Chapter 10

Geotechnical

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Appendix

Appendix 10.A: Guidelines for Geotechnical Investigations
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# Acronyms

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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
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<td>Authority</td>
<td>California High-Speed Rail Authority</td>
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<td>BDS</td>
<td>Bridge Design Specifications</td>
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<td>Caltrans</td>
<td>California Department of Transportation</td>
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<tr>
<td>CBC</td>
<td>California Building Codes</td>
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<tr>
<td>CEG</td>
<td>Certified Engineering Geologist</td>
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<tr>
<td>CGS</td>
<td>California Geological Survey</td>
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<td>CHSTP</td>
<td>California High-Speed Train Project</td>
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<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
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<td>CPT&lt;sub&gt;u&lt;/sub&gt;</td>
<td>Cone Penetration Test with pore pressure measurements</td>
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<td>FLH</td>
<td>Federal Lands Highway</td>
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<td>GBR</td>
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<td>GTGM</td>
<td>Geotechnical Technical Guidance Manual</td>
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<td>HST</td>
<td>High-Speed Train</td>
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<td>IGM</td>
<td>Intermediate Geomaterials</td>
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<td>LOTB</td>
<td>Logs of Test Borings</td>
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<td>MASW</td>
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<td>M-O</td>
<td>Mononobe-Okabe</td>
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<td>MCE</td>
<td>Maximum Considered Earthquake</td>
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<td>OBE</td>
<td>Operating Basis Earthquake</td>
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<td>OCS</td>
<td>Overhead Contact System</td>
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<td>PDA</td>
<td>Pile Driving Analyzer</td>
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<td>PDDM</td>
<td>Project Development Design Manual</td>
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<td>PGA</td>
<td>Peak Ground Acceleration</td>
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<td>SASW</td>
<td>Spectral Analysis of Surface Waves</td>
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<td>SSI</td>
<td>Soil-Structure Interaction</td>
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<td>USGS</td>
<td>United States Geological Survey</td>
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10 Geotechnical

10.1 Scope

This chapter provides guidance, geotechnical criteria, and requirements for the geotechnical engineering design for earthwork, embankments, and bridges/aerial structures, abutments, underground structures, and culverts for the California High-Speed Train (HST) trackway.

10.2 Regulations, Codes, Standards, and Guidelines

Refer to the General chapter for requirements pertaining to regulations, codes, and standards. Elements of HST infrastructure, based on their importance to HST, shall be classified as Primary Type 1, Primary Type 2, and Secondary. Definitions of these elements can be found in the Seismic chapter. Design of geotechnical work specified in this chapter applies to Primary Type 1 and Type 2 structures, while the Secondary structures are designed according to the requirements set forth in the jurisdiction of the local County, City, or third party.

Geotechnical design work, in general, shall be in accordance with the AASHTO LRFD Bridge Design Specifications (BDS) with State of California Caltrans Amendments, these geotechnical design criteria, and the requirements of the following standards:

- American Association of State Highway and Transportation Officials (AASHTO) Guidance
  - AASHTO Guide Specifications for Design and Construction of Segmental Concrete bridges
  - AASHTO Guide Specifications for Thermal Effects in Concrete Bridge Superstructures
- California Department of Transportation (Caltrans)
  - Caltrans Seismic Design Criteria (CSDC)
- California Building Code (CBC)
- American Society of Civil Engineers (ASCE), Geotechnical Baseline Reports for Construction – Suggested Guidelines, prepared by Essex, 2007
- Federal Highway Administration (FHWA) Guidelines
  - FHWA Project Development and Design Manual (PDDM)
  - FHWA Drilled Shaft Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016
  - Technical Manual for Design and Construction of Road Tunnels – Civil Elements, FHWA-NHI-09-010
10.3 General Requirements

The Geotechnical Designer shall be a licensed Geotechnical Engineer in the State of California with a minimum of 15 years of design and practical field experience in geotechnical and seismic engineering related to aerial and underground structures. The Geotechnical Designer shall conduct work necessary to perform supplemental geotechnical investigation and complete the design for the California High-Speed Train Project (CHSTP). The Geotechnical Designer shall develop geotechnical designs and construction excavation support systems in accordance with the requirements set forth in this chapter. Elements of the work include, but are not limited to, the following:

- Review of existing geotechnical information, including but not limited to the Geotechnical Baseline Report for Bidding (GBR-B), the preliminary Geotechnical Data Report (GDR), and the preliminary Geotechnical Engineering Design Report (GEDR).
- Evaluate the requirements of the work and perform additional geotechnical explorations, laboratory testing, and geotechnical analyses to supplement the existing data in support of its final design and proposed means and method of construction.
- Perform additional field testing to measure in situ field shear wave velocity for each geological site and at river crossings, creeks, and locations where compressible soils and high groundwater are expected. Submit the results to the California High-Speed Rail Authority (Authority) for updating final ground motion analyses.
• Prepare final Geotechnical Data Report (GDR) and Geotechnical Engineering Design Report (GEDR), and Geotechnical Baseline Report for Construction (GBR-C) as stated herein.

• Perform professional engineering support of the final structural design and design of temporary support works.

• Perform construction inspection and provide construction support to the Contractor related to geotechnical related works.

The Geotechnical Designer shall prepare the Geotechnical Reports in accordance with the criteria set forth in this chapter. Geotechnical work shall be conducted under the direction of the Geotechnical Designer. Geotechnical reports, calculations, and drawings shall be initialed and stamped by the Geotechnical Designer. In addition, the Geotechnical Designer shall be responsible for the following:

• Overseeing geotechnical design and construction support of bridges, embankments, retaining walls, roadways, tunnels, underground stations, roadways, and other geotechnical related facilities.

• Determining if more stringent criteria are appropriate and/or required by applicable codes, manuals, or other references (in addition to those listed). Addressing such criteria as part of the final design.

• Approving construction under his/her design control.

10.4 Subsurface Investigation and Data Analysis

The Geotechnical Designer shall interpret the existing geotechnical data and perform subsurface investigations, field and laboratory testing, fault displacement mapping, and rock slope mapping as may be necessary to satisfy itself as to the nature of the following:

• Soil, rock, groundwater, and subsurface conditions

• The geological and hydro-geological formations and seismic hazards within and attributes of the project site

• Variations in the subsurface and groundwater conditions across the project site and adjacent areas that can potentially be impacted by construction or train operations (e.g., ground movements or estimate of high-speed train induced ground vibration).

In addition, the Geotechnical Designer shall undertake soil resistivity testing. The ability of soils to conduct electricity may have a significant impact on the corrosion of buried structures and the design of grounding systems. Accordingly, subsurface investigations shall include conducting appropriate investigations to obtain soil resistivity values. The following information and methodologies are required:
• Soil resistivity readings shall be obtained to determine the electric conduction potential of soils at each traction power facility (supply/paralleling/switching station) locations, which are spaced at approximately 5-mile intervals.

• Resistivity measurements shall be obtained in accordance with the Institute of Electrical and Electronics Engineers (IEEE) Standard 81-1983 – IEEE Guide for Measuring Earth Resistivity using the four-point method for determining soil resistivity. IEEE states that the four-point method is more accurate than the two-point method.

*Appendix 10.A – Guidelines for Geotechnical Investigations* provides guidance for the expected level, frequency, and reporting of geotechnical investigation envisioned as necessary to fully satisfy the requirements of the Project.

Interpretations and necessary investigations and testing shall consider the methods of construction, critical combinations of loading, and all other factors impacting evaluation.

A Geotechnical Investigation Plan (GIP) shall be prepared by the Geotechnical Designer to supplement and update existing subsurface information available for final design of the structures. The investigation shall follow *Appendix 10.A – Guidelines for Geotechnical Investigations*.

The plan shall include the criteria or rationale used in developing the plan and shall identify locations of explorations, together with their depths, sampling intervals, and a description of both the field and laboratory testing program utilized. This plan shall be submitted to the Authority prior to commencing geotechnical investigations.

The requirements for the field and laboratory investigations to be performed by the Geotechnical Designer shall be the following:

• Perform additional subsurface investigations to supplement existing geotechnical data for the design of elements along the proposed alignment.

• Supervision – Boring and in-situ testing and inspection, and laboratory classification and testing, shall be performed by California licensed geologists or geotechnical engineers under the direct supervision of a design professional licensed in California with a minimum of 10 years experience in the performance and supervision of geotechnical investigations.

• Location and Ground Surface Elevation – The Geotechnical Designer shall determine the coordinate location and ground surface elevation for each boring and field investigation site, and shall show the coordinates, and station and offset, and the elevation for each individual boring log or investigation record. Coordinates and station and offset shall be referenced to the Project survey control. Elevations shall be referenced to the CHSTP datum and horizontal control system.

• Laboratories shall be Caltrans certified and equipment used for field testing shall have documentation of calibration within the last year.
• Information obtained using a pocket penetrometer or field torvane shall not be relied upon as the primary means for development of geotechnical parameters.

• Soil samples and rock cores shall be kept and maintained in a readily accessible storage facility within 100 miles of the project site during construction. No disposal of the soil samples and rock cores shall be made until it is instructed by the Authority after completion of the project. These samples shall be available for viewing by the Authority or its designees within two business days of a request. Untested samples shall not be disposed of or released to a third party at any time without the written authorization of the Authority.

• For rock slopes, tunnels through rock within 15 feet above the tunnel crown and 10 feet below the tunnel invert, and rock excavations at the portals and substructures, oriented cores with down hole camera logging shall be performed to obtain structural geological parameters such as orientations (dip/strike), roughness, infilling, spacing, etc. of structural discontinuities (bedding, joints, fault zones, shear zones, breccias, etc.)

• Borehole Site Cleanup – Backfilling of borings, test pits, Cone Penetration Tests (CPTs), rotosonic holes, wells, and probe holes shall be performed in accordance with the provisions of applicable local, state, or federal laws and regulations, and permit requirements. Restoration of pavement shall be performed in accordance with street use permit requirements.

• Test holes shall be backfilled in a manner that ensures against subsequent settlement or heave of the backfill. Upon completion of field investigations, surplus materials, temporary structures, and debris resulting from the drilling work performed on land and in water shall be removed and disposed of from the site.

• Final boring and rock core logs shall be prepared using gINT Geotechnical and Geoenvironmental software.

• No geological/hydro-geological data and seismic hazard evaluation results shall be released to a third party without the approval of the Authority.

10.5 Geotechnical Reports

Geotechnical reports including the GDR, GEDR, and GBR-C shall be prepared, signed, and stamped by the Geotechnical Designer.

10.5.1 Geotechnical Data Report (GDR)

Geotechnical investigation of the subsurface conditions, including laboratory and field testing, shall be performed to describe the geologic features of the project area. A summary of geotechnical data and findings, including a summary of existing information and that of the preliminary design level of subsurface investigation, results of the final field subsurface investigations including mappings, if any; and laboratory testing data, shall be prepared as the
GDR. The GDR shall contain factual information that has been gathered in the preliminary
design of subsurface investigations and the final subsurface investigations. The GDR shall
contain the following information:

- Project description
- Description of desk study results gathered from existing available data
- Description and discussion of the site exploration program
- Locations and results of subsurface investigations (borings, CPTs, Geophysical Testing,
etc.) including photo documentation of core hole core samples and investigation sites.
- A detailed description of geological and subsurface conditions (including a description of
  site stratigraphy, geologic hazards, and groundwater conditions)
- Rock parameters including orientation and nature of jointing, bedding, etc.
- Description of surface water (springs, streams, etc.) and groundwater conditions
- Seismic setting including location of nearby faults
- Boring and rock core logs with soil descriptions and field test results
- Groundwater level measurements from monitoring wells and piezometers
- Ground movement measurements from inclinometers
- Description and results of field/in situ testing and rock mapping
- Description and results of laboratory tests
- Material properties
- Chloride content, acidity (pH value) and sulfate content of the surface water, groundwater,
  and soils
- Statistical analysis for test results per geotechnical layer
- Results of field and laboratory testing
- Logs of borings, CPTs, trenches, and other site investigations
- Standards for laboratory and field testing

10.5.2 Geotechnical Engineering Design Report (GEDR)

The findings and evaluations of subsurface data along with geotechnical and foundation
analyses and design recommendations shall be documented in the form of a GEDR which
serves as the basis for final geotechnical design. The GEDR shall include, but is not limited to,
the following:

- Project description including surface conditions and current use
- Regional and site geology
• Regional and site seismicity
• A summary of subsurface explorations, including field and laboratory testing, and
  locations (map with coordinates) of borings, wells, and other in-situ testing sites
• Detailed description of geological and subsurface conditions (including a description of site
  stratigraphy) along with geological profile and cross-sections
• Seismic design criteria including design earthquake, magnitude, and peak ground and
  bedrock acceleration
• Evaluation of seismic and geologic hazards including, but are not limited to,
  liquefaction, lateral spreading, pre-historic landsliding and ground subsidence due to long-
  term pumping of groundwater or withdrawal of petroleum and gas, if any
• Subsurface material properties
• Data and complete discussions of geotechnical analyses, designs, and studies.
• Recommended design parameters for soil and rock types
• Conclusions and recommendations for foundation types for structures (with appropriate
  design parameters), soil and rock cut slopes, fill embankments, retaining walls,
  requirements for backfill materials
• Potential groundwater impact and dewatering requirements
• Instrumentation and monitoring requirements during and after construction
• Potential settlement, horizontal deflection problems and mitigation measures
• Potential soil and rock slope and retaining wall stability problems and analysis results
  along with mitigation measures
• Anticipated ground behavior and categorization of ground during excavation, filling and
  foundation, and retaining structure construction; particular attention shall be paid to
  identifying and mitigating impacts due to excavating near the groundwater table.
• Blasting and excavation methods as related to the design of cut slopes, including a
  discussion of blast design parameters that are related to the geotechnical conditions
• Consideration for, discussion of, and rationale for protection of existing structures, water
  bodies, and environmentally or historically sensitive areas
• Discussion on induced vibration and noise from the selected construction equipment and
  procedures and the effects on adjacent structures and landowners
• Evaluation of in-situ stress conditions (if applicable)
• Evaluation of load bearing capacity of the encountered soil/rock types
• Stability analyses in agreement with applicable codes and standards
• Evaluation, if excavated material can be used as fill/backfill material
• Geotechnical recommendations including earthwork/sitework; ground stabilization for foundation support; stabilization of unstable soil and rock slopes; and foundation options for aerial structures, underground structures, retaining walls, hydraulic structures, and other structures

• Construction considerations given to issues related to construction staging, shoring needs, potential installation difficulties, temporary slopes, earthwork constructability issues, dewatering, etc.

• Long-term and construction monitoring needs

10.5.3 Geotechnical Baseline Report for Construction (GBR-C)

A Geotechnical Baseline Report for Construction (GBR-C) shall be developed, upon completion of subsurface investigations, to verify design assumptions and finalize the design. As part of the final design and construction planning process, the Geotechnical Designer shall interpret the various baselines expressed in the GBR-B, consider those baselines in the development of the design and construction approaches, and fill in any missing information in the GBR-B accordingly. An electronic form of the GBR-B shall be used to record modifications or clarifications in the track-change mode of a computerized word processing software program. In its completed form, the GBR-C will serve as the physical baselines established by the Authority and the Contractor as well as the behavioral baselines described by the Contractor consistent with its design approach, equipment, means and methods.

The GBR-C shall include, but is not limited to, the elements listed in the “Geotechnical Baseline Reports for Construction – Suggested Guidelines” prepared by ASCE (Essex, 2007). The GBR-C shall be limited to interpretive discussion and baseline statements, and shall make reference to information obtained in the Geotechnical Data Report (GDR), Geotechnical Baseline Report for Bidding (GBR-B), drawings, and specifications.

10.6 Aerial Structure / Bridge Foundations and Stations / Miscellaneous Structures

Foundation design shall be based on project-specific information developed for the location(s) and foundation type planned. It shall be carried out in accordance with the AASHTO Specifications or other Standards or Codes referred to in Section 10.2 of this chapter provided that these are comparable and equivalent to or complement AASHTO Specifications, and as modified below. For structures subject to the jurisdiction of local authorities, soil parameters, such as design bearing and frictional values for foundations, shall not exceed the limits given by the local building codes, except for deviations as provided for in the codes.
10.6.1 Geotechnical Data

Type and depth of foundations shall be determined from available geotechnical data and additional geotechnical investigations at the locations of the foundations. Use of presumptive values shall not be allowed for final design.

Foundations in rivers and creeks shall take into consideration flood levels and maximum scour depth as determined by the Drainage chapter.

10.6.2 Load Modifiers, Load Factors, Load Combinations, and Resistance Factors

The design shall be in accordance with the concepts and general methodology of AASHTO LRFD BDS with Caltrans Amendments. See the Structures chapter for load factors and load combinations. Load resistance factors walls and shafts shall be in accordance with AASHTO LRFD BDS with Caltrans Amendments.

10.6.3 Allowable Foundation Settlements and Displacements

Requirements for tolerable foundation settlements and displacements presented herein shall supersede criteria indicated in AASHTO LRFD BDS with Caltrans Amendments. For deep foundations, allowable settlements or displacements are measured at the top of the foundation (i.e., 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 Operating Basis Earthquake (OBE) load combination (such as those resulting from post-liquefaction downdrag, seismic compaction, etc.) shall not exceed the settlement limits denoted in Table 10-1. For approach embankments, the Service 1 settlement limits and OBE load combination are applicable to settlements that occur after the placement of track.
Table 10-1: Maximum Settlement Limits \(^{(4)}\) for Service 1 and OBE Load Cases

<table>
<thead>
<tr>
<th>Settlement Criteria</th>
<th>Non-Ballasted Track</th>
<th>Ballasted Track</th>
</tr>
</thead>
<tbody>
<tr>
<td>Differential Settlement Between Adjacent Supports (^{(1)})</td>
<td>(\leq \frac{L}{1500}) ((L) = smaller span in inches), but no greater than 3/4 inch</td>
<td>N/A (^{(3)})</td>
</tr>
<tr>
<td>Differential Settlement Between Abutment and Approach Embankment (^{(2)}) (^{(5)})</td>
<td>(\leq 3/8) inch over 62 feet</td>
<td>(\leq 3/4) inch over 62 feet</td>
</tr>
<tr>
<td>Differential Settlement Between Abutment and Tunnel Portal (^{(5)})</td>
<td>(\leq 3/8) inch over 62 feet</td>
<td>N/A (^{(3)})</td>
</tr>
<tr>
<td>Uniform Settlement at Piers and Abutments</td>
<td>(\leq 3/4) inch</td>
<td>N/A (^{(3)})</td>
</tr>
</tbody>
</table>

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. Embankment shall be instrumented and monitored for a period of at least 12 months following completion of the structure. The Geotechnical Designer shall demonstrate future compliance with the residual settlements by extrapolation from the monitored data.
3. Not applicable based on the assumption that ballasted track will not be used for bridges, aerial structures, or tunnels.
4. The 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.).
5. Differential settlement shall be measured along the track (surface profile uniformity) in the vertical plane of each rail at the mid-point of a 62-foot long chord.

Refer to the Structures chapter’s section on Track-Structure Interaction for additional performance requirements for allowable deformations for the track.

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 and that foundation element are capacity protected in accordance with the Seismic chapter.

10.6.4 Aerial Trackway Foundations

Aerial trackway structure and bridge foundations shall be either shallow or deep foundations.

10.6.4.1 Shallow Foundations

Shallow foundations shall be spread footings, combined footings, or mat foundations. They shall be used where there is competent bearing layer near the surface, no highly compressible layers below, and calculated settlements are within acceptable limits outlined in this chapter.

Geotechnical design of abutment and bent/pier shallow foundations shall be carried out in accordance with AASHTO LRFD BDS with Caltrans Amendments, Articles 10 and 11, and as supplemented in this chapter. Unless otherwise specified, see the Structures chapter for LRFD load factors and load combinations.
Geotechnical design of retaining wall shallow foundations shall be carried out in accordance with Section 10.7 of this chapter, AASHTO LRFD BDS with Caltrans Amendments Articles 10 and 11, and as supplemented in this chapter. See the Structures chapter for LRFD load factors and load combinations and this chapter for additional three service load conditions.

A. Bearing of Soil/Rock

The nominal bearing resistance for shallow foundations shall be determined based on existing available geotechnical data and the geotechnical subsurface conditions of the foundation soil or rock. For all types of shallow soil foundations, the factored uniform bearing stress at the strength limit state, based on the effective footing dimension method in accordance with AASHTO LRFD BDS with Caltrans Amendments Articles 10.6.1.3 and 11.6.3.2, shall not be greater than the factored nominal bearing resistance. For all types of shallow rock foundations, the factored bearing stress at the strength limit state, based on the linearly distributed pressure method in accordance with AASHTO LRFD BDS with Caltrans Amendments, Article 11.6.3.2, shall not be greater than the factored nominal bearing resistance.

For abutment shallow soil/rock foundations, the bearing stress at the Service 1 limit state, based on the linearly distributed pressure method, shall not be greater than the site specific nominal bearing resistance according to AASHTO LRFD BDS with Caltrans Amendments.

B. Stability

The entire soil bearing area under the shallow foundations shall remain in compression at all times under the normal loading condition \(^{(1)}\).

Under exceptional loads \(^{(2)}\), the soil pressure distribution at the base shall have a compression width of at least \(2/3\) of the foundation width.

Under ultimate loads \(^{(3)}\), the soil pressure distribution shall have a compression width of at least \(1/2\) of the foundation width.

Notes:

\(^{(1)}\) Normal Loads \(= DC + DW + L + CF + E + WA + LF_2 + 0.6TU\)
\(^{(2)}\) Exceptional Loads \(= DC + DW + L_1 + CF_1 + LF_1 + E + WA + WS + WL_1\)
\(^{(3)}\) Ultimate Loads \(= DC + DW + E + WA \) (buoyancy only) + MCE

For loading definitions, refer to the Structures chapter.

- DC = Dead load of structural components and permanent attachments
- DW = Dead load of non-structural components and non-permanent attachments
- CF = Centrifugal force (multiple trains)
- CF_1 = Centrifugal force (single train)
- E = Earth pressures, including EV, EH, and ES
- L = Multiple trains of LLRR or LLV, whichever governs
- L_1 = Single train of LLRR or LLV, whichever governs
- LF_1 = Braking forces (apply braking to one train) for LLV loading
- LF_2 = Acceleration and braking forces (apply braking to one train, and acceleration to the other train) for LLV loading
- MCE = Maximum Considered Earthquake (refer to the Seismic chapter)
C. Allowable Foundation Settlements and Displacements

Total settlements and differential settlements of shallow foundations under the service limit state shall not exceed those specified in Table 10-1. See the Structures chapter for service limit state load combinations.

D. Benching

Where footings are to be constructed on inclined surfaces with slopes greater than 1 Vertical: 10 Horizontal (1V:10H), the surface shall be benched (Section 10.8.4).

E. Bottom of Footings

The depth of footings shall be determined based on the characteristics of the foundation materials and in consideration of the possibility of undermining. Footings not exposed to the action of stream or river current shall be founded on firm foundation below the frost level or a firm foundation that is made frost resistant by over excavation of frost-susceptible material below the frost line and replaced with material that is not frost susceptible.

In cases where spread footings are used in streams and rivers, the following additional design requirements shall be considered:

- **Footings on Soils** – The bottom of footings on soils shall be set at least 10 feet below the river bottom unless otherwise stated in this chapter. The potential shift of the stream or river channel shall be considered when determining the channel bottom. The top of footings on soil shall be below the total scour depth determined for the 100-year flood, and the bottom of footings on soil shall be below the estimated depth of the check flood (500-year) scour elevation.

- **Footings on Rock** – The bottom of footings shall be at least 3 feet below the surface of scour-resistant rock (i.e., rock not subject to scour attack) with the top of the footings at least below the rock surface.

- **Footings on Erodible Rock** – The foundation design of footings on erodible rock shall be based on:
  - Assess weathered rock or other potentially erodible rock formations for scour.
  - An analysis of intact rock cores, including rock quality designations and local geology, hydraulic data, and anticipated structure life.
10.6.4.2 Deep Foundations

Deep foundations shall be bored or driven piles and Cast-in-Drilled-Hole (CIDH) piles (also known as drilled shafts). These shall be used when shallow foundations cannot be used to carry the applied loads safely and economically and meet the required settlement criteria. Alternative deep foundation systems such as micropiles, rammed aggregate piers, and proprietary systems shall not be allowed for support of aerial structures and bridges.

The top of deep foundations, including top of drilled shafts or pile caps where multiple shafts or piles are employed, shall be a minimum of 3 feet below the lowest adjacent finished grade.

A. Ultimate Pile Load Capacities

The ultimate pile axial capacity shall be determined based on appropriate values of skin friction plus end bearing developed from the results of site-specific geotechnical investigations, and shall be verified by test piles and load testing as described herein.

The adequacy of the bearing capacity of the drilled shafts and bored or driven piles shall be verified regarding (1) the factual soil parameters at the respective locations and depth of the foundations, and (2) the groundwater table. Refer to Section 10.6.4.3 on Test Piles and Load Tests for verification of assumptions for deep foundation design.

Pile foundations shall be designed in such a way that plastic hinges do not occur in the piles.

B. Settlements and Horizontal Displacements

Total settlements of deep foundations shall not exceed those specified in Table 10-2. These values shall be verified. Piles/drilled shafts and connection to pile caps shall be checked for the estimated deflection from the lateral load. Tolerable lateral deflection shall be no greater than 1/2 inch.

For deep foundations, soils exhibiting potential liquefaction and lateral spreading in an earthquake, ground improvement may be considered to improve the foundation stability. Where ground improvement measures are prohibitively costly and impractical, consideration shall be given to designing a combined system composed of improved ground and strengthening of the foundation.

C. Lateral Load Capacity

Piles and drilled shafts shall be designed to adequately resist lateral loads transferred to them from the structure without exceeding the deformation which creates a stress outside the allowable stress range of the structure or overstressing the foundation elements. The lateral load resistance of the individual and group of piles and drilled shafts shall be analyzed. The analysis shall consider nonlinear soil pressure-displacement relationships, soil-structure interaction, group action, groundwater, and static and dynamic load conditions. The performance of the piles and drilled shafts shall include determination of vertical and horizontal movements, rotation, axial loads, shear, and bending moment for the foundation elements.
The lateral load capacity of piles and drilled shafts shall be verified by means of pile load tests in the field as described herein.

D. Wave Equation Analyses
The constructability of a pile design and the development of pile driving criteria shall be performed using a Wave Equation Analysis for Piles (WEAP) computer program in accordance with AASHTO Standard Specifications for Highway Bridges. Analysis shall be conducted for hammers and pile types proposed for use and for each bridge foundation. Wave equation analysis shall not be used as the sole basis for determining pile capacity or pile acceptance.

E. Pile Group Effects
Generally for piles or drilled shafts constructed in groups, the spacing of pile centers shall not be less than 2.5 pile diameters (or pile size). Piles or drilled shafts in any one group shall be of the same diameter. Pile group effects shall be considered with regard to the bearing capacity, settlement, and lateral resistance.

Multiple rows of piles/drilled shafts will have less resistance than the sum of the single individual piles/drilled shafts because of pile-soil-pile interactions that take place in the pile group (also called shadowing effect). The shadowing effect results in the lateral capacity of the pile group being less than the sum of the lateral capacities of the individual piles comprising the group. Consequently, lateral loaded pile groups will have group efficiencies less than unity, depending on the pile spacing.

When the P-Y method of analysis is used to evaluate a lateral loaded pile group, reduce the values of P by a multiplier ($P_m$) as shown in Table 10-2.

Table 10-2: Pile Load Modifiers, $P_m$, for Multiple Row Shading

<table>
<thead>
<tr>
<th>Pile Center-to-Center Spacing (in direction of loading)</th>
<th>Pile Load Modifiers, $P_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Row 1</td>
</tr>
<tr>
<td>3D</td>
<td>0.75</td>
</tr>
<tr>
<td>5D</td>
<td>1.0</td>
</tr>
<tr>
<td>7D</td>
<td>1.0</td>
</tr>
</tbody>
</table>

F. Down Drag (Negative Skin Friction) Effects
The design of piles and drilled shafts shall take into consideration the effect of negative skin friction as induced by dewatering, liquefaction, construction of embankments or from pile installation methods. When down drag (negative skin friction) is considered, it shall be treated as an addition to the nominal loads.
The nominal pile resistance available to support the downdrag and nominal loads shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to downdrag (i.e., neutral plane\(^1\)). If measures are proposed for reducing the effect of negative skin friction by means of a slip coating (e.g., bitumen, geotextile coating, etc.), then consideration shall be given to the long term value of residual negative skin friction that may develop. Instrumented pile load tests and dynamic tests shall be undertaken to verify design assumptions and to estimate the available nominal resistance to withstand the downdrag plus the nominal loads.

G. Uplift

Friction piles may be designed to resist uplift in non-liquefiable soils in accordance with recommendations in the GEDR. Resistance factors are per AASHTO LRFD BDS with Caltrans Amendments.

Should any loading or combination of loadings produce uplift on any pile, the pile to pile cap or pile to invert slab connection or footing shall be designed to resist uplift forces and other tension zones caused by the uplift condition.

H. Scour

For design of deep foundations to support bridges and aerial structures, geotechnical analyses shall be performed assuming that the soil above the estimated scour line based on the 100-year flood has been removed and is not available for bearing or lateral support.

10.6.4.3 Test Piles and Load Tests

A. Indicator Piles/Test Piles, Method Test Shafts and Load Test Shafts

An adequate number of indicator piles (1), test piles (2) and method test shafts (3)/load test shafts (4) shall be specified. These shall include advanced test piles/shafts tested to ultimate load to verify design assumptions. The locations and length of the indicator/test piles and method shafts/load test shafts shall be shown on the plans. Indicator piles/test piles and method test shaft/load test shafts shall be located to cover conditions of pile type, sizes, pile/shaft capacity, and soil conditions which will be encountered. Test piles that pass the load test in an undamaged condition may be utilized as production piles in the work. However, method test shafts/load test shafts shall be considered sacrificial and shall not be used as production drilled shafts.

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\(^1\) Neutral plane is the location where the downward acting forces are equal to the upward acting forces and where there is no movement between the pile and the soil. At this location, the pile and the soil settle equally.
Notes:

1. Indicator Pile – An individual pile that is tested and observed to determine its behavior during driving.

2. Test Pile – An individual pile that is tested and observed under static axial compressive or tension load, under lateral load, and under dynamic load tests.

3. Method Test Shaft – A drilled shaft that is excavated to verify construction methods so that drilling and support of excavation can be evaluated for each site.

4. Load Test Shaft – A method test shaft with reinforcing placed, any casing or other excavation support system withdrawn, and full concrete placement, followed by gamma ray testing or crosshole sonic testing to verify concrete placement. Method test shaft is then observed under static axial compressive or tension load, under lateral load and under dynamic load tests.

As a minimum, indicator piles, test piles, and method test shafts shall be located on the following basis:

- One indicator pile and one test pile per 300 driven piles.
- One indicator pile and one test pile at each pile location separated by a distance of 500 feet or less from other indicator pile/test pile locations.
- One method test shaft per 50 drilled shafts.
- One method test shaft and one load test shaft at each shaft location separated by a distance of 500 feet or less from other method test shaft/load test locations.
- Test programs as indicated elsewhere in this chapter.

B. Load Tests

An appropriate number of deep foundations (driven piles and drilled shafts) shall be tested to ultimate or design loads to verify design assumptions. The location and length of the test deep foundations shall be such that they will cover conditions of types and capacity of the deep foundations as well as soil conditions which will be encountered. These load tests shall be conducted on test piles, method test shafts, and production piles/drilled shafts.

Load tests, if conducted, may be used to increase the resistance factor that is specified in AASHTO Standard Specifications for Highway Bridges. Loading test methods shall be in accordance with the technical specifications applicable to the Contract. In general, static load test capacity of piles shall be tested for compressive, lateral, and tensile loads in accordance with the following ASTM International Standards:

- ASTM D3966, Test Method for Deep Foundations Under Lateral Load

Alternative load test methods such as Standard Test Method for High Strain Dynamic Testing of Piles (ASTM D4945), Osterberg Cells, Statnamic Load Test (ASTM D7383), etc., may be used.
However, these substitutive test methods shall be verified by a conventional loading test of similar piles or drilled shafts.

**Drilled Shafts** – An adequate number of load tests shall be specified in the following. These shall include Load Test Shafts tested to ultimate load (load tests) to verify design assumptions. The locations and length of the test shafts shall be shown on the plans. Method test shafts shall be located to cover the shaft type, shaft capacity, and soil conditions which will be encountered.

The Geotechnical Designer shall perform a test shaft program consisting of method test shafts (1) to confirm adequacy of drilling methodology and equipment, and (2) load tests to verify compressive, lateral, and tensile load capacities per site as described in the following. A location is considered to be a different site if any of the following are true:

- The location is more than 2,000 feet from the method test shaft location where the subsurface conditions are similar.
- The geologic character of the predominantly bearing formation and side resistance is different.
- At each of the main piers of a long span (more than 300 feet) bridge where there are a large number of drilled shafts (greater than 8) in each pier foundation, particularly where the geology may differ on either side of a natural drainage feature.
- The average calibrated resistance (unit load transfer in side resistance or end bearing) in the zone providing the majority of the axial resistance varies from the test location by a factor of two or more.
- Sequence, type of construction, and type of shafts are changed.

Once approval has been given to constructing production drilled shafts, no change shall be permitted in the methods and equipment used to construct the satisfactory method test shaft without production of additional method test shafts and written approval of the Geotechnical Designer.

**Driven Piles** – An indicator pile program consisting of indicator piles, test piles and load tests shall be conducted at each bridge or aerial structure site where driven piles are to be installed. Perform dynamic monitoring using a Pile Driving Analyzer (PDA) on indicator piles conforming to the requirements of ASTM D4945. Perform static load tests to verify compressive, lateral, and tensile loads of individual piles. Indicator piles may be installed as production piles provided PDA test results demonstrate the required capacity is achieved.

To utilize the increase in capacity due to setup in cohesive soils, PDA measurements shall be recorded using Case Pile Wave Analysis Program (CAPWAP) during restrike of piles to determine setup. PDA results and revised criteria for the restrike shall be applied to only the piles in that group. Piles shall be re-struck no sooner than 48 hours after installation.

The Engineer inspecting the PDA testing shall have at least five years of experience in the monitoring of the driving of piles with PDA and in performing analyses with CAPWAP in
similar type of soil conditions. The Engineer performing PDA related analyses shall be a
geotechnical engineer licensed in the State of California.

The Geotechnical Designer shall be on-site during PDA testing of initial and restrike pile
installation. The Geotechnical Designer shall evaluate data to establish driving criteria for
production pile installation.

C. Integrity Testing

Integrity testing consisting of gamma-gamma or Crosshole Sonic Logging (CSL) or both shall be
performed on drilled shafts larger than 24 inches in diameter. Gamma-gamma and CSL tests
shall be reviewed and approved by the Geotechnical Designer as well as any remedial measures
or repairs that may be needed. In addition, integrity testing is required on driven piles. ASTM
D5882, Test Method for Low Strain Impact Integrity Testing shall be performed on piles and
drilled shafts 24 inches in diameter or more.

10.6.5 Other Design Considerations

10.6.5.1 Foundation Cover

Soil cover over top of foundations of piers or abutments shall have a minimum thickness of 3
feet. In rivers and creeks, the soil cover shall be such that it will be at least 3 feet below the
maximum estimated scour depth or at least 10 feet below the river bottom for shallow
foundations supported by soils, whichever is the deeper.

In urban areas and adjacent to highways, railroads, transit systems, the elevation at the top of
the foundations shall be in compliance with the requirements set forth by the local authorities to
allow for adequate depth for utilities and surface drains.

10.6.5.2 Foundation Thickness

Spread footings for piers and abutments shall have a minimum thickness of 3 feet.

The thickness of a pile cap shall be the larger of 3.5 feet or the depth required to develop the full
compressive, tensile, flexural, and shear capacity of the pile reinforcement.

10.6.5.3 Piles/Drilled Shafts

Minimum penetration depth of piles and drilled shafts into competent bearing soils shall be 10
feet. In the event that the piles and drilled shafts are embedded in rock, the minimum
penetration depth shall vary between 3 feet to 10 feet, varying linearly with the unconfined
compressive strength of the rock as follows:
Table 10-3: Minimum Penetration Depth in Rock

<table>
<thead>
<tr>
<th>Rock Unconfined Compressive Strength (psi)</th>
<th>Embedded Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 75</td>
<td>10</td>
</tr>
<tr>
<td>≥ 750</td>
<td>3</td>
</tr>
</tbody>
</table>

End bearing soil below the pile/drilled shaft tip shall be competent materials, having a thickness of at least 3 x D (where D is either the pile diameter or drilled shaft width) and shall demonstrate the adequacy of resisting punching shear failure and settlements.

10.6.6 Station and Miscellaneous At-Grade Structures

10.6.6.1 Shallow Foundations

Per Caltrans BDS (ASHTO LRFD BDS, Article 10.2 Definitions): “Shallow Foundation – A foundation that derives its support by transferring load directly to the soil or rock at shallow depth.”

Design of shallow foundations, e.g., spread and strip footings in addition to mat foundations, shall be based on project-specific information developed for the location(s) and foundation type(s) planned. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical Data Report; use of presumptive values shall not be allowed. Designs of shallow foundations supporting rail structures or attached appurtenances shall be as required in Caltrans BDS (AASHTO LRFD BDS, Article 10.6) and in accordance with FHWA-SA-02-054 (Geotechnical Engineering Circular No. 6 Shallow Foundations). Shallow foundations for support of structures under the purview of the Building Code, buildings not directly supported off the aerial trackway structure, shall be designed in conformance with the requirements of the Building Code – Footings and Foundations. Shallow foundations shall have a minimum ground cover of 2 feet as measured from the top of footing to finished grade.

Shallow foundations shall be designed to limit total settlement to no more than 3/4-inch. Differential settlements shall be no more than 1/2-inch between adjacent supports or L/1000 (where L is the distance between two supports in inches), whichever is less.

10.6.6.2 Deep Foundations

Where shallow foundations cannot be used due to presence of soft, compressible soils, deep foundations such as piling can be considered. Design of deep foundations shall be in accordance with AASHTO LRFD Design Specifications.

10.6.6.3 Miscellaneous At-Grade Structure Foundations

Design of foundations for miscellaneous structures shall be in accordance with the requirements above for shallow foundations, excepting that presumptive values may be used. These include, but are not limited to miscellaneous structures such as light standards, retaining walls less than...
5 feet in height and are not supporting any structures, and other lightly loaded and uninhabited structures.

Cantilever signs and signals shall be supported on drilled shaft foundations. Design for cantilever signals and signs shall be performed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO 2001). Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed as stated in this chapter.

Foundation design for noise barriers shall be conducted in accordance with Caltrans Memo To Designer 22-1, Soundwall Design Criteria. Seismic issues related to foundation design such as downdrag and lateral spread due to liquefaction shall be addressed as stated in this chapter.

10.7 Retaining Walls and Trenches

The criteria set forth in this section govern the static load design of retaining walls and trenches (retaining walls with a continuous base slab between them). The design shall conform to the applicable requirements set forth in Article 11 of AASHTO LRFD BDS with Caltrans Amendments, FHWA’s Earth Retaining Structures Reference Manual, and the sections specified in this chapter. For permanent surcharge loads, refer to Section 10.11.5. For design loads of the HST, refer to the Structures chapter.

Retaining walls can be classified as either a “fill wall” or a “cut wall.” Acceptable fill walls include standard cantilever walls, mechanically stabilized earth walls, reinforced soil slope embankment, and modular gravity walls ( gabions and crib walls). Acceptable cut walls include soil nail walls, cantilever soldier-pile walls, slurry walls, secant pile/tangent pile walls, and ground anchored walls (other than nail walls).

10.7.1 Design

Design of retaining walls shall consider the following conditions of external instability where applicable:

- Sliding in connection with a horizontal displacement of the structure
- Overturning or excessive settlement
- Failure of the structure base (allowable soil pressure exceeded)
- Overall stability behind and under the structure (soil shear failure)
- Liquefaction potential of the supporting ground

For geotechnical design, refer to Article 11 of AASHTO LRFD BDS with Caltrans Amendments.

Design of mechanically stabilized earth structures and reinforced soil slope embankments shall be in accordance with the LRFD version of FHWA’s manual FHWA-NHI-10-024/25 “Design and
Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes”, Volumes 1 and 2. Embedded metallic strip reinforcing elements shall not be used since they are potentially susceptible to stray current corrosion that may cause significant loss of reinforcement over the life of infrastructure supporting track.

10.7.2 Unacceptable Walls

Unacceptable retaining walls include mortar rubble gravity walls, timber or metal bin walls, “rockery” walls, and other wall types not specifically listed in Section 10.7.

10.7.3 Stability of Retaining Walls

Retaining walls, abutment walls, and basement walls shall be evaluated and designed for internal, external (sliding and overturning), and global stability. In addition to the static loads, the retaining walls shall be designed to resist the dynamic (seismic) earth pressure (ED), hydrodynamic force (WAD) and (water pressure), if submerged, under the seismic loading conditions.

Except for abutment walls where they shall be designed using the Service-1 Limit State loads, geotechnical designs for retaining walls and basement walls shall be performed in accordance with AASHTO LRFD BDS with Caltrans Amendments. Earth pressures used in design of the walls and abutments shall be selected consistent with the requirement that the wall/abutment movements shall not exceed tolerable displacement and settlement set forth in this chapter.

Retaining walls that are not restrained from rotation at the top, which are located where Peak Ground Acceleration (PGA) values (i.e., from MCE ground motion) are less than or equal to 0.30g, shall be designed for only active pressures, surcharge loads, and inertial forces of the wall itself; additional dynamic (seismic) earth pressures shall not be considered. For walls containing cohesionless material as backfill, seismic active pressures shall be estimated using the Mononobe-Okabe (M-O) method (Mononobe and Matsuo, 1929) only under the following conditions:

- The material behind the wall can be reasonably approximated as a uniform, cohesionless soil within a zone defined by a 3H:1V wedge from the heel of the wall
- The backfill is not saturated and in loose enough condition such that it can liquefy during shaking
- The combination of horizontal acceleration coefficient ($K_h$) and vertical acceleration coefficient ($K_v$) and backslope angle, $i$, do not exceed the friction angle of the soil behind the wall as specified by:

$$
\phi \geq i + \arctan \left( K_h / (1 - K_v) \right)
$$

For wall geometry or site conditions for which the M-O Method is not suitable, the Generalized Limit Equilibrium (GLE) Method shall be used to determine seismic active earth pressures.
Horizontal seismic coefficient ($K_h$) shall be estimated using the Bray et al. (2010) method assuming a wall movement of 1 inch. 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:

- $\Delta K_{AE}$ = Incremental seismic pressure coefficient
- $K_{AE}$ = Total seismic pressure coefficient
- $K_A$ = Active pressure coefficient

The incremental dynamic (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 ($c$-$\phi$) 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 (for MCE ground motion) are less than or equal to 0.35g, walls shall be designed for only at-rest pressures, surcharge loads, 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.

The no-seismic-load options mentioned above shall be limited to internal and external seismic stability design of the retaining wall and to the condition that no liquefaction and severe strength loss in sensitive clays occur that can cause wall instability. If the wall is part of a bigger slope, overall seismic stability of the wall and slope combination shall still be evaluated.

### 10.7.4 Base Pressure

Soil bearing pressures shall be determined based on the applicable backfilled bearing materials. In order to minimize differential settlement and excessive outward tilting of walls, walls shall be proportioned so that the base pressure on soil under the footing is as nearly uniform (within 10 percent) as practical under the long term loading.

---

10.7.5 Hydrostatic Pressure (Buoyancy)
Refer to the Structures chapter for design criteria for water loads (hydrostatic pressure) (buoyancy).

The use of tiedowns, tension piles, or other elements specifically designed to resist uplift forces shall be permitted. The use of augercast piles shall not be allowed as an anti-buoyancy hold down structure. The use of tension elements to resist buoyancy shall not compromise waterproofing and shall be designed to prohibit corrosion and be designed with the same design life as the rest of the structure.

10.7.6 Settlements and Horizontal Deformations
Retaining walls directly supporting HSTs shall be designed not to exceed settlement of 3/4-inch and horizontal deformation of 1/2-inch. To avoid long-term deflections in the track, track structures shall not be constructed until the majority of estimated retaining wall settlement has already occurred. Use of ground improvement methods may be required to expedite settlement, mitigate lateral deformations, as well as potential seismic hazards such as liquefaction and seismic instability.

10.7.7 Drainage
Adequate drainage behind retaining walls shall be included in the design and implemented during construction. An exception to this requirement is for trenches and underground structure walls where the top of trackway subgrade is below the groundwater table/flood level. These walls shall be designed to resist full hydrostatic pressures, both laterally and vertically (buoyancy).

10.7.8 Backfill
Backfill behind retaining walls shall be cohesionless and drained. 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 full hydrostatic pressure in addition to earth pressures.

The compaction of the backfill shall meet a minimum of 95 percent degree of compaction using the Modified Proctor Test in accordance with ASTM (D-1557) or AASHTO T180. Care shall be taken not to damage the walls during compaction using light compactor or hand-held tamper.

10.8 Embankments for HST Trackway
For roadway and site embankments, refer to the Civil chapter. For design loads, refer to the Structures chapter.
Embankments shall be engineered. Design of embankments shall focus on settlement of support ground and stability of embankment. Care shall be taken to avoid possible landslides within the embankment areas.

At each embankment, the following shall be evaluated:

- Slope stability
- Liquefaction potential of support ground
- Bearing capacity and plastic flow evaluation
- Construction of embankment shall not lead to reactivation of existing landslides or the formation of new ones
- Creep considerations
- Drainage considerations to avoid eroding the slope, scouring the toe, and clogging the water course
- Impact of Rayleigh-wave vibration induced by the high-speed train on the track-ground system composed of ballast/subballast or non-ballasted slab, embankment fill, and supporting subgrades
- Assessment of prepared subgrade, subballast/bearing base layers, and trackway; in particular (1) high dynamic effects on low embankments (less than 6.5 feet) / foundation soils and (2) critical speed issues of embankments over soft, compressible foundations with undrained shear strength less than 600 psf.

### 10.8.1 Slope Inclination

**Fill** – 2H:1V or flatter. Steeper slopes may be designed using geosynthetics (geogrids or geofabric) reinforcement to enhance the slope inclination.

**Cut** – 2H:1V or steeper if justified by slope stability analyses. Refer to Section 10.9.

### 10.8.2 Safety Factors

The stability of an embankment slope shall be determined using the Service-1 limit state. For the Service-1 static slope stability, the resistance factor is simply the inverse of the factor of safety (FOS). Table 10-4 shows the minimum required factors of safety for embankment slopes.
Table 10-4: Minimum Required Factors of Safety for Embankment Slopes

<table>
<thead>
<tr>
<th>Loading Conditions</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal (Permanent) (1)</td>
<td>≥1.50</td>
</tr>
<tr>
<td>Temporary (open less than 1 year)</td>
<td>≥1.30</td>
</tr>
<tr>
<td>Earthquake (OBE and MCE)</td>
<td>≥1.0 (2)</td>
</tr>
</tbody>
</table>

Notes:
1. The factor of safety shall be in accordance with the requirements set forth by the local agencies.
2. The stability of embankment slopes under earthquakes shall be analyzed by using the pseudo-static analysis, under the following conditions:
   - $K_v = 0$

Where:
- $K_h$ = Horizontal seismic coefficient
- $K_v$ = Vertical seismic coefficient

10.8.3 Settlements

Once the embankments are designed to meet safe allowable bearing pressures and satisfy stability, settlements of the embankments during and after construction shall be evaluated. Settlement assessment shall be performed in the following critical areas:

- Approaches to bridge abutments
- Soft and organic layers beneath the embankment
- Subsiding areas

The vertical deformation ‘settlement’ of embankments (which also affects overlying trackbed 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 10-1. Conventional settlement analyses shall consider ‘immediate’, ‘consolidation’, and ‘secondary’ components of settlement against the requirements of CHSTP. For analysis of embankments, calculation procedures in the following references shall be used to assess soil settlement:

- Soil Slope and Embankment Design Manual, chapters 4 and 8, FHWA-NHI-05-123, 2005
Geotechnical evaluations for embankments and their foundations shall include the settlement contribution from surcharge/track load, high-speed train induced vibration, and additional loading and/or ground deformation due to earthquakes.

Once the embankments are designed based on safe bearing pressures and satisfying stability, the ‘residual’ settlement estimates and differential displacements between locations along the length of the embankments shall be evaluated to assess potential serviceability problems for the trackbed.

Residual settlement occurring after construction of the “permanent way” tracks shall be limited along general track segments as shown in Table 10-5.
Table 10-5: Maximum Residual Settlement Limits

<table>
<thead>
<tr>
<th>Residual Settlement (1)</th>
<th>Non-Ballasted Track</th>
<th>Ballasted Track</th>
</tr>
</thead>
<tbody>
<tr>
<td>Differential Settlement (2)</td>
<td>≤ 3/8 inch over 62 feet</td>
<td>≤ 3/4 inch over 62 feet</td>
</tr>
<tr>
<td>Uniform Settlement</td>
<td>≤ 5/8 inch</td>
<td>≤ 1-1/8 inch</td>
</tr>
</tbody>
</table>

Notes:

(1) Embankment shall be instrumented and monitored for a period of at least 12 months following completion of the structure. The Geotechnical Designer shall demonstrate future compliance with the residual settlements in Table 10-5 by extrapolation from the monitored data.

(2) Differential settlement shall be measured along the track (surface profile uniformity) in the vertical plane of each rail at the mid-point of a 62-foot long chord.

Embankments shall be designed and constructed so as not to exceed the maximum residual settlement set forth in Table 10-5. "Residual" settlements occur after the monitoring period and completion of the embankments and shall be limited along the general track segments.

If the predicted differential displacements are excessive and exceed track profile tolerances, then embankment designs shall be modified and ground improvement designed if needed to act as a foundation system. Where predicted settlement movements and their duration are excessive, consideration shall be undertaken to change the design from an embankment to a viaduct or other structure.

Settlement of earth structures is time-dependent and will vary by segment, the time duration “waiting (leaving) period” shall be evaluated and established following initial fill embankment placement before releveling of subgrade. After this evaluation and establishment of the waiting period, subsequent construction of the overlying trackbed “permanent way” is allowed to take place. An illustration of various settlement parts related to time is shown in Figure 10-2. To meet CHSTP design and performance requirements, a settlement survey program shall be developed 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 10-2: Different Settlement Parts by Time

Notes:

Reference: Figure no. 22 of UIC-719R (2008)
Commentary: 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 0.004 to 0.008 in (or 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.

10.8.3.1 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/measures shall be considered for geotechnical design. The selection of mitigation methods/measures shall follow the process described in detail in FHWA’s Ground Improvement Reference Manuals Volumes I and II; FHWA-NHI-06-019,020 dated 2006.

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

For embankments higher than 30 feet (measured from existing ground surface to top of slope), design shall include mid-slope benches to mitigate surface erosion and to facilitate future access for maintenance reasons. Slope benches shall be at least 6 feet wide with a 4 to 6 percent slope towards the low end of the slope with a lined drainage channel. For embankments higher than 30 feet, slope benches shall be designed at every 25 to 30 feet in height connected to the surrounding ground surface for access.

At the top surface of the embankment, transverse cross-slope for drainage shall be 24:1 towards the outer edges of the embankment foreslopes (see Figure 10-3).

When an embankment is constructed next to an existing slope, the existing slope shall be benched in steps (typically 5 to 10 feet wide and no greater than 4 feet deep) to assure the fill embankment is keyed into the existing slope (see Figure 10-3). Drainage measures shall be placed on these benches to facilitate and convey groundwater to discharge outlets.

**Figure 10-3: Typical Section Earthwork Cut/Fill**

![Diagram of typical section earthwork cut/fill](image)

10.8.5 Particular Requirements

10.8.5.1 Foundation Support

If the height of the embankment is less than or equal to 6.5 feet, as measured from the flat top of the subballast at the side edge of the embankment to the existing ground surface, and the foundation subgrades are loose and soft, compressible soils, they shall be removed and replaced with backfill and compacted to ensure settlement criteria.

For embankment heights greater than 6.5 feet over loose, soft, and compressible subgrade soils, the global stability and settlement induced by the embankment load shall be determined and ground improvement implemented, if necessary, to improve stability and achieve settlement criteria.
10.8.5.2 Embankments in Wet Conditions

In case an embankment is located in a wet area where the groundwater table is permanently or periodically at ground level, the embankment shall be constructed on a layer of drainage material as depicted on Figure 10-4. This material shall not swell or deteriorate when immersed in water. It shall be well graded with no more than 10 percent passing the No. 200 sieve. The grading of the drainage material shall be designed according to Sherard’s filter criteria (Sherard et al., 1984). A layer of geosynthetic cloth shall be placed below the drainage material to provide a better support to the drainage material.

Figure 10-4: Earthwork Embankment in Wet Conditions

10.8.5.3 Embankment in Flood Plains

Where an embankment is located in a floodplain, the highest flood water level shall be determined from the 100-year flood. The embankment shall be, in addition to the drainage layer arrangement in Section 10.8.5.2, designed to protect the slopes within the highest water level with a layer of drainage layer and protection riprap as depicted on Figure 10-5. The drainage material shall be designed to comply with Sherard’s filter criteria. This layer shall extend up to the highest flood water level plus 2 feet and be underlain by a layer of geosynthetic membrane.

Figure 10-5: Drainage Layer under Embankments in Floodplain / High Water
10.8.5.4 Embankments over Active Fault Locations

Where possible, embankments shall be located outside of active fault lines and founded on competent grounds. If this cannot be avoided, the embankments shall be designed with layers of geosynthetic cloth with drain rock or concrete mat at the bottom and containment earthworks wide enough to accommodate the potential rupture offsets and subsequent re-alignment.

10.8.5.5 Embankments on Potentially Liquefiable Soils/Compressible Soils

Where embankments are underlain by soft compressible soils or loose saturated soils that indicate high potential of liquefaction under OBE and MCE earthquakes, mitigation shall be required. The following soil improvement methods should be considered to mitigate soil liquefaction and increase the consistency of the foundation subgrade:

- 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)
  - Rammed aggregate piers (Replacement or Displacement type)
- Mixing/Solidification
  - Permeation Grouting
  - Deep soil mixing
  - Jet grouting
- Surcharge with wick drains (for soft compressible soils)
- Lime columns for soft compressive clays
- Drainage (only used in combination with other ground improvement methods listed above)
  - Passive or active dewatering systems
  - Pipe Pile Stone Columns (drainage in combination of vibration and displacement)

Ground improvement design shall be in accordance with FHWA Ground Improvements Reference Manual Volumes 1 and 2, FHWA-NHI-06-019 and FHWA-NHI-06-020.
10.8.5.6 Rayleigh-Wave Induced Vibration by High-Speed Trains on Embankments and Structures

High-speed trains will produce compressive (P) waves, shear (S) waves, and Rayleigh (R) waves, of which Raleigh waves (moving parallel to the ground surface) are the primary source of vibrational energy. This vibrational energy could have a substantial destructive and fatiguing effect on the HST track-ground system composed of rails, embankments, and foundation subgrades. In addition, ground vibrations generated by high-speed trains are of great concern because of the possible damage they can cause to buildings or other structures near the track and the annoyance to the public living in the vicinity of the track. Particularly in soft-soil regions, where the wave speed is comparable to the speed of the trains, a strong increase of the vibration level can occur. The impact of the high-speed train-induced ground vibration on the track-ground system shall be evaluated and mitigated accordingly to avoid long term degradation of the HST track-ground system and all adjacent structures. For design purposes, the following shall be required:

- Vibration induced stability of the embankment shall be verified,
- Tracks shall be supported by well compacted ballast/subballast or slab track,
- Embankments supporting the track shall be adequately compacted, and
- Subgrade underlying the embankment shall be competent and firm, and if soft compressible soils are present, they shall be stabilized with ground treatment to increase its overall stiffness with undrained shear strength and \( E_v^2 \) exceeding 15 psi and 6,500 psi, respectively. \( E_v^2 \) is the subgrade stiffness determined from the 2\(^{nd} \) loading of a plate load test according to ASTM_D1883-67.

In addition, an instrumentation program shall be devised to investigate the effect of the stabilization measures before and after the measures are conducted.

10.8.5.7 Embankment Prepared Subgrade

Material and thickness of the prepared subgrade for each track type (ballasted and non-ballasted) shall be as noted in the “Thickness of Prepared Subgrade” table of Figure 10-6. For non-ballasted track where the embankment height is low (less than 6.5 feet as measured from the flat top of the subballast at the side edge of the embankment to the existing ground surface), excavation below existing grade is not required to achieve a 6.5-foot thick prepared subgrade if it can be demonstrated that \( E_v^2 \) of the existing subgrade is \( \geq 11,500 \) psi after the foundation soil is proof-rolled. In this case, the thickness of the prepared subgrade can be reduced to the available thickness, but it shall not be less than 14-inch thick and \( E_v^2 \geq 11,500 \) psi shall be provided.

10.8.5.8 Transition of Embankments to Structures

Embankments adjacent to the aerial structure abutments, tunnel portals, cut-and-cover structures, and cut sections with an abrupt topographic change shall be designed to minimize the differential settlement and to provide a smooth transition in the structural stiffness between
different infrastructures. Provide a smooth transition by stiffening the subballast bearing base layer and the approach fill with soil cement as depicted on Figures 10-6, 10-7, and 10-8.
**Figure 10-6: Transition from Concrete Slab to Embankment**

<table>
<thead>
<tr>
<th>Thickness of Prepared Subgrade Material</th>
<th>Gradation for Prepared Subgrade Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well Graded Soils Containing 5% to 15% Fines</td>
<td>Grain Size (mm)</td>
</tr>
<tr>
<td>Ballasted Track</td>
<td>D(50)</td>
</tr>
<tr>
<td>Non-Ballasted Track</td>
<td>D(100)</td>
</tr>
<tr>
<td></td>
<td>D(200)</td>
</tr>
<tr>
<td></td>
<td>D(500)</td>
</tr>
<tr>
<td></td>
<td>D(1000)</td>
</tr>
</tbody>
</table>

**NOTES:**

1. Transitions shall be designed to minimize the differential settlement and to provide a smooth transition in the structural stiffness between different infrastructures.

2. Embankments shall be designed specifically taking into account the construction sequence and the geometrical, geological and geotechnical conditions of the site.

3. The minimum subballast (supporting ballasted track) thickness shall be 9".

**LEGEND:**

- MPD: Modified Proctor Density (AASHTO T180)
- EV2: Deformation Modulus of Second Loading

![Diagram of Transition from Concrete Slab to Embankment](image-url)

Subballast (Type I Gravel) 100% MPD
Prepared Subgrade 95% MPD
Cement Treated Gravel (Type 3 Gravel) 3% Cement, 95% MPD (See Figure 10-8)
Invert (Concrete Slab) Tunnel or Trench
Figure 10-7: Transition from Cut to Embankment

<table>
<thead>
<tr>
<th>THICKNESS OF PREPARED SUBGRADE</th>
<th>GRADEATION FOR PREPARED SUBGRADE MATERIAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>MATERIAL</td>
<td>GRAIN SIZE (mm) PERCENTAGE PASSING</td>
</tr>
<tr>
<td>WELL GRADED SOILS CONTAINING 5% TO 15% FINES</td>
<td>P(D)</td>
</tr>
<tr>
<td>BALLASTED TRACK</td>
<td>P(DMAX)</td>
</tr>
<tr>
<td>14&quot;</td>
<td>P(D)</td>
</tr>
<tr>
<td>NON-BALLASTED TRACK</td>
<td>P(D/2)</td>
</tr>
<tr>
<td>6'-6&quot;</td>
<td>P(D/5)</td>
</tr>
<tr>
<td></td>
<td>P(D/10)</td>
</tr>
<tr>
<td></td>
<td>P(D/20)</td>
</tr>
<tr>
<td></td>
<td>P(D/50)</td>
</tr>
<tr>
<td></td>
<td>P(D/100)</td>
</tr>
<tr>
<td></td>
<td>P(D/200)</td>
</tr>
<tr>
<td></td>
<td>P(D/500)</td>
</tr>
<tr>
<td></td>
<td>P(D/1000)</td>
</tr>
</tbody>
</table>

D = NOMINAL GRAIN SIZE
Dmax = 1.25D IF D ≥ 50 mm;
Dmax = 1.58D IF D < 50 mm;

NOTES:
1. TRANSITIONS SHALL BE DESIGNED TO MINIMIZE THE DIFFERENTIAL SETTLEMENT AND TO PROVIDE A SMOOTH TRANSITION IN THE STRUCTURAL STIFFNESS BETWEEN DIFFERENT INFRASTRUCTURES.
2. EMBANKMENTS SHALL BE DESIGNED SPECIFICALLY TAKING INTO ACCOUNT THE CONSTRUCTION SEQUENCE AND THE GEOMETRICAL, GEOLOGICAL AND GEOTECHNICAL CONDITIONS OF THE SITE.
3. THE MINIMUM SUBBALLAST THICKNESS SHALL BE 9".

LEGEND:
MPD MODIFIED PROCTOR DENSITY (AASHTO T180)
E_v2 DEFORMATION MODULUS OF SECOND LOADING
Figure 10-8: Transition from Aerial Structure to Embankment

**Notes:**
1. Transitions shall be designed to minimize the differential settlement and to provide a smooth transition in the structural stiffness between different infrastructures.
2. Embankments shall be designed specifically taking into account the construction sequence and the geometrical, geological and geotechnical conditions of the site.
3. The minimum subballast thickness shall be 9".
4. The minimum thickness shall be equal to the combined thickness of the subballast and the prepared subgrade and no less than 1'-11".
5. Length L shall be 4H or 65'-whichever is greater.
6. Prepared subgrade thickness is shown on Figures 10-6 and 10-7.

**Legend:**
MPD = Modified Proctor Density (AASHTO T180)

Ev2 = Deformation modulus of second loading
10.8.5.9 Embankments in Cut Sections

Embankment design in cut sections shall include selection of appropriate earthworks for a given setting based on design constraints and potential conflicts, geotechnical subsurface investigations, and surface and groundwater issues. Figure 10-3 shows a typical embankment in a cut section.

10.8.5.10 Drainage (Surface and Subsurface)

Control of surface and ground water is essential to avoid surface erosion and potential slope instability. Provision shall be made in the design for an adequate system of surface and subsurface drainage and surface protection that incorporates sufficient capacity for the following:

- Design rainfall run-off to prevent long term erosion
- Build-up of groundwater that could result in slope instability

Notwithstanding the requirements of available relevant standards, consideration shall be given to the long term performance of the drainage and erosion control system for each embankment of fill under local conditions.

Where horizontal drains are to be used, a protective measure shall be devised to protect the drains from freeze/thaw. A long term maintenance program shall be developed by the Geotechnical Designer in order to safeguard the long term functionality of the horizontal drains.

Geotechnical design 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.

10.8.6 Soil Materials Used for Embankments

For design purposes, evaluation of soil suitability for re-use within the body of embankments shall be based on the following guidelines:
Table 10-6: Soil Material Suitability for Engineered Fill in Embankments

<table>
<thead>
<tr>
<th>Acceptable (1)</th>
<th>Unacceptable (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1-a</td>
<td>A-4 (CBR &lt;10)</td>
</tr>
<tr>
<td>A-1-b</td>
<td>A-2-7</td>
</tr>
<tr>
<td>A-2-4</td>
<td>A-5</td>
</tr>
<tr>
<td>A-2-5</td>
<td>A-6</td>
</tr>
<tr>
<td>A-2-6</td>
<td>A-7-5</td>
</tr>
<tr>
<td>A-3</td>
<td>A-7-6</td>
</tr>
<tr>
<td>A-4 (CBR &gt;10)</td>
<td>*</td>
</tr>
</tbody>
</table>

Notes:

1. Source: Per ASTM D3282 / AASHTO Subgrade Soil Group System
2. Refer to the Trackwork chapter and Standard Specifications for Trackbed layers of subballast and prepared subgrade.
3. Rockfill is not acceptable for track embankment material.
4. In addition to the AASHTO criteria, the maximum soil particle size is limited to 3 inches.
5. 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 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 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** – soil types susceptible to frost, such as silt or clay, shall not be used for embankments in regions where cold conditions (below freezing temperatures) can occur in order to reduce the potential to cause unacceptable disturbances to track geometry upon freeze/thaw cycles.
- **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.
- **Slake Durability of Rock** – based on the slake durability behavior in wetting and drying cycles.

### 10.9 Cut Slopes

Cut slopes include soil, Intermediate Geomaterials (IGM), and rock slopes, and shall be designed per the following sections. Sloped excavations during construction shall be designed

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4 Corrosion potential is the potential of a corroding surface in an electrolyte relative to a reference electrode measured under open-circuit conditions.
and constructed in compliance with local, state, and federal regulations, including but not limited to Occupational Safety and Health Administration (OSHA) and Cal-OSHA requirements.

### 10.9.1 Design of Cut Slopes

Design of cut slopes shall consider the following:

- Impact of slope instability to the HST facility operations and integrity (short term and long term)
- Slopes within existing pre-historic landslide areas
- Locations where liquefaction-related lateral spreading conditions are present
- Rock slopes with adversely oriented and kinematically unstable structural discontinuities such as joints, bedding planes, shear planes, gouges, and faulted zones

At each cut slope location, the following shall be evaluated:

- Locations where evidence of prior landsliding is present
- Slopes composed of quick, sensitive, and expansive clays

At each cut slope, the following shall be evaluated:

- Slope stability (static and seismic)
- Construction of the cut slope shall not lead to reactivation of existing landslides or the formation of new ones

For design of rock slopes, refer to Appendix 10.C – Guidelines for Rock Slope Engineering.

### 10.9.1.1 Design Requirements

**Slope Inclination (Typical)**

- **Soil cut** – 3H:1V slope or steeper if justified by slope stability analyses
- **IGM cut** – 2H:1V slope or steeper if substantiated by slope stability analyses
- **Rock cut** – 1H:1V slope or steeper if justified by slope stability analyses

### 10.9.1.2 Safety Factors

For design criteria for stability of cut slopes, please refer to Section 10.8.2.

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5 The slope inclination design guidelines stated herein do not apply to the cut slopes in pre-historic landslide areas, prior landslide locations, and potential liquefaction related lateral spreading conditions, slopes composed of sensitive, quick, and expansive clays.
10.9.2 Drainage (Surface and Subsurface)

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 slopes shall be performed similar to those stated in Section 10.8.5.10 and Section 10.10.2. Impermeable coverings such as shotcreting (with or without ground reinforcements), stone-pitching, etc., shall be considered to protect rock slopes from degradation and deterioration due to weathering.

Subsurface drainage systems such as cut-off drains, horizontal drains, french drains, etc., shall be designed to permanently lower groundwater table to enhance overall stability of the slopes.

10.9.3 Slope Stability Mitigation Methods for Cut Slopes

Where the minimum required factors of safety cannot be achieved or the alignment cannot be relocated away from unstable slopes, the Geotechnical Designer shall design measures to enhance slope stability. Slope stability mitigation measures for cut slopes include the following:

- Soil Cuts
  - Flattening the slopes (if permitted by right-of-way) with vegetation cover
  - Buttressing the toe of the slopes
  - Stabilizing the slope with ground reinforcements such as soil nails and soil anchors with or without shotcrete
  - Covering the slope face with stone pitching, concrete, or shotcreting
  - Debris flow diversion walls
  - Retaining walls such as soldier pile walls, secant pile and tangent piles, gabion walls, etc.
  - Drainage and subdrainage measures
  - Ground improvements such as deep soil cement mixing or jet grouting
  - A combination of any of the above

- Rock Cuts
  - Rock scaling and dentition
  - Rock fall ditches
  - Rock fall retention meshes
  - Rock fall detention fences
  - Rock dowels and anchors
Shotcreting

A combination of any of the above

10.10 Existing Slopes

The Geotechnical Designer shall evaluate existing slopes for potential instability. At a minimum, the Geotechnical Designer shall mitigate unstable slopes to ensure that they will not pose a detrimental impact to the alignment.

10.10.1 Protection of Existing Slopes

The Geotechnical Designer shall be responsible for maintaining the stability of existing slopes during the course of construction. Slope instability that occurs during construction shall be repaired by the Geotechnical Designer at its own expense.

10.10.2 Drainage (Surface and Subsurface)

Erosion control and drainage measures shall be evaluated, considered and designed for existing slopes. Erosion of slopes presents a significant maintenance issue and overall stability concern. Rock and soil strata that are susceptible to erosion and/or freeze/thaw shall be mapped and delineated for existing and new fills and cuts. Slope protection measures shall be evaluated on site-specific conditions, such as surface and subsurface conditions, cut geometry, and susceptibility of erosion or deterioration. Each cut and fill slope that requires erosion control and drainage measures shall be evaluated for the following:

A. Reduction of water flow across slope

Where slope revegetation cannot be sufficiently established, reduce the quantity of water flowing over the slope from upland areas by means of drainage or interceptor ditches across the top of the slope and down the ends of the slope. At the base of the slope, water shall be directed to a discharge point. Coordinate discharge point drainage with existing facilities.

Drainage or interceptor ditches shall be lined or unlined and capable of carrying water generated from upland areas based on the 100-year storm. Lining materials shall be cast-in-place concrete, pre-cast concrete, reinforced shotcrete, or asphalt. Rock check dams to slow flows shall be designed and installed based on flow calculations.

B. Slope Revegetation

Where the slope can be made to support vegetation, local plantings shall be used to establish root systems to stabilize the surface of the slope and prevent deterioration of the slope. Design and provide systems of degradable woven blankets to temporarily hold plantings in place and minimize erosion until vegetation has established a stable root system.
C. **Slope Armor**

Where slopes will not support vegetation, slope cover/protection or permanent facing shall be used to protect the slope. Such measures as mattress-shaped steel wire mesh containers, gabions, articulated concrete blocks, fabric formed concrete, shotcrete, geosynthetic cells filled with gravel, and rip-rap (crushed stone) placed on a graded filter shall be evaluated, designed and installed. Stone sizes shall be designed based on design water flows.

D. **Subsurface Water Control**

Design of subsurface water drainage features shall be evaluated as water control measures. Design shall consider the use of horizontal drains, blanket drains, trench drains and geocomposites for both cut and fill slopes. Design shall consider outlet design and address long-term performance and maintenance requirements for the drainage system.

E. **Springs and Water Seepage**

Any springs and water seepage identified in the field shall be contained by means of drainage systems. Design shall consider long-term performance and maintenance requirements for the drainage system.

### 10.11 Cut-and-Cover Underground Structures

The cut-and-cover underground structures include subways, cross-passages, sump pump structures, stations, building basements, vaults, ventilation structures, and other structures of similar nature.

Underground structures shall include waterproofing protection, drainage systems and/or dewatering pumps as needed to prohibit water buildup in the underground structures.

#### 10.11.1 Structural Systems

The structural system for cut-and-cover line structures shall be single and/or multi-cell reinforced concrete box structures, with walls and slabs acting one-way in the transverse direction to form a frame. Walls that provide temporary support of excavation shall not be used as part of the permanent structure. Expansion joints are required at locations of major change in structural sections such as from line structure to station. Construction joints shall have continuous reinforcing steel and non-metallic waterstops.

#### 10.11.2 Hydrostatic Pressure (Buoyancy)

Refer to the *Structures* chapter for water loads (hydrostatic pressure) (buoyancy) for design criteria for buoyancy.

Refer to Section 10.7 on Retaining Walls and Trenches for types of systems to be allowed to resist buoyancy.
10.11.3 Temporary Support of Underground Structures

Equivalent static loads and deformations may be used to design temporary support systems such as wales, struts, and braces recognizing the short duration of these systems. These loads shall be provided by the Geotechnical Designer and shall be shown on the shoring design calculations and drawings.

In locations where adjacent buildings and their foundations create an interaction configuration in conjunction with temporary ground support structures that would significantly influence the seismic response of the adjacent buildings themselves, the combined group of temporary ground support and building structural configurations shall also be analyzed as a single permanent structure.

Refer to the Structures chapter for more structural detail.

10.11.4 Temporary Lateral Loading Conditions

Soil Pressures – The Geotechnical Designer shall have the responsibility of determining earth pressures of temporary earth support; however, the earth pressures shall not be less than those calculated assuming the active case. Pressures shall consider the impacts due to compaction. The temporary design of the wall shall not allow for overstressing of the wall.

Water Pressures – The temporary earth support system shall be designed to a construction term water level that is not lower than the existing groundwater level with consideration given to the potential of elevated groundwater conditions due to ground water re-injection activities.

Surcharge Loads – The earth support system shall include surcharge loads including, but not limited to traffic, construction material and equipment, and building loads.

Earthquake Loads – Refer to the Structures chapter.

Temporary Excavation Support Systems – Excavation and backfill sequence and strut installation and strut removal sequence shall be in accordance with the Designer of Record’s design requirements.

Temporary earth support may remain in place or be removed following completion of the structure. Temporary earth support walls left in place shall be cut off at a depth not higher than 5 feet below grade or top of structure whichever is higher. Removal of temporary earth support walls shall be permitted. The settlement analysis shall indicate that removal will not cause settlement and lateral movement of adjacent structures, sidewalks, streets, and utilities. Tiebacks used to retain temporary support walls shall be de-tensioned prior to abandonment.

10.11.5 Permanent Lateral Loading Conditions

Soil Pressures – Permanent underground structures shall be designed for earth pressures as given in Section 10.7.3 on Stability of Retaining Walls. The at-rest pressures shall be used in the
design of cut-and-cover underground structures. In addition, hydrostatic pressures and seismic loadings shall also be included in the design of the underground structures.

Surcharge Loads – Loads from adjacent building foundations shall be used in the design of cut-and-cover underground structures unless these existing buildings are founded on piles or permanently underpinned at a depth below the zone of influence of the cut-and-cover structures. Horizontal distribution of loads from foundations of existing buildings shall be determined in accordance with AASHTO LRFD BDS with Caltrans Amendments, Article 3.11.6.

10.11.6 Deformation Limits for Support of Excavation Systems

Excavation support systems shall be designed to limit wall deformations that would otherwise lead to ground settlements, resulting in damage to the support systems or any superimposed structures and adjacent structures/utilities. Ground settlement and lateral deformation shall be limited to less than 3/4 inch and 1/2 inch, respectively. The Geotechnical Designer shall analyze the support of the excavation system taking into account the ground conditions, wall stiffness, requirements for wall bracing systems, global stability, and sequence of construction including timing of support installations to determine the deflection and settlements for open cut excavation methods.

10.11.7 Dewatering

Concrete placement of a cut-and-cover structure below a groundwater table shall be either by tremie concrete or placed in the dry. When placement in the dry method is chosen, a dewatering/groundwater control system shall be designed to permit placement of all structural elements in the dry. The bearing subgrade shall be kept dry and stable with no flowing, standing and/or piping of the groundwater permitted. Water levels within the limit of excavation shall be maintained a minimum of 5 feet below subgrade. Tremie seals, grouting, and other similar methods shall be permitted as part of dewatering/groundwater control methods.

Design and installation of a groundwater recharge system to protect nearby structures and utilities shall be performed to mitigate excessive ground settlements induced by dewatering. In addition, the dewatering system shall be designed so that the construction dewatering recharge system will not adversely impact existing fresh water aquifers.

10.12 Seismic Design

Seismic design requirements are also covered in the Seismic chapter and the Structures chapter. The geotechnically-focused elements of the seismic design criteria are presented in this section. Structures shall be designed to resist seismically induced forces and deformations due to ground motions resulting from an earthquake, and to meet the performance criteria specified in this document. Foundations shall be designed to address inertial loads from superstructures, liquefaction, lateral spread, and other seismic effects such that it will not experience damage.
under the design earthquakes. Earth retaining structures shall be evaluated and designed for
seismic stability internally and externally. Cut slopes in soil and rock, fill slopes, and
embankments having impact on the operations of HSTs shall be evaluated for instability due to
design seismic events and associated geologic hazards.

### 10.12.1 Design Earthquakes

For seismic design guidelines and performance requirements, refer to the *Seismic* chapter.

### 10.12.2 Liquefaction of Foundation Soils

Liquefaction may cause partial or total loss of shear strength of soils, thereby causing
foundation instability, flow slides, lateral spreading and ground settlements. The Geotechnical
Designer shall evaluate the possibility of ground failures caused by liquefaction, the potential
impacts to foundations and structures, and mitigation measures to satisfy performance
requirements.

Liquefaction-triggering evaluations shall be performed for sites that meet 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 of soils for liquefaction with a measured Standard Penetration Test
(SPT) resistance, corrected for overburden pressure and hammer energy (N1)60-cs, less
than 33 blows/ft., or a cone tip resistance qc1N-cs (defined as the normalized cone tip
resistance with clean sand equivalence) of less than 185 ton per square feet, or a geologic
unit is present at the site that has been observed to liquefy in past earthquakes.

Liquefaction-induced movement/settlement shall be estimated and compared with the
allowable deformation values required in this chapter. The Geotechnical Designer shall
develop mitigation measures accordingly to meet the allowable deformation values set forth in
this chapter.

Guidelines for evaluation of soil liquefaction triggering potential are presented in *Appendix 10.B
– Guidelines for Geotechnical Earthquake Engineering*.

Where potential for liquefaction exists under OBE and MCE earthquakes (as confirmed by
liquefaction studies by the Geotechnical Designer) and its impact on foundations/structures is
not acceptable, the following remedial measures shall be considered:

- Liquefiable soils shall be removed; or

- Soil improvement techniques shall be used (see Section 10.8.5.5); or
Deep foundations such as piles or drilled shafts shall be used, and shall be designed to resist and accommodate the liquefaction-induced ground movements and force demands, taking into account the reduced soil properties as a result of liquefaction.

10.12.2.1 Compositional Criteria for Liquefaction Susceptibility for Soils

A. Sandy Soils
Sandy soils with few amounts of fines that meet the above-mentioned two criteria shall require liquefaction triggering evaluations.

B. Silty and Clayey Soils
Whether silty and clayey soils meet the criteria for liquefaction susceptibility shall be evaluated 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). The Modified Chinese Criteria for clayey soils in the Youd et al. (2001) method shall not be used.

For fine-grained soils that do not meet the above criteria for liquefaction, the effect of cyclic softening resulting from seismic shaking shall be evaluated and its impact on foundations/structures shall be analyzed and considered in the design.

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.

C. 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 be considered susceptible to liquefaction and its liquefaction potential shall be evaluated as such.

10.12.3 Underground Structures
Seismic design of underground structures shall be based primarily on the ground deformation approach specified herein. During earthquakes, underground structures move together with the surrounding soil/rock mass. The structures shall therefore be designed to accommodate the deformations imposed by the ground, taking into consideration the soil-structure interaction effect.

Seismic effects on underground structures take the form of deformations that in general cannot be changed significantly by stiffening the structures. The structures shall instead be designed and detailed to withstand the imposed deformations without losing the capacity to carry applied loads and to meet the performance goals of the structures. Shear capacity degradation and compressive strains shall be evaluated. If necessary, additional confinement reinforcement shall be added to increase ductility and shear capacity.
Underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial, and curvature deformations. The ovaling/racking deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation (Figure 10-9). The axial and curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis (Figure 10-10).

**Figure 10-9: Tunnel Transverse Ovaling and Racking Response to Vertically Propagating Shear Waves**

![Tunnel Transverse Ovaling and Racking Response to Vertically Propagating Shear Waves](image)

**Figure 10-10: Tunnel Longitudinal Axial and Curvature Response to Traveling Waves**

![Tunnel Longitudinal Axial and Curvature Response to Traveling Waves](image)
10.12.4 Effect of Ground Deformation

10.12.4.1 Transverse Ovaling Deformations

For bored circular tunnels, using either the precast concrete segmental lining or cast-in-place concrete lining, there are two general approaches to determining the effects of seismic ovaling deformation.

The first approach is based on closed form solution that accounts for soil-structure interaction effect. The closed form solution is based on the following assumptions: (1) the tunnel is of completely circular shape (without decks or walls inside) with uniform lining section, (2) surrounding soil is uniform, and (3) there is no interaction effect from adjacent tunnels or other structures.

The second approach is a numerical modeling approach that relies on mathematical models of the structures (including adjacent structures if relevant) to account for structural properties, varying soil stratigraphy and properties, loadings and deformations more rigorously. These structural models are generally run on computers with specialized software. If the actual soil-structure systems encountered in the field are more complex than the assumed conditions described above for the closed form solution approach which could lead to unreliable results, then the numerical modeling approach shall be adopted.


10.12.4.2 Transverse Racking Deformations

For box type underground structures such as cut-and-cover tunnels and stations, and mined station sections that behave in similar manner as a rectangular structure during earthquake shaking, seismic design of the transverse cross section of the structure shall consider two loading components:

- The racking deformations due to the vertically propagating shear waves, which are similar to the ovaling deformations of a circular tunnel lining (see Figure 10-9).
- Inertia forces due to vertical seismic motions.

There are two general approaches to determining the effects of seismic racking deformations:

The first approach is based on semi-closed form solution that has been calibrated with a series of numerical analyses for a number of soil-structure configurations. The semi-closed form solution is based on the following assumptions: (1) the tunnel is of rectangular shape, (2) surrounding soil is reasonably uniform, and (3) there is no interaction effect from adjacent tunnels or other structures.

The second approach is a numerical modeling approach that relies on mathematical models of the structures (including adjacent structures if relevant) to account for structural properties, varying soil stratigraphy and properties, loadings and deformations more rigorously. These
structural models are generally run on computers with specialized software. If the actual soil-
structure systems encountered in the field are more complex than the assumed conditions
described above for the semi-closed form solution approach leading to unreliable results, then
the numerical modeling approach shall be adopted.

Refer to FHWA-NHI-09-010 Report, “Technical Manual for Design and Construction of Road
Tunnels”, Chapter 13 for general guidelines on transverse racking analysis for box type
structures.

10.12.4.3 Longitudinal Axial/Curvature Deformations

The evaluation procedures for the longitudinal response (due to axial/curvature deformations)
of tunnel structures shall be based on the procedures outlined in Section 13.5.2 of the FHWA-
NHI-09-010 Report, “Technical Manual for Design and Construction of Road Tunnels”. The
Free-Field Deformation procedure in section 13.5.2.1 of the Road Tunnel Manual may be used to
determine the strains related to axial and longitudinal deformation of the tunnel under seismic
ground motions. Supplement the analysis with Numerical Modeling Approaches similar to
those in Section 13.5.2.3 of the Technical Manual where there are abrupt changes in structural
stiffness or geological properties.

For the Free-Field Deformation analysis, the combined axial and bending strains shall be
calculated from the P-Waves (primary waves), S-Waves (shear waves), and R-Waves (Rayleigh
waves) using the formulae given in Section 13.5.2.1 of the Technical Manual. The parameters
associated with each class of wave are to be developed and provided by the Geotechnical
Engineer/Seismologist.

Numerical modeling approach shall be used to investigate the effects of abrupt changes in
structural stiffness or geological properties. Structural stiffness change locations can include the
tunnel breakouts at the portals; where egress and ventilation shafts may join the tunnel; and
other local hard spots. Geological changes requiring numerical modeling include area of abrupt
change in soil stiffness along the alignment. These include the interfaces between liquefiable
and non-liquefiable soils and the interfaces between soft soil and rock.

The effect of spatial variations of ground motions on long structures resulting from the effects of
wave passage and local soil overburden shall be considered. The wave-passage effect results
from different arrivals of seismic waves at different parts of the structure. The wave-passage
effect can be accounted for by assuming a time lag of the ground-motion time histories between
any two locations along the tunnel alignment. This time lag can be estimated by dividing the
distance between the two locations by the horizontal wave travelling velocity (in the ground)
$V_H = 2 \text{ km/sec}$ along the tunnel alignment.

The effect of local soil overburden is specified in Section 10.12.4.4.
10.12.4.4 Site Response Analysis

Variations of local site conditions at different locations along the proposed tunnel alignment will have a major effect on the seismic response of the tunnel structures. The requirements and guidelines for evaluating the local site response effect on design ground motions are defined below.

Site response analyses shall be based on numerical modeling of the soil layering configuration, using site-specific soil properties along the tunnel alignment.

Several analysis methods are available for evaluating the effect of local soil conditions on ground response during earthquakes. The equivalent-linear one-dimensional total stress method shall be used. The non-linear one-dimensional total and effective stress method, the two-and three-dimensional equivalent-linear total stress methods, and the two- and three-dimensional non-linear total and effective stress methods shall also be used.

The one-dimensional site response analysis described above can be used for developing ground displacement profile for the evaluation the ovaling/racking effects on the seismic behavior of a tunnel’s transverse section.

To evaluate the tunnel’s seismic performance in the longitudinal direction, the effect of subsurface variability in soil conditions along the tunnel alignment must be taken into consideration. When the soil/rock strata are highly variable and not horizontally layered, response analysis shall be performed with two-dimensional or three-dimensional modeling techniques.

For any numerical programs to be used (e.g., by finite element or finite difference methods), the Geotechnical Designer shall, prior to final design of any structural elements, verify the accuracy of such programs by a written report and with calculations that explain the theory, the input values, and the results.

10.12.5 Soil-Structure Interaction for Bridges and Aerial Structures

For bridges and aerial structures, the following primary soil-structure interaction effects shall be considered:

- The influence of foundation stiffness on structural response.
- The inertial structural loads imparted to the foundation system – termed as the inertial effect.
- The ground displacement loads imparted to the foundation system (resulting from both free-field soil displacement and ground-failure conditions such as lateral spreading or permanent seismically-induced embankment/slope movements if applicable) – termed as the kinematic effect.

The soil-foundation-structure interaction problem can be solved using either a coupled or uncoupled analysis. The coupled analysis examines the behavior of the entire soil-foundation-
structure system simultaneously in a single, complex model, in which non-linear soil behavior is described by a continuum model and/or non-linear soil springs (e.g., p-y, t-z, and q-z). In the uncoupled analysis, the effect of foundation stiffness on structural response is examined by replacing the foundation in the structural model with a set of spring (or stiffness matrix).

At a minimum, the soil-foundation-structure interaction effects shall be considered using the uncoupled approach using the stiffness matrix approach. In the event that a more detailed representation of the complex interactions between the superstructure, foundation and the surrounding soil is required, a fully coupled analysis shall be conducted.

10.12.5.1 Pile/Drilled Shaft Design Subject to Ground Displacements

Ground displacement loading can be divided into two categories: (1) free-field ground displacement and (2) displacement due to unstable ground such as liquefaction induced lateral spread or unstable embankments/slopes. Ground displacements impose forces acting along the length of the piles and pile cap and therefore shall be considered in the design. For the free-field ground displacements, the resulting forces can be estimated by imposing the estimated free-field ground displacement profile on the pile through p-y springs. Proper selection of the non-linear p-y properties of the surrounding soil is crucial for the design. The displacement profile can be estimated from a site response analysis. In competent sites, the free-field ground displacements generally do not govern the pile design because the curvature of the ground displacement is small. This effect, however, has to be considered for piles in soft soils and for sudden changes in soil stiffness with depth. The effect is particularly significant for large diameter piles or drilled caissons in soft soils.

Similarly, seismic soil instability resulting from geotechnical seismic hazards can produce large soil movements adversely affecting the performance of deep foundations. The p-y procedure described above is also applicable for this case. The ground displacements resulting from unstable ground require detailed analysis using site-specific data and shall be provided by the Geotechnical Designer.

The overall evaluation procedure for pile design in liquefied soil deposits would essentially be the same as that described above. However, the choice of p-y characteristics must properly consider liquefaction effects of the soils.

Computer program LPILE has the ability to impose a soil displacement profile against the pile by adjusting the location of the base of the soil springs (p-y). For calculation of loads and deformation demands on bridge foundations and abutment resulting from liquefaction induced spreading ground, refer to Caltrans Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading (2011).

10.12.5.2 Effective Support Motions

Due to the complex interaction between soil, pile, and structures, the effective support motions (i.e., the near field ground motions) at the foundation/structure interface differ from those in the free field. For regular shallow footings and flexible pile-supported footings (relative to the
surrounding ground), using free-field motions as the support motions in the structure response analysis is reasonable. For very large and stiff foundations, such as large gravity caissons, very stiff battered pile groups, or large diameter drilled shaft foundations, the effective support motions at the foundation/structure interface may differ considerably from the free-field motions. When this situation occurs, a more refined analysis taking into account the presence of the foundation and the soil-pile/shaft kinematic interaction effect shall be performed to derive the effective support motions.

### 10.13 Formation Supporting Track Structures

Formation, defined as layers comprising subballast, prepared subgrade/subgrade, and earth fill, provides the base for track structure which is composed of rail track and ballast. The formation shall be designed to be safe against shear failure, and accumulated/plastic deformations under repetitive axle loads of the trains. The subballast and prepared subgrade/subgrade provide support to the track structure and bear additional stresses due to static and dynamic effects of moving wheel loads. The load is transmitted through the subballast, prepared subgrade/subgrade, and earth fill to foundation soils.

The ballast under the rail track serves as a stress disperser. Below the ballast is the subballast overlying the prepared subgrade/subgrade. This subballast layer (also referred to as the blanket layer in the UIC standards) shall be of adequate thickness to reduce the induced stresses to an acceptable level at the top of prepared subgrade/subgrade to avoid shear failure. The subballast shall have adequate strength under dynamic loads and vibrations, high resilient modulus, reasonable plastic strain accumulation characteristics under repeated wheel loads, etc. Therefore, the material shall be permeable enough to avoid any positive pore pressure build-up under repeated load. It shall consist of durable particles and should not be sensitive to moisture content. In addition, it shall resist break-down and abrasion from cyclic stresses produced by the train repetitive loading.

- **Subballast** – The subballast shall conform to the following design requirements:
  - It shall be coarse, granular, and well graded as per Standard Specifications.
  - Gap-graded material shall not be permitted.
  - It shall meet the minimum Resistance (R-value), Sand Equivalent and Durability Index requirements set forth in Standard Specifications.
• **Subgrade** – Below the subballast is the subgrade layer, which in its most complete form, shall consist of a sandy gravel layer with gradation sizes per Standard Specifications. The upper part of the subgrade shall be formed into a prepared subgrade layer, which normally has a cross slope. This layer is made of imported or treated material depending on the quality of the upper part of the embankment or the bottom of the cut. It shall have a gradation as specified in Figure 10-6. Its deformation modulus, $E_v^2$, from the 2nd loading in the plate load test shall not be less than 11,500 psi.

• **Earth Fill** – Underlying the subgrade is the fill (embankment fill/retaining structure backfill) on top of the existing foundation soils. This earth fill shall be designed against slope failure and settlement/deformation as provided earlier in this chapter.

### 10.13.1 Determination of the Thickness of the Trackbed Layers

Trackbed layers are composed of ballast and subballast which are placed on top of the prepared subgrade/subgrade. The dimensioning of trackbed layers shall take into account both the following:

- Desirable bearing capacity
- Problems of frost penetration

The total thickness (ballast layer plus sub-ballast layer) varies according to the following:

- Bearing capacity of the prepared subgrade
- Level of frost protection required
- Type of tie and the tie spacing
- Traffic characteristics (tonnage supported, axle-load and speed)

The thickness of the ballast varies depending on the train types, sleeper types, or whether non-ballasted tracks are used. The minimum thickness of subballast shall be 9 inches. For the prepared subgrade, a minimum thickness of 14 inches is required for ballasted tracks, whereas, a minimum thickness of 6 feet-6 inches of prepared subgrade is required for support of non-ballasted tracks unless otherwise stated in Section 10.8.5.7.

### 10.13.2 Design of Formation

Knowledge of cumulative plastic deformation for foundation soils under repeated loading is essential for the proper design of HST tracks. Excessive foundation soil plastic deformation will produce high maintenance costs and unwanted ride quality.

Design methods of formation, particularly for subballast thickness, are used in different railway systems. They are based on different properties of soil used in embankment construction which governs the behavior of the soil (viz. percentage of fines less than 75 microns) present in the soil, CBR value of the soil, undrained shear strength of the soil, etc. Methods such as the Association...
of American Railroads (AAR) method (Li and Selig, 1998) may be used for design of the formation.

10.13.2.1 Rail Deflections
Rail deflections as a result of dynamic amplification due to high-speed trains shall be considered. These deflections are a function of (1) axle load of the train, (2) thickness of the embankment fill, (3) elastic properties of the sub-soil/foundation subgrade and the damping in the system, (4) train speed, and (5) both upward and downward rail deflections during the train passages. At certain speed of the train, “resonance” phenomena may cause rail deflections that are far larger than the static values.

Rail deflections induced by high-speed trains as a result of the dynamic amplification shall not exceed 1/8-inch and 1/2-inch for non-ballasted and ballasted trackways, respectively. Deformation analyses shall be performed to verify the rail deflections are within the required limits. If such limit cannot be achieved, consideration shall be given to increasing the thickness/stiffness of the prepared subgrade, subballast/bearing base layer and/or stabilizing the foundation subgrade.

10.13.2.2 Existing Embankments/Retaining Structures over Soft Grounds
In addition to checking against shear/bearing failure, design of high-speed train track formation over existing embankments underlain by soft ground shall be performed to evaluate the structural integrity of the formation supporting the trackways. The velocity of a high-speed train may approach or exceed the characteristic wave velocity of the dynamic system comprising the underlying soft ground, the formation, and the moving load. As the train’s velocity reaches some “critical velocity”, large deformations may occur. These motions could be dangerous for the train and the integrity of the track structure, and potentially costly in terms of track maintenance and performance. It is therefore vital to design the embankments which provide a dynamic stiffness that will limit track deflections to acceptable levels (see Section 10.8.3.1).

Analytical methods are used to model train-induced dynamic motion. Of these methods, the Winkler model may be used as it is a very prevalent and simple numerical model. In the Winkler mode, the embankment/rail/foundation material structure is simplified as a beam on an elastic or visco-elastic foundation, represented by a series of discrete springs and dashpots. The solution of the model may be used to calculate the critical velocity \( V_{cr} \) (Kenny, 1954) which is equal to:

\[
V_{cr} = \sqrt[4]{\frac{4kEI}{\rho^2}}
\]
Where:

1. $k =$ Spring constant per unit length of the beam
2. $E =$ Modulus of elasticity of beam
3. $I =$ Moment of inertia of beam
4. $\rho =$ Mass per unit length of beam

For design, the critical velocity of the embankments/retaining structures shall exceed 1.7 that of the design speed of the train.

10.13.2.3 Drainage of Formation

Water contained in the formation layers cause detrimental conditions in the track. Therefore, it is necessary to contain and reduce water content in the formation layers by the following measures:

- Removal of vegetation growth on surface
- Cleaning ballast bed and establishing cross fall slope at top of formation, subballast, and prepared subgrade/subgrade layers
- Provision of longitudinal drains and drainage outfall facilities
- Arrangement of lateral side drainage facilities
Appendix 10.A: Guidelines for Geotechnical Investigations

10.A.1 Purpose

These guidelines represent a preferred, but not necessarily the only actions required for the development of additional geotechnical investigations. These guidelines convey a minimum standard of care in performing geotechnical investigations and are not intended as prescribed site investigation criteria or checklists.

10.A.2 Geotechnical Investigation Guidelines

Geotechnical investigations are to be performed by a Geotechnical Designer in collaboration with an engineering geologist, both of which are licensed in the State of California. The level of geotechnical investigation performed shall consider the engineering needs and amount of information necessary to achieve performance criteria, complete the design, and mitigate construction risks. Guidelines for advancing the geotechnical investigations are described in the following sections.

The Geotechnical Designer/engineering geologist shall be required to present the investigation results in a Geotechnical Data Report (GDR) document that contains the factual information/data gathered during the geotechnical investigations. The GDR shall minimally contain the following information:

- Summarize and reference to separate geologic hazards report
- Description and discussion of the site exploration program
- Logs of borings, trenches, and other site investigations
- Description and discussion of field and laboratory test programs
- Results of field and laboratory testing

The high cost component of geotechnical investigations is borehole drilling; therefore, planning of the geotechnical investigations shall maximize the use of existing geologic and subsurface data, and optimize the use of geophysical testing and Cone Penetration Tests (CPTs) where warranted in order to minimize the amount and cost of drilling required and still achieve a level of knowledge commensurate with good engineering practice and judicious judgment for similar locations and applications. Geotechnical investigations shall not begin until project specific information is gathered as set forth in the following sections.

10.A.2.1 Standards and Key Geotechnical Investigation Reference Documents

The ASTM test methods, Caltrans Manual, and FHWA manuals are considered the most comprehensive and applicable guideline documents for geotechnical investigation of the CHSTP as well as federal transportation projects. Chapter 6 of the 2008 FHWA Project
Development and Design Manual (PDDM) provides an overview of practice for geotechnical work and direction for understanding policies and standards for geotechnical work performed by the Federal Lands Highway (FLH). The PDDM also provides a portal to technical information and presents a high-level source of technical guidance with regard to what needs to be accomplished. The corresponding 2007 FHWA Geotechnical Technical Guidance Manual (GTGM) provides guidance as to how the work shall be done. The GTGM also provides guidance for activities where standards and standard practices do not exist and provides access to and guidance for the use of new technologies. Chapter 3 of FHWA-NHI-09-010 presents good geotechnical investigation techniques and parameters for planning, design, and construction of road tunnels. For soil and rock logging, classifications, and presentation, refer to 2010 Caltrans Soil and Rock Classification, Classification, and Presentation Manual.

**10.A.2.2 Geotechnical Investigation Goals**

The goals of geotechnical investigations project are to:

1. Perform additional subsurface investigations to supplement existing geotechnical data for design of structural elements including bridges, retaining walls, at-grade structures, cut-and-cover tunnels, large culverts, mast arm supports (OCS, signals), wayside equipment, and signs along the proposed alignment.

2. Identify the distribution of soil and rock types within the project limits and assess how the material properties will affect the final design and construction of the project elements.

3. Define the groundwater and surface water regimes, especially, the depth, and seasonal and spatial variability of groundwater or surface water within the project limits. The locations of confined water-bearing zones, artesian pressures, and seasonal or tidal variations shall also be identified.

4. Identify and characterize any geologic hazards that may be present within or adjacent to the project limits (e.g., faults, landslides, rockfall, debris flows, liquefaction, soft ground or otherwise unstable soils, seismic hazards). These items are vital pieces of the overall geotechnical exploration process, and the investigators must ensure that these elements are addressed.

5. Assess surface hydrological features (infiltration or detention facilities) that are required for the project, as well as determine pond slope angle and infiltration rates to enable estimation of the size and number of those facilities.

6. Identify suitability of onsite materials as fill and/or the suitability of nearby materials sources.

7. For structures including bridges and cut-and-cover tunnels, large culverts, signs, signals, walls, or similar structures, provide adequate subsurface information for final design and construction.
8. For tunnels, trenchless technology, or ground improvement, provide adequate information to determine the viability of construction methods and potential impacts to adjacent facilities.

9. For landslides, rockfall areas, and debris flows, provide adequate information to determine stabilization or containment methods for design and construction.

10. Develop design soil properties for engineering evaluations, including dynamic analysis.

11. Perform chemical assessment of groundwater and soil for the impact evaluation of existing soil and groundwater on foundation materials.

12. Substantiate the various baselines expressed in the Geotechnical Baseline Report for Bidding (GBR-B), consider those baselines in the development of the design and construction approaches, and fill in any missing information in the GBR-B accordingly to develop a Geotechnical Baseline Report for Construction (GBR-C).

10.A.2.3 Sequence of Geotechnical Investigations

Details on performing geotechnical investigations are provided in Section 10.A.2.4 and shall follow the general sequence listed below.

1. Review the scope of project requirements to obtain a clear understanding of project goals, objectives, constraints, values, and criteria. This information may consist of:
   - Project location, size and features
   - Project element type (bridge, tunnel, station, embankment, retaining wall, etc.)
   - Project criteria (alignments, potential structure locations, approximate structure loads, probable bridge span lengths and pier locations, and cut and fill area locations)
   - Project constraints (context-sensitive design issues, right-of-way, environmental and biological assessments and permitting)
   - Project design and construction schedules and budgets

2. Review of available geologic and geotechnical data.

3. Initiate and prepare geotechnical investigations plans. Identify the anticipated required analyses and key engineering input for the analyses.

4. Perform field reconnaissance and geological mapping.

5. Finalize the Geotechnical Investigation Plan (GIP) and submit to the Authority. Obtain permits and rights-of-entry.

6. Perform exploration and laboratory testing for final design.

7. Compile and summarize data for use in performing engineering analyses, and prepare geotechnical data reports, geotechnical engineering reports, and geotechnical baseline report for construction.
10.A.2.4 Planning Geotechnical Investigations

The planning process for geotechnical investigations requires evaluating the appropriate number, depth, spacing, and type of exploration holes, as well as sampling intervals and testing frequencies. The involvement of engineering geologists (supporting the Geotechnical Designer) is critical throughout the investigation process, from initial exploration planning through the characterization of site conditions, to assure consistency for geologic interpretation of subsurface conditions in support of developing parameters for use in phased engineering design and construction.

The geotechnical investigation program shall be carried out in phases.

10.A.2.4.1 Desk Study

Review of subsurface conditions based on existing geological and subsurface data.

All relevant available information on the project site shall be reviewed. Available data may consist of reports, maps, journal articles, aerial photographs, historical records of previous investigations by agencies, as-built plans from construction of existing facilities, and communication with individuals with local knowledge. A Geologic Hazards Report shall be prepared by a California Certified Engineering Geologist (CEG) in advance of geotechnical investigations. The report shall be reviewed and utilized as a basis for geologic characterization and potential geologic hazards, and for siting of proposed subsurface exploration points. The results of the geologic and seismic hazard evaluation shall be collaborated with the Geotechnical Designer. Other sources of available information include the California Geological Survey (CGS), the United States Geological Survey (USGS), Caltrans archived Logs of Test Borings (LOTBs), the GIS database developed as part of the CHSTP, and data in individual city and county records and archives.

10.A.2.4.2 Field Reconnaissance

Field reconnaissance shall be conducted jointly by the Geotechnical Designer and the CEG after the desk study is completed. The following factors shall be evaluated by the field reconnaissance:

- **Geologic Report Reviews** – The Geotechnical Designer and Engineering Geologist responsible for the geotechnical investigations shall review and become familiar with geologic site characterizations and any identified geologic hazards provided in geologic hazards evaluation reports.

- **Environmental Considerations** – Potential impacts the project may have on subsurface materials, landforms, and the surrounding area shall be identified, and assessed to determine if project areas are governed by special regulations or have protected status.

- **Explorations** – The type(s) and amount of exploration and the kinds of samples that would best accomplish the phased project needs shall be evaluated.
• **Drilling Logistics** – The type, approximate locations, and depths of geotechnical explorations shall be defined, and approximate routes of access to each exploration location shall be determined. Make note of any feature that may affect the geotechnical investigation program, such as accessibility, structures, overhead utilities, evidence of buried utilities, or property restrictions. Evaluate potential water sources for use during borehole drilling operations. Evaluate potential concerns that may need to be addressed while planning an exploration program (permits, buried or overhead utilities clearance, equipment security, private property, etc.).

• **Permits** – The various types of permits that may be required shall be assessed, and all applicable jurisdictions shall be considered, which could include partner agencies, adjoining properties including railroads, Caltrans, regulatory agencies, and state and local government agencies. Local government agencies requirements could include regulations, codes, and ordinances from city, county, and departments of public works having jurisdiction. Permits could include right-of-entry, drilling and well permits, special use permits, lane closure and traffic control plans, utility clearances, etc.

10.A.2.4.3 General Subsurface Profiles

The general subsurface profiles, once developed, will present an overall geologic conditions of the areas under study and allow the Geotechnical Designer (in collaboration with the engineering geologist) to determine the locations of supplementary explorations for final design and construction.

10.A.2.4.4 Carry Out Geotechnical Investigations In Stages

For areas where there are no existing subsurface investigation data, conduct geophysical testing such as Spectral Analysis of Surface Wave (SASW), Multi-channel Analysis of Surface Wave, (MASW), Suspension PS Logging, Cross-hole Seismic Logging, seismic refraction tests, seismic reflection tests, or a combination of the above to measure shear wave and P-wave velocities in situ and to generalize the subsurface conditions prior to drilling CPTs and borings. The sequence of site investigation shall be as follows:

• **Geophysical testing** – To determine the general subsurface conditions for areas with no available existing geologic data.

• **CPTs** – To confirm the general subsurface conditions with measurements of pore water pressure and shear wave velocities with depth by means of using a combination of seismic cones, CPTu, and CPTs.

• **Borings** – To refine the general subsurface conditions after CPTs are performed. Install observation wells or piezometers and inclinometers where necessary to confirm groundwater table levels and ground movement in the field. Perform suspension PS logging or cross-hole seismic logging at deep boreholes (180 feet or deeper) in structures located over river crossings or unusual geologic conditions, and other boring locations selected by the Geotechnical Designer in collaboration with the engineering geologist.
1 Unusual Geologic Conditions – Structures that are subject to and founded on the following geologic conditions:
2  • Soft, collapsible, or expansive soil
3  • High groundwater table (within 5 feet below ground surface)
4  • Soil having moderate to high liquefaction and other seismically induced ground deformation potential
5  • Soil of significantly varying type over the length of the structure
6  • Fault Zones
7  • Unusual geologic conditions shall be defined within the Geotechnical Reports.

10.A.2.5 Surface Explorations

Standards for surface exploration methods are provided in PDDM Section 6.3.2.2, and technical guidance is provided in GTGM Section 3.2.2. Geologic field mapping of surficial soil and rock units and measurements of rock discontinuities shall begin by observing, measuring, and recording of exposed rock structure data at existing road cuts, drainage courses, and bank exposures, as well as portal locations where profiles transition from underground segments to elevated structures or at-grade reaches. Where rock exposures exist, mapping shall include initial characterization of rock mass rating, weathering, texture, overall quality, and discontinuity characteristics.

The objective of these observation and data collection efforts is to confirm the general types of soil and rock present, and topographic and slope features. For rock slopes, performance of slopes and the rockfall history are important indicators of how a new slope in the same material will perform. In addition to plotting data on a site plan or large-scale topographic map, preparation of field-developed cross sections is a valuable field method.

10.A.2.6 Subsurface Explorations

Relative advantages (economy, data quality, data collection time) of various methods of subsurface investigation should be considered in selecting the exploration plan. For example, geophysical methods and CPTs, which are relatively cheap and faster in operations, shall be conducted first, then followed by conventional test borings in specific situations.

Standards for performing subsurface explorations are provided in PDDM Section 6.3.2.2, and technical guidance is provided in GTGM Section 3.2.2. A guideline for the type of equipment and frequency of use for various types of investigations is presented in GTGM Exhibit 3.2-E. Additional guidance is contained in Caltrans (2007) logging manual.

The scope of the investigations shall reflect the anticipated subsurface and surface conditions and the preliminary results presented in the GDR during the bidding phase. Some factors that may impact the prioritization (sequence order ranking), method, number, and depth of subsurface explorations include the potential geologic hazards identified and geology (soil and rock units), landslides, slope stability, rockfall, rip-ability, fill suitability, expansive soils, compressible or collapsible soils, groundwater and hydrogeology, ground-borne vibration and noise transmissivity, erosion, temporary shoring, and excavation slopes. The level of
investigation, priority, and scope of work for each component shall be developed in accordance
with the geotechnical investigation guidelines set forth in these guidelines.

- **Geophysical Methods** – Spectral Analysis of Shear Wave (SASW), Multi-channel Analysis
  of Shear Wave (MASW), suspension logging, or cross-hole seismic logging shall be
  conducted to measure in situ shear wave and primary (P) wave velocities with depth. Shear
  wave and P-wave velocities are the key dynamic properties for seismic design and shall be
  measured in situ during geotechnical investigations.

  Standards for geophysical methods are provided in PDDM Section 6.3.2.3.2. The primary
  source supporting the guidance is FHWA DTFH68-02-P-00083 Geophysical Methods
  Generally, geophysical methods are used as a reconnaissance investigation to cover large
  areas and/or to supplement information between boreholes. These exploration techniques
  are most useful for extending the interpretation of subsurface conditions beyond what is
determined from small-diameter borings. The methods presented in FHWA (2003) shown as
Exhibit 3.2-F of the GTGM are some of the most common. The reliability of geophysical
results can be limited by several factors, including the presence of groundwater, non-
homogeneity of soil stratum thickness, gradation or density, and the range of wave
velocities within a particular stratum. Subsurface strata that have similar physical properties
can be difficult to distinguish with geophysical methods. Geophysical methods are also
applicable for testing ground-borne vibration transfer mobility of subsurface conditions,
and assessment of this parameter is considered important for HST systems. The reference
document for this testing is titled, “High-Speed Ground Transportation Noise and Vibration

- **Cone Penetration Test, Seismic Cones, and Piezocone Penetrometer Test** – CPT is a
  specialized quasi-static penetration test where a cone on the end of a series of rods is
  pushed into the ground at a constant rate and continuous or intermittent measurements are
  made of the resistance to penetration of the cone. This test can be used in sands or clays,
  fibrous peat or muck that are sensitive to sampling techniques, but not in rock, dense to
  very dense sands, or soils containing appreciable amounts of gravel, and cobble. The CPT is
  relatively inexpensive in comparison to borings, but it can only be used to supplement
  sampled borings because no samples are obtained so that no positive identification of soil
types can be made out of the CPTs.

  Piezocones are electric penetrometers that are capable of measuring pore-water pressures
during penetration. When equipped with time-domain sensors, cones can also be used to
measure shear wave velocity.

  Tests are conducted in accordance with ASTM D 5778 (Standard Test Method for Electronic
  Friction Cones and Piezocone Penetration Testing of Soils). References: Guides to CPT
  (Robertson, 2010), TRB-NCHRP synthesis report 368 (2007), and FHWA-SA-91-043.

  Many correlations relating CPT data to soil types and engineering properties have been
published. These correlations can be used for design of spread footings and piles.
Test Borings – Guidance for selection of the applicable exploration methods is presented in PDDM Exhibit 6.3-A (borings). Methods for exploratory borings shall be in accordance with AASHTO and ASTM standards. Detailed information on drilling and sampling methods is given in NHI132031 which lists applicable American Association of State Highway and Transportation Officials (AASHTO) and ASTM drilling and sampling specifications and test methods. Additional references include AASHTO MSI-1, FHWA GEC-5, FHWA-ED-88-053, National Highway Institute (NHI) 132012, NHI132035, USACE EM 1110-1-1804, USACE EM 1110-1-1906, FHWA-FL-91-002, and Caltrans (2007).

For the rotary wash drilling method, the drilling fluid in boreholes shall be kept above the groundwater level at all times. Rapid fluctuations in the level of drilling fluids shall be avoided. The boreholes shall be thoroughly cleaned prior to taking samples. Drill cuttings shall be collected and disposed of in accordance with applicable regulations.

Disturbed samples can be used for determining the general lithology of soil deposits, for identifying soil components and general classification purposes, and for determining grain size, Atterberg limits, and compaction characteristics of soils. The most commonly used in-situ test for surface investigations is the Standard Penetration Test (SPT), AASHTO T 206. The use of automatic hammers for SPT is highly recommended, and standard drop height and hammer weight must be maintained. The SPT values obtained with non-automatic hammers are discouraged and are allowed when calibrated by field comparisons with standard drop hammer methods. The SPT dynamic analyzer shall be used to calibrate energy of the SPT equipment at the site at least at the start of the project and bi-weekly for long-duration site investigations. More frequent use of the SPT dynamic analyzer is encouraged.

Undisturbed samples shall be obtained in fine-grained soil strata for use in laboratory testing to determine the engineering properties of those soils. Specimens obtained by undisturbed sampling methods may be used to develop the strength, stratification, permeability, density, consolidation, dynamic properties, and other engineering characteristics of soils. Disturbed and undisturbed samples can be obtained with a number of different sampling devices, as summarized in Table 7 of FHWA GEC-5 and Table 3-4 of NHI 132031.

It will be the responsibility of the Geotechnical Designer to obtain enough testable samples of rock and soil to complete the agreed-upon laboratory testing program. The quantity of each type of test conducted shall be proposed by the geotechnical investigation consultant to adequately characterize each soil or rock unit encountered. Therefore, adequate subsurface exploration and sampling will be necessary to obtain sufficient sample quantity for subsurface characterization.

- Sandy or Gravely Soils Sampling – The SPT (split-spoon) samples shall be taken at 5-foot intervals or at significant changes in soil strata. Continuous SPT samples with a gap of at least 6 inch between two consecutive tests are recommended in the top 15 feet of borings made at locations where spread footings may be placed in natural soils. SPT
bagged samples shall be sent to lab for classification testing and verification of field
visual soil identification. Modified California (MC) and/or California (C) samplers shall
not be used in these soils.

- **Silt or Clay Soils and Peat Sampling** – The SPT and undisturbed thin wall tube samples
shall be taken at 5-foot intervals or at significant changes in strata of cohesive soils.
Hydraulic (Osterberg) thin-walled piston samplers shall be used in collecting soft to
very soft clays. Take alternate SPT and tube samples in same boring or take tube
samples in separate undisturbed boring. Tube samples shall be sent to lab to allow
consolidation testing (for settlement analysis) and strength testing (for slope stability
and foundation-bearing capacity analysis). The tube samples shall be retrieved by
pushing soil out in the same direction that it entered the tube (i.e., through the top of the
tube sampler; do not reverse and push it back out of the bottom). Field vane shear
testing is also recommended to obtain in-place shear strength of soft clays, silts, and
peat.

- **Rock Sampling** – Continuous cores shall be obtained in rock or shales using double- or
triple-tube core barrels. In structural foundation investigations, core a minimum of 10
feet into rock to ensure it is bedrock and not a boulder. Core samples shall be sent to the
lab for possible strength testing (unconfined compression) if for foundation
investigation. Percent core recovery and rock quality designation (RQD) value shall be
determined in field or lab for each core run and recorded on the boring log. Additional
guidelines for rock coring are described later in this section and in the reference
manuals.

- **Groundwater in Borings** – Water level encountered during drilling, at completion of
boring, and at 24 hours after completion of boring shall be recorded on the boring log. In
low permeability soils such as silts and clays, a false indication of the water level may be
obtained when water is used for drilling fluid and adequate time is not permitted after
boring completion for the water level to stabilize (more than one week may be required).
In such soils, a plastic pipe water observation well shall be installed to allow monitoring
of the water level over a period of time. Seasonal fluctuations of water table shall be
determined where fluctuation will have significant impact on design or construction
(e.g., borrow source, footing excavation, excavation at toe of landslide). Artesian
pressure and seepage zones, if encountered, shall also be noted on the boring log. In
landslide investigations, slope inclinometer casings can also serve as water observation
wells by using leaky couplings (either normal aluminum couplings or PVC couplings
with small holes drilled through them) and pea gravel backfill. The top 1 foot or so of
the annular space between water observation well pipes and borehole wall shall be
backfilled with grout, bentonite, or sand-cement mixture to prevent surface water
inflow, which can cause erroneous groundwater level readings.

- **Probes, Test Pits, Trenches, and Shafts** – Guidance for selection of the applicable
exploration methods is presented in PDDM Exhibit 6.3-B (probes, test pits, trenches, and
shafts), and GTGM Section 3.2.3.5. The recommended primary reference is NHI 132031.
Additional guidance is contained in AASHTO MSI-1 and Caltrans 2007. Exploration pits and trenches performed by hand, backhoe, or dozer allow detailed examination of the soil and rock conditions at shallow depths and relatively low cost. Exploration pits can be an important part of geotechnical explorations where significant variations in soil conditions occur (vertically and horizontally), large soil and/or non-soil materials exist (boulders, cobbles, debris) that cannot be sampled with conventional methods, or buried features must be identified and/or measured. Upon completion, the excavated test pit shall be backfilled and compacted with the excavated material or other suitable soil material, and the surface shall be restored to its previous or approved condition.

- **Soil Resistivity Testing** – The ability of soils to conduct electricity can have a significant impact on the corrosion of buried structures and the design of grounding systems. Accordingly, subsurface investigations shall include conducting appropriate investigations to obtain soil resistivity values. The following information and methodologies are recommended.
  - Soil resistivity readings shall be obtained to determine the electric conduction potential of soils at each traction power facility (supply/paralleling/switching station), which are spaced at approximately 5-mile intervals.
  - Resistivity measurements shall be obtained in accordance with Institute of Electrical and Electronics Engineers (IEEE) Standard 81-1983 - IEEE Guide for Measuring Earth Resistivity using the four-point method for determining soil resistivity. IEEE states that the four-point method is more accurate than the two-point method.

- **Standards for Boring Layout and Depth** – Standards for boring layout and depth with respect to structure types, locations and sizes, and proposed earthwork are provided in these guidelines.

- **Standards for Sampling and Testing From Borings** – Minimum standards for disturbed and undisturbed soil and rock are presented in Exhibit 6.3-D of PDDM, and Section 3.2.3.3 of GTGM.

- **Rock Coring** – Standards for soil and rock classification are provided in PDDM Section 6.3.2.3.4, and technical guidance is provided in GTGM Section 3.2.3.4. The International Society of Rock Mechanics (ISRM) classification system shall be followed for rock and rock mass descriptions, as presented in FHWA GEC-5 FHWA-IF-02-034. The primary source supporting the standards and guidance is NHI 132031, and a secondary source is AASHTO MSI-1. Because single-tube core barrels generally provide poor recovery rates, the double- or triple-tube core barrel systems shall be used. To protect the integrity of the core from damage (minimize extraneous core breaks), a hydraulic ram shall be used to expel the core from the core barrel. Rock cores shall be photographed in color as soon as possible after being taken from the bore hole and before laboratory testing.

  If rock is encountered in boreholes within the planned depth of drilling, continuous rock coring shall be performed in accordance with the following procedures. Rock coring shall be performed using a double or triple tube HQ coring system or a larger-diameter, double or
triple-tube coring system. The HQ system produces cores 2.4 inches in diameter. The advantage of the triple tube system is that a split liner is used to contain the core, which results in relatively minimal disturbance to the core. Where weak rock zones are encountered, soil sampling techniques may be used instead of coring to recover samples that would be relatively undisturbed and suitable for testing. These techniques include the use of samplers such as the Pitcher or MC samplers. The potential difficulty with these samplers is that they can be easily damaged by hard, gravel-size particles that are often mixed with the softer, clay-like matrix of the weathered rock. These difficulties will need to be considered when planning the exploration program.

Rock core samples shall be placed in plastic core bags or double wrapped in plastic wrap and placed in wooden core boxes and transported to a storage facility at the end of each day. An adequate number of core boxes shall be maintained on site at all times during field exploration activities. The core shall be photographed, taking at least one photo for each core box, and close-ups taken of special features such as shear zones or other features of special interest. The core box label shall be clearly visible within the photo. An experienced geologist shall study the core and edit the borehole log based on their observations. Cores boxes shall be maintained throughout the design process and construction, with cores that have been removed for testing duly indicated in the appropriate locations in each box.

In some rock slope applications, it is important to understand the precise orientation of rock discontinuities for the design. Standards for using orienting-recovered rock core are presented in NHI 132031. In special cases, boreholes can be photographed/imaged to visually inspect the condition of the sidewalls, distinguish gross changes in lithology, and identify fracture zones, shear zones, and joint patterns by using specialized television cameras. Refer to AASHTO MSI-1, Section 6.1.2.

- **Care and Retention of Samples** – Technical guidelines for soil and rock retention are provided in GTGM Section 3.2.3.7, and geotechnical boring and sample identification, handling and storage guidelines are provided in each Contract.

### 10.A.2.7 Soil and Rock Classification

Standards for soil and rock classification are provided in PDDM Section 6.3.2.4, and technical guidance is provided in GTGM Section 3.2.4. Soils shall be classified in accordance with the ASTM Unified Soil Classification System (USCS). Rock and rock mass descriptions and classification shall follow the ISRM classification system presented in FHWA GEC-5. Material descriptions are based on the visual-manual method, and materials classifications are based on laboratory index tests (ASTM D 2487). Additional guidance is contained in Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2007).

### 10.A.2.8 Exploration Logs

Standards for preparing exploration field logs are provided in PDDM Section 6.3.2.5, and technical guidance is provided in GTGM Section 3.2.5.
Field Logs – Field logging shall be performed by a geologist or engineer under the direct supervision of a California registered geotechnical engineer or CEG. Logging shall be performed in accordance with ASTM D 5434. The location information (e.g., station, offset, elevation, and/or state plane coordinates) of all the explorations are to be recorded on the field logs. Exploration locations shall be located at the time of drilling by GPS with at least sub-10-foot accuracy. The explorations shall eventually be located by a licensed land surveyor. Required documentation for test pits shall include a scale drawing of the excavation, and photographs of the excavated faces and spoils pile. Drilling and sampling methods and in-situ measurement devices that were used shall also be documented. The field logs shall contain basic reference information at the top, including project name, purpose, specific location and elevation, exploration hole, number, date, drilling equipment, procedures, drilling fluid, etc. In addition to the logging descriptions of soil and rock encountered during exploration, the depth of each stratum contact, discontinuity, and lens shall be recorded. The reason for terminating an exploration hole and a list/description of instrumentation (if any) or groundwater monitoring well installed shall be written at the end (bottom) of each exploration log.

Final Logs – Exploration logs shall be prepared with the gINT boring/test pit log software platform, using the formatted boring record template standardized by Caltrans (illustrated as Figures 5-12 and 5-13 in the Caltrans logging manual, 2007 version). An explanation key, known as the Boring Record Legend shall always accompany exploration logs whenever they are presented. The standardized legends to be used for CHSTP are illustrated as figures 5-14 through 5-16 of Caltrans (2007). The final edited log shall be based on the initial field log, visual classification, and the results of laboratory testing. The final log shall include factual descriptions of all materials, conditions, drilling remarks, results of field and lab tests, and any instrumentation. Where groundwater observation wells or piezometers are installed, several measurements are usually necessary within a one-week timeframe following drilling to verify that measured groundwater levels or pressures have achieved equilibrium. As a minimum, final boring logs shall contain the information shown in NHI132031. AASHTO MSI-1 provides additional guidance regarding documentation for boring logs.

10.A.2.9 In-Situ Testing

Standards for performing in-situ testing are provided in PDDM Section 6.3.2.6, and technical guidance is provided in GTGM Section 3.2.6. The primary reference is NHI1 32031. In-situ testing is very beneficial for projects where obtaining representative samples suitable for laboratory testing is difficult. Field in-situ borehole tests can be correlation tests, strength and deformation tests and permeability tests. Correlation tests primarily consist of SPTs performed in accordance with ASTM D 1596 and AASHTOT 206, and Dynamic CPTs are performed in accordance with ASTM D 3441.

- In-situ soil tests may consist of the following:
Cone Penetration Test (CPT) – Refer to Section 10.A.2.6.

Pressuremeter Test – This test measures state of stress in-situ and stress/strain properties of soils by inflating a probe placed at a desired depth in a borehole. Tests are completed in accordance with ASTM D 4719. Reference FHWA-IP-89-008.

Flat-Plate Dilatometer Test – This test uses pressure readings from an inserted plate at the base of a borehole to determine stratigraphy and obtain estimates of at-rest lateral stresses, elastic modulus, and shear strength of loose to medium dense sands (and to a lesser degree, silts and clays). Tests are completed in accordance with ASTM D 6635. Reference FHWA-SA-91-044. Care and judgment shall be undertaken for this test as it often provides information that is difficult to interpret or relate to parameters needed for engineering design.

Field Vane Shear Test (VST) – This test is used on very soft to medium stiff cohesive soil or organic deposits to measure the undrained shear strength, remolded strength of the soil and soil sensitivity. Field vane shear test may provide more reliable estimate of peak and residual shear strength in cohesive soils, as disturbance from sampling and testing in laboratory is avoided. Tests are completed in accordance with ASTM D 2573 and AASHTO 223. VST is often regarded as a valuable test to estimate peak and residual shear strength in cohesive soils as disturbance from sampling and testing in the laboratory can be avoided.

Hydrogeologic testing in-situ may consist of the following:

- Permeability Tests – Several in-situ hydraulic conductivity tests exist, with the most commonly used methods being the pumping test and the slug test. The selection of the appropriate aquifer test method for determining hydraulic properties by well techniques is described in ASTM D 4043. In general, refer to NHI 32031, BOR Geology Manual, and NAVFACDM-7.1.

- Pumping Test – The pumping test requires not only a test well to pump from, but also one to four adjacent observation wells to monitor the changes in water levels as the pumping test is performed. Pumping tests are typically used in large-scale investigations to more accurately measure the permeability of an area for the design of dewatering systems. Refer to ASTM D 4050.

- Slug Test – The slug test is quicker to perform and much less expensive, because observation wells are not required. It consists of affecting a rapid change in the water level within a well by quickly injecting or removing a known volume of water or solid object, known as a slug. The natural flow of groundwater out of or into the well is then observed until equilibrium in the water level is obtained. Refer to ASTM D 4044.

- Packer Tests – These tests are performed in a borehole by placing packers above and below the soil/rock zone to be tested. One method is to remove water from the material being tested (Rising Water Level Method). Another method is to add water to the borehole (Falling Water Level Method and Constant Water Level Method). A third
method utilizes water under pressure rather than gravity flow. The coefficient of permeability that is calculated provides a gross indication of the overall mass permeability. Refer to FHWA-TS-89-045 and NHI 32031.

- **Open Borehole Seepage Tests** – Methods include "Falling Water Level," "Rising Water Level," and "Constant Water Level" and are selected based on the relative permeability of the subsurface soils and groundwater conditions. Further detail is provided in Chapter 6 of NHI 32031.

- **Infiltration Tests** – Two types of infiltrometer systems are available: sprinkler type and flooding type. Sprinkler types attempt to simulate rainfall, while the flooding type is applicable for simulating runoff conditions. Applications for these tests include the design of subdrainage and dry well systems. The most common application is the falling head test, performed by filling (flooding) a test pit hole and monitoring the rate at which the water level drops. Refer to ASTM D 4043.

Handling and disposal (or permitted discharge to storm sewer system) of water generated from hydrogeologic field testing shall be the responsibility of the Geotechnical Designer conducting the investigation work.

If the Geotechnical Designer intends to use field tests not covered in the current ASTM or referenced standards, the proposed test methods shall be submitted to the Authority for approval prior to start of testing.

**10.A.2.10 Laboratory Testing of Soil and Rock**

Standards for performing laboratory testing are provided in PDDM Section 6.3.2.7 and technical guidance is provided in GTGM Section 3.2.7. Sufficient laboratory testing shall be performed to represent in-situ conditions. Exhibit 3.2-J of the GTGM provides a guideline for estimating laboratory test requirements for the different types of geotechnical analysis. Chapters 7 through 10 of NHI 132031, GEC-5, and Chapters 2 and 3 of NHI 132012 provide overviews of testing and correlations, as well as criteria to consider when planning the scope of testing programs. Additional references include AASHTO MSI-1, NHI 132012, NHI 132035, USACE EM 1110-2-1906, FHWA-FL-91-002; and Kulhawy and Mayne (1990). Exhibits 3.2-K (soil) and Exhibit 3.2-L (rock) of GTGM present a summary of the predominant laboratory tests. The proposed workplans for laboratory testing programs shall be submitted for review. Testing shall be done at a Caltrans approved facility.

If the Geotechnical Designer proposes to use laboratory tests not covered in the current ASTM or referenced standards, a variance of test methods shall be submitted to the Authority for approval prior to commencement.

**10.A.2.11 Instrumentation and Monitoring**

Standards for installing and monitoring geotechnical instrumentation are provided in PDDM Section 6.3.2.8, and technical guidance is provided in GTGM Section 3.2.8. Instrumentation is
used to augment standard investigation practices and visual observations where conditions
would otherwise be difficult to evaluate or quantify due to location, magnitude, or rate of
change. The quantity and locations of proposed geotechnical instrumentation shall be selected
to suit the anticipated conditions consistent with project objectives and design requirements.
The geotechnical exploration work plan shall include instrumentation work detailing locations,
installation procedures, and methods to be used. The work plan shall be submitted to the
Authority prior to commencement. Additional information about inclinometers and
piezometers are presented in Cornforth (2005).

10.A.3 Project Features Requiring Geotechnical Investigations

10.A.3.1 General

The CHSTP will require geotechnical investigations of the various project features. The
referenced standards and technical guidance documents shall be utilized, in addition to the
primary and secondary references, where listed. Guidelines for the approximate number and
depth of various exploration methods are included. In addition to the general guidelines, the
scope of the investigation for the various project features shall also reflect the anticipated
subsurface and surface conditions, as well as the design phase level (whether preliminary or
final). Some factors that may impact the method, number, depth, and prioritization of
subsurface explorations include type of soil or rock, landslides, slope stability, rockfall,
rippability, fill suitability, expansive soils, compressible soils, groundwater and hydrogeology,
ground-borne vibrations, erosion, engineering design needs, temporary shoring, and excavation
slopes.

The scope of investigation work for each component shall be developed in accordance with the
guidelines contained in this section. The quantity, locations, and depths of proposed
geotechnical exploration shall be selected to suit the anticipated conditions consistent with
phased project objectives and design requirements. The geotechnical exploration work plan
shall include information detailing methods to be used and proposed schedule. The work plan
shall be submitted to the Authority prior to commencement. If the Geotechnical Designer
proposes to use exploration methods or frequencies that differ from the guidelines set forth
herein or are not covered in the current reference standards, a variance for the proposed
alternate exploration plans shall be submitted to the Authority for approval prior to
commencement.

The geophysical testing and CPTs provide advantages over conventional test borings under
specific situations and should be considered first.

10.A.3.2 Rail Alignment and Earthwork

Standards for investigations for the at-grade rail alignment and earthwork are provided in
PDDM Section 6.3.1.2.1, and technical guidance is provided in GTGM Section 3.1.2.1.
Explorations are made along the proposed at-grade rail alignment for the purpose of defining the geotechnical properties of materials. This information is used to:

- Design cut and fill slopes
- Assess material suitability for embankment construction
- Define the limits of potential borrow materials
- Assess the suitability of foundation materials
- Evaluate settlement or slope stability problems
- Quantify the depths of topsoil and volumes of material to be removed
- Design remedial measures in areas of poor materials
- Aid the designer of the rail roadbed subgrade section
- Identify geologic hazards such as liquefaction and landslides

For cuts and fills, test borings shall be advanced at least every 200 feet (erratic conditions) to 400 feet (uniform conditions) along the project alignment where cuts or fills are anticipated. For large cuts or fills (e.g., 30 feet or more in height) an additional boring near the top of the proposed cut and toe of the proposed fill to evaluate cut/fill feasibility and overall stability may be necessary. Depths of the borings shall be at least three times the vertical height of the fill (or 40-foot minimum depth) and at least 15 feet below the base of the cut. If soft or poor soils are encountered, additional depth to competent material or 10 feet into rock will be needed to define the subsurface conditions.

### 10.A.3.3 Structures

Standards for structures and geotechnical hazards are provided in PDDM Section 6.3.1.2.3, and technical guidance is provided in GTGM Section 3.1.2.3 and Exhibit 3.1-B Guideline “Minimum Boring” Criteria. Structures and geotechnical hazards will primarily consist of the following:

- Bridges and aerial structures (viaducts)
- Stations
- Buildings
- Retaining walls
- Tunnels and portals
- Large culverts
- Mast-arm supports (OCS, signals, message signs)
- Landslides
- Faults
For bridges, one boring shall be drilled at the substructure unit under 100 feet in width and two borings per substructure unit over 100 feet in width, both drilled to a depth of 20 feet below pile/shaft tip elevation or two times maximum pile group dimension, whichever is greater or to a depth of a minimum of 10 feet into bedrock. In addition, at least one seismic cone, suspension PS logging, or SASW shall be conducted at each bridge to measure shear wave and P-wave velocities in situ, each to a depth of 100 feet or deeper. The number of the seismic cones, suspension loggings, and SASW shall increase if the bridge is of multiple long spans (greater than 350 feet) and/or if the bridge is located in erratic soil conditions with soft, compressible and loose saturated soils.

For buildings and stations, one boring shall generally be made at each corner and one in the center. This may be reduced for small buildings. For extremely large buildings and stations or highly variable site conditions, one boring shall be taken at each support location. Refer to building foundation manuals and CBC (codes) for additional guidance in planning geotechnical investigations. In addition, areas of influence of the building/station and/or of surrounding geologic or geotechnical issues shall be considered in defining the extent of explorations.

For retaining walls, the minimum site exploration will be one boring or one CPT (or both) at 100 to 200 foot intervals, each drilled to a depth of 0.75 to 1.5 times wall height or to a competent stratum if potential deep stability or settlement is a problem. The boring and CPT can be interchangeable and located at the front of and some in back of the wall face.

Due to the extreme variability of conditions under which tunnels are constructed and the complexity of the projects, it is difficult to provide specific recommendations for tunnel investigation criteria. In general, boring footage is typically on the order of 1.5 to 3.0 linear feet of borehole per route foot of tunnel, and site exploration budgets are typically on the average of three percent of the estimated tunnel cost. Criteria shall be established for each project reach on an individual basis and be based on the complexity of the geology and the length and depth of the tunnel. FHWA-IF-05-023 and U. S. National Committee on Tunneling (USNCTT, 1995) shall be considered the primary references.

For culverts, a minimum of 1 boring per major culvert drilled to a competent stratum or to a depth of twice the culvert height, whichever is less.

Standard foundations for sign bridges, cantilever signs, cantilever signals, and strain pole standards are based on allowable lateral bearing pressure and angle of internal friction of the foundation soils. The determination of these values may be estimated by SPT and CPT. One CPT or one boring shall be made at each designated location. Cones shall be drilled to at least 50 feet into firm ground. Borings shall extend 50 feet into suitable soil or 5 feet into competent rock. Deeper borings may be required for posts with higher torsional loads or if large boulders are anticipated. Other criteria are the same as for bridges.

In addition to the above structures, any structure such as signage or other design features shall be addressed with regard to their potential influence and evaluated, as needed.
10.A.3.4 Landslides – Slope Instability

Standards for investigations for landslides are provided in PDDM Section 6.3.1.2.4, and technical guidance is provided in Section 3.1.2.4 and Exhibit 3.1-B of the GTGM. A minimum of three borings shall be advanced along a line perpendicular to centerline or planned slope face to establish geologic cross sections for stability analysis. The number of cross sections depends on the extent of the slope stability problem. For active slides, place at least one boring each above and below the sliding area. The borings shall be extended to an elevation below active or potential failure surfaces and into hard stratum, or to a depth for which failure is unlikely because of geometry of the cross section. If slope inclinometers are used to locate the depth of an active slide, they must extend to a depth below the base of the slide. Observation wells and/or piezometers at selected depths will also be required to determine the groundwater table in the soil/rock mass.

10.A.3.5 Faults

At locations where active faulting is suspected to be coincident with or within the area of CHSTP operations and facilities, a geologic reconnaissance will be required to ground-truth mapped fault traces. This reconnaissance shall be carried out by means of interpretations of aerial photos, LiDAR data, satellite imagery, and topographic information. The locations shall be reviewed in the field to assess the presence of geomorphic features associated with faulting such as escarpments, pressure ridges, sag ponds, seeps/springs, vegetation contrasts, or deflected drainages. All such features shall be documented on a geologic field map. If sufficient field data is available to document that the fault or fault zone is outside the footprints of the high-speed train operations, no further fault evaluation is required. Otherwise, a site specific investigation including paleo-seismic trenching will be necessary.

If existing paleo-seismic trenching data is available, it may be reviewed and used as a basis for locating the fault and providing its rupture characteristics for final design; however, if either a known active fault or suspected active fault is located near or at the location of a project facility, exploratory trenching across the fault will be required to assess its rupture characteristics for input to final design.

10.A.3.6 Materials Sources

Standards for investigations for materials sources are provided in PDDM Section 6.3.1.2.2, and technical guidance is provided in Section 3.1.2.2 and Exhibit 3.1-B of the GTGM. Borings shall be spaced every 100 to 200 feet. The depth of exploration shall extend 5 feet beyond the base of the deposit, or to a depth required to provide the needed quantity of borrow material. These investigations shall evaluate the quality and quantity of materials available at existing and prospective sources within the vicinity of a project. These materials could include gravel base, crushed surfacing materials, pavement and concrete aggregates, riprap, wall backfill, borrow excavation, and select backfill materials. The evaluation may consider existing government-
owned material sources, existing commercial material sources, expansion of existing sources, and development of new material sources.

10.A.3.7 Hydrological Features – Infiltration and Detention Facilities

For surface hydrological features (infiltration or detention facilities) that may be needed, at least one boring per site shall be obtained to assess feasibility and define groundwater conditions. Boring depths will depend on the nature of the subsurface conditions encountered and the depth of influence of the geotechnical feature. Borings shall extend at least 20 feet below the likely base elevation of the facility, or five times the maximum anticipated ponded water depth, whichever is greater. Observation wells and/or piezometers shall be installed and monitored for at least 1 year to assess yearly highs and lows for the groundwater.

10.A.3.8 Pavement

Pavements are not a significant component of the HST trackway alignment design but will be an extensive design element for station areas, access roads, grade separations, and surface road reconstruction. Standards for investigations for pavement subgrade are provided in PDDM Chapter 6, Section 6.3.1.2.5 and Chapter 11, and technical guidance is provided in GTGM Section 3.1.2.5. Other sources supporting investigation standards and guidance are NHI 132031, AASHTO MSI-1, and FHWA GEC-5.

10.A.4 References

7. Federal Highway Administration (FHWA):
- Project Development and Design Manual (Draft) – Chapter 6 - Geotechnical, April 2011.


Appendix 10.B: Guidelines for Geotechnical Earthquake Engineering

10.B.1 Purpose

These guidelines represent a preferred, but not necessarily the only required actions needed for a particular design feature associated with earthquake engineering. These guidelines convey a minimum standard of care in performing earthquake engineering design. These are not intended as a prescribed design criteria or checklist.

10.B.2 Seismic Design Criteria

Seismic design criteria for geotechnical earthquake engineering have been established in terms of two levels of project performance criteria: No Collapse Performance Level (NCL) and Operability Performance Level (OPL) as noted in the Seismic chapter of the Design Criteria.

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 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 high-speed train (HST) operation, shall be evaluated for instability due to design seismic events and associated geologic hazards.

10.B.2.1 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. These compositional criteria are presented in Section 10.12.2 of the Geotechnical chapter.

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 Designer shall consider...
the allowable deformation and the long-term, post-construction performance requirements for earth and fill conditions.

For fine-grained soils (especially soils that are potentially sensitive) that do not meet the compositional criteria for liquefaction, the impact of cyclic softening resulting from seismic shaking shall be evaluated.

Considering the range of criteria currently available in the literature, the Geotechnical Designer shall consider performing cyclic triaxial or simple shear laboratory tests on undisturbed soil samples to assess cyclic response for critical cases.

For gravels, 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.

### 10.B.2.2 Liquefaction Triggering Evaluations

Liquefaction-triggering evaluations shall be performed for sites that meet the two design criteria established in the Geotechnical chapter:

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. Where CPT data are unavailable, SPT values can be used as the 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 modeled well by the simplified methods.

### 10.B.2.2.1 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. 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 1.05.
10.B.2.2.2 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 10B.2.8. Liquefaction-induced total ground settlement of saturated granular deposits shall be estimated using Zhang et al. (2002) and at least one of the following methods: Ishihara and Yoshimine (1992), Idriss and Boulanger (2008), and Cetin et al. (2009). 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.

10.B.2.2.3 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.

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.

10.B.2.2.4 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 (H2) and the thickness of the non-liquefiable crust (H1) 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 (H2) 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.

10.B.2.3 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.

10.B.2.3.1 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 10B.2.9.1 below 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 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 Designer shall compare the estimated lateral spread values with the allowable deformation values and develop mitigation plans described in Section 10B.2.4, if necessary. The Geotechnical Designer shall consider the long-term, post-construction performance requirements for earth-and-fill conditions.

10.B.2.4 Analysis for Design of Liquefaction Mitigation

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. In general, options for mitigations are divided into two categories: ground improvement options and structural options.

10.B.2.5 Ground Improvement Options

Refer to Section 10.8.5.5 of the Geotechnical chapter.

10.B.2.6 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 in Section 10B.2.7.

### 10.B.2.7 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 shall be used. However, the liquefaction susceptibility and triggering analyses performed as part of this procedure shall be based on Section 10B.2.1 and Section 10B.2.2, respectively. Similarly, the lateral spread estimates shall be based on Section 10B.2.3. The Geotechnical Designer shall compare the estimated lateral spread values with the allowable deformation values and develop mitigation plans described in Section 10B.2.4, if necessary. The Geotechnical Designer 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.
10.B.2.8 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 Designer shall compare the estimated settlement values with the allowable deformation values and develop mitigation plans described in Section 10B.2.4, if necessary. The Geotechnical Designer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

10.B.2.9 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 Designer shall compare the estimated deformation values with the allowable deformation values and develop mitigation plans described in Section 10B.2.4, if necessary. The Geotechnical Designer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

10.B.2.9.1 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 the Geotechnical Designer. 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 10B.2.2.3 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, $k_v$ 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
1. Minimum horizontal acceleration in a pseudo-static analysis for which FOS is 1.0. Using the
2. estimated $k_y$ values, deformations shall be estimated using simplified methods such as Makdisi
3. and Seed (1978) and Bray and Travasarou (2007). Other methods such as Newmark time history
4. method or more advanced methods involving numerical analysis may be used, but shall be
5. checked against the simplified methods.

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

10.B.2.9.2 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, the allowable deformation set forth in the
Geotechnical chapter 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. A new failure
plane shall be searched for the pseudo-static analysis. The analysis shall look for both circular
and non-circular failure surfaces.

10.B.2.10 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.

10.B.2.11 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.

10.B.3 References

1. American Association of State Highway and Transportation Officials (AASHTO) LFRD Bridge Design Specifications with California (Caltrans) Amendments
   - Section 3 “Loads and Load Factors”
   - Section 10 “Foundations”
   - Section 11 “Abutments Piers and Walls”
4. American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318.
6. ASTM International.


20. Bridge Welding Code ANSI/AASHTO/AWSD1.5-95


22. California Department of Transportation (Caltrans) Bridge Design Manuals, latest edition
   - Bridge Design Specification (BDS) - AASHTO LRFD Bridge Design Specification, 2005, with Caltrans Interim Revisions
   - Bridge Memo to Designers Manual (MTD)
   - Bridge Design Practices Manual (BPD)
   - Bridge Design Aids Manual (BDA)
   - Bridge Design Details Manual (BDD)
   - Standard Specifications
   - Standard Plans
   - Seismic Design Memorandum
   - Seismic Design Criteria (CSDC) ver. 1.4


33. Empelmann, Whittaker, Los and Dorgarten, “Taiwan High Speed Rail Project – Seismic Design of Bridges Across the Tuntzuchiao Active Fault”, 13th World Conference on Earthquake Engineering, Paper No. 140, Vancouver, B.C., Canada, 2004


35. European Standard EN 1990 annex A2: Application to Bridges


37. FIB (Federation Internationale du Beton) Task Group 7.4, Seismic Design and Assessment Procedures for Bridges, Chapter 8: Design for Active Fault Crossings, primary author Ian Billings.
Federal Highway Administration (FHWA) Publications:


Ghasemi, H., “Bolu Viaduct: Damage Assessment and Retrofit Strategy”, The 36th Joint Meeting US-Japan Panel on Wind and Seismic Effects (UJNR), Session 1, National Institute of Standards and Technology, Gaithersburg, Maryland, USA, 2004


53. National Cooperative Highway Research Program (NCHRP),


63. Taiwan High Speed Rail (THSR) Corporation: Volumes 1 and 3.


Appendix 10.C: Guidelines for Rock Slope Engineering

10.C.1 Purpose

These guidelines convey a minimum standard of care for performance of rock slope engineering design, mapping, and construction.

10.C.2 Design

Rock slopes are typically composed 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 are typically controlled by the discontinuities. Analytical techniques for rock slope stability assessment shall consider the kinematic stability of blocks or groups of blocks sliding upon the discontinuities, toppling, or in terms of wedge failure. Limit equilibrium methods that calculate a factor of safety 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.

For rock mass consisting 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 & 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 percent of fallen rocks. If right-of-way is not available to size catchment basins to achieve 99 percent 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:

- Acoustic sensing
- Electromagnetic sensing
1. Seismic sensing
2. Visual sensing, using cameras

Input data and parameters used in rock slope stability analyses shall take into consideration geology, groundwater and rainfall, and proposed geometry/topography. Rock engineering parameters shall be developed for use in slope stability analyses.

When available, empirical or historical data and direct observation within the geologic unit or 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 shear strength parameters shall be selected, depending upon the rate of loading, and permeability characteristics of the rock. 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.

10.C.3 Rock Slope Mapping and Condition Survey Requirements

The results of the mapping and condition surveys shall be used by the Geotechnical Designer to develop design and construction recommendations for treatment of exposed rock slopes and design of new rock cut slopes.

Under supervision of the Geotechnical Designer, qualified personnel trained in geology or engineering geology shall supervise and perform the rock slope mapping activities and data collection. A Certified Engineering Geologist (CEG) licensed in the State of California with at least 5 years of experience in rock slope design shall conduct slope condition surveys and rock mapping. Prior to mapping, the CEG shall be familiar with the local and regional geology. The mapping teams shall be knowledgeable of the rock units and structural and historical geologic aspects of the areas to be mapped.

10.C.4 Rock Slope Mapping


Field observation data shall be recorded on approved forms similar to the one depicted on Figure AI-9a and b of the Rock Slope Reference Manual, FHWA-HI-99-007 and in field notebooks. Parameters described in the Rock Slope Reference Manual, FHWA-HI-99-007 (pages AI-3 to AI-14) shall be recorded. The following methods/assessments shall be used in the rock slope mapping:

- Use Project stationing to describe the location of rock mapping or rock slope condition observations. Record observation locations to within plus or minus 3 feet of actual Project
stations. Also, designate observation locations with a sequential numbering system.
Orientation data shall be referenced to Project north (as shown on the plans).

- Color digital photographs shall be taken of each mapping area and window. A scale shall
  be included in the window mapping photograph. Photographs shall be mounted on an
  8 1/2 x 11-inch sheet and labeled.
- Feature specific photographs shall be taken, with a minimum of one photograph per
  window, and labeled.
- After the geologic mapping for a window has been completed, evaluate the rock slope at
  each mapping window using the Rock Slope Hazard Rating System presented in Chapter

10.C.5 Rock Excavation

Rock excavation surfaces shall be mapped to ensure that the final excavation surfaces are
examined and to aid in the discovery of unanticipated adverse geologic conditions. The
mapping shall serve as a permanent record of the geologic conditions encountered during
construction.

For rocks that are prone to weathering and deterioration when exposed by excavation
processes, the Geotechnical Designer in collaboration with the CEG shall develop measures to
protect the rock surfaces to preserve the strength and character of the material.

Rock excavation may be done either by mechanical equipment; by using explosives in drill-and-
blast operations, or both. However, blasting shall not be allowed in urban areas unless
otherwise permitted per local building ordinances. If permitted, blasting of rock shall be
undertaken by controlled blasting techniques (cushion [trim], pre-splitting, smooth-wall
blasting, and line drilling). The Geotechnical Designer in collaboration with the CEG shall select
the rock excavation method to minimize vibration, over-breaks, fly rock and air blast. The
Contractor shall repair any blast and vibration induced damage.

10.C.5.1 Quality Assurance During Blasting Operation

The Geotechnical Designer in collaboration with the CEG shall do the following:

- Obtain copies of applicable codes, standards, regulations, and ordinances, and keep readily
  accessible copies at the project field office at times.
- Retain a blasting specialist who shall be responsible for supervision of field blasting
  operations and personnel, and have a minimum 15 years of blasting experience with 10
  years experience in responsible charge of blasting operations. Such a blasting specialist
  shall possess required federal, state and local licenses and/or permits.
- Prepare a blasting plan for the areas to be excavated by means of controlled blasting. The plan shall describe the necessary items to excavate the rock using the controlled blasting techniques selected by the Geotechnical Designer.

- The Blasting Plan shall be prepared and signed by the blasting specialist.

### 10.C.5.2 Damage Repair

Damage to existing structures or property caused by the blasting shall be repaired by the Contractor.

The Geotechnical Designer shall notify the Authority immediately of any blasting-induced damage.

### 10.C.5.3 Fly Rock Control

The Contractor shall control fly rock at all times during construction.

### 10.C.5.4 Notification

The Contractor shall notify each adjoining property owner, the Authority, local agencies where applicable, in writing, prior to each blast. Indicate the date and time of the proposed blast, and include any safety precautions required of the adjoining property owner.

### 10.C.5.5 Photography

Photographs shall be taken before, during, and at the end of construction of excavated surfaces. Photos shall be properly labeled with date, subject, direction of view, vantage point, and photographer.

### 10.C.6 References