall be evaluated through examination of a series (different years) of aerial photographs (preferably stereographic, if available), geologic/landslide hazard maps, and topographic maps.

The method of analyses shall first take into consideration the style of slope failure that has occurred in the formation to be analyzed as well as the recognized behavior of that formation. Soil and rock engineering parameters shall also consider the proven performance values for the subject formations. Locating the slip surface is a critical step in the characterization of a landslide. Techniques that shall be considered for locating slip surfaces include surficial mapping, down-hole logging of large diameter borings, continuous sampling of borings, and monitoring with inclinometers. Slope stability analyses conducted for landslides shall consider sliding along the existing slip surface as well as sliding upon potential new slip surfaces.

Shear strength parameters for the stability analyses of landslides shall be based on the shear strength along the existing slide plane. The shear strength along an existing slide plane can be significantly lower than that of the surrounding intact soil or rock, because slippage has resulted in residual strength and slickensides (reorientation of grained-soil particles that decreases frictional resistance). Shear strength parameters used in landslide analyses shall consider values back-calculated from slope stability analysis of the pre-slide configuration and groundwater conditions. Back-calculated values should be compared with data obtained from laboratory testing. In performing laboratory testing to evaluate shear strength of a landslide slip surface, the shearing shall be performed along a pre-sheared surface using either a direct shear or ring shear test. It is preferable that the pre-sheared surface be that of the actual landslide slip surface, such that the laboratory specimen is prepared from a block sample taken directly from the slide plane in a large-diameter boring. If this is not possible, then an intact specimen can be used that is pre-cut along the slip surface with a knife prior to shearing. The shear test displacement shall be large enough such that no appreciable reduction in shearing resistance occurs with additional shearing (residual strength condition is achieved). With the direct shear apparatus, this typically involves repeated cycles of shear, with the shear strength plotted against the aggregate of the unidirectional shear displacement. Further background on these procedures can be found in Blake et al. (2002).

Additional guidance on the evaluation of landslides is provided in FHWA (2005).

3.8.7 Seismic Analysis for Design of Slopes

Seismic analysis and design of slopes shall be performed in accordance with Section 6.10 of this TM.

3.8.8 Slope Deformations

The potential for slope deformation shall be considered in the design. The potential mechanisms of deformation include settlement because of consolidation or compression, settlement because of collapse upon wetting, shrink and swell caused by changes in moisture content, and creep-type shear displacement. Section 6.6.11 of this TM addresses settlement due to consolidation and compression.

The potential for shrink and swell resulting from changes in moisture content and/or stress shall be addressed in design of cut slopes. Analysis shall be conducted to assure that movements due to shrink and swell are less than the maximum displacements permitted for the project. Removal of expansive soil or rock and replacement with non- or very low-expansive material shall be performed as needed to meet maximum allowable displacement criteria. Constraints on acceptable fill material types described in Section 6.6 of this TM are such that shrink and swell should not pose a problem for fill slopes and embankments.

Creep-type shear displacements occur in the downward direction of many slopes and are typically characterized displacement rates on the order of 1 inch per year or less. Creep may occur in a relatively shallow zone (i.e., the upper 5 to 10 feet) because of the effects of seasonal wetting/drying or freeze/thaw cycles. Deeper-seated creep movement can occur in association with historic, prehistoric, or incipient landslides. In existing slopes, creep may be observed by slope inclinometer measurements, or may be evidenced by tilted or curved trees. Creep potential shall be considered in slope stability design, particularly when evidence of creep is observed in the field, or where past experience has shown site soil or rock to be creep prone. Where creep movements could result in adverse effects to HST facilities, mitigation methods shall be employed in the design.

3.8.9 Drainage and Erosion Control for Slopes

Drainage provisions and permanent erosion control facilities to limit erosion (including soil erosion and rock slope degradation) are required for design of cut slopes. Surface drainage shall be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. Erosion control for cut and fill slopes shall be performed in accordance with Section 6.6.10 of this TM.

3.8.10 Slope Stability Mitigation Methods

Where the minimum required FOS cannot be achieved or the alignment cannot be relocated away from unstable slopes, measures to enhance slope stability or mitigate the effects of potentially unstable slopes are warranted. In addition to the discussion below, Section 9 of FHWA (2005) shall be used as guidance for slope stability mitigation methods. Methods of slope stability mitigation not described herein or in FHWA (2005) may be considered provided they can be proved effective through analysis and previous experience.

Subsurface drainage can enhance slope stability by reducing pore water pressures and increasing effective stress, and by reducing the driving stresses from saturated soils and water-filled cracks. Methods of subsurface drainage for slope stabilization include horizontal drains, wells, and trench drains. For long-term applications, passive (gravity flow) forms of drainage such as horizontal drains and gravity flow trenches are generally preferred over active systems such as wells that require pumping.

Unstable slopes can be mitigated through removal of the weak and unstable material and through replacement with a suitable compacted fill. This removal and replacement method is typically combined with the installation of subsurface drainage measures. Slope stability can be enhanced through grading that reduces the driving forces and/or increases the resisting forces. Buttresses constructed in the toe area of as slope can utilize weight and higher shear strength to increase resisting forces. Driving forces can be reduced by removal of material near the top of the slope.

Slope stability enhancements could include adding structural inclusions such as soldier piles, soil nails, anchors, dowels, or geosynthetics. The efficacy of any slope stability mitigation method shall be demonstrated through stability analyses using the limit equilibrium methods described earlier.

For cases where debris flow-type failures from existing natural slopes could impact HST facilities, debris flow diversion walls may be considered as a mitigation method. Culverts that are fed by drainages and tributaries should be sized with consideration for debris flow potential. Debris-flow diversion walls shall be designed to withstand the impact from debris flows and to control and redirect them such that there is no adverse impact to HST facilities.

3.9 GROUND IMPROVEMENT

Ground improvement shall be considered as a design alternative for conditions such as, but not limited to, the following:

- As an alternative to deep foundations where poor (i.e., compressible, weak, liquefiable) soils are present
- Mitigation of excessive settlements due to soft and/or compressible soils
- Slope stabilization
- Liquefaction mitigation
- Expedite settlement by enhanced drainage
- Make soil less permeable
- Improve ground behavior for tunneling
- Temporary support (underpinning) for existing foundations

Methods of ground improvement generally consist of the following:

- Vertical drains
- Vibro-compaction
- Deep dynamic compaction
- Stone columns
- Compaction grouting
- Jet grouting
- Permeation grouting
- Deep soil mixing

Ground improvement design shall be performed in accordance with FHWA (2006) “Ground Improvements Reference Manual, Volumes 1 and 2, FHWA-NHI-06-019 and FHWA-NHI-06-019.”

Any ground improvement program shall be reviewed and approved by the program management team (PMT). Ground improvement shall be designed to achieve performance requirements with respect, global stability, bearing capacity, and settlement. The ground improvement design shall include a construction quality assurance program that specifies field observation and testing methods necessary to verify the design objectives are achieved.

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 two 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 the two 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 movement/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 movement/settlement between mitigated and non-mitigated soils. Additional measures may be required to limit differential movement/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 HSTs 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

Limits on site response analyses are presented in TM 2.9.6 for 30% design.

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 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 assessed that liquefaction will be triggered, the engineering consequences of liquefaction shall be evaluated. In addition to triggering criteria for liquefaction, the designer shall also consider the allowable deformation values described in Section 6.3.5 of this TM and the long-term, post-construction performance requirements for earth and fill conditions.

3.10.7.1 Compositional Criteria for Liquefaction Susceptibility of Silty and Clayey Soils

Evaluation of whether silty and clayey soils meet the criteria for liquefaction susceptibility shall be performed primarily 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. According to Bray and Sancio (2006), field-based methods for liquefaction triggering can be used for silty and clayey soils with PI $\leq$ 12. Using the above criteria in evaluation of liquefaction triggering at the 30% design level potentially includes some conservatism. Therefore, for transitional soils, it is recommended to perform "undisturbed" sampling and cyclic testing to characterize these transitional soils in cases where the design and its costs are significantly affected by this evaluation in later stages of design.

For fine-grained soils (especially soils that are potentially sensitive) that do not meet the above criteria for liquefaction, cyclic softening resulting from seismic shaking shall be performed. 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 cyclic response for critical cases.

3.10.7.2 Criteria for Liquefaction Susceptibility of Gravels

Gravel layers bounded by lower permeability layers shall be considered potentially susceptible to liquefaction, and their liquefaction susceptibility shall be 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 shall be evaluated as such. Field investigation methods appropriate for soil layers containing gravels include Becker Hammer Penetration Test (BPT), Large Sampler Penetration Test (LPT), and small interval SPT. Seed et al. (2003) discusses different methods for performing liquefaction analysis in coarse and gravelly soils.

3.10.8 Liquefaction-Triggering Evaluations

Liquefaction-triggering evaluations should be performed for sites that meet both of the following two criteria:

- The estimated maximum groundwater elevation at the site is within 75 feet of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile characterized in the upper 75 feet as having soils that meet the compositional criteria for liquefaction with a measured SPT resistance, corrected for overburden pressure and hammer energy $(N_1)_{60\text{-}cs}$, less than 33 blows/foot, or a cone tip resistance $q_{c1N\text{-}cs}$ of less than 185, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

CPT and/or CPTu (with pore water pressure measurement) shall be used as the primary method of field investigation for liquefaction analysis where it can be advanced without premature refusal. SPTs shall be used as the primary liquefaction evaluation method where borings are performed. LPT, shear wave velocity (Vs), or BPT shall be used in soils difficult to test using SPT and CPT methods, such as gravelly soils. In addition, small interval SPT (blow counts measured for every 1 inch) shall be used in gravelly soils. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modelled well by the simplified methods.

3.10.8.1 Simplified Procedures

The three simplified methods by Youd et al. (2001), Seed et al. (2003), and Idriss and Boulanger (2008) shall be used for liquefaction-triggering analysis for each boring and/or CPT. However, the Modified Chinese Criteria for clayey soils in the Youd et al. (2001) method shall not be used. Results in terms of FOS shall be reported. Results of these analyses shall be interpreted and applied to design using engineering judgment. The following should be considered as guidelines to develop engineering judgment.

If the FOS values among the three methods are within 20% of each other, an average FOS shall be reported for that particular boring and/or CPT. If the FOS values from these three methods are off by more than 20% and also have significant cost consequences, some additional work may be warranted, such as an assessment of what method best applies to this specific case, additional soil-specific field and laboratory testing, and review by an expert panel.

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

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 evaluate 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 modelling 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 to evaluate the sensitivity of its results to any uncertain parameters or modelling assumptions. Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, approval by the engineering management team's (EMT's) geotechnical engineer is required to use nonlinear effective stress methods for liquefaction evaluation.

3.10.8.3 Liquefaction-Induced Movement/Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Methods to estimate movement/settlement of unsaturated granular deposits are presented in Section 6.10.14 of this TM. Liquefaction-induced total ground settlement of saturated granular deposits shall be estimated using at least two of these methods: Ishihara and Yoshimine (1992), Zhang et al. (2002), Idriss and Boulanger (2008), and Cetin et al. (2009). Other methods (i.e., more recently developed methods that are an improvement) may be used if justified and approved by the PMT. Where relatively closely spaced borings or CPTs are available (i.e., boring or CPT data are available at each structural support location), differential settlements can be estimated based on the settlements calculated for adjacent borings/CPTs. Where only sparse or widely-spaced borings or CPT data are available, differential settlement between two adjacent supports shall be assumed to one-half of the total settlement (Martin and Lew, 1999). The corrected SPT blow counts and CPT tip resistance values for estimating movements/settlements 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 cyclic stress ration (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.

It should be noted that all of these estimates are free-field settlements, and structural movement/settlements resulting from soil liquefaction are more important in most of the cases (Bray and Dashti, 2010). Structural movement/settlements may also result from shear-induced movements. Hence, methods that are used for estimating lateral ground movements may be required.

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

3.10.8.4 Liquefied Residual Strength Parameters

Unless soil-specific laboratory performance tests are conducted as described later in this section, residual strengths of liquefied soil shall be evaluated using at least two of these procedures: Seed and Harder (1990), Idriss and Boulanger (2008), Olson and Stark (2002), and Kramer and Wang (2011). Design liquefied residual shear strengths shall be based on weighted average of the results; Ledezma and Bray (2010) may be used as a reference to select a reasonable weighting scheme. Other methods for estimating liquefied residual shear strength (i.e., more recently developed methods that are an improvement) may be used if justified and approved by PMT. 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, two of the above empirical methods 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.

3.10.8.5 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 that is not within a few hundred feet of a free face shall be made using the method outlined by Ishihara (1985) and shall be compared against results by the method presented in Youd and Garris (1995). It is emphasized that settlement may occur, even with the absence of surface manifestation. The Ishihara (1985) 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., 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 either a ground surface slope gradient of 0.1% exists or a free face conditions (such as an adjacent river bank) exists. Use Shamoto et al. (1998) as a method to assess the maximum distance from the free face where lateral spreading displacements could occur. 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

If there is a free face condition, the post-liquefaction flow failure FOS of an earth slope or sloping ground shall be estimated per Section 6.10.15.1 of this TM before estimating liquefaction-induced lateral movements. If the post-liquefaction stability FOS is less than 1.0 then empirical or analytical methods cannot generally be used to reliably predict the amount of ground movement.

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), and Idriss and Boulanger (2008). Analysts shall be aware of the applicability and limitations of each method. Lateral displacements shall be evaluated using the Zhang et al. (2004) method and at least 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, the Newmark sliding block analyses shall be used. The Newmark analyses shall be conducted similar to that described in the seismic slope stability section, except that estimation of the yield acceleration (k_y) shall consider strength degradation due to liquefaction. In addition, numerical methods using finite elements and/or finite difference approach may be used.

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 of this TM and develop mitigation plans described in Section 6.10.9 of this TM, 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 evaluate the extent of liquefaction and potential consequences such as bearing failure, slope stability, and/or vertical and/or horizontal deformations. Similarly, the engineer shall 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 Iai et al. (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 and 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

There are many different methods of ground improvement. The five primary methods of ground improvement (and some examples of each of them) to be considered for soil liquefaction mitigation are:

- Replacement
    - Excavate and replace with compacted fill
- Vibratory Densification
    - Vibro-compaction
    - Vibro-replacement stone columns (combination of vibration and displacement)
    - Deep-dynamic compaction
- Displacement Densification/Reinforcement
    - Compaction grouting
    - Displacement piles
    - Vibro-replacement stone columns (combination of vibration and displacement)
- Mixing/Solidification
    - Permeation Grouting
    - Deep soil mixing
    - Jet grouting
- Drainage
    - Passive or active dewatering systems
    - (Earthquake drains are not permitted for use)

The implementation of these techniques shall be designed to fully, or partially, eliminate the liquefaction potential, depending on the requirements of the engineered facility under consideration. 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 produce acceptable structural performance (consistent with the requirements for the two design earthquakes) in terms of liquefaction/lateral spread-induced displacements and structural damage. The appropriate means of structural mitigation may depend on the magnitude and type of liquefaction-induced soil deformation or load.

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

- Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils
- Post-tensioned slab foundation (appropriate only for small, lightly loaded structures)
- Continuous spread footings having isolated footings interconnected with grade beams
- Mat foundation (appropriate only for small, lightly loaded structures)

Details, applicability, and limitations of these techniques can be found in Martin and Lew (1999). Additional requirements for design of piles in liquefied soil are presented below.

3.10.11 Seismic Considerations for Lateral Design of Piles in Liquefiable Soils

Seismic considerations for lateral design of pile/shaft design in 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 Boulanger, et al. (2007) and Brandenberg et al. (2007).

The displacement based approach for evaluating the impact of liquefaction-induced lateral spreading loads on deep foundation systems that follow recently published Caltrans guidelines titled "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading," dated February 2011 (http://dap3.dot.ca.gov/shake_stable/references/Guidelines%20on%20Foundation%20Loading-Jan%202011.pdf) shall be used. However, the liquefaction susceptibility and triggering analyses performed as part of this procedure shall be based on Section 6.10.6 and Section 6.10.7, respectively of this TM. Similarly, the lateral spread estimates shall be based on Section 6.10.8.

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 of this TM 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.

Numerical methods incorporating finite element and/or finite difference techniques may be used to assess pile response in laterally spreading soils.

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.

For retaining walls that are not restrained from rotation at the top in locations where peak ground acceleration (PGA) values are less than or equal to 0.30g, walls shall be designed for only active pressures and inertial forces of the wall itself, and additional seismic earth pressures shall not be considered. For walls containing cohesionless materials as backfill, seismic pressures shall be estimated using the Mononobe-Okabe (M-O) method (Mononobe and Matsuo, 1929). Horizontal seismic coefficient (k_h) shall be estimated using the Bray et al. (2010) method assuming a wall displacement of 1 inch. For the 30% design phase and final design, PGA values associated with two 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 a triangular distribution with the resultant acting at 0.33H 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 where the M-O solution does not converge (see Figure 7.8 of NCHRP Report 611), 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 0.35g, walls shall be designed for only at-rest pressures and inertial forces from the wall itself, 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 (k_h) shall be estimated using Bray et al. (2010) assuming a wall movement of 1 inch.

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 geotechnical engineer shall compare the estimated settlement values with the allowable deformation values described in Section 6.3.5 of this TM 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.

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 of this TM 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 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 6.10.7 of this TM shall be used. In addition, strength reduction due to cyclic degradation versus strength increase due to the effects of rate of loading shall be considered for normally consolidated clayey layers and non-liquefiable sandy layers. Chen et al. (2006) have discussed the effects of different factors on the dynamic strength of soils. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium FOS is less than 1.1, 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 FOS for this decoupled analysis is greater than 1.1 for liquefied conditions, yield acceleration 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 FOS 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 the Newmark time history method or more advanced methods involving numerical analysis may also be used, but shall be checked against the simplified methods.

For final design, more advanced methods involving numerical analyses may be used to better characterize the initiation of liquefaction and pore-pressure generation and the subsequent reduction in strength.

3.10.16.2 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 seismic coefficient (k_h) that acts upon the critical failure mass. A horizontal seismic coefficient, k_h, estimated using Bray and Travasarou (2009) and a vertical seismic coefficient, k_v, equal to zero shall be used for the evaluation of seismic slope stability. The Bray and Travasarou (2009) method requires an estimate of allowable deformation to compute k_h. Therefore, for the MCE, an allowable deformation of 6 inches may be used and for the OBE, the allowable deformation presented in Table 6.3.5-1 shall be used. For these conditions, the minimum required FOS is 1.0. 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.

3.10.16.3 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 FOS 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), Bray and Travasarou (2007), and Rathje and Saygili (2008), or dynamic stress-deformation models. These methods shall not be employed to estimate displacements if the post-earthquake static slope stability FOS 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 on 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, 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's geotechnical engineer.

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

Downdrag loads on foundations shall be evaluated in accordance with Article 3.11.8 of the AASHTO LRFD Bridge Design Specifications and as specified herein. The AASHTO LRFD Bridge Design Specifications, Article 3.11.8, recommends the use of the non-liquefied skin friction in the layers within and above the liquefied zone that do not liquefy, and a skin friction value as low as the residual strength within the soil layers that do liquefy, to calculate downdrag loads for the extreme event limit state.

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 TM.

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. AASHTO LFRD Bridge Design Specifications with California (Caltrans) Amendments
   - Section 3 “Loads and Load Factors”
   - Section 10 “Foundations”
   - Section 11 “Abutments Piers and Walls”
2. AASHTO, Manual on Subsurface Investigations, MSI-1, 1988.
3. American Association of State Highway and Transportation Officials (AASHTO) LFRD Bridge Design Specifications, 4th Edition, 2007.
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34. CHSTP TM 2.6.7 – Earthwork and Track Bed Design
35. CHSTP TM 2.9.1 - Geotechnical Investigation Guidelines, R0
36. CHSTP TM 2.9.2 - Geotechnical Report Preparation Guidelines, R0
37. CHSTP TM 2.9.3 - Geotechnical and Seismic Hazard Analysis Guidelines, R0
38. CHSTP TM 2.9.6 - Interim Ground Motions, R0
39. CHSTP TM 2.10.4 - Interim Sesimic Design Criteria,R0
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6.0 DESIGN MANUAL CRITERIA

6.1 GENERAL

Geotechnical criteria prescribed herein address the design, methodology, assumptions, analytical procedures, and integrity of the final design. Subject to the restrictions imposed by licensing laws in the State of California, analysis and designs shall be completed 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. 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.

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 TM 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 GEC5 (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. Dynamic properties of soil and rock shall be assessed for consideration of seismic actions and design.

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 in collaboration with the geotechnical engineer 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.). The geologic model shall serve as a fundamental tool to develop the subsurface exploration plan 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.

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)
- 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. Groundwater characterization shall include evaluation of:

- Historically high groundwater levels
- 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
- Potential for groundwater level rise resulting from anticipated rise of sea level due to climate change
- 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) for zone of 100 percent saturation should be used to provide an estimate of reasonable groundwater level.

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 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 1, Chapter 2 of GEC5. Additional guidelines that shall be considered are summarized in Section 10 of AASHTO LRFD (2007). The properties resulting from LRFD-based evaluations shall be consistent those obtained with general geotechnical practice and not overly conservative or unconservative.

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. Additional guidelines that shall be considered for correlations are presented in Manual on Estimating Soil Properties for Foundation Design by Electric Power Research Institute (EPRI) Report EL-6800, (EPRI, 1990). 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.

GEC5 shall be used to assess the design properties for the intact rock and the rock mass as a whole. However, GEC5 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 controlled by regular, systematic discontinuities in the rock mass.

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 GEC5, which provides guidance for analyzing data and resolving inconsistencies. The geotechnical engineer shall also use GEC5 to assess variability for a given engineering property in a particular geologic unit and how that variability influences the selection of the final design values.

Development of the geotechnical model outlined in Section 6.2.5.2 of this TM shall include an estimate of the scatter surrounding average physical properties of soil and rock units. The geotechnical engineer shall provide upper and lower reasonable estimates of key engineering properties to describe the uncertainty associated with estimates of the median properties. The upper and lower reasonable estimates are not upper and lower bounds, but instead represent approximately 16th and 84th percentile 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, increases in stress due to fill or foundation loads, decreases in stress due to excavation, or other factors.

Geotechnical evaluations for design shall keep in mind that resistance factors have been developed assuming statistical mean values for soil properties. 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 consistent with engineering judgment. 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 the 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.

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

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

Where shallow foundations are not feasible (i.e., they cannot meet the required bearing capacity or settlement criteria) or cost effective, deep foundations shall be used. Cast-in-drilled-hole (CIDH) 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. Where larger bridge spans and higher foundation loads are required, caissons, barrette, or diaphragm wall foundations may be considered.

Scour – The selection of foundation types and design of foundations shall consider the effects of scour on the capacity requirements and size (dimensions, embedment, and length) of foundations. The capacity of deep foundations shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes shall be evaluated by analysis methods TM 2.6.5.

6.3.3 Analysis for Foundation Design

Except where noted herein, geotechnical analysis for foundation design shall be performed in accordance with the AASHTO LRFD Bridge Design Specifications with California (Caltrans) Amendments, Customary U.S. Units, as adapted and modified by this and other technical memoranda. Caltrans Amendments require that abutment foundations be designed using Service 1 limit state and Working Stress Design (WSD) per Caltrans 2000 Bridge Design Specifications dated November 2003. LRFD loads, load groups, and limit states for aerial viaduct and bridge structure design are defined in TM 2.3.2.

6.3.4 Allowable 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, allowable settlements or displacements are measured at the top of the foundation: the pile cap, the pile head, or the ground surface for drilled-shaft pier extensions. For structure foundations, settlements calculated from the Service 1 load combination plus any settlements resulting from the OBE load combination (such as those resulting from post-liquefaction downdrag, seismic compaction, etc.) shall not exceed the settlement limits denoted in Table 6.3.5-1. For approach embankments, the Service 1 settlement limits are applicable to settlements that occur after the placement of track.

Further performance requirements for allowable deformations are prescribed in TM 2.10.10.

Table 6.3.5-1 Settlement Limits[4] for Service 1 and OBE Load Cases

Settlement Criteria	Non-Ballasted Track	Ballasted Track
Differential settlement between adjacent supports [1]	<= L/1500 and 3/4”, where L = smaller span	N/A [3]
Differential settlement between the abutment and approach embankment [2]	<= 3/8” over 62 feet	<= 3/4" over 62 feet
Differential settlement between the abutment and tunnel portal	<= 3/8” over 62 feet	N/A [3]
Uniform settlement at piers and abutments	<= 3/4”	N/A [3]

Notes:

1. The additional forces imposed on the structural system by differential settlements shall be calculated and considered as part of dead load in the design.
2. Prior to placement of tracks, the approach embankment shall be instrumented and monitored for a period of at least 6 to 12 months to ensure the embankment is in compliance with the settlement requirements set forth in the table above.
3. Not applicable based on the assumption that ballasted track will not be used for bridges, aerial structures or tunnels.
4. The settlements are calculated from the Service 1 load combination plus any settlements resulting from the OBE load combination (such as those resulting from post-liquefaction downdrag, seismic compaction, etc.).

No specific settlement or displacement limits are required for the extreme event MCE loading case, only that the structure shall not collapse. For deep foundations, the maximum relative horizontal displacement between the bottom (i.e. toe of pile) and top (i.e. pile cap) of the foundation resulting from OBE loading shall not be more than 1.75 inches.

The settlements and displacements noted in the table above are considered minimum performance criteria. The Structural design may require that foundations be designed to more stringent criteria for certain structures depending upon specific performance requirements.

6.3.5 Abutments and Abutment Foundations

Bridge abutments have components of both foundation design and retaining wall design. It should be noted that Caltrans Amendments require that abutment foundations be designed using Service 1 limit state and Working Stress Design (WSD) per Caltrans 2000 Bridge Design Specifications dated November 2003. The retaining wall aspects of abutments shall be designed in accordance with Section 6.7 of this TM, and AASHTO LRFD Bridge Design Specifications with California Amendments, Sections 10 and 11.

6.3.6 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 of this TM.

6.4 Foundations for Buildings and other At-Grade Structures

This section describes the methods that shall be applied in foundation analysis and design of buildings and other at-grade structures such as signals, signs, and noise barriers.

6.4.1 Buildings

Foundations and retaining walls for buildings shall be designed in accordance with the provisions of the 2010 CBC, California Code of Regulations, Title 24, Part 2, California Building Standards Commission (2010 CBC), and TM 2.5.1. In absence of site-specific data, presumptive values provided in Chapter 18 of the 2010 CBC for allowable foundation bearing pressure, lateral earth pressures, and sliding coefficients shall be used. Seismic issues related to foundation design such as seismic earth pressures, downdrag and lateral spread due to liquefaction shall conform to the limits provided in Table 3.3.7-1 of this TM.

6.4.2 Noise Barriers

Foundation design for noise barrier shall be conducted in accordance with Caltrans Memo To Designers 22-1, Soundwall Design Criteria. Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed per Section 6.10 of this TM.

6.4.3 Signs and Signals

Cantilever signs and signals shall be supported on drilled-shaft foundations. Design for cantilever signals and cantilever signs shall be performed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO, 2001). The foundation design will require ultimate and allowable downward and uplift capacities. In addition, lateral capacities of shafts shall be provided. Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed per Section 6.10 of this TM.

6.5 Tunnels and Other Underground Structures

This section describes the methods that shall be applied in geotechnical and geological analysis for design of tunnels and other underground structures. Tunnels and other underground structures include bored tunnels (i.e., in rock and/or soft ground), mined tunnels, underground chambers, cut-and-cover tunnels, portals, shafts, and tunnel crossovers. U-shaped structures are addressed in Section 6.7 of this TM.

6.5.1 Site Investigation

Site investigations shall be planned and conducted in sufficient detail to evaluate the subsurface conditions of the rock and/or soil medium, and groundwater regime within which tunnels and/or underground structures will be constructed. Site investigations shall be planned and executed in accordance with TM 2.9.1, and Chapter 3, FHWA-NHI-10-034 – Technical Manual for Design and Construction of Road Tunnels-Civil Elements (FHWA 2009). The site investigations shall be sufficient in scope to explore and characterize the site conditions at the specific location and elevation of the proposed underground excavation.

Sufficient data shall be developed to characterize the geotechnical design and construction issues sufficiently for analysis and appropriate mitigations in design. Such data shall be presented in the GDR and the GBR for all CAHST tunnel segments as outlined in TM 2.9.2 and FHWA (2009) Chapters 4.2 and 4.4.

6.5.1.1 Laboratory Testing

Soil samples shall be described and classified using ASTM D 2488 guidelines for field classification and ASTM D 2487 based on laboratory test results. Rock (both hard and soft) shall be classified using ASTM D 5878 and be in conformance with GEC5, Evaluation of Soil and Rock Properties (FHWA 2002), which is based on the ISRM guidelines. Laboratory testing shall be performed in accordance with Section 6 of TM 2.9.1. Sufficient laboratory testing shall be performed to represent in-situ rock and soil conditions of the project.

In addition to laboratory testing identified in TM 2.9.1, specialized testing of rock for underground excavations mined by TBM shall also include drilling rate index, bit wear index, and cutter life Index, as described in Chapter 3 of FHWA (2009). Additional testing may include cherchar abrasion index, and punch penetration test for use by TBM designers. The abrasion characteristics of soils (Abrasion Value for Soils AVS) shall be assessed by applying the Norwegian Institute of Technology (ibid NTNU) SAT.

Petrographic analysis shall be conducted on representative rock samples. The petrographic analyses shall be conducted on rock thin sections prepared for analysis under a polarizing microscope to identify principal mineral constituents (especially quartz content and presence of asbestos), textural relationships, alteration/metamorphism, percentages, and other unusual properties that may affect TBM performance.

6.5.2 Characterization

6.5.2.1 Soil Classification

Descriptions of soils shall also be in accordance with TM 2.9.1 and Chapters 3.5 and 7.2 of FHWA (2009). For tunnels and other underground excavation, special attention shall be given to documenting soil grain-size characteristics and stratification features, both of which strongly influence ground behavior for excavations. Terzaghi (1950) classified soils in the Tunnelman's ground classification according to the anticipated soft ground behavior based on soil identification (grain size) and whether the excavation is above or below groundwater. Emphasis shall focus on cohesionless soils (i.e., composition, gradation and density) and on cohesive soils (i.e., consistency and strength) with respect to the proposed excavation and with respect to groundwater conditions (e.g., perched and confined conditions, permeability, and evidence of artesian conditions or groundwater barriers, i.e., faults).

6.5.2.2 Rock Mass Classification

Rock mass classifications shall be evaluated from the geotechnical and geological data collected during the field investigations to describe the rock mass conditions that shall predominate within the proposed tunnels and underground excavations in rock in accordance with FHWA (2009). Terzaghi (1946) proposed a qualitative description of rock mass classes and successful applications of various tunnel support systems that prevent rock masses from dropping from the tunnel roof. If used for the CHSTP, Terzaghi's rock mass classifications shall be implemented only for preliminary estimates of tunnel support requirements based on professional judgment. The more recently proposed numerical classifications of rock shall be used for the CHSTP design recommendations. The numerical rock quality designation (RQD) proposed by Deere and Deere (1989), the tunneling quality index (Q) proposed by Barton et al. (1974) of the Norwegian Geotechnical Institute, and the rock mass rating (RMR) by Z.T. Bieniawski (1989) shall be developed for site-specific application to rock tunnel support and lining of the CHSTP. The numerical rock mass classifications shall be used for evaluating and demonstrating the design of proposed rock support systems for the tunnel excavations.

6.5.2.3 Geologic Structure

Analysis of geologic structure shall be performed for all proposed excavations in rock. Rock discontinuities, which typically control the behavior of a rock mass with respect to slope stability and underground stability, shall be analyzed in outcrops by geologic mapping, in rock-core logging as outlined in TM 2.9.1, and using in-situ methods of logging as outlined in AASHTO MSI-1, Section 6.1.2. All structural mapping and logging methods are to document in-situ geologic structural trends for use in structural analyses of the rock mass and for design of tunnel and underground excavation interim and final supports.

Compiled discontinuity orientations defined by "strike", "dip", and "dip direction" shall be analyzed using Rocscience's software program Dips V6.0 (Rocscience, 2010), or other equivalent analysis software. The resulting stereonet plots of data shall be used in estimating appropriate orientation adjustments (R_A) for calculating the RMR of rock. The discontinuity data shall also be used for estimating structurally controlled roof and wall failures in excavations (e.g., wedge failures) and for qualitative estimates of the slope stability at rock portals including basal slip, wedge, and toppling failures as outlined in 8.5.2 of this TM.

6.5.2.4 Hydrogeology

The hydrogeology of both soil and rock sites shall be evaluated for tunnels and other underground structures and shall be in accordance with FHWA (2009) including groundwater elevations of static and perched water zones derived from published and research sources, geotechnical field investigations, and groundwater investigations of proposed excavations. The site characterization shall include the checklist for GBRs included in Table 4-3 of FHWA (2009). The hydrogeologic data shall be used to define static groundwater elevations, seasonal fluctuations, flow directions, hydraulic conductivity, perched and confined aquifer conditions (artesian), porous medium or fractured medium, pH, temperature, and water chemistry.

Hydraulic conductivities shall be calculated based on data collected in accordance with TM 2.9.1, including pumping and slug tests, packer tests, open borehole seepage tests, and infiltration tests. For groundwater monitoring, monitoring wells and piezometers shall be installed and monitored for at least one year, but preferred for multiple years (i.e., wet and dry years). Procedures for calculating hydraulic conductivities from pumping test data and effective hydraulic conductivities from borehole packer tests and falling head tests shall be implemented in accordance with FHWA (2009) Chapter 3.5.6, and FHWA (2002) - Subsurface Investigations – Geotechnical Site Characterization – Reference Manual.

Where groundwater inflow or dewatering is a concern, hydraulic conductivity testing shall be conducted by the designer as part of the subsurface investigations in accordance with TM 2.9.1 and shall include permeability tests, pumping tests, slug tests, packer tests, open borehole seepage tests, and/or infiltration tests. These tests are further described in ASTM D 4043 and shall form the basis of understanding groundwater occurrence, hydraulic pressures, and groundwater flow characteristics.

The designer shall identify conditions that could result in design changes, construction delays, and unanticipated construction costs due to unanticipated groundwater conditions. Unexpected groundwater conditions can include but are not limited to instantaneous inflows and sustained flows higher than estimated, groundwater barriers, flowing saturated soils, geothermal waters, and gas-bearing water. The designer shall investigate and quantify all potential groundwater conditions that could result in changed conditions for the project and fully develop and explain the variability of potential hydrogeology conditions in the GDR and the GBR for each project structure according to TM 2.9.2.

Water-level measurements and/or hydraulic pressures shall be measured for predicting uplift pressures and hydraulic pressures on tunnel lining systems. Measurements and pressures shall be taken at least one diameter above and below the tunnel structure. Groundwater characterization shall account for potential variations resulting from seasonal changes, rainfall, irrigation, and other factors.

6.5.3 Design Issues

6.5.3.1 Groundwater Impacts

Influences of dewatering on existing structures (e.g., settlement) and on project excavations shall be included in design of below-ground excavations and for calculating uplift pressures for slab design. The designer shall provide calculations for estimates of inflows including initial flows (e.g., flush flows) and sustained flows (long term), which are dependent on the occurrence of groundwater (static, artesian, fracture systems, etc.), hydraulic conductivity, hydraulic head, volume of water reservoir source, and groundwater barriers such as aquitards, aquicludes, and faults.

6.5.3.2 Seepage Control

Undrained and drained tunnel designs shall be considered. Tunnels (i.e., bored and mined) may be designed as undrained (i.e., with waterproofing) with the objective of eliminating impacts to groundwater and surface water resources due to groundwater drawdown. Drained tunnels may be viable for specific conditions such as short tunnels or where pre-construction grouting of the rock mass is applied to minimize long-term inflows. Conditions for design of undrained and drained tunnels are outlined in TM 2.4.5 for static loads resulting from groundwater pressures. For guidance on watertightness and drainage for tunnel structures refer to TM 2.4.5, Section 6.2.5. Cut-and-cover tunnels and U-wall trackway structures shall be designed as undrained to accommodate groundwater conditions (including seasonal changes) at the site and the designer's proposed excavation support and lining design.

6.5.3.3 Induced Ground Settlement (Movement)

The potential for induced settlement shall be evaluated using FHWA (2009) guidelines outlined in Section 7.5, which provide methods for calculating movement either due to a groundwater depression during dewatering or due to ground loss during tunnel excavation. The designer shall calculate the settlement trough depth, width, and shape to estimate the potential surface settlement and effects on surface structures. The settlement calculation shall include both single-bore and multiple-bore tunnels, where proposed.

Potential damage to structures due to ground settlement shall be evaluated using the guidance provided in Section 7.6 of the FHWA (2009). The relationships presented in FHWA shall be used for initial estimates of structural damage as part of tunnel lining and TBM design and planning for ground support to mitigate construction impacts. Mitigation methods of ground settlement shall be included in the design process as outlined in Section 7.6.

6.5.3.4 Gassy Ground Hazard

Tunnels and underground structures shall be designed to protect against potential hazardous conditions due to the presence of explosive, corrosive, or poisonous gasses (e.g., methane, petroleum-derived gases, hydrogen sulfide) entering tunnels. Identification and evaluation of potential gassy ground shall be part of the GDR and GBR for the tunnel. Special attention shall be given to areas of petroleum-bearing geologic materials, especially within or near known active or abandoned oil fields. The California Division of Oil and Gas and Geothermal Resources (DOGGR) maintains records of active and abandon oil fields and wells. Research of oil wells and petroleum-bearing areas at DOGGR shall be part of the source information to be reviewed as outlined in TM 2.9.1 and shall be investigated and evaluated as a potential geologic hazard with reference to TM 2.9.3. Mitigation of gassy conditions shall be included in the design of the tunnel-lining system, ventilation system, and electrical and mechanical components for use in the tunnel. The designer shall include protections including gas-resistant waterproofing, impermeable membrane, full-time ventilation system, and gas-detection system monitoring both the crown and invert spaces, where gases could collect. Concrete and steel shall be protected against corrosive gases or gas-saturate groundwater. Designs shall conform to all local fire department requirements for confined space and fire safety.

6.5.3.5 Seismic Loads

Tunnels and underground structures shall be designed to resist the effects of ground shaking (i.e., ovaling/racking, longitudinal curvature, and axial straining) and permanent ground deformations (i.e., seismic slope instability, liquefaction, and lateral spreading) that result from the design earthquakes. These analyses shall be performed in accordance with TM 2.10.4 and TM 2.3.2. Additional guidance can be found in FHWA (2009) Chapter 13. Seismic ground shaking parameters shall be developed in accordance with TM 2.9.6, and ground failure potential shall be evaluated in accordance with Section 6.10 of this TM.

6.5.3.6 Static Loads

Earth loads shall be addressed in accordance with the TM 2.4.5, TM 2.3.2, FHWA (2009) Chapters 6 and 7, and the following provisions. The designer shall account for earth loads that include rock, soil, and groundwater.

Groundwater loads for design shall represent the full hydrostatic head or height of the column of water above the excavation (refer to TM 2.4.5). The tunnel lining shall support the full hydrostatic head along the length of any particular tunnel.

6.5.4 Excavation Issues

6.5.4.1 Rock Tunnels and Chambers

In accordance with Chapter 6 of FHWA (2009), design and construction of tunnels and other underground excavations in rock shall consider all potential rock stresses, failure modes, and difficult ground including but not restricted to wedge failures, rock burst, stress-induced failures, tunnel face or roof instability, squeezing ground, swelling ground, mixed-face conditions, high horizontal stresses, ground displacements, groundwater, and any combination of conditions that can adversely impact tunnel construction and performance. The designer shall evaluate and address all site conditions and material properties that can influence the behavior of rock in which an excavation is planned, including intact rock strengths, discontinuities, rock mass classifications, deformation modulus, abrasiveness, in-situ stresses, fault zones, gassy ground, flowing and running ground, water inflows, water pressures, geothermal conditions, and water chemistry.

Ground support shall be designed for initial and final support to resist all induced rock stresses and shall be considered with all applicable options of support, including rock bolts, ribs and lagging, shotcrete, lattice girder, spiles and forepoles, and precast segmented lining systems.

The designer shall apply methods of groundwater flow control in accordance with Chapter 6.7 of FHWA (2009). Groundwater controls may include pre- and post-construction grouting, ground freezing, and the use of pressurized-face TBMs operated in closed mode. Inflow of groundwater will be limited to not more than 100 gallons per minute for sustained flows. However, allowable inflow rates may be set by negotiation with local jurisdictions.

6.5.4.2 Portals and Shafts

Portal and shaft excavations shall be designed to be stable (i.e., static stability) during excavation and upon being put into service in accordance with Section 8 – Analysis and Design for Slopes. All cut slopes and excavations shall be designed for any possible mode of failure to meet minimum FOS that are 1.3 or greater for short-term construction (temporary slopes or construction slopes) and 1.5 or greater for permanent or final design slopes. These factors of safety shall apply to both soil and rock slopes for all portal and shaft excavations.

The overburden materials, weathered rock, rock, and pre-existing landslides shall be investigated and evaluated for physical conditions (lithology and geologic structure) and strength properties (friction angle – φ; and cohesion-c) in accordance with TM 2.9.1 and FHWA (2009) for use in calculating FOS for excavated slopes and openings. The applicable modes of failure shall be evaluated by site-specific field investigations in accordance with TM 2.9.1, TM 2.9.3 and FHWA (2009). The designer shall evaluate all ground conditions affecting stability of the portal or shaft that may include overburden excavation, weathered rock, and unweathered rock. The modes of failure shall be modeled using two-dimensional geologic cross sections and/or three-dimensional modeling for use in conducting slope stability analysis using limit equilibrium methods outlined in Section 6.8 of this TM to evaluate the FOS of each slope and slope-support methods.

Initial ground support for shaft excavations will depend on the site conditions, whether the excavation is above or below groundwater, and construction preferences of the construction contractor, and, therefore, are not detailed in this document. However, support systems shall consider the following methods:

- Soldier piles and lagging in soils without water
- Ring beams and lagging or liner plate
- Precast concrete segmental shaft lining
- Steel sheet pile walls in soils with or without water
- Diaphragm walls cast in slurry trenches, which can minimize settlement and dewatering effects
- Secant pile walls or soil-mix walls instead of diaphragm walls

6.5.4.3 Cut-and-Cover Structures

Cut-and-cover tunnels shall be analyzed and designed in accordance with AASHTO LRFD Bridge Design Specifications with Caltrans Amendments, and Chapter 5 of the FHWA (2009) Technical

Memorandum for Design and Construction of Road Tunnels – Civil Elements. Site characterization for cut-and-cover tunnels shall be in accordance with TM 2.9.1 and Chapter 3 of FHWA (2009), which can apply to soil or rock excavations; however, most cut-and-cover applications are expected at soil sites.

6.6 TRACK BED EMBANKMENTS AND EMBANKMENT FOUNDATIONS

6.6.1 Design Overview for Embankments

The geotechnical analyses and design guidelines included in this section supersede the technical guidelines provided in TM-2.6.7. Additional guidance is provided in TM 2.6.7 and TM 2.10.10.

For trackway type selection purposes during the design phase, the feasibility of selecting ground supported trackwork (on fill embankment or in cut) will depend on ability to meet the project performance criteria, and shall also consider cost and construction schedule. Other track guideway types for consideration and comparison against embankment-supported track include viaduct- or retaining-wall supported track. Embankment/fill design considerations shall also be linked to the earthwork material availability and handling strategy on a regional basis for CHSTP, including proportioning of cut and fill with the goal to balance quantities, where feasible.

6.6.2 Design Considerations

The embankments and foundations for trackways shall be designed considering the durability and longevity over the 100-year design life and the ability to meet applicable levels of required criteria, that may vary depending on track segment, train speed and loading, frequency of train traffic, and track use such as mainline, stations, sidings, yards, etc., as described in TM 2.1.5. Geotechnical designs shall consider that embankment track substructure must meet geometric accuracy for maintaining the overlying track surface geometry (profile and alignment) and satisfy stability. This includes resistance to static load and dynamic load (passing trains), as well as extreme action/loading events resulting from seismic shaking, heavy precipitation, frost action, etc.

Design guidelines for the “track bed” (layers, dimensioning, and materials) overlying soil embankments or trackways in cut are provided in TM 2.1.5. Track designers shall provide minimum subgrade stiffness criteria as well as anticipated loading and required bearing capacity information to the geotechnical engineers regarding the required subgrade surface (in cut segments) and embankment fill surface minimum stiffness values (subgrade modulus and modulus of deformation) for evaluation during site exploration and design.

Designs for trackway on embankment or in cut sections shall be coordinated with other project features that might interfere with or impact the design or construction of CHSTP elements. This includes coordination with other design disciplines (including structural and civil, hydrology and hydraulics, and systems) to select appropriate earthworks for a given setting based on design constraints and potential conflicts, geotechnical subsurface investigations, and surface and groundwater issues. General earthwork terms with a typical track cross section are depicted on Figure No. 6-1.

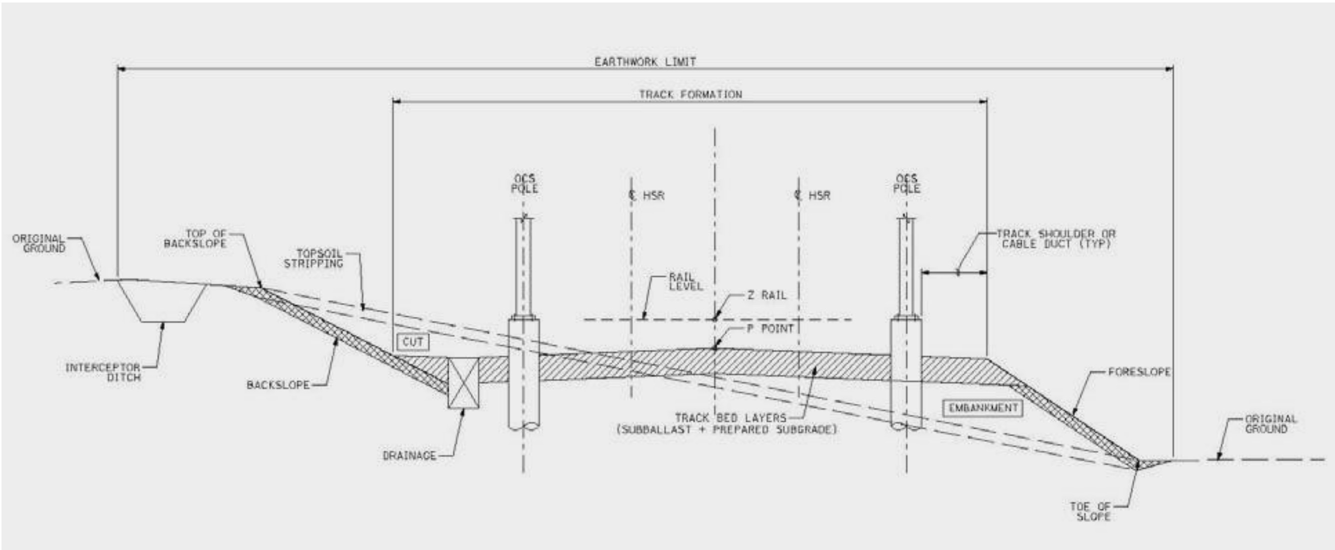


Figure 6-1: General Earthwork Cross Section and Terminology

6.6.3 Soil Material Suitability for use as Engineered Fill in Embankments

For design purposes, evaluation of soil suitability for re-use within the body of track embankments shall be based on the following guidelines which supersede the definitions for “Soil Quality Classes” presented in UIC 719R and Section 6.3.2 (Table 6-4) of TM 2.6.7.

Table 6.6.3-1 Soil Material Suitability for Engineered Fill in Embankments (per ASTM D 3282 / AASHTO subgrade soil group system)

Acceptable [1]	Unacceptable [2]
A-1-a	A-4 (CBR <10)
A-1-b	A-2-7
A-2-4	A-5
A-2-5	A-6
A-2-6	A-7-5
A-3	A-7-6
A-4 (CBR >10)	*

Notes:

* Rockfill is not acceptable for track embankment material.

1. In addition to the AASHTO criteria, the maximum soil particle size is limited to 3 inches.
2. Potential embankment fill source materials from groups A-2-7, A-5, A-6, and A-4 (with CBR <10) that can be shown by analysis and testing to meet all requirements (including strength, stability, settlement/deformation, long-term durability, etc.) shall be submitted for consideration of acceptability on a case-by-case basis. This includes marginal soil types from these groups that can be modified using soil amendments or additives such as cement, lime, hydraulic binders, etc., to be rendered suitable for use provided they meet all requirements (described above) as demonstrated by analysis and testing programs, including laboratory trial batching and field test sections.

Soil suitability evaluations shall also consider potentially detrimental properties as follows:

- Frost susceptibility – In order to reduce the potential to cause unacceptable disturbances to track geometry upon freeze/thaw cycles, soil types susceptible to frost, such as silt or clay, shall not be used for embankments in regions where cold conditions can occur .
- Corrosivity – Soil suitability shall also consider corrosion potential. Corrosive soils that are potentially detrimental to buried metal and/or concrete features (such as OCS poles, pipes/culverts, geogrid reinforcement, etc.) shall not be used.

6.6.4 Embankment Fill Design

Embankment foreslope inclinations shall be limited to 2H:1V or flatter for preliminary engineering design.

For embankments with dimensions exceeding 30 feet in height (measured from original ground to top of slope), designs shall include mid-slope benches for purposes of drainage and facilitating future access for maintenance reasons. Slope benches shall be 6 feet wide minimum, with 6% gradient toward the low end of the fill slope, and shall include a lined gutter channel at the drainage surface. For tall embankments, slope benches shall be laid out on average of every 30 feet in height (allowance from 26- to 32-foot range is considered acceptable) and shall be connected to the surrounding ground surface for access.

At the top surface of fill embankment or cut subgrade level (immediately underlying track roadbed section), the design section shall have a transverse cross slope drainage gradient of at least 4%, preferably sloped toward the outer edges of the embankment foreslopes. The 4% minimum cross slope at subgrade surface must be met even after long-term settlement. A general track cross section depicting the required cross-slope drainage at the subgrade surface is shown below in Figure 6-2.

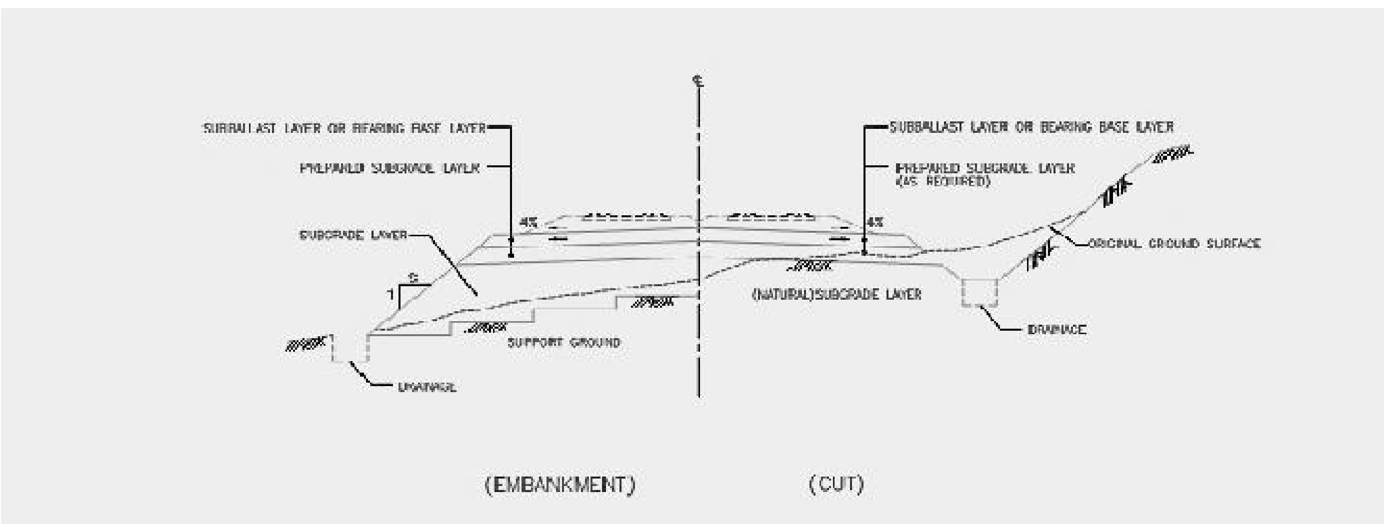


Figure 6-2: Track Cross Section with Sloped Subgrade Surface (4% gradient)

The proposed geometry and dimensions for track embankment sections shall be confirmed by geotechnical analyses (short-term, and long-term conditions) for stability and settlement/movement in accordance with CHSTP TM guidelines. Embankment engineering guidelines that shall be considered for geotechnical design are summarized in FHWA Soil Slope and Embankment Design Manual FHWA/NHI-05-123, 2005.

Design shall consider potential problems associated with movements, including internal deformations (compression of fill materials) within the embankments, as well as external deformation in underlying foundation / native soils below the embankment. Internal deformation within embankments shall be controlled by design and use of fill materials (in accordance with CHSTP TM guidelines and referenced specifications) that have the ability to resist the expected loads. Deformation considerations for the embankment shall consider both vertical as well as lateral deformation movements. Vertical deformation movements are referred to as settlements. Lateral deformation movements can result in rotation of embankment earth structures at abutments that is commonly referred to as tilting. Design solutions for deformation problems are provided in Sections 7.6 and 7.7 of FHWA Soils and Foundations Reference Manual FHWA/NHI-06-088, Volume I, dated 2006, UIC 719R 3rd Edition dated 2008, and FHWA Soil Slope and Embankment Design Manual FHWA/NHI-05-123 dated 2005.

The track substructure (foundation/embankment system) shall be analysed for stability by the geotechnical designer since overstressing of embankment or foundation soil may result in failures. For assessment of load due to the earth embankment structure, designers shall assume fill soil compacted to at least 95% of maximum density per ASTM D 1557 for estimation of soil unit weight. Potential failure modes that shall be considered include bearing capacity, displacement failure, translatory failure, rotational sliding failure (extending through the foundation), and lateral squeezing. Analytical procedures for use to assess stress distribution in soil foundations underlying embankment fills is given in Section 7.3 of FHWA/NHI-06-088, Volume I, dated 2006. Guidelines for additional stability assessments for both fill slopes and natural or cut slopes are provided in section 6.8 of this TM.

For FOS against bearing capacity failure, the level of stress in the subgrade material (directly underlying track structure) due to loading of track structure components and ballast, if present, plus loading from trains shall not exceed an allowable bearing pressure that includes a minimum FOS of 2.5.

For stability and settlement analysis, consideration shall be given to additional actions/loading due to dynamic load from passing trains and also extreme events (seismic shaking, liquefaction, and related strength loss due to seismic load, etc.). Seismic design guidelines for embankments and earth structures are provided in Section 6.10 of this TM. The seismic case evaluations and associated analyses should be displacement based leading to estimates of potential lateral deformations of embankments/slopes and ground settlement.

Embankment section designs shall avoid having trackways straddle the cut/fill line on side-hill sections of mainline segments, where feasible. Where embankments are to be located on slopes or where a new fill is to be placed against an existing embankment, the slopes of the original hillside or existing embankment shall be benched in order to provide a notched interface between the new fill and the existing ground. Bench widths are expected to be variable depending on the slope angle; however, bench heights shall be limited to 4 feet. A keyway shall be excavated to provide support for the toe of new fill slopes constructed against slopes.

Design recommendations shall be provided to control subsurface drainage since it is integral to the performance and stability of earth structures as well as track segments in cuttings (cut ground). Standard design guidelines for longitudinal (sideline) subdrainage features at track shoulder subgrade zones are provided in UIC 719R, Section 2.8.

For reaches of earth embankment at transition zones (immediately adjacent to bridge and viaduct abutments, tunnels, cut-and-cover structures, and cut sections with an abrupt topographic change) the approach embankment shall be designed to minimize the potential for differential settlement and to provide a smooth transition in the structural stiffness between different infrastructure features.

6.6.5 Embankment Strengthening and Stabilization Using Geogrids

For mainline track segments, embankment design for fill sections greater than 5 feet in height, and with slope inclinations steeper than 2.5H:1V shall include horizontal layers of geogrid-reinforcing material extending a minimum of 8 feet from the outer edge surface of fill foreslopes inward toward the center of the embankment body. The geogrid reinforcing is required in order to improve strength/resistance of the foreslopes of the fills and to enhance overall durability of the earth structures for HST mainline track segments. The maximum vertical spacing between consecutive layers of geogrid reinforcement shall be 1.5 feet. A generalized embankment slope section detail is shown below in Figure 6-3.

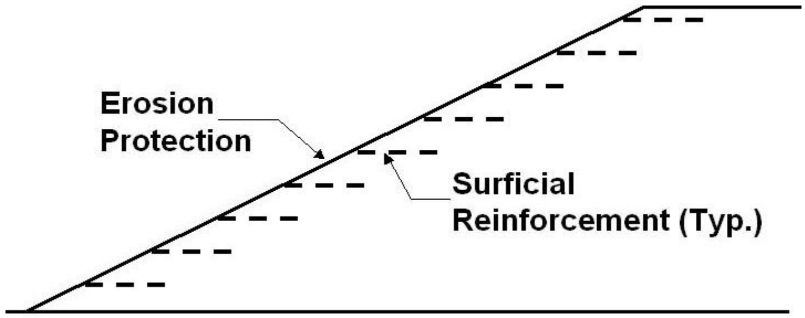


Figure 6-3: Geogrid-Reinforced Embankment Foreslope

Embankment designs shall consider the need for additional layers of continuous horizontal geogrid reinforcing across the full width of embankments to strengthen the body of fills and control deformation for segments where only poorer quality fill types (per Table 6.3.5-1, second column) are available, and/or where there are areas of weak foundation conditions (based on site exploration and geotechnical analysis). Geotechnical evaluation methods for use in the design for geogrid-reinforced embankments and control of embankment deformation are presented in the following technical guidance reference documents:

- Geosynthetic Design and Construction Guidelines, Chapter 7, FHWA-HI-95-038, 1998
- Soils and Foundations Reference Manual, Chapters 6 and 7, FHWA-NHI-06-088, Volume I, 2006
- Soil Slope and Embankment Design Manual, Chapter 8, FHWA-NHI-05-123, 2005

The designer shall select a geogrid material with adequate tensile strength for the proposed use and shall give consideration to other important aspects and properties (such as durability, degradation resistance, creep behavior, high modulus, protective polymer coatings, other mechanical properties that are time dependent, etc.) to meet CHSTP requirements. Metallic reinforcing elements shall not be used, since they are potentially susceptible to stray current corrosion that causes significant loss of section over the life of infrastructure supporting track.

6.6.6 Special Requirements for Embankments at Floodplains and at Fault Crossings

For design of embankment segments located in floodplains, the level of highest water shall be established by the hydrology and hydraulics studies based on the levels of known floods. The embankment fill section shall be designed to protect foreslopes located within the highest water level zone with a surface layer of drainage material and riprap protection as well as use of an underlying "drainage layer" as shown in Figure 6-4. The drainage layer shall extend upward along the foreslopes to the estimated high flood water level plus 1.5 feet. The granular drainage material shall contain less than 5% fine-grained material (passing the No. 200 sieve) and comply with Terzaghi's filter criteria, as summarized by Cedegren (1989):

$$\frac{D_{15}(filter)}{D_{85}(soil)} < 5 < \frac{D_{15}(filter)}{D_{15}(soil)}$$

and

$$\frac{D_{50}(filter)}{D_{50}(soil)} \leq 25$$

Where D_{15}, D_{50}, and D_{85} are the particle sizes at which 15, 50 and 85%, respectively, of the material by weight is smaller. D_{15}(filter) denotes the D_{15} value for the filter material, D_{15}(soil) denotes the D_{15} value for the fill or subgrade material in contact with the filter, and so forth.

Additionally, the filter material should not be gap or broadly graded. The design process for riprap shall follow the approach provided in NCHRP Report 568, Riprap Design Criteria, Recommended Specifications, and Quality Control, NCHRP, 2006.

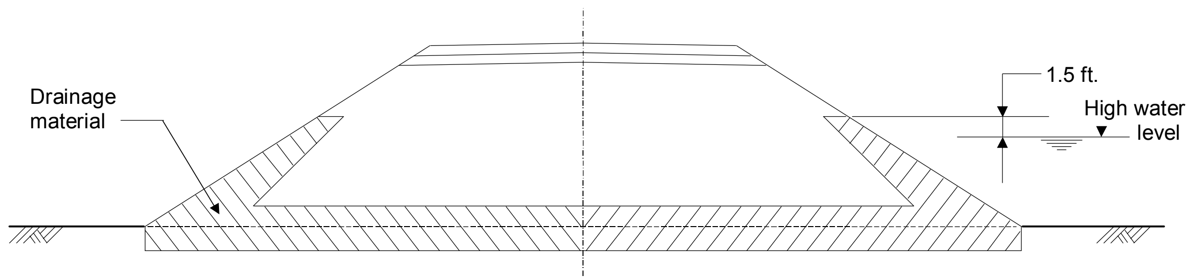


Figure 6-4: Drainage Layer under Embankment in Floodplain

For embankments located in track segments in wet areas where the water table is permanently or periodically at ground level, the embankment shall be constructed on a layer of drainage material as shown in Figure 6-5. This material shall not deteriorate or swell when immersed in water. It shall be well graded with no more than 10% passing the No. 200 sieve. The grading of the drainage material shall comply with Terzaghi's filter criteria against the subgrade original ground material as described above. The thickness of this drainage layer shall be related to the topography of the wet zone, but no less than 1.5 feet. In flat ground areas, the thickness of the layer shall be such that, after consolidation settlement of the bearing subgrade soil, the height of the drainage layer shall be at least 9 inches above the natural ground.

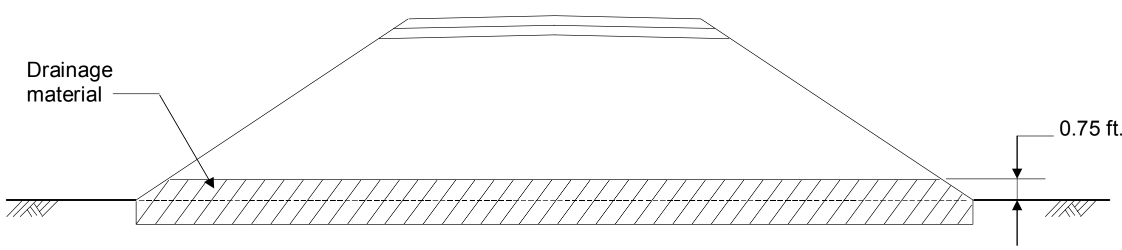


Figure 6-5: Drainage Layer under Embankments in Wet Locations

For locations where transverse box culvert drainage structures or pipes will be constructed within the body of trackway embankments, the embankments shall be designed to minimize the potential for differential settlement and to provide a smooth transition in the fill stiffness between these different infrastructure features at transition zones.

Earth structure designs at these locations shall include "approach embankments," and the design details shall specifically take into account the geometrical, geological, and geotechnical conditions of the site and the anticipated construction sequence. Design examples for transitions from earthworks to culverts or other railway infrastructure features are given in UIC 719R, and Innovative Track Systems INNOTRACK Guideline for Subgrade Reinforcement with Geosynthetics, Section 9.4 of Report D2.2.6 Project No. TIP5-CT-2006-031415, dated 2009.

For earthquake fault crossings at locations where track segments on embankment or in cut cross earthquake faults classified as hazardous (subject to ground movement/displacement due to potential fault rupture) as defined in TM 2.10.6, the earthworks supporting trackway infrastructure shall be designed according to guidelines in TM 2.10.6.

6.6.7 Embankment Settlement (Magnitude and Rate) and Tolerable Deformations/Movements

The vertical deformation settlement of embankments (which also affects overlying track bed structure) is a combination of the settlement movement of the foundation on which it is resting plus settlement of the embankment fill, as shown in Figure 6-6. Conventional settlement analyses shall consider immediate, consolidation and secondary components of settlement against the requirements of CHSTP. For analysis of embankments, calculation procedures that shall be used to assess soil settlement are given in the following references:

- Soil Slope and Embankment Design Manual, Chapters 4 and 8, FHWA-NHI-05-123, 2005
- Soils and Foundations Reference Manual, Chapter 7, FHWA-NHI- 06-088, Volume I, 2006

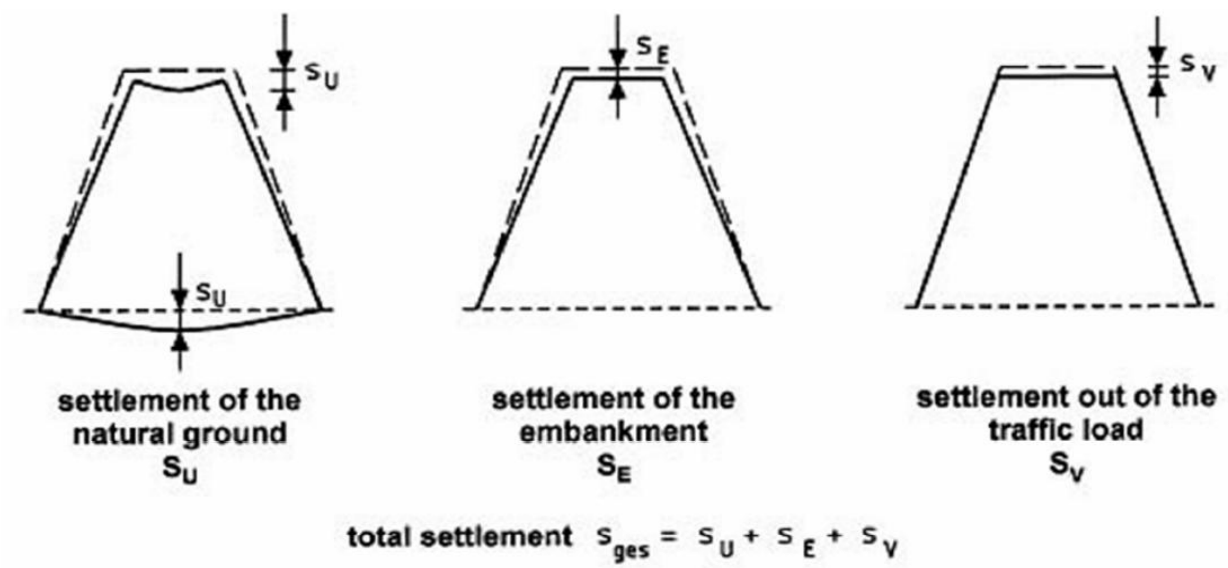


Figure 6-6: Settlements of Embankments

Reference: Figure No. 21 of UIC-719R (2008)

Geotechnical evaluations for embankments and their foundations shall also include the settlement contribution from surcharge/track load, and additional loading and/or ground deformation due to earthquakes.

Once the embankments are designed based on safe allowable bearing pressures and satisfying stability, the residual settlement (following track installation) estimates and differential displacements between locations along the length of the embankments shall be evaluated.

Settlement occurring after construction of the permanent way tracks shall be limited along general track segments as follows:

Table 6.6.7-1 Settlement Criteria - Residual Settlement After Placement of Tracks

Residual Settlement [1]	Non-Ballasted Track	Ballasted Track
Differential settlement [2]	≤ 3/8 inch over 62 feet	≤ 3/4 inch over 62 feet
Uniform settlement	≤ 5/8 inch	≤ 1-1/8 inch
Rate of settlement (per year)	≤ 3/16 inch	≤ 3/4 inch

Notes:

1. Prior to placement of tracks, embankment sections shall be instrumented and monitored for a period of at least 6 to 12 months to ensure compliance with these requirements for residual settlement.
2. Differential settlement along track segments is measured along the track (surface profile uniformity) in the vertical plane of each rail at the mid-point of a 62-foot-long chord.

If the predicted differential displacements are excessive and exceed profile tolerances, then embankment designs shall require further modification, and/or ground improvement may be needed for the foundation systems. Where predicted settlement movements and their duration are excessive, change the design from an embankment to a viaduct or other structure shall be considered.

Settlement estimates shall show not only how fast construction should proceed (appropriate timeframe for when installation of overlying permanent way track structure can begin) but also shall demonstrate that any ongoing settlements, which occur after the rail line is opened, can be rectified economically by shimming and/or adjusting track fasteners (vertical adjustment capacity approximately 0.25 inch), or other routine track maintenance throughout the long-term design life of the earth structure; if not, advance mitigation alternatives shall be considered. For the purpose of this section, “long-term” shall be defined as 100 years. Clearances over rail tracks and roadways shall include an allowance for anticipated short-term and long-term movements of earth structures.

Considering that settlement of earth structures is time dependent and will vary by segment, the geotechnical engineers shall evaluate and establish the time duration waiting (leaving) period following initial fill embankment placement before releveling of subgrade and subsequent construction of the overlying track bed permanent way is allowed to take place. An illustration of various settlement parts related to time is shown in Figure 6-7. Based on international experience for other HST systems, the waiting period duration is typically 6 months to 12 months. To meet CHSTP design and performance requirements, a periodic settlement survey program shall be developed by the geotechnical engineer and then implemented during and after the construction phase to monitor settlement at the acceptance check timeframe after laying track, and then long term residual settlement as part of the track maintenance program.

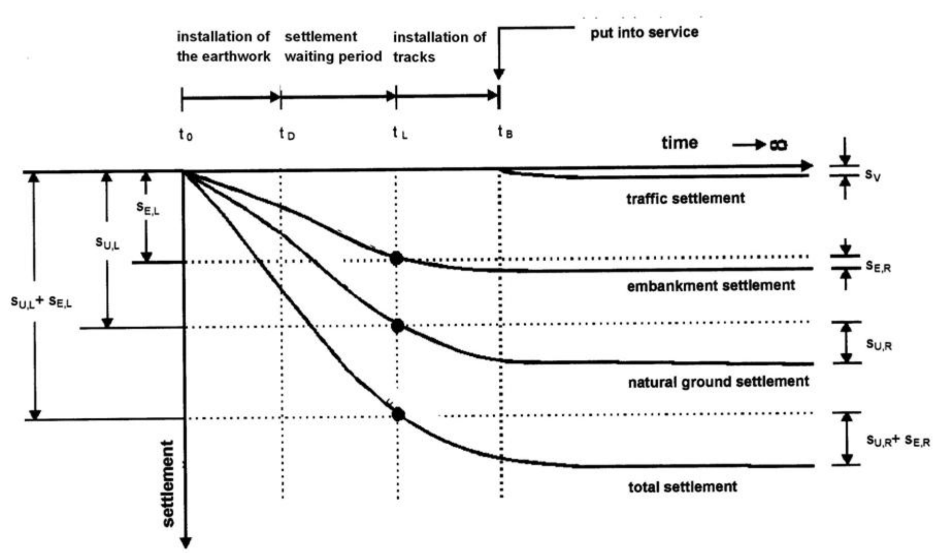


Figure 6-7: Different Settlement Parts by Time
Reference: Figure no. 22 of UIC-719R (2008)

6.6.8 Embankment Foundation Settlement Mitigation and Foundation Modification Using Ground Improvement Methods

For track embankment segments or at-grade trackway features that do not meet settlement criteria or indicate stability problems, advanced mitigation measures such as pre-loading, over-excavation and replacement, or other ground improvement methods shall be considered for geotechnical design.

Ground improvement measures may also be necessary for advance mitigation of potential seismic hazards (such as liquefaction or seismic stability) or other geologic hazards such as collapsible soils, potential hydro-consolidation, regional subsidence, etc. The selection of mitigation methods or candidate ground improvement options for preliminary design shall follow the process described in detail in the FHWA Ground Improvement Reference Manuals, Volumes I and II, FHWA-NHI-06-019/020, dated 2006.

A settlement monitoring program shall be developed and implemented by the geotechnical engineer during the construction phase for any mitigation method selected. InSAR techniques shall be considered as possible methods for large scale regional monitoring in addition to traditional surveying and the use of geotechnical instrumentation during and after construction.

For track segments located in relatively large-scale geographic areas where deep-seated regional subsidence is an ongoing problem with expected duration to continue over some or all of the design life, typical ground improvement measures may not be economically feasible. The geotechnical engineer shall identify the approximate regional boundary limits for these segments and shall provide information to the track and civil designers regarding expected range in total magnitude and estimated rate (inches per year) of future regional subsidence movements.

6.6.9 Evaluation of Earthwork-Related Factors for Shrink/Swell (Shrinkage and Bulking) Estimation

The geotechnical engineers shall provide shrinkage/swell factors for the anticipated cut and embankment fill soils for purposes of earthwork quantity computations. Available reference sources in common use for approximate factors (earthwork shrink/swell) are listed as follows:

- Shrink/Swell Factors for Common Materials - Exhibit 4.6-F, FHWA Geotechnical Technical Guidance Manual (draft) 2007
- Geotechnical Design Manual M46-03 - State of Washington Department of Transportation, Chapter 10 Soil Cut Design, Table 10-1 Approximate Shrink/Swell Factors, WADOT Manual dated September 2005

Earthwork quantity estimation shall also consider embankment overbuild (higher elevation than design profile) that may be necessary on a segment-by-segment basis to allow for short-term and long-term settlement movement of the embankment and/or underlying foundation soils supporting trackway embankments.

6.6.10 Erosion Control for Embankment Features

Geotechnical studies for design shall provide recommendations to the engineering designers for erosion control needs. Evaluations shall be based on characterization of embankment materials, potential water sources, railway geometrics, and slope design. Design recommendations shall be provided to control surface drainage when integral to the design or performance of the earth structures, such as surface drainage ditches on slopes, interceptor ditches, and drainage channels. Geotechnical evaluation to support selection and preliminary design for erosion control shall follow the processes described in the reference document titled Design and Implementation of Erosion and Sediment Control – Reference Manual, FHWA NHI-05-013, 2006.

The design details or requirements shall be incorporated in the geotechnical report and construction plans. Geotechnical discipline shall coordinate with the hydrology and hydraulics and civil design disciplines for erosion control since they provide project-wide drainage design for the control of surface drainage. If long-term erosion control measures will include establishing vegetation on slopes, then consideration shall be given to the use of erosion mats or other stabilization methods for slope inclinations steeper than 3H:1V.

Geotechnical design recommendations shall also include evaluation of temporary construction erosion control requirements on cut-and-fill slopes when integral to geotechnical design or performance. For example, the requirement to provide bench drainage during construction of slopes may be required to ensure construction-phase stability.

6.7 RETAINING WALLS, FILL WALLS, AND REINFORCED SOIL SYSTEMS

6.7.1 Definitions and Wall Types Including Acceptable and Unacceptable Walls

Walls shall be classified as either a “fill wall” or a retained “cut wall.” Examples of fill walls include standard cantilever walls, MSE walls, and modular gravity walls (gabions, bin walls, and crib walls). Cut walls include U-walls, 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, California (Caltrans) Amendments, and the FHWA Earth Retaining Structures Reference Manual (FHWA 2008). 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. Selection of appropriate earth retention system for a given setting shall be 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, OCS poles, 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, pull-out capacity of the structural members or soil reinforcement from the soil.

Retaining walls 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.

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 movement due to settlement for walls supporting tracks needs to be within acceptable limits to meet structural and track movement tolerance requirements provided in Sections 6.3 and 6.6 of this TM, as well as wall aesthetics (visual appearance) to be established by the structural designer.

6.7.3 Limit States and Resistance Factors

Geotechnical designs for retaining walls shall be performed in accordance with AASHTO LRFD Bridge Design Specifications with California (Caltrans) Amendments. However the Amendments confirm that abutment foundations are not subject to LRFD design approach and so conventional WSD shall be used. 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, non-gravity cantilevered walls, anchored walls, MSE walls, and prefabricated modular walls.

6.7.4 External Loads and Stability Analysis

AASHTO LRFD Section 11with California Amendments shall be used for evaluation of stability for retaining walls and abutments. Those provisions include calculation methods for various wall types and shall include analyses for overturning, bearing resistance, external stability (soil failure), 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. 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 per Section 6.7.7 of this TM.

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. An exception to this requirement is for U-walls (retaining walls with continuous base slab between them) that are used where the top of trackway subgrade is below the groundwater table/flood level. No permanent dewatering shall be assumed for design of U-wall sections that are undrained structures subject to hydrostatic pressures, both laterally and vertically (buoyancy).

Retaining wall drainage 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 satisfies 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 that intersect 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, 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 may 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 Soil Systems

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

6.7.7 Settlement and Horizontal Deformation Tolerances

Settlement issues, especially differential settlement, are of primary concern in the selection of walls. 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, tracks, and any potentially affected nearby structures, i.e., both structural and aesthetic.

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 has 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 RSS embankments and 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 the FHWA 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. Numerous facing systems and geosynthetic reinforcements are available, however embedded metallic strip reinforcing elements shall not be used, since they are potentially susceptible to stray current corrosion that causes significant loss of section over the life of infrastructure-supporting track.

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 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 the FHWA 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 SLOPES

6.8.1 Overview

This section addresses the analysis and design of slopes, including cut slopes, fill slopes (embankment slopes), landslides, and natural slopes. In addition to the provisions herein, sloped excavations shall be designed and constructed in accordance with any and all local, state and federal regulations, including but not limited to OSHA and Cal-OSHA requirements.

The provisions contained herein supersede any slope analysis and design provisions contained in TM 2.6.7.

6.8.2 Slopes Requiring Evaluation

Slope stability shall be evaluated where any of the following conditions are present:

- Where slope stability related geologic hazards are present as defined in TM 2.9.3
- Soil and sedimentary rock slopes steeper than 5:1 (H:V) and igneous and metamorphic rock slopes steeper than 3:1 (H:V), and any flatter slope where the following adverse conditions could exist:
    - Where there is a potential that adversely oriented and kinematically unstable geologic joints, bedding, or slip surfaces are potentially weak
    - Where evidence of prior landsliding is present (see TM 2.9.3 for evaluation guidelines)
    - Where quick or sensitive clay conditions are present
    - Where liquefaction-related lateral spreading conditions are present, they shall be evaluated using the methods described in Section 6.10.
    - Any other conditions that the GE or CEG feel warrants slope stability evaluations

With respect to the location of a slope relative to the project right-of-way, the following criteria shall be used to assess when slope stability evaluations are required:

- Where the movement of a potential slide mass either directly or indirectly could affect HST facility operations or integrity, for example, where a slope failure slide mass could intersect or envelop tracks, stations, or appurtenant facilities
- Where instability of slopes away from the HST facilities could impact operations or integrity, for example, landslides, debris flows or rockfalls that may originate away from HST facilities, but that could be deposited within SHT facilities

6.8.3 Design and Analysis

The following sections provide requirements for design and analysis of soil and rock slopes. Static slope stability analysis shall be considered in the Service 1 Load Case. Slope performance is expected to be such that normal operation of the HST facilities is maintained. Requirements for seismic analysis and design of slopes are provided in Section 6.10 of this TM.

6.8.3.1 Loads, Unit Weights, and Surcharges

Where structures, trains, or other non-earth materials are present within a slope area, their corresponding surcharge loads shall be included in slope stability analysis. Loading shall be established in accordance with TM 2.3.2. Soil loads shall be taken at their nominal values; no load factors are applied to soil in slope stability analyses.

6.8.3.2 Slope Stability Analysis Methods

Analysis methods for soil and rock are presented below. In some cases, geomaterials may exhibit behavior that is intermediate between soil and rock. Such materials are sometimes referred to as intermediate geomaterials (IGM). Since slope stability methods specific to IGMs are not generally available, both soil and rock methodologies shall be applied for IGMs, and the most conservative result shall govern design.

6.8.4.2.1 Soil

Static soil slope stability shall be evaluated by calculating a FOS using limit equilibrium procedures with the method of slices. Permissible methods of limit equilibrium analysis include Spencer’s (1967) for any slip surface shape, and Taylor’s (1937) or modified Bishop’s (1955) for circular slip surfaces. Both circular and non-circular potential failures surfaces shall be considered in the analyses. Search routines shall be used to find the slip surface with the lowest FOS. The slip surface with the lowest FOS is considered the “critical slip surface”. Slip surfaces with FOS greater than the critical slip surface shall also be reviewed and considered in the design. Widely used and well-validated computer programs for slope stability analysis shall be used such as SLIDE by RocScience, SLOPE/W by GEO-SLOPE International, XSTABL, UTEXAS3 by the University of Texas, or other equivalent software.

The FOSs may also be calculated using finite element or finite difference methods that employ the φ-c (strength) reduction method (Dawson et al., 1999). However, conventional method of slices limit equilibrium methods must be run in conjunction with φ-c reduction analyses. The φ-c reduction method does not require an assumed slip surface (as does the method of slices), which can be advantageous when subsurface stratigraphy or other factors could lead to slip surfaces with irregular geometry.

Infinite slope stability analyses shall be performed for soil slopes where shallow (6 feet or less) downslope seepage parallel to the slope face could develop. Such a hydrogeologic condition could result where a shallow layer is underlain by a less permeable layer (i.e., residually weathered soil over bedrock), and rainfall potential or temporary submergence is sufficient to saturate the less permeable layer. Infinite slope stability analyses shall be conducted in accordance with the analytical methods presented in Section 5.5 of FHWA (2005).

6.8.4.2.2 Rock

For the purpose of slope stability analysis, rock slopes are characterized herein by groups based on the anisotropic or isotropic characteristics of the rock mass. Therefore, the first step in analysis of the rock slope is to establish if the rock mass is anisotropic or isotropic.

The first group assumes the rock mass consists of heterogeneous rock masses with structural anisotropic systems of relatively regular discontinuities in the form of joint sets, bedding, fissures, or foliation. The strength and slope stability of these types of rock masses is typically controlled by the discontinuities, and analytical techniques for slope stability assessment shall consider the kinematic stability of blocks or groups of blocks sliding upon the discontinuities, or toppling. Limit equilibrium methods that calculate a FOS shall be used. These analyses shall consider blocks that are kinematically permissible as evaluated by the Markland (1972) method, block theory (Goodman and Shi, 1985), or rock slope engineering techniques described by Hoek and Bray (1981) and Wyllie and Mah (2004). If computer software is used for rock slope stability analyses, it shall be well validated and widely accepted.

The second group assumes the rock mass consists of homogeneous and isotropic rock masses with irregular and/or closely spaced discontinuities that do not have well defined systematic planes of weakness. The evaluation of the stability of these types of slopes shall be based on the non-circular limit equilibrium techniques described above for soil, except that a suitable rock strength model shall be used such as General Hoek-Brown criterion (Hoek et al., 2002; Wyllie and Mah, 2004; Hoek, 2010).

Where rock slopes existing upslope of HST facilities and have the potential to shed rock pieces over time, an evaluation of the rock fall hazard shall be performed in accordance with the procedures outlined in the FHWA and Oregon DOT (2001) Rockfall Catchment Area Design Guide. Computer programs that model rockfall physics such as the Colorado Rockfall Simulation Program (CRSP III) or RocFall (by RocScience), or other equivalent software, may be used in conjunction with the FWHA procedures. Rockfall catchment basin width and inclination shall be designed to retain 99% of fallen rocks. If right-of-way is not available to size catchment basins to achieve 99% rockfall retention, additional mitigation measures such as rockfall protection walls, wire mesh, cable drape, or catchment fences shall be used in the design. In areas where rock fall is a critical problem, a railway slide fence with electronic warning system shall be installed in conjunction with an appropriate catchment ditch and rock fall retention system described above. Other warning systems for rockfall events that may be considered are as follows:

1. Acoustic sensing
2. Electromagnetic sensing
3. Seismic sensing
4. Visual sensing, using cameras

6.8.4.2.3 Input Data and Parameters

Input data and parameters used in slope stability analyses for both soil and rock shall take into consideration geology, groundwater and rainfall, and proposed geometry/topography. Soil engineering parameters shall be developed for use in slope stability analyses in accordance with Section 6.2 of this TM.

When available, empirical or historical data and direct observation within the geologic unit on the past performance of similar slopes shall be considered in slope stability evaluations. In particular, when assessing existing landslides, shear strength parameters back-calculated from previous failures shall be considered.

Drained or undrained shear strength parameters shall be selected, depending on the rate of loading and the permeability characteristics of the soil or rock. In general, undrained strengths should be used for relatively short-term loads and end-of-construction cases. Long-term stability should generally use drained strengths. In the analysis of existing landslides, residual shear strengths shall be used for existing landslide slip planes. FHWA (2005) Section 4 should be consulted for additional guidance on the selection of shear strength parameters.

6.8.4.2.4 Minimum Factors of Safety

The FOS calculated with the methods above for soil and rock shall meet the following minimum requirements. For the Service 1 static slope stability case, the FOS is simply the inverse of the resistance factor (φ); FOS=1/φ.

Slopes shall be designed so that the minimum FOS of any slip surface (or φ–c reduction scenario) is 1.3 or greater for short-term construction cases and 1.5 or greater for long-term static cases. If a slope must be designed to meet the standards of another agency, such as a city, county, or right-of-way holder that has more stringent requirements, then the more stringent requirement shall govern.

Short-term construction cases could include:

- Sidewalls of sloped temporary excavations
- Temporary surcharge fill slopes
- Temporary back cuts
- The end-of-fill placement (sometimes referred to as the end-of-construction case) for fills over fine-grained foundation soils that will behave undrained or partially undrained during construction, and will gain strength over time
- Construction stages prior to the end-of-construction if the staging is such that an intermediate stage could be critical. Such conditions could arise when significant consolidation of fined-grained soils is allowed to occur between stages

6.8.4 Requirements for New Slopes

New fill slopes in soil shall be no steeper than 2H:1V. If 2H:1V slopes cannot be achieved because of geometric and/or right-of-way restrictions, the design will likely have to incorporate retaining walls.

Refer to Section 6.6 Track Bed Embankments and Embankment Foundations for slope benching and geosynthetic reinforcement requirements.

Limitations on the inclination of rock slopes and cut slopes in soil are not imposed. Maximum inclinations shall be established based on achieving the minimum FOS and rockfall protection requirements described earlier. For soil cut slope configurations over 30 feet in height, designs shall include mid-slope benches for purposes of drainage and of facilitating future access for maintenance reasons. Slope benches are typically 6 feet wide with 6% gradient toward the low end of the cut slope, and include a lined gutter channel at the drainage surface. For deep cut slopes in soil, slope benches shall be laid out on average of every 30 feet in height (allowance from 26- to 32-foot range is considered acceptable) and shall be connected to the surrounding ground surface for access.

6.8.5 Landslides

Evaluations of the stability of landslides require special attention and specialized techniques. Geomorphic evidence of landsliding shall be evaluated through examination of a series (different years) of aerial photographs (preferably stereographic, if available), geologic/landslide hazard maps, and topographic maps.

The method of analyses shall first take into consideration the style of slope failure that has occurred in the formation to be analyzed as well as the recognized behavior of that formation. Soil and rock engineering parameters shall also consider the proven performance values for the subject formations. Locating the slip surface is a critical step in the characterization of a landslide. Techniques that shall be considered for locating slip surfaces include surficial mapping, down-hole logging of large diameter borings, continuous sampling of borings, and monitoring with inclinometers. Slope stability analyses conducted for landslides shall consider sliding along the existing slip surface as well as sliding upon potential new slip surfaces.

Shear strength parameters for the stability analyses of landslides shall be based on the shear strength along the existing slide plane. Shear strength parameters used in landslide analyses shall consider values back-calculated from slope stability analysis of the pre-slide configuration and groundwater conditions. Back-calculated values should be compared with data obtained from laboratory testing. In performing laboratory testing to evaluate shear strength of a landslide slip surface, the shearing shall be performed along a pre-sheared surface using either a direct shear or ring shear test. It is preferable that the pre-sheared surface be that of the actual landslide slip surface, such that the laboratory specimen is prepared from a block sample taken directly from the slide plane in a large-diameter boring. If this is not possible, then an intact specimen can be used that is pre-cut along the slip surface with a knife prior to shearing. The shear test displacement shall be large enough such that no appreciable reduction in shearing resistance occurs with additional shearing (residual strength condition is achieved). With the direct shear apparatus, this typically involves repeated cycles of shear, with the shear strength plotted against the aggregate of the unidirectional shear displacement. Further background on these procedures can be found in Blake et al. (2002).

Additional guidance on the evaluation of landslides is provided in FHWA (2005).

6.8.6 Seismic Analysis for Design of Slopes

Seismic analysis and design of slopes shall be performed in accordance with Section 6.10 of this TM.

6.8.7 Slope Deformations

The potential for slope deformation shall be considered in the design. The potential mechanisms of deformation include settlement because of consolidation or compression, settlement because of collapse upon wetting, shrink and swell caused by changes in moisture content, and creep-type shear displacement. Section 6.6.11 of this TM addresses settlement due to consolidation and compression.

The potential for shrink and swell resulting from changes in moisture content and/or stress shall be addressed in design of cut slopes. Analysis shall be conducted to assure that movements due to shrink and swell are less than the maximum displacements permitted for the project. Removal of expansive soil or rock and replacement with non- or very low-expansive material shall be performed as needed to meet maximum allowable displacement criteria. Constraints on acceptable fill material types described in Section 6.6 of this TM are such that shrink and swell should not pose a problem for fill slopes and embankments.

Creep-type shear displacements occur in the downward direction of many slopes and are typically characterized displacement rates on the order of 1 inch per year or less. Creep may occur in a relatively shallow zone (i.e., the upper 5 to 10 feet) because of the effects of seasonal wetting/drying or freeze/thaw cycles. Deeper-seated creep movement can occur in association with historic, prehistoric, or incipient landslides. In existing slopes, creep may be observed by slope inclinometer measurements, or may be evidenced by tilted or curved trees. Creep potential shall be considered in slope stability design, particularly when evidence of creep is observed in the field, or where past experience has shown site soil or rock to be creep prone. Where creep movements could result in adverse effects to HST facilities, mitigation methods shall be employed in the design.

6.8.8 Drainage and Erosion Control for Slopes

Drainage provisions and permanent erosion control facilities to limit erosion (including soil erosion and rock slope degradation) are required for design of cut slopes. Surface drainage shall be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. Erosion control for cut and fill slopes shall be performed in accordance with Section 6.6.10 of this TM.

6.8.9 Slope Stability Mitigation Methods

Where the minimum required FOSs cannot be achieved or the alignment cannot be relocated away from unstable slopes, measures to enhance slope stability or mitigate the effects of potentially unstable slopes are warranted. In addition to the discussion below, Section 9 of FHWA (2005) shall be used as guidance for slope stability mitigation methods. Methods of slope stability mitigation not described herein or in FHWA (2005) may be considered provided they can be proved effective through analysis and previous experience.

Subsurface drainage can enhance slope stability by reducing pore water pressures and increasing effective stress, and by reducing the driving stresses from saturated soils and water-filled cracks. Methods of subsurface drainage for slope stabilization include horizontal drains, wells, and trench drains. For long-term applications, passive (gravity flow) forms of drainage such as horizontal drains and gravity flow trenches are generally preferred over active systems such as wells that require pumping.

Unstable slopes can be mitigated through removal of the weak and unstable material and through replacement with a suitable compacted fill. This removal and replacement method is typically combined with the installation of subsurface drainage measures. Slope stability can be enhanced through grading that reduces the driving forces and/or increases the resisting forces. Buttresses constructed in the toe area of as slope can utilize weight and higher shear strength to increase resisting forces. Driving forces can be reduced by removal of material near the top of the slope.

Slope stability enhancements could include adding structural inclusions such as soldier piles, soil nails, anchors, dowels, or geosynthetics. The efficacy of any slope stability mitigation method shall be demonstrated through stability analyses using the limit equilibrium methods described earlier.

For cases where debris flow-type failures from existing natural slopes could impact HST facilities, debris flow diversion walls may be considered as a mitigation method. Culverts that are fed by drainages and tributaries should be sized with consideration for debris flow potential. Debris-flow diversion walls shall be designed to withstand the impact from debris flows and to control and redirect them such that there is no adverse impact to HST facilities.

6.9 GROUND IMPROVEMENT

Ground improvement shall be considered as a design alternative for conditions such as, but not limited to, the following:

- As an alternative to deep foundations where poor (i.e., compressible, weak, liquefiable) soils are present
- Mitigation of excessive settlements due to soft and/or compressible soils
- Slope stabilization
- Liquefaction mitigation
- Expedite settlement by enhanced drainage
- Make soil less permeable
- Improve ground behavior for tunneling

- Temporary support (underpinning) for existing foundations

Methods of ground improvement generally consist of the following:

- Vertical drains
- Vibro-compaction
- Deep dynamic compaction
- Stone columns
- Compaction grouting
- Jet grouting
- Permeation grouting
- Deep soil mixing

Ground improvement design shall be performed in accordance with FHWA (2006) "Ground Improvements Reference Manual Volumes 1 and 2, FHWA-NHI-06-019 and FHWA-NHI-06-019." Ground improvement shall be designed to achieve performance requirements with respect, global stability, bearing capacity, and settlement. The ground improvement design shall include a construction quality assurance program that specifies field observation and testing methods necessary to verify the design objectives are achieved.

6.10 Geotechnical Earthquake Engineering

6.10.1 Seismic Design Criteria

Seismic design criteria for geotechnical earthquake engineering have been established in terms of two 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 the two 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 movement/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 movement/settlement between mitigated and non-mitigated soils. Additional measures may be required to limit differential movement/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 HST 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

Limits on site response analyses are presented in TM 2.9.6 for 30% design.

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 assessed that liquefaction will be triggered, the engineering consequences of liquefaction shall be evaluated. In addition to triggering for liquefaction, the geotechnical engineer shall also consider the allowable deformation values described in Section 6.3.5 of this TM and the long-term, post-construction performance requirements for earth and fill conditions.

6.10.6.1 Compositional Criteria for Liquefaction Susceptibility of Silty and Clayey Soils

Evaluation of whether silty and clayey soils meet the criteria for liquefaction susceptibility shall be performed primarily 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.

For fine-grained soils (especially soils that are potentially sensitive) that do not meet the above criteria for liquefaction, cyclic softening resulting from seismic shaking shall be performed. 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 cyclic response for critical cases.

6.10.6.2 Criteria for Liquefaction Susceptibility of Gravels

Gravel layers bounded by lower permeability layers shall be considered potentially susceptible to liquefaction, and their liquefaction susceptibility shall be 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 shall be evaluated as such. Field investigation methods appropriate for soil layers containing gravels include the Becker Hammer Penetration Test (BPT), Large Sampler Penetration Test (LPT), and small interval SPT. Seed et al. (2003) discusses different methods for performing liquefaction analysis in coarse and gravelly soils.

6.10.7 Liquefaction Triggering Evaluations

Liquefaction-triggering evaluations shall be performed for sites that meet both of the following two criteria:

- The estimated maximum groundwater elevation at the site is within 75 feet of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile is characterized in the upper 75 feet as having soils that meet the compositional criteria for liquefaction with a measured SPT resistance, corrected for overburden pressure and hammer energy $(N_1)_{60\text{-}cs}$, less than 33 blows/ft., or a cone tip resistance $q_{c1N\text{-}cs}$ of less than 185, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

CPT and/or CPTu (with pore water pressure measurement) shall be used as the primary method of field investigation for liquefaction analysis where it can be advanced without premature refusal. SPT shall be used as the primary liquefaction evaluation method where borings are performed. LPT, shear wave velocity (Vs), or BPT shall be used in soils difficult to test using SPT and CPT methods, such as gravelly soils. In addition, small interval SPT (blow counts measured for every 1 inch) shall be used in gravelly soils. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modelled well by the simplified methods.

6.10.7.1 Simplified Procedures

All three simplified methods by Youd et al. (2001), Seed et al. (2003), and Idriss and Boulanger (2008) shall be used for liquefaction-triggering analysis for each boring and/or CPT. The Modified Chinese Criteria for clayey soils in the Youd et al. (2001) method shall not be used. Results in terms of FOS shall be reported. Results of these analyses shall be interpreted according to the following. If the FOS values between the three methods are within 20% of each other, an average FOS shall be reported for that particular boring and/or CPT. If the FOS values from these three methods vary by more than 20% and use of the more conservative results for design would have significant cost consequences, some additional evaluations may be warranted. The additional evaluations shall include an assessment of which method best applies to this specific case, additional soil-specific field and laboratory testing, and/or review by an expert panel.

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

6.10.7.2 Liquefaction-Induced Movement/Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Methods to estimate movement/settlement of unsaturated granular deposits are presented in Section 6.10.14 of this TM. Liquefaction-induced total ground settlement of saturated granular deposits shall be estimated using at least two of the following methods: Ishihara and Yoshimine (1992), Zhang et al. (2002), Idriss and Boulanger (2008), and Cetin et al. (2009). Other methods (i.e., more recently developed methods that are an improvement) may be used if justified and approved by the PMT. Where only sparse or widely-spaced borings or CPT data are available, differential settlement between two adjacent supports shall be assumed to one-half of the total settlement (Martin and Lew, 1999). The corrected SPT blow counts and CPT tip resistance values for estimating movements/settlements shall include all corrections, including the corrections for fines. However, it should be noted that 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.

It should be noted that all of these estimates are free-field settlements, and structural movement/settlements resulting from soil liquefaction are more important in most of the cases (Bray and Dashti, 2010). Structural movement/settlements may also result from shear-induced movements. Hence, methods that are used for estimating lateral ground movements may be required.

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

6.10.7.3 Liquefied Residual Strength Parameters

Unless soil-specific laboratory performance tests are conducted as described later in this section, residual strengths of liquefied soil shall be evaluated using at least two of these procedures: Seed and Harder (1990), Idriss and Boulanger (2008), Olson and Stark (2002), and Kramer and Wang (2011). Design liquefied residual shear strengths shall be based on weighted average of the results; Ledezma and Bray (2010) may be used as a reference to select a reasonable weighting scheme. Other methods for estimating liquefied residual shear strength (i.e., more recently developed methods that are an improvement) may be used if justified and approved by PMT. 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, two of the above empirical methods 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.

6.10.7.4 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 that is not within a few hundred feet of a free face shall be made using the method outlined by Ishihara (1985) and shall be compared against results by the method presented in Youd and Garris (1995). It is emphasized that settlement may occur, even with the absence of surface manifestation. The Ishihara (1985) 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% or more exists, or a free face conditions (such as an adjacent river bank) exists. Use Shamoto et al. (1998) as a method to assess the maximum distance from the free face where lateral spreading displacements could occur. 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

If there is a free face condition, the post-liquefaction flow failure FOS of an earth slope or sloping ground shall be estimated per Section 6.10.15.1 of this TM before estimating liquefaction-induced lateral movements. If the post-liquefaction stability FOS is less than 1.0 then empirical or analytical methods cannot generally be used to reliably predict the amount of ground movement.

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) and Idriss and Boulanger (2008). Analysts shall be aware of the applicability and limitations of each method. Lateral displacements shall be evaluated using the Zhang et al. (2004) method and at least 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, the Newmark sliding block analyses shall be used. Newmark analyses shall be conducted similar to that described in the seismic slope stability section, except that estimation of the yield acceleration (k_y) shall consider strength degradation due to liquefaction. In addition, numerical methods using finite elements and/or finite difference approach may be used.

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 of this TM 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

During the liquefaction evaluation, the engineer shall evaluate the extent of liquefaction and potential consequences such as bearing failure, slope stability, and/or vertical and/or horizontal deformations. Similarly, the engineer shall 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 Iai et al (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 and 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

There are many different methods of ground improvement. The five primary methods of ground improvement (and some examples of each of them) to be considered for soil liquefaction mitigation are:

- Replacement
  - Excavate and replace with compacted fill
- Vibratory Densification
  - Vibro-compaction
  - Vibro-replacement stone columns (combination of vibration and displacement)
  - Deep dynamic compaction
- Displacement Densification/Reinforcement
  - Compaction grouting
  - Displacement piles
  - Vibro-replacement stone columns (combination of vibration and displacement)
- Mixing/Solidification
  - Permeation grouting
  - Deep soil mixing
  - Jet grouting
- Drainage
  - Passive or active dewatering systems
  - (Earthquake drains are <u>not</u> permitted for use)

The implementation of these techniques shall be designed to fully, or partially, eliminate the liquefaction potential, depending on the requirements of the engineered facility under consideration. 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 produce acceptable structural performance (consistent with the requirements for the two design earthquakes) in terms of liquefaction/lateral spread-induced displacements and structural damage. The appropriate means of structural mitigation may depend on the magnitude and type of liquefaction-induced soil deformation or load.

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

- Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils
- Post-tensioned slab foundation (appropriate only for small, lightly loaded structures)
- Continuous spread footings having isolated footings interconnected with grade beams
- Mat foundation (appropriate only for small, lightly loaded structures)

Details, applicability, and limitations of these techniques can be found in Martin and Lew (1999). Additional requirements for design of piles in liquefied soil are presented below.

6.10.10 Seismic Considerations for Lateral Design of Piles in Liquefiable Soils

Seismic considerations for lateral design of pile/shaft design in 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. 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. (2007) and Boulanger, et al. (2007).

The displacement-based approach for evaluating the impact of liquefaction-induced lateral spreading loads on deep foundation systems that shall follow Caltrans' "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading," dated February 2011 (http://dap3.dot.ca.gov/shake_stable/references/Guidelines%20on%20Foundation%20Loading-Jan%202011.pdf) shall be used. However, the liquefaction susceptibility and triggering analyses performed as part of this procedure shall be based on Section 6.10.6 and Section 6.10.7, respectively. Similarly, the lateral spread estimates shall be based on Section 6.10.8. The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values described in Section 6.3.5 of this TM and develop mitigation plans described in Section 6.10.9 of this TM, if necessary. The geotechnical engineer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

Numerical methods incorporating finite element and/or finite difference techniques may be used to assess pile response in laterally spreading soils.

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.

For retaining walls that are not restrained from rotation at the top in locations where PGA values are less than or equal to 0.30g, walls shall be designed for only active pressures and inertial forces of the wall itself, but additional seismic earth pressures shall not be considered. For walls containing cohesionless materials as backfill, seismic pressures shall be estimated using the Mononobe-Okabe (M-O) method (Mononobe and Matsuo, 1929). Horizontal seismic coefficient (k_h) shall be estimated using the Bray et al. (2010) method assuming a wall movement of 1 inch. For the 30% design phase and final design, PGA values associated with two 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 a triangular distribution with the resultant acting at 0.33H from the bottom (i.e., an upright rather than inverted triangle). This pressure shall be added to the active earth pressure for the design. For higher angles of sloping back fills where the M-O solution does not converge (see Figure 7.8 of NCHRP Report 611) 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 0.35g, walls shall be designed for only at-rest pressures and inertial forces from the wall itself, but 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 (k_h) shall be estimated using Bray et al. (2010) assuming a wall movement of 1 inch.

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 of this TM 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 of this TM 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 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 6.10.7 of this TM shall be used. In addition, strength reduction due to cyclic degradation versus strength increase due to the effects of rate of loading shall be considered for normally consolidated clayey layers and non-liquefiable sandy layers. Chen et al. (2006) have discussed the effects of different factors on the dynamic strength of soils. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium FOS is less than 1.1, 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 FOS for this decoupled analysis is greater than 1.1 for liquefied conditions, k_y 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 FOS 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). Other methods such as Newmark time history method or more advanced methods involving numerical analysis may be used, but 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.

6.10.15.2 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 seismic coefficient (k_h) that acts upon the critical failure mass. A horizontal seismic coefficient, k_h, estimated using Bray and Travasarou (2009) and a vertical seismic coefficient, k_v, equal to zero shall be used for the evaluation of seismic slope stability. The Bray and Travasarou (2009) method requires an estimate of allowable deformation to compute k_h. Therefore, for the MCE, an allowable deformation of 6 inches may be used, and for the OBE, the allowable deformation presented in Table 6.3.5-1 shall be used. For these conditions, the minimum required FOS is 1.0. 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.

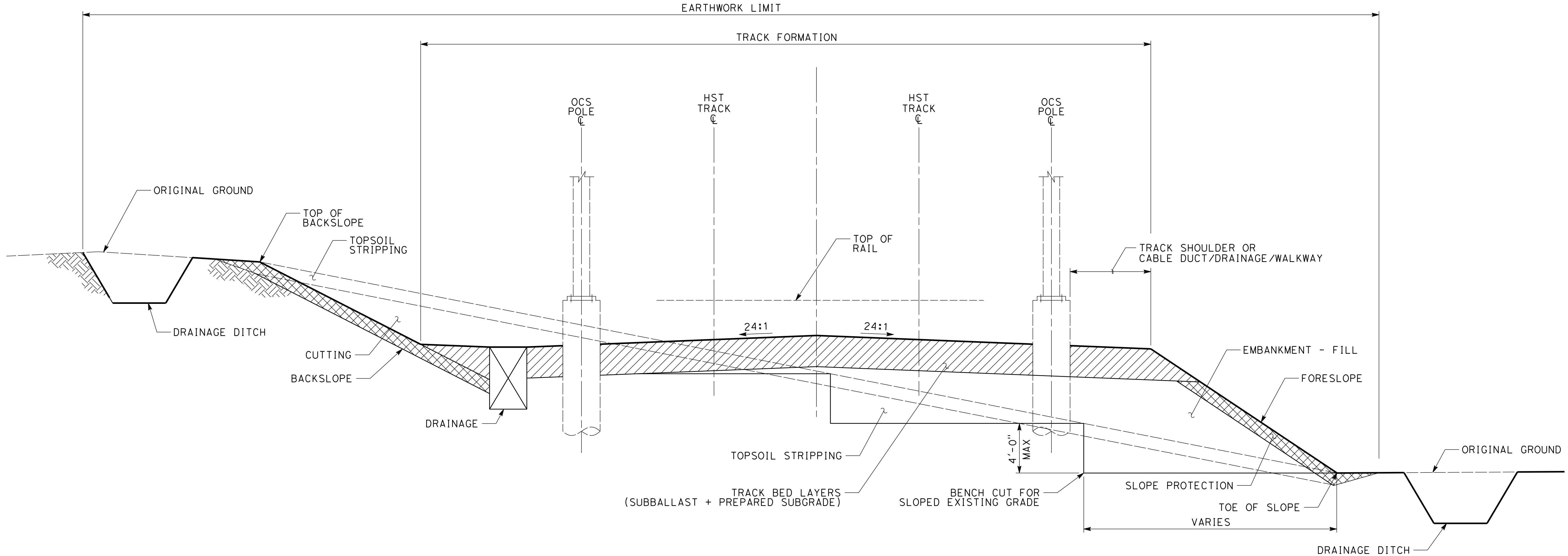
6.10.15.3 Seismic Slope Deformations

Deformation analyses shall be performed where an estimate of the magnitude of seismically induced slope deformation is required, and the pseudo-static slope stability FOS 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), Bray and Travasarou (2007), and Rathje and Saygili (2008), or dynamic stress-deformation models. These methods shall not be employed to estimate displacements if the post-earthquake static slope stability FOS 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

Downdrag loads on foundations shall be evaluated in accordance with Article 3.11.8 of the AASHTO LRFD Bridge Design Specifications and as specified herein. The AASHTO LRFD

Bridge Design Specifications, Article 3.11.8, recommends the use of the non-liquefied skin friction in the non-liquefied layers above and between the liquefied zone(s), and a skin friction value as low as the residual strength within the soil layers that do liquefy, to calculate downdrag loads for the extreme event limit state.



TYPICAL SECTION EARTHWORK CUT/FILL
SCALE: 1/4"=1'-0"

4 0 4 8
1/4"=1'-0"

REV	DATE	BY	CHK	APP	DESCRIPTION

DESIGNED BY B. VALENTI
DRAWN BY D. SO
CHECKED BY T. LEE
IN CHARGE J. CHIRCO
DATE 06/22/11

PARSONS BRINCKERHOFF

CALIFORNIA HIGH-SPEED RAIL AUTHORITY

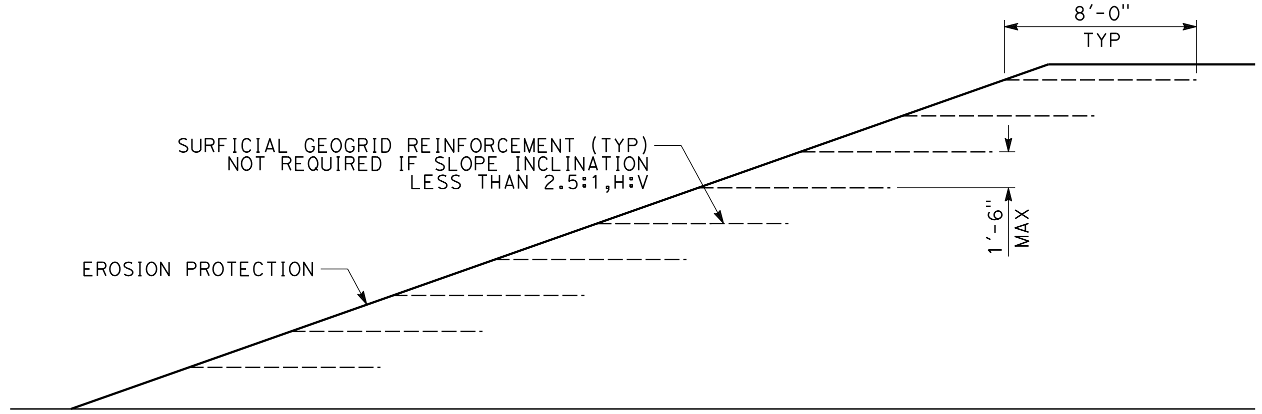
CALIFORNIA HIGH-SPEED TRAIN PROJECT
TECHNICAL MEMORANDUM

EARTHWORK
TYPICAL EMBANKMENT

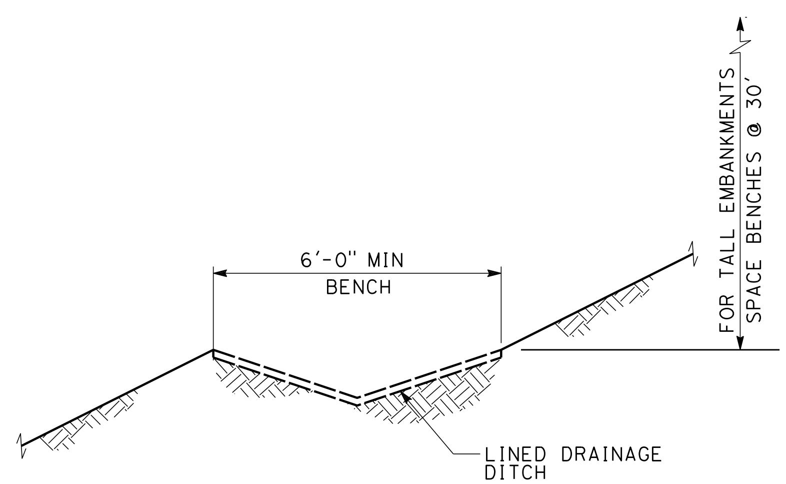
CONTRACT NO. 13259
DRAWING NO. TM 2.9.10-A
SCALE AS SHOWN
SHEET NO.

NOTE:

1. PROVIDE DETAILS AS SHOWN FOR ALL EMBANKMENTS THAT SUPPORT HIGH SPEED RAIL.



SLOPE DETAIL
ALL EMBANKMENTS GREATER THAN 5' IN HEIGHT
SCALE: 1/4"=1'-0"



SLOPE DETAIL
MID SLOPE BENCH FOR EMBANKMENT AND CUT
HIGHER/DEEPER THAN 30'
SCALE: 1/2"=1'-0"

REV	DATE	BY	CHK	APP	DESCRIPTION

DESIGNED BY B. VALENTI
DRAWN BY D. SO
CHECKED BY T. LEE
IN CHARGE J. CHIRCO
DATE 06/22/11

PARSONS BRINCKERHOFF

CALIFORNIA HIGH-SPEED RAIL AUTHORITY

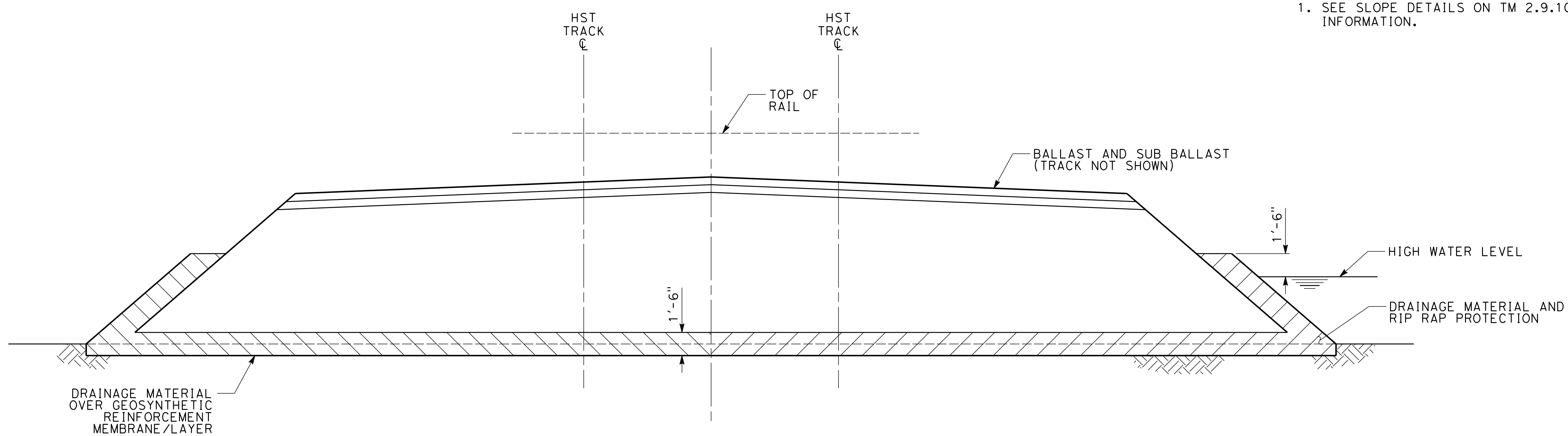
CALIFORNIA HIGH-SPEED TRAIN PROJECT
TECHNICAL MEMORANDUM

EARTHWORK
EMBANKMENT TYPICAL DETAILS

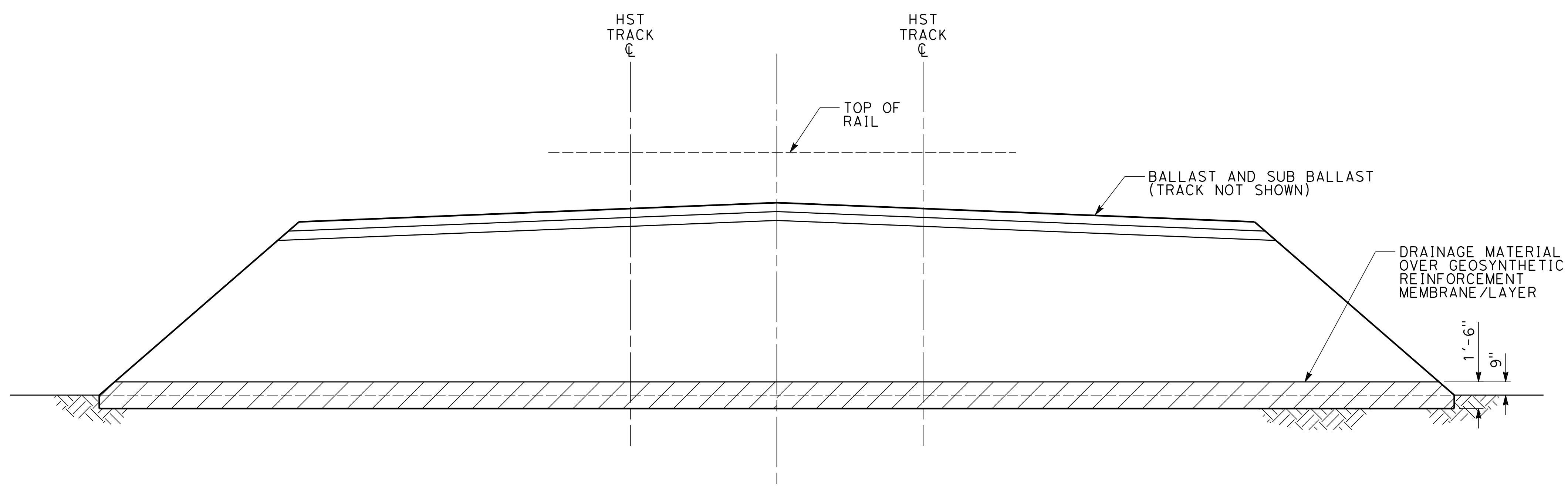
CONTRACT NO. 13259
DRAWING NO. TM 2.9.10-B
SCALE AS SHOWN
SHEET NO.

NOTE:

1. SEE SLOPE DETAILS ON TM 2.9.10-B FOR ADDITIONAL INFORMATION.



DRAINAGE LAYER UNDER EMBANKMENT IN FLOODPLAIN
SCALE: 1/4"=1'-0"



DRAINAGE LAYER UNDER EMBANKMENT IN WET LOCATIONS

TYPICAL SECTION EARTHWORK EMBANKMENT
IN FLOOD PLAIN / HIGH WATER
SCALE: 1/4"=1'-0"

REV	DATE	BY	CHK	APP	DESCRIPTION

DESIGNED BY	B. VALENTI
DRAWN BY	D. SO
CHECKED BY	T. LEE
IN CHARGE	J. CHIRCO
DATE	06/22/11

PARSONS BRINCKERHOFF

CALIFORNIA
HIGH-SPEED RAIL AUTHORITY

CALIFORNIA HIGH-SPEED TRAIN PROJECT
TECHNICAL MEMORANDUM

EARTHWORK
EMBANKMENT IN FLOODPLAIN / HIGH WATER

CONTRACT NO.	13259
DRAWING NO.	TM 2.9.10-C
SCALE	AS SHOWN
SHEET NO.	