California High-Speed Rail Authority

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Book III, Part A.1
Design Criteria Manual Changes

Chapter 4

Track Geometry

Revision	Date	Description
0	02 Mar 12	Initial Release, R0
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Acronyms

E / e	Equilibrium Superelevation
Ea	Actual Superelevation
Eu	Unbalanced Superelevation
HST	High-Speed Train
OCS	Overhead Contact System

4 Track Geometry

4.1 Scope

This chapter provides design criteria of geometric design requirements for mainline tracks, station tracks, yard tracks, turnouts, and crossovers on dedicated high-speed rail corridors of standard gauge (4'-8 ½").

4.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards. Applicable codes and regulations include but are not limited to the following:

- Code of Federal Regulations (CFR) Title 49, Part 213, Track Safety Standards
- California Public Utilities Commission (CPUC) General Orders (GOs) 26D and 118
- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering

4.3 Types of Rail Corridors

4.3.1 Dedicated High-Speed Rail Corridors

Dedicated High-Speed Rail Corridors are segments of right of way within the High-Speed Train (HST) System where tracks are used exclusively for HST operations, designated as such in the operating rules and where the main tracks are physically separated from other railroad tracks. There is no operation of freight trains or other passenger trains within these corridors. The operation of trains and equipment used for Maintenance of Infrastructure work is permitted in these corridors.

4.3.2 Shared Corridors

Shared Corridors are segments along the HST System where the HST right of way is shared with other transportation system(s) including highway, freight or passenger rail.

Where HST tracks are shared with other passenger trains, design criteria for the maximum practicable design speed shall be used. At locations where tracks are shared with freight trains, the alignment standards for freight operations shall be checked and the more stringent criteria shall be applied.

4.4 Horizontal Alignment

Alignments for HST operation shall be designed to minimize the use of curves and to permit the maximum practical design speed.

When curves are used, the largest practical radii shall be used. Where the maximum design speed cannot be achieved, the highest achievable speed shall be used to define the geometry of the alignment.

The horizontal alignment shall be developed along track centerlines. It shall consist of tangents and circular curves connected by transition spirals of appropriate lengths.

When possible, double track alignment shall be designed with a constant distance in between track centerlines. Segments along straight line tracks shall be parallel and circular curves on adjacent, parallel tracks shall be concentric.

4.4.1 Selection of Design Speed

The speed to be used for the design of the alignment shall be the system design speed, not the operating speed, planned to be used at the time of start of operations. The purpose of determining design speed is to find the appropriate superelevation and spiral length for a particular curve in the alignment. The highest anticipated speed, superelevation, and unbalanced superelevation shall be used.

The maximum design speed for a curve shall be the same throughout the entire length of the curve from tangent points. Separate design speeds shall not be used for separate portions of a curve. If a speed limitation exists for any segment of the curve, then the design speed for the entire curve shall be the lower speed.

Refer to the *General* chapter for maximum allowed design and operating speeds.

4.4.2 Minimum Lengths of Alignment Segments

The minimum allowed segment length (L), in feet, shall be calculated by the following formula:

$$L = V \times 44/30 \times t$$

Where:

V = design speed (miles per hour)

t = attenuation time (seconds)

t ≥ 2.4 seconds (Recommended)

1.8 seconds (Minimum)

1.0 seconds (on diverging route of turnouts)

Minimum segment lengths shall apply to horizontal and vertical alignment segments. Where alignment segments overlap, each change shall be treated as a separate alignment element for the purpose of calculating minimum segment lengths. See Section 4.6, Combined Horizontal and Vertical Curves for further information. The segment length requirement will govern only where other design considerations for the individual alignment elements do not require longer segment lengths.

Minimum segment lengths for various design speeds are presented in Table 4-1. Additional values, for design speeds not shown, can be obtained from the formula provided in this section, rounded up to the nearest integer.

Table 4-1: Minimum Segment Lengths at Various Speeds

Design Speed (miles per hour)	Minimum Segment Lengths (in feet) for times of		
	2.4 seconds	1.8 seconds	1.0 seconds
250	880	660	367
220	774	581	323
200	704	528	293
175	616	462	257
150	528	396	220
125	440	330	183
110	387	290	161
90	317	238	132

4.4.3 Minimum Radii

The minimum allowed curve radius shall be derived from the following formula:

$$R = 4V^2_{max} / (Ea + Eu)$$

Where:

R = Radius (feet)

V_{max} = Maximum design speed (miles per hour)

Ea = Actual superelevation (inches) Ea_{max}= 6 inches

Eu = Unbalanced superelevation (inches) Eu_{max}= 3 inches

Table 4-2 presents minimum values of curve radii for various design speeds. When possible, recommended values shall be used. Additional curve radii for design speeds not shown on Table 4-2 can be calculated with the formula provided above, using Ea= 6 inches for Minimum values and Ea= 3 inches as Recommended values. The minimum curve radius for tracks located outside the perimeter of the yards shall not be less than the value specified in Section 4.14.

Table 4-2: Recommended and Minimum Curve Radii

Design Speed (miles per hour)	Minimum Radius based on Superelevation Limits	
	Recommended (feet)	Minimum (feet)
250	45,000	28,000
220	35,000	22,000
200	30,000	18,000
175	22,000	14,000
150	16,000	10,000
125	10,500	7,000
110	8,100	5,400
90	5,500	3,600

4.4.4 Curves with Small Central Angles

For small central angles the radius shall be sufficiently large to provide the time-based minimum arc and spiral segment lengths. There is no limitation on maximum acceptable curve radius. In general, larger radii are preferable to smaller radii as the superelevation and unbalance values become smaller as radius increases. It is desirable that the radius selected results in the length of the simple curve portion being about equal to or longer than the length of spiral. Since each portion is an alignment segment, if each segment is equal in length, the entire curve with spirals should have a minimum length not less than three times the Minimum Segment Length for the design speed of the curve. Double (back-to-back) spirals or curves with long spirals and short arc lengths shall not be used.

4.4.5 Superelevation

Superelevation is the maximum difference in height between outer and inner rails on curved track, measured at the center of the rail head surface. Superelevation is used to counteract, or partially counteract, the centrifugal force acting radially outward on a train when it is traveling along the curve. A state of equilibrium is reached when the centrifugal force acting on a train is equal to the counteracting force pulling on a train by gravity along the superelevated plane of the track.

4.4.5.1 Equilibrium (Balanced) Superelevation

Equilibrium superelevation (E) may be derived by the simplified formula:

$$E = 4.0\ V^2 / R$$

Where:

E = Equilibrium superelevation (inches)

V = Design speed (miles per hour)

R = Radius of curve (feet)

E is also expressed as:

$$E = Ea+Eu$$

Where:

Ea = actual superelevation (inches)

Eu = unbalanced superelevation (inches)

Thus: $E = (Ea+Eu) = 4.0\ V^2 / R$

4.4.5.2 Actual Superelevation

Actual superelevation (Ea) shall be accomplished by maintaining the top of the inside (or low) rail at the “top of rail profile” while raising the outside (or high) rail by the amount of the Ea. The inside rail is designated as the “grade rail” and the outside rail is designated as the “line rail”.

The Ea shall be determined to the nearest 1/4 inch by the formulas above. For any curve calculation on the main track which yields less than 1/4 inch of required superelevation, 1/4 inch shall be specified.

Curves within special trackwork shall not be superelevated. Yard tracks and other low speed tracks on which trains or equipment will normally be stationary for long periods shall not be superelevated. Yard lead tracks and other running tracks shall be superelevated as described in the discussion of those type tracks.

It is recommended that the Ea be limited to 6 inches.

4.4.5.3 Unbalanced Superelevation

Unbalanced superelevation (Eu), also referred to as cant deficiency, is the amount of superelevation not applied to the curve. Eu can also be defined as the difference between the equilibrium superelevation (E) and the Ea.

$$Eu=E-Ea$$

Where:

Ea = actual superelevation that is applied to the curve

Eu = unbalanced superelevation

The maximum Eu shall be limited to 3 inches.

4.4.5.4 Ride Quality and Superelevation

Ride quality on curves is determined by the amount of lateral acceleration which in curve design is expressed as Eu. Curves shall not be superelevated to balance the design speed, the calculated average speed, or the maximum operating speed. Eu values shall be kept between 1 and 3 inches for ride comfort and smooth running of the vehicles through curves.

- Minimum Eu shall be 1.0 inch, except where Ea+Eu is less than 2.0 inches, in which case Ea and Eu shall be set to be approximately equal.
- Maximum Eu shall be 3.0 inches, based on a lateral acceleration limit of 0.05g.

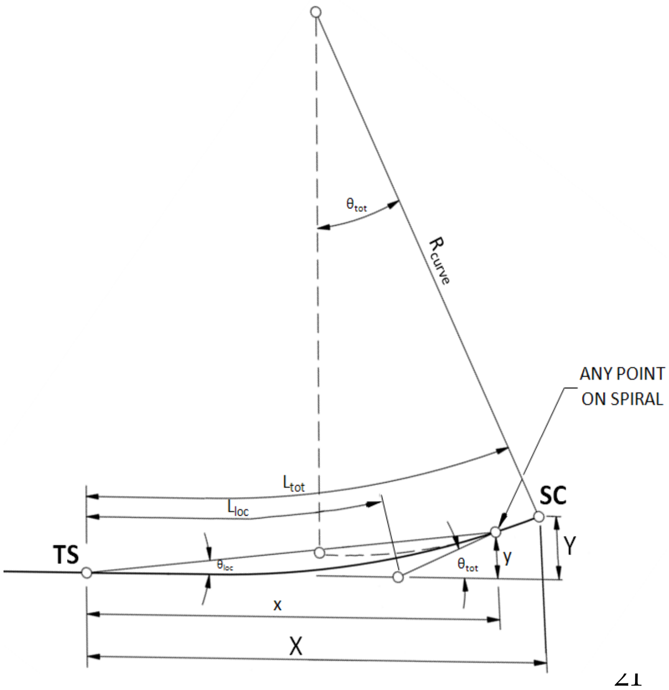
4.4.6 Spiral Curves

Spiral curves shall be used to transition from tangent tracks to tracks on circular curves and to gradually develop full track superelevation. Figure 4-1 illustrates the geometry of spiral transition curves.

Spiral transition curves used in high-speed track alignment shall be of the following types:

- Half-Sine spiral curves (variable rate transitions)
- Clothoid spiral curves (constant rate transitions)

Figure 4-1: Spiral Curves Definition



Where:

TS	Tangent Spiral point: the point of change from tangent to spiral
SC	Spiral Curve point: the point of change from spiral to circular curve
R_{curve}	Radius of circular curve, in feet
L_{loc}	Spiral length from TS to a specific location.
L_{tot}	Total length of spiral from TS to SC (or SCS) in feet
x =	Distance from TC point to any point on the curve, measured along the extended initial tangent
X=	Total x at the end of the transition curve from TS to SC
y=	Tangent offset distance to any point along the spiral, measured perpendicular to the extended initial tangent.
Y =	Tangent offset of the SC point.
θ_{loc} =	Spiral angle at any point along the spiral
θ_{tot} =	Total spiral angle

4.4.6.1 Half-Sine Spirals

Half-Sine spirals (also known as Sine Half-Wavelength Diminishing Tangent Curves and Cosine Spirals) provide a variable rate of change in curvature between the tangent and circular curved track. Half-Sine spirals shall be used on:

- All curves along HST mainline tracks
- Curves having design speeds of 110 mph or more
- Curves associated with turnouts having design maximum speed of 110 mph or more

Half-Sine spirals are defined by the following formulas (angles in these formulae are in radians):

$$y=\frac{X^2}{R_{curve}}\left[\frac{\alpha^2}{4}-\frac{1}{2\pi^2}\left\{1-\cos\alpha\pi\right\}\right] \quad \alpha=\frac{x}{X}$$

$$R_{loc}=\frac{2R_{curve}}{\left[1-\cos\left(\pi\frac{L_{loc}}{L_{tot}}\right)\right]}$$

$$Ea_{loc}=0.5Ea_{curve}\left[1-\cos\left(\pi\frac{L_{loc}}{L_{tot}}\right)\right]$$

Where:

Ea_{loc}= Variable actual superelevation at a specific location along the spiral, in inches ($Ea_{loc}=Ea_{curve}$ at the SC location)

Ea_{curve}= Actual superelevation at the SC and throughout the circular curve, in inches

4.4.6.2 Clothoid Spirals

Clothoid spirals provide a constant rate of change in curvature between the tangent and the connecting circular curve. Clothoid spirals shall be used on tracks having design speed lower than 110 mph. Clothoid spirals may be used on large radius curves that require small amounts or no superelevation and small unbalanced superelevation.

Clothoid spiral are defined by the following formulas:

$$\theta_{loc} = \frac{L_{loc}^2}{2R_{curve}L_{tot}}$$

$$R_{loc} = \frac{R_{curve}}{\left(\frac{L_{loc}}{L_{tot}}\right)}$$

$$Ea_{loc} = Ea_{curve}\left(\frac{L_{loc}}{L_{tot}}\right)$$

4.4.6.3 Spiral Lengths

The length of the spirals shall be the longest length determined by calculating the length requirements per Table 4-3. These lengths are the following:

- Length determined by allowed rate of change in superelevation, which controls the speed of car rotation around the track centerline (roll).
- Length determined by allowed rate of change in Eu, which controls the acceleration caused by centrifugal force not balanced by the Ea (lateral jerk).
- Length determined by limitation on twisting over the vehicle body.
- Length needed to achieve Attenuation Time

Table 4-3: Recommended and Minimum Length of Spiral (Ls)

Half-Sine (Variable Change) Spirals (1)		
Spiral Design Factor	**Recommended**	**Minimum**
Superelevation	1.63 Ea V	1.30 Ea V
Unbalance	2.10 Eu V	1.57 Eu V
Twist (2)	140 Ea	118 Ea
Minimum Segment	2.64 V	2.20 V
Clothoid (Linear Change) Spirals		
Superelevation	1.47 Ea V	1.17 Ea V
Unbalance	1.63 Eu V	1.22 Eu V
Twist	90 Ea	75 Ea
Minimum Segment	2.64 V	2.20 V

Notes:
(1) Longer lengths of half-sine spirals are due to the variability in the ramp rate.
(2) Provides maximum twist rates identical to the twist rate of the clothoids.

Where:
Ls= Spiral length (feet)
Ea = Actual elevation (inches)
Eu = Unbalanced elevation (inches)
V = maximum speed of the train (mph)

After calculation and selection of length, based on the governing requirement, the spiral length should then be rounded up to a convenient value for further calculation and use in the alignment.

4.4.6.4 Special Situations

Spirals on Large Radius Curves – Clothoid spirals may be used instead of half-sine spirals regardless of track type or design speed if the following conditions are met: The required superelevation and unbalanced superelevation are both under 1.0 inch at the maximum design speed; and the "Minimum Segment" length for the spiral is more than twice the length required by any other factor.

Spirals may be omitted if the following conditions are met:

- The required superelevation is zero (balancing superelevation for the maximum speed less than 0.5 inches); and
- The calculated offset of the curve due to application of the spiral is less than 0.05 feet in ballasted track or less than 0.02 feet in non-ballasted track.

Reverse Curves – Reverse curves shall only be allowed when there is insufficient distance between spiral curves to provide the minimum required length of tangent segment. In these cases, the spirals shall be extended to provide a reversing curve.

Compound Curves – Compound curves shall not be used on mainline tracks.

4.5 Vertical Alignment

The vertical alignment is defined as the top of rail profile grade. In curves with superelevation, the vertical alignment is the top of the low rail.

Vertical alignment shall be designed to have the smoothest practical profile while optimizing earthwork, structures, tunnels, and drainage. Use of multiple short grades and multiple changes in grade within any particular change of elevation ("sawtooth profiles") shall be avoided to the extent practical. In addition to increasing operational costs and difficulty by requiring frequent changes in power, a line with multiple changes in grade is aesthetically unappealing. As a check on the reasonableness of the profile developed, it shall be drawn up at a highly condensed horizontal scale so that the vertical changes are exaggerated, otherwise, the alignment can appear deceptively smooth. Changes in top of rail profile gradients shall be connected by vertical curves.

4.5.1 Grades

Grades are expressed in absolute values. Grades shall be as low as practical. In areas of relatively flat terrain, the grades should not exceed the recommended values per Table 4-4. In mountainous terrain, grades should be minimized in order to maximize operating efficiency which most often means lower gradients than the surrounding terrain.

The average grade over any 6.0 miles of line should not exceed 2.5 percent.

Maximum gradient shall not exceed 2.5 percent on ballasted track and 3.5 percent on non-ballasted track. When these limit values are used, the low end of the grade shall not be less than 2.0 miles beyond the end of a passenger station platform.

Maximum gradient through passenger station platform shall be 0.25 percent.

Minimum gradient through cuts, tunnels, and trenches shall be 0.25 percent.

Maximum segment length of continuous 3.5% grade shall not exceed 20,000 feet.

In areas occupied by turnouts and other special trackwork, grades up to 1.75 percent in ballasted track and 3.50 percent in non-ballasted track may be used where the use of lower grades would result in the requirement for lower speed turnouts.

For grade limitation at phase breaks, refer to the *Traction Power Supply System* chapter.

Table 4-4: Recommended and Maximum Grades

Track type and conditions	Recommended	Maximum
Ballasted	1.25%	2.50%
Non-ballasted	1.25%	3.50%
Ballasted track through turnouts and other special trackwork	0.50%	1.75%
Non-ballasted track through turnouts and other special trackwork	1.25%	3.50%
Mainline tracks through Station Platforms	0%	0.25%

4.5.2 Vertical Curves

Vertical Curves shall be Parabolic. The length of vertical curves shall be rounded up to nearest 100-foot increment where practical.

4.5.2.1 Vertical Curve Acceleration Rates

The acceleration value to be used for vertical curves shall not exceed 0.90 ft/sec^2.

4.5.2.2 Minimum Vertical Curve Lengths (L_{VC})

The minimum vertical curve lengths (L_{VC}), in feet, on lines carrying HSTs only shall be the longer of the following:

$$L_{VC} = 3.5\ V \text{ or } L_{VC} = 2.15\ V^2\,(\Delta\%/100)\ /\ 0.90\ \text{ft/sec}^2\text{, but not less than } 200\ \Delta\ \%$$

Where:

V = Design speed (miles per hour)

Δ% = algebraic difference of the gradients (in %)

4.5.2.3 Vertical Curves in Shared Corridors

Where HST tracks closely parallel lines for other passenger or freight trains such that a common profile is desirable, the longest vertical curve length determined by separate calculation for each type of traffic shall determine the vertical curve length to be used for all tracks. The length of vertical curve for the other systems shall be based on the standards of the systems involved.

4.6 Combined Horizontal and Vertical Curves

Horizontal and vertical curves may overlap. It is preferred to avoid overlap of vertical curves and spiral curves. Overlaps may be used if this consideration causes an increase in cost, increases the height of fill or aerial structures, or results in other aspects of the alignment being reduced below recommended values. For example, when there is a vertical curve within the body of a horizontal curve, the parts of the horizontal curve outside of the vertical curve will be treated as separate segments when calculating segment lengths. Horizontal and vertical

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segment ends may coincide if it is not practical to separate them by the minimum segment length distance.

4.7 High-Speed Turnouts (60 mph and faster)

Turnout geometries are presented for the following speeds: 60 mph, 80, mph, 110 mph, and 150 mph. The requirements of this section are limited to geometric considerations only. Track components for turnouts and other special trackwork can be found in the *Trackwork c*hapter. Other spatial considerations, including distance between track centers and space beside and around turnouts can be found in the *Trackway Clearances* chapter.

High-speed turnout and crossover designs are based on the following criteria:

- Eu not to exceed 3 inches
- Minimum time over any turnout segment or curve connected to a turnout, including spirals on the frog end of turnouts and spirals into a curve on the diverging track that is adjacent to the turnout, about 1.0 second
- Maximum Virtual Transition Rate at switch point: 4.5 inches/second
- Ratio of entry radius to turnout body radius: Not less than 2:1.
- Curved frogs
- Spirals shall be kept out of frogs

Figure 4-2: High-Speed Turnouts

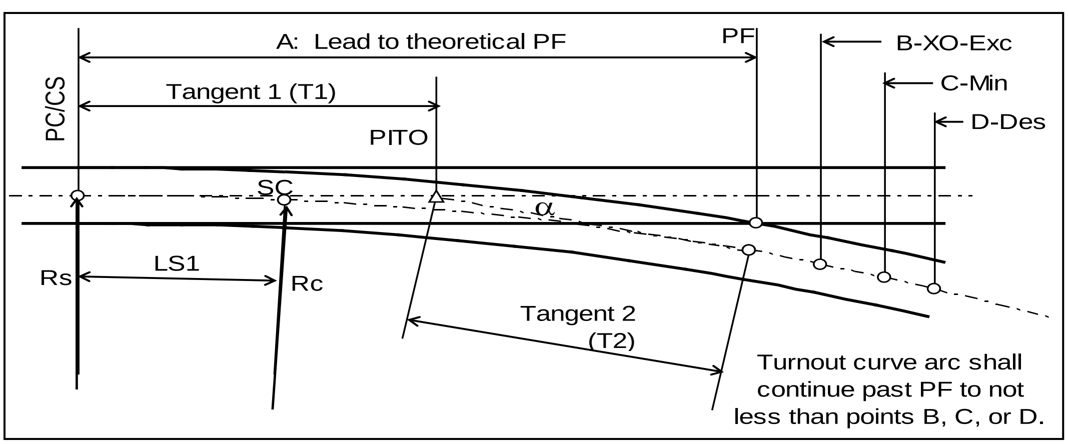


Table 4-5: High-Speed Turnouts

Geometry of Turnout and its Segments, in feet unless stated otherwise				
Design Speed	60 mph	80 mph	110 mph	150 mph
Turnout Entry Radius	10,000.00	18,000.00	34,000.00	80,000.00
Turnout Body Radius	5,000.00	9,000.00	17,000.00	32,000.00
Switch Spiral Length and Desirable Frog End Spiral Length	90.00	120.00	160.00	220.00
A. Distance to Theoretical Point of Frog (Zero Point, also called Fine Point)	237.53	318.53	436.76	610.07
Angle at Theoretical Point of Frog	2d27m49s	1d 50m12s	1d20m14s	0d58m27s
Derived Frog Number (AREMA method)	23.25	31.2	42.8	58.8
Tangent 1 (T1)	128.06	171.67	333.14	461.99
Tangent 2 (T2)	109.48	146.87	276.93	363.30
Turnout Body Curve Arc Length, SC to PF	147.50	198.51	276.74	375.18
B. Distance to point of 5.85 ft. separation	262.62	352.18	482.98	673.52
C. Distance to point of 7.00 ft. separation	285.48	382.85	525.11	731.34
D. Distance to point of 8.00 ft. separation	303.85	407.49	558.97	777.81

Notes:
Values in table are for illustration purposes, and so are generally given to 2 decimal places. This is not to be construed as the necessary limit for the alignment calculations.

To provide for future OCS design and construction, sufficient distance is required between:

1. two adjacent points of switches of adjacent universal crossovers
2. point of switch of turnout and adjacent point of switch of crossover

The preferred distance between adjacent switch points along the main tracks is 1,400 feet. The minimum distance between adjacent switch points along the main track is 1,000 feet. Placement of high-speed turnouts in relation to alignment features shall be based on 1.0 seconds of run time of the slower alignment element, whether another turnout or the end of a spiral or vertical curve.

4.8 Low and Medium Speed Turnouts (55 mph and slower)

Turnouts to storage and refuge tracks, yard connection tracks, and within yards and any other low and medium speed locations shall use AREMA standard frogs. The standard turnout sizes to be used shall be Numbers: 9, 11, 15, and 20.

Number 11 turnouts shall be used as the standard yard turnout, and as the minimum size turnout to be installed in main tracks with speeds of 125 mph or less and in station tracks. Main line turnouts to yard Leads or other tracks shall be no less than Number 20 turnouts if the conditions allow it. Turnouts from station tracks to stub end storage tracks shall be Number 11.

Number 9 turnouts may be used in yard tracks where geometric constrains make the use of Number 11 turnout impractical.

Figure 4-3: Low and Medium Speed Turnouts

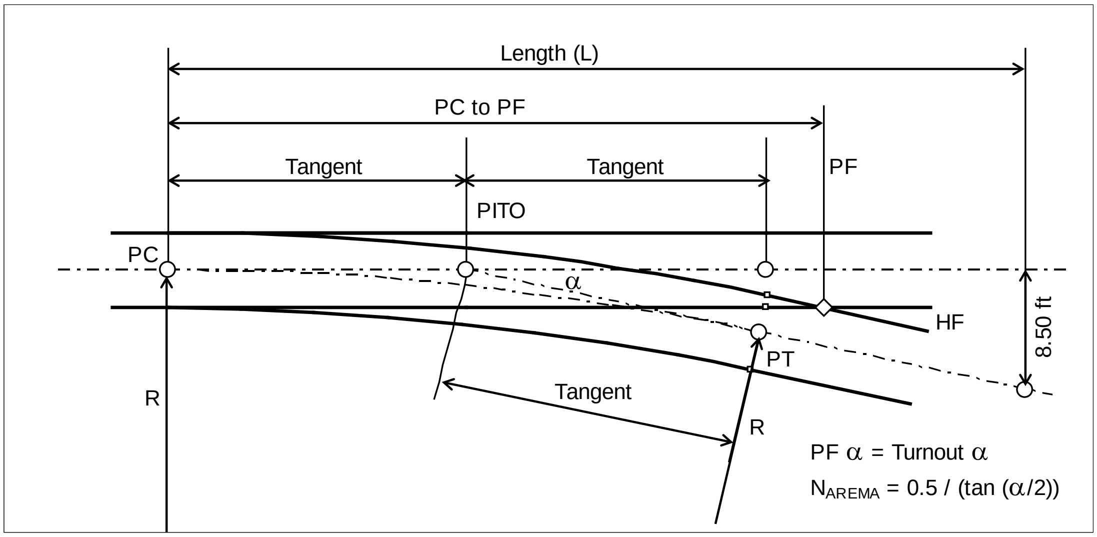


Table 4-6: Low and Medium Speed Turnouts

Number	9	11	15	20	24
Defined Angle (degrees/minutes/seconds)	6d21m35s	5d12m18s	3d49m06s	2d51m51s	2d23m13s
Radius	620 feet	950 feet	1750 feet	3250 feet	4650 feet
Tangent	34.44 feet	43.18 feet	58.33 feet	81.25 feet	96.87 feet
Lead, PC to ½ inch PF	77.19 feet	95.43 feet	129.58 feet	176.25 feet	210.87 feet
PC to 8.5 feet separation	110.71 feet	136.49 feet	185.69 feet	251.14 feet	300.79 feet
Tangent Rail, ½ inch PF to Curve PT	8.31 feet	9.07 feet	12.92 feet	13.75 feet	17.13 feet
Maximum Diverging Speed	20 mph	25 mph	35 mph	45 mph	55 mph
Unbalance at Max. Diverging Speed	2.58 inches	2.63 inches	2.80 inches	2.49 inches	2.61 inches

Notes:
Values in table are for illustration purposes and so are generally given to 2 decimal places. This is not to be construed as the necessary limit for the alignment calculations.

Spatial considerations, including space beside and around turnouts can be found in the *Trackway Clearances* chapter. For track components, see the *Trackwork* chapter.

To provide for future OCS design and construction, sufficient distance is required between:

1. two adjacent points of switches of adjacent universal crossovers
2. point of switch of turnout and adjacent point of switch of crossover

The preferred distance between adjacent switch points along the main tracks is 600 feet. The minimum distance between adjacent switch points along the main track is 400 feet. This requirement does not apply for the yard turnouts. Run time considerations are not relevant to the location and spacing of low and medium speed turnouts. Vehicle twist and relative end offsets are the controlling factors. It is recommended to provide at least 75.00 feet of straight track in advance of a switch. Where practical, these turnouts shall be spaced so that the length between turnouts is at least equal to the sum of vehicle truck centers plus one end overhang. Where the usage of switches that are point to point is such that trains are unlikely to use both turnouts, the switch points may be placed closer, down to 30 feet apart. It is desirable that the track off the frog end of the turnout be straight to at least the end of the switch tie set, which may be taken as the point at which the tracks are 8.50 feet apart. In the development of crossovers, track ladders, and track fans, it will be seen that these values are not always achievable.

4.9 Non-Standard Turnouts

Turnouts on curves or in locations where standard turnouts cannot be used shall be designed as special cases. These turnouts shall be designed such that the lateral forces and rates of change in these forces are similar to those in standard design turnouts.

For all turnouts, the Eu shall not exceed 3.0 inches on either side of the turnout.

For high-speed turnouts, the following governs:

- Switch end spiral having a transition rate not more than 4.5 inches per second
- Eu at the point of switch: 1.5 inches
- If the curve does not continue beyond the turnout on the frog end, a frog end spiral having a transition rate of not more than 4.5 inches per second shall be applied.
- Minimum time over any turnout segment or curve connected to a turnout shall be approximately 1.0 second, and not less than 0.9 seconds.

For low and medium speed turnouts, compound internal curves shall not be used. If a curved frog is used, the end of the curve shall be outside the casting portion of the frog.

4.10 High-Speed Crossovers

Crossovers in high-speed turnouts are more complex, as the curve continues through the frog. In order to place crossovers for 60 mph or faster between tracks at the standard track center spacing of 16.50 feet, the frog end spiral must be shortened to keep the spiral out of the frog. The length of the 2 spirals combined achieves the minimum 1.0 second run time when they are considered as 1 design element. Figure 4-4 shows the normal relationship between crossover components in a crossover between 16.50 feet track centers.

Figure 4-4: High-Speed Crossovers

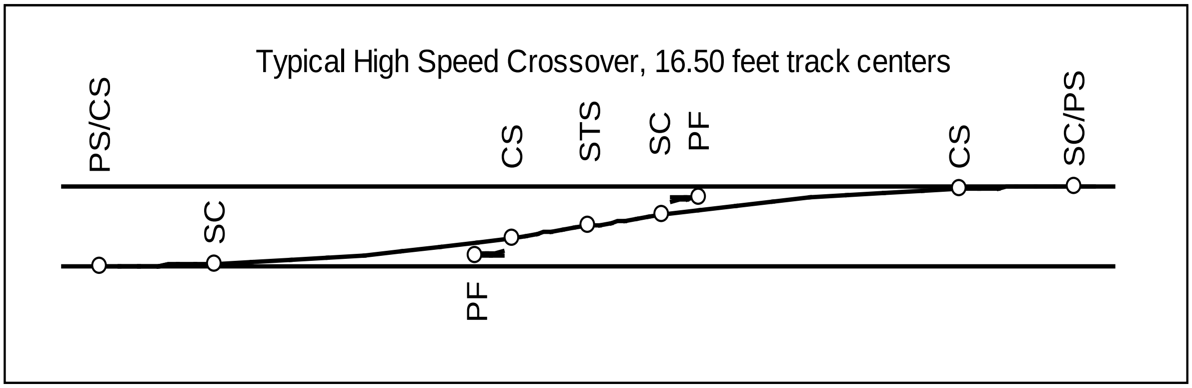


Table 4-7: High-Speed Crossovers – 16.50 feet Track Centers

Geometry of Turnout and its Segments, in feet unless stated otherwise				
Design Speed	60 mph	80 mph	110 mph	150 mph
Track Centers	16.50	16.50	16.50	16.50
Total Length along main track	618.74	829.97	1,138.63	1,583.92
Total Length along Crossover Track	619.05	830.20	1,138.80	1,584.04
Turnout Entry Radius	10,000.00	18,000.00	34,000.00	80,000.00
Turnout Body Radius	5,000.00	9,000.00	17,000.00	32,000.00
Switch Spiral Length	90.00	120.00	160.00	220.00
Frog Spiral Length	45.00	62.00	85.00	115.00
Angle at STS	3d01m31s	2d 15m15s	1d38m28s	1d11m49s
Length of Entry Curve	0.00	0.00	0.00	0.00
Length of Turnout Body Curve	173.52	233.10	324.40	457.02

Notes:
Values in table are for illustration purposes and so are generally given to 2 decimal places. This is not to be construed as the necessary limit for the alignment calculations.

For high-speed crossovers between track centers of between 16.50 feet and 21.50 feet, longer spirals between turnouts may be used, but with the limitation that they be kept out of the frog. Where the track centers are 21.50 feet or greater, a full length spiral shall be used. High-speed crossovers shall not be used between tracks having track centers of under 16.50. For other spatial requirements see the *Trackway Clearances* chapter.

4.11 Low and Medium Speed Crossovers

The essence of a crossover is 2 turnouts connected at their frog ends. This occurrence is common. The distance of concern in crossovers is the central tangent, shown as “Tangent” in

Figure 4-5. For close track centers and small turnout numbers, this distance can be less than the truck centers plus one end overhang that is the minimum tangent distance between reversing curves.

Figure 4-5: Low and Medium Speed Crossovers

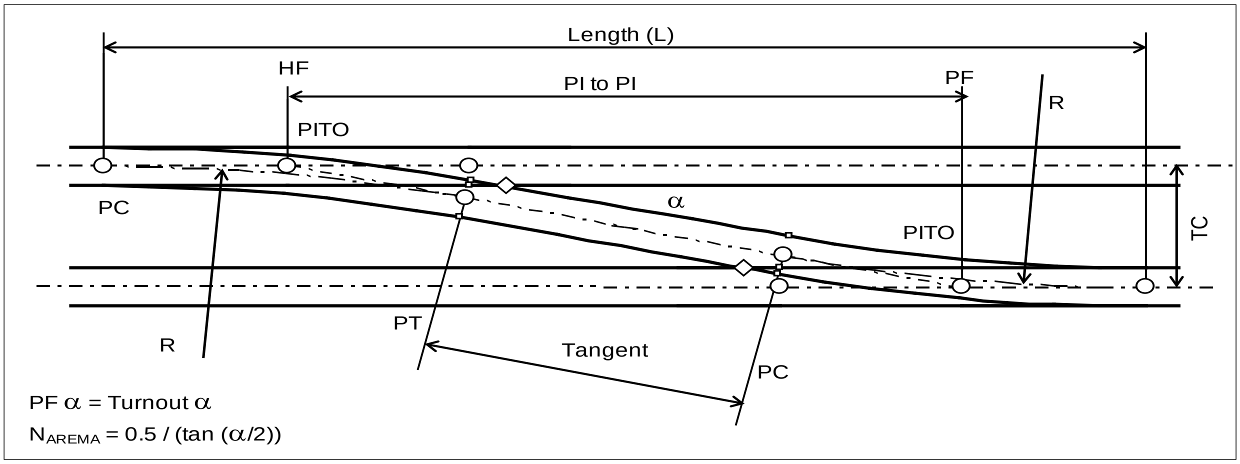


Table 4-8: Low and Medium Speed Crossovers

Number	9	11	15	20	24
Defined Angle	6d21m35s	5d12m18s	3d49m06s	2d51m51s	2d23m13s
Radius	620 feet	950 feet	1750 feet	3250 feet	4650 feet
Allowed Speed	20 mph	25 mph	35 mph	45 mph	55 mph
Length (L) end to end of crossover, 15.00 feet track centers	203.47 feet	251.02 feet	341.42 feet	462.31 feet	553.60 feet
PITO to PITO distance on tangent, 15.00 feet track centers	134.58 feet	164.66 feet	224.75 feet	299.82 feet	359.85 feet
Change in length per 1.00 foot change in track centers, either of the above	8.972 feet	10.978 feet	14.983 feet	19.988 feet	23.990 feet
Tangent length on diagonal, 15.00 feet track centers	66.53 feet	78.98 feet	108.58 feet	137.69 feet	166.41 feet
Change in length per 1.00 foot change in track centers	9.03 feet	11.02 feet	15.02 feet	20.01 feet	24.01 feet

Notes:

Values in table are for illustration purposes and so are generally given to 2 decimal places. This is not to be construed as the necessary limit for the alignment calculations.

Since small radii curves in turnouts result in short component life and working the equipment to near its limits of movement is undesirable, it is recommended that the turnouts in crossover be Number 11 or larger. It is also recommended to keep the track centers at 15.00 feet or larger for this and other reasons.

4.12 Double Crossovers (Scissors Crossovers)

Where space is constrained and crossovers allowing universal moves are desired, crossovers may be overlapped to form a double crossover. This form of crossover is sometimes called a scissors crossover, as on some systems the term "double crossover" means two single crossovers of opposite hand placed in succession.

Double (scissors) crossovers shall be used only where their use keeps other aspects of the alignment from being reduced to less than minimum values due to their high cost and maintenance requirements. Double crossovers using high-speed turnouts shall not be used unless the track centers are wide enough that the crossing diamond may be straight, and preferably where the crossing angle is equal to or less than that in a Number 15 double crossover.

The following double crossovers may be used:

- Number 9 (Shall not be used in main tracks)
- Number 11 at 15.00 feet or larger track centers (Shall not be used in main tracks)
- Number 15 at 15.00 feet or larger track centers

Double (scissors) crossovers with frog angles larger than that of a Number 15 turnout require movable center frogs, and therefore should be used only where use of smaller crossovers affects run time.

4.13 Track Layout along Station Platforms

Tracks along passenger platforms shall be of sufficient length to avoid delay of through trains by trains making the station stop. Turnouts connecting the platform track with the main track shall permit speeds not less than the train would be running if decelerating to or accelerating from the station stop. Other than the main line turnouts, the normal train operation into or out of the platform shall not pass through the curved side of turnouts.

Platform tracks shall be tangent through the platform length and to a distance of not less than 75 feet beyond the ends of the platform~~.~~ If the platform track must be curved, the largest practical radius of curve shall be used, and other means used to provide for accessibility in accordance with Americans with Disabilities Act (ADA) requirements.

Other tracks connected to platform tracks shall turn out of the tangent portion of the platform tracks. Turnouts shall be placed not less than 75 feet beyond the ends of the platform.

See Standard Drawings for Typical Station Connection Tracks Layouts.

Figure 4-6: Detail of Station Entry/Exit High-Speed Turnout and Return Curve

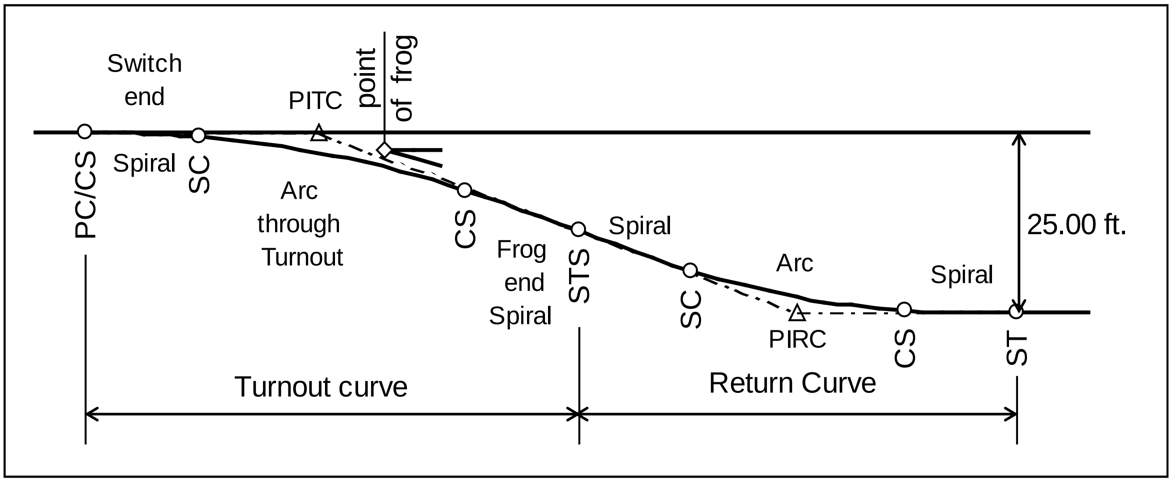


Table 4-9: Geometry of Station Entry/Exit High-Speed Turnouts and Return Curves

Geometry of Connection and its Segments, in feet unless stated otherwise			
Design Speed	60 mph	80 mph	110 mph
Platform Track Offset	25.00	25.00	25.00
Turnout Entry Radius	10,000.00	18,000.00	34,000.00
Turnout Body Radius	5,000.00	9,000.00	17,000.00
Switch Spiral Length	90.00	120.00	160.00
Frog Spiral Length	90.00	120.00	160.00
Return Curve Radius	4,000.00	7,000.00	13,500.00
Curve Spiral Length	90.00	120.00	160.00
Total Length along main track	743.65	991.80	1,364.60
Total Length along Platform Track	744.25	992.25	1,364.92
Angle at STS	3d44m07s	2d 48m04s	2d02m17s
Length of Entry Curve	0.000	0.000	0.000
Length of Turnout Body Curve	213.47	290.02	404.71
Length of Return Curve	170.78	222.24	320.21

Notes:
Values in table are for illustration purposes and so are generally given to 2 decimal places. This is not to be construed as the necessary limit for the alignment calculations.

The turnouts for storage and refuge tracks at passenger stations will depend upon the operational requirements. Turnouts smaller than Number 11 shall not be used. Spirals need not be applied to the return curve for a stub end track. If the track is for yard access instead of to storage, a spiral appropriate to the design speed of the access track shall be applied.

4.14 Access Tracks to Yards and Maintenance Facilities

The criteria contained in this section are intended for the geometric design of tracks connecting the mainline to yards, Maintenance of Infrastructure (MOI), terminal layup and storage facilities. See section 4.13 for the geometric design of tracks connecting the mainline to station platforms.

Site constraints may lead to large distances between mainline access points and these facilities. For the purpose of minimizing time required to clear revenue tracks, these tracks shall be designed much like a secondary mainline railroad. The design speeds of the turnouts that are used between the mainline and these tracks shall be 60 mph. The design speed of crossovers between main tracks associated with these turnouts shall be 60 mph unless they also serve another purpose that requires a higher speed.

The minimum length between mainline turnout fouling point and the first yard or MOI turnout shall be not less than 1600 feet. The following are the minimum/maximum design parameters for these tracks

- Design speed: 60 mph, site conditions permitting. Where conditions do not permit 60 mph, a lower design speed may be used. This lower design speed shall be as high as site conditions permit.
- Minimum Curve Radii: 900 feet
- Maximum Actual Superelevation (E_a): 3 inches
- Maximum Unbalanced Superelevation (E_u): 3 inches
- Spiral Lengths (Clothoid): 62 feet per inch of superelevation or unbalanced superelevation, whichever gives the greatest length
- Minimum Length of Tangent between curves in the same direction: 0 feet. Compound curves must be joined by spirals of length equal to 62 feet per inch of change in superelevation or unbalance, whichever gives the greater length
- Minimum Length of Tangent between reversing curves. The length may be reduced by one-half the combined lengths of the adjacent spirals. $L_{min} = 9{,}400{,}000 / (R_1)^2 + 9{,}400{,}000 / (R_2)^2$, but not less than 40 feet
- Recommended Turnouts: not less than Number 15
- Minimum Turnouts: Number 11

- Minimum Track Centers, not including allowance for Overhead Contact System (OCS) poles, drainage, walkways, roadways, or other facilities that will be placed between tracks in some areas: 15.00 feet
- Minimum Track Centers on small radius curves may need to be larger than the values given above. If the following calculation results in a larger value, this value shall be used: 14.75 + 1,100 / Radius (in feet), but not less than 15.00 feet
- Maximum Grade: 2.50 percent
- Vertical Curves: 100 feet minimum length with a rate of change of not more than 1.00 percent per 100 feet

4.15 Yards Tracks

The specific track arrangement for each yard will depend upon the purpose of the yard and tracks in the yard. Therefore the basic layout will be determined by operational requirements. The requirements developed in this chapter are therefore limited to those of a general nature except for those relating to geometric constraints due to:

- Curvature related constraints due to vehicle characteristics
- Track length constraints due to train and individual vehicle length
- Profile and grade related issues

Other than the tracks connecting the yards to the revenue tracks, the design parameters for these tracks are speed-independent.

4.15.1 Connecting and Switching Tracks Inside Yards

The following standards apply to tracks on which trains will not be stored or left standing but are installed for the purpose of connections between yard tracks and yard access tracks within the area designated as yards, all types, and other low speed tracks.

- Minimum Curve Radii: 620 feet
- Minimum Length of Tangent between curves in the same direction: 0 feet (compound curve)
- Minimum Length of Tangent between reversing curves: $L_{min} = 9{,}400{,}000 / (R_1)^2 + 9{,}400{,}000 / (R_2)^2$, but not less than 40 feet.
- Minimum Turnout Number: 9 (internal radius 620 feet). If in a track with high volume traffic, the minimum shall be a Number 11.
- Minimum Track Centers: 15.00 feet
  - On small radius curves, minimum track centers shall be increased if the following calculation results in a larger value: 14.75 + 1,100 / Radius (in feet), but not less than

15.00 feet. This value does not include allowance for OCS poles, drainage, walkways, roadways, or other facilities that will be located between tracks.

- Track centers shall be increased for OCS poles, light poles, drainage, signal masts, equipment cases, walkways, service aisles or other facilities placed between tracks.Maximum Grade: 2.50 percent
- Minimum Length of Vertical Curve: 50 feet with a rate of change of not more than 2.00 percent per 100 feet.

For additional criteria on walkways and service aisles see *Civil* chapter.

4.15.2 Servicing and Storage Tracks

The following standards apply to those portions of tracks on which trains or equipment will be left standing, serviced, or stored and do not apply on the approach portions of those tracks. These standards apply only to the usable length of track and any overrun distances or, in the case of stub end tracks, the portion between usable length and the bumping post or other end of track device.

- **Usable Length of Track** – The usable length of track is defined as the length of track which is usable for its defined purpose. Usable length does not include space for bumping posts or other end of track devices, defined set back from the end of track device, defined set back from signals, space occupied by road crossings, turnouts to other tracks, and any other feature that render the equipment on the track inaccessible to service, if the purpose of the track is to hold equipment while being serviced, or unusable for storage if the purpose of the track is to store passenger trains or other equipment.

  Usable length of track for train servicing and storage tracks is defined based on the maximum potential train length. Sufficient length beyond train length to hold a switch engine shall also be provided. Minimum length shall be 1400 feet.

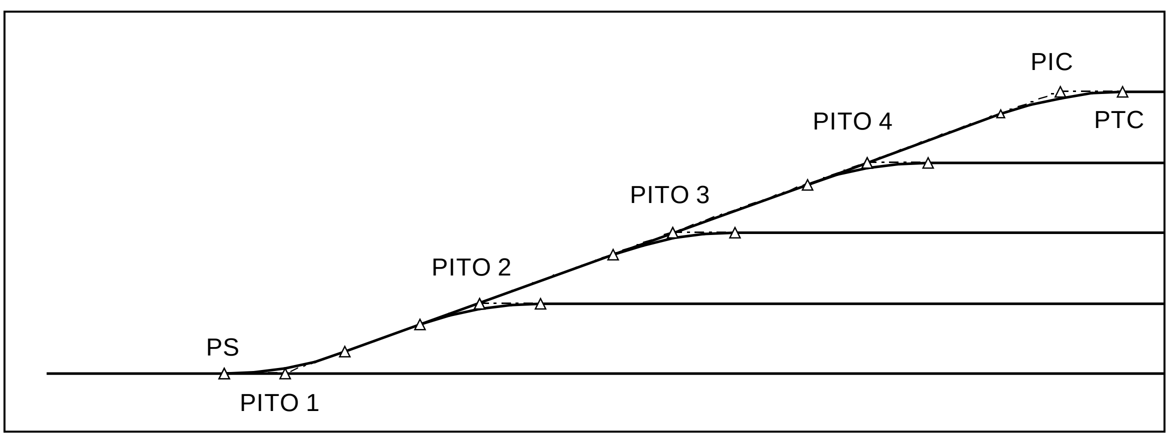
  Usable length of track for other purposes: For tracks not intended to hold full length trains, the usable length shall be defined by the length of equipment that it is intended to hold plus some allowance for placement of equipment, and desirably additional length sufficient to hold a switch engine. Minimum length shall be 75 feet plus the length to be occupied by equipment.
- **Minimum Curve Radii for curves within the usable length** – 10,000 feet
- **Minimum Grade within the usable length** – between 0.00 percent and 0.20 percent or between 0.30 percent and 0.50 percent down from the access point. In the case of double ended tracks, the low end access track shall not be lower than the highest point within the portion designated as usable length.
- **Minimum Length of Vertical Curves** – 50 feet minimum length with a rate of change of not more than 1.00 percent per 100 feet.

- **Minimum Track Centers, between tracks on which servicing of equipment will be performed** – alternating spacing of 28.00 feet and 20.00 feet. These track centers provide space between tracks for roadways on the wider centers and cart paths or walkways on the narrower centers. However, these do not include allowances for OCS poles, light poles, drainage, signal masts, electrical cases, inspection platforms and pits, or other facilities that may interfere with the use of the aisles as traveled ways. Wider track centers shall be provided where these facilities are needed.
- **Minimum Track Centers, between tracks on which no servicing of equipment will be performed** – 15.00 feet. Wider track centers shall be provided if OCS poles, light poles, drainage, signal masts, electrical cases, major walkways or other facilities must be placed between tracks.

4.15.3 Simple Track Ladders

A track ladder is a series of turnouts used to connect a group of parallel tracks to each other in conjunction with either an approach track or a stub end track to permit equipment to be accessed or shuttled between tracks. The most common form of connection of multiple parallel tracks is a straight ladder, also called a simple ladder. A simple ladder is a series of turnout connected end to end so as to access all the parallel tracks. Its primary advantage is its simplicity in design, construction and maintenance. Its disadvantage is its length when more than a few tracks are involved.

Figure 4-7: Simple Ladder (4 Tracks Illustrated)



Calculation of the points on these ladders is straightforward. The Point of Switch (PS) to Point of Intersection of the Turnout (PITO) 1 dimension is a property of the turnout used. The PITO 1 to PITO 2 and PITO 2 to PITO 3 and so forth lengths parallel to the tracks are simply track spacing divided by the tangent of the frog angle of the turnout. PI to PI lengths along the ladder track are track spacing divided by the sine of the frog angle of the turnout. When summed and the length of the final curve tangent added, the length of the entire ladder is determined.

Dimensions for the basic ladder connecting tracks at 15.00 centers using Number 11 turnouts are as follows:

- Between PITOs parallel to the lead track: 164.66 feet
- Between PITOs on the ladder track: 165.34 feet
- Total distance, PS entry turnout to curve PT for the case illustrated: 745.01 feet
- Total PS to PS distance for double ended tracks with 1,500 feet clear length: 2,990 feet
- Length utilized by ladder for each additional track: 329.33 feet (double ended)

When more than a few tracks are involved, the total length of this arrangement quickly becomes impractical, particularly where track centers are large. Thus, the need for compound ladders to shorten the overall yard length.

4.15.4 Double Angle Track Ladders

Considerable space can be saved by use of double angle ladder tracks, as the larger angle considerably reduces the length required to achieve the required offsets. The following is provided for assistance in understanding the design of multi-track ladder tracks.

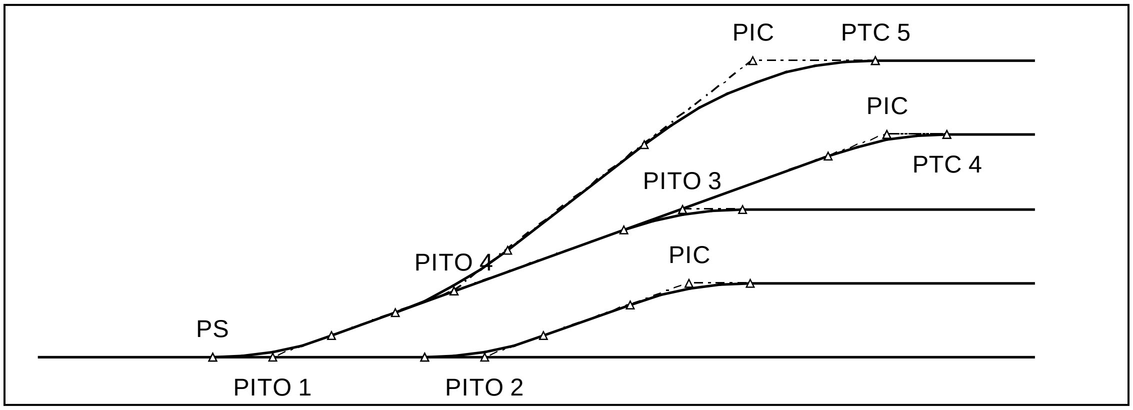
First, look at the situation with the same number and spacing of tracks as used in the simple ladder illustration. The single frog angle ladder using Number 11 turnouts and 15.00 feet track centers was 745 feet long from first point of switch to the point of development of the full width. By taking only one track off the outside, the length is reduced to approximately 580 feet, a saving in length of over 320 feet if the yard is double ended.

This method can be carried forward with additional tracks to whatever extent is necessary. The greater the number of tracks, the greater is the saving in length. For the illustrated six diverging track arrangement, the length from beginning point to end of last curve is about 734 feet, using Number 11 turnouts. The same number of tracks using a simple ladder would utilize approximately 1074 feet. Thus, for a double ended arrangement, the length saving is 680 feet.

The greater the number of tracks, the greater is the savings in overall yard length. For large numbers of tracks, the arrangement can be carried at least one step further to go to a triple ladder. Figure 4-8 and Figure 4-9 illustrate the nature of these savings.

When developing this form of track arrangement, the need to provide space for switch machines must not be overlooked. In addition, with these more complex track ladder arrangements, consideration must be given to the location of OCS poles since complex track layouts equate to complex overhead wiring layouts, including the need for wire termination poles and downguys.

Figure 4-8: Double Ladder (4 Diverging Tracks Illustrated)

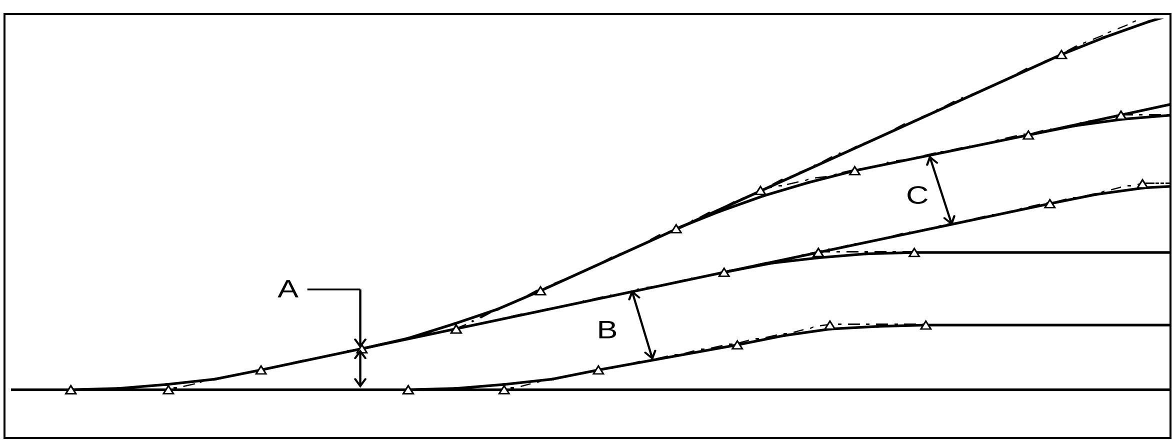


The following considerations shall be used in the development of these designs:

- **Separation at switch point** – Recommended: 9.00 feet, Minimum: 9.0 feet.
- **Space between track centers** – Recommended: 20.0 feet, Minimum 18.0 feet.
- **Space between track centers with switch points approximately opposite** – Recommended: 25.0 feet; Minimum 20.0 feet; if at least one switch machine can be turned away.

The above considerations are required to provide space for the switch tie sets of adjoining turnouts to fit together without overlapping. While overlapping tie sets are constructible, these are undesirable because they create the need for non-standard, site-specific ties and fixtures that add to yard cost and complexity. These space requirements generally will provide adequate clearance for switch machines to be located clear of adjacent tracks. However, the specifics of each yard layout may create localized conditions of interference. Ultimately the yard ladders must be laid out with dimensionally accurate switch machines and tie layouts, and adjacent roads and facilities must be overlaid to verify fit.

Figure 4-9: Double Ladder, Track Space Requirements (6 Diverging Tracks Illustrated)



Chapter 6

Rolling Stock and Vehicle Intrusion Protection

Revision	Date	Description
0	02 Mar 12	Initial Release, R0
1	Jun 2013	Revision 1
2	Feb 2014	Revision 2

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Acronyms

Authority	California High-Speed Rail Authority
Caltrans	California Department of Transportation
CHSTP	California High-Speed Train Project
HST	High-Speed Train
OCS	Overhead Contact System
TCL	Track Centerline

6 Rolling Stock and Vehicle Intrusion Protection

6.1 Scope

This chapter provides the separation requirements for the California High-Speed Train (HST) alignment adjacent to, crossing over, and crossing under other transportation systems. The requirements are established to protect the HST operating infrastructure from intrusion by rolling stock and vehicles from adjacent transportation systems (i.e., passenger and freight rail tracks, and state and local highways/roadways). Application of these requirements will be determined by a site-specific hazard analysis, in conformance with the hazard management process in the CHSTP Safety and Security Management Plan. This chapter is intended to serve primarily as the basis of the track, earthwork, and structural design and mitigation measures where minimum clearances cannot be met. This chapter does not define requirements for access control devices. Refer to the *Civil* chapter for protection of the Authority's right-of-way and facilities against trespass by unauthorized persons and animal intrusion.

6.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards. Applicable codes and regulations include but are not limited to the following list:

Code of Federal Regulations (CFR)

- Title 49, Part 213, Section 361 for protection of the right-of-way for Class 8 and 9 tracks
- Title 49, Part 214, Railroad Workplace Safety

California Public Utilities Commission (CPUC) General Order (GO) No. 26-D

Federal Railroad Administration (FRA) guidelines regarding the separation and protection of adjacent transportation systems and conventional railroads

- High-Speed Passenger Rail Safety Strategy published by FRA (November 2009)

American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering

California Department of Transportation (Caltrans), Highway Design Manual, Traffic Manual and Standard Plans

6.3 Protection of HST Operating Infrastructure from Vehicle Intrusion

The HST operating infrastructure, which includes the operating envelope, traction power facilities, wayside power cubicles, communication cabinets, cable troughs, piers and walls supporting HST structures, shall be protected from intrusion in order to preserve safe and reliable operations. The limits of the HST operating envelope is defined as the area from the outer face of the Overhead Contact System (OCS) pole foundations in width and from top of the OCS poles to the trackbed supporting the HST tracks in height. In locations where the HST operating envelope is located within an open trench, on retained fill, or on an aerial structure, the limit of operating envelope shall be extended to the outer face of retaining walls, trench walls, abutments and piers of aerial structures.

6.3.1 Protection Against Intrusion of Conventional Trains

Passenger and freight trains that operate in shared corridors or adjacent to the HST system shall be prevented from entering into the HST operating infrastructure by lateral separation or by a physical barrier (e.g., earth berms, ditches, or reinforced concrete walls) when lateral separation between railway systems is insufficient and where supported by site-specific hazard analysis.

6.3.1.1 Protection Measures without Physical Barriers

The preferred protection is to locate HST operating infrastructure at a sufficient distance from conventional railroad systems to avoid intrusion. A lateral distance of 102 feet or greater measured between the closest existing or future planned track centerlines (TCL) of the conventional railroad and HST system does not require a physical barrier for intrusion protection. Alternatively, when the HST alignment is on embankment and its trackbed is 10 feet or higher than the freight/conventional railroad top of rail, use of a physical barrier for intrusion protection of HST operating infrastructure is not required.

6.3.1.2 Protection Measures with Physical Barriers

When lateral separation between the closest existing or future planned TCLs of the conventional railroad and HST system is less than 102 feet, physical barriers shall be installed based on site specific hazard analysis. The intrusion protection shall be designed to mitigate the risk of a train derailment from adjacent conventional railroad intruding into the HST operating envelope. For train collision loads, refer to the *Structures* chapter. For grounding and bonding of reinforced concrete barrier refer to the Overhead Contact System and Traction Power Return System, and Grounding and Bonding Requirements chapters. The intrusion protection is achieved by the following measures:

- HST At or Below Grade
    - Use of a minimum 10-foot-high berm, or 10-foot-deep ditch, or a 5-foot-deep ditch and a 5-foot-high berm combination, as an intrusion protection measure when lateral separation between the closest TCLs of the conventional railroad and HST system is less than 102 feet. Refer to Standard and Directive Drawings for typical sections of various intrusion protection measures.
    - Use of a minimum 10-foot-high reinforced concrete barrier as an intrusion protection measure when lateral separation between the closest TCLs of the conventional railroad and HST system does not allow construction of a 10-foot high berm/ditch. Refer to Standard and Directive Drawings for placement of the wall within HST right-of-way.

- When there is a concrete barrier as an intrusion protection between a conventional railroad and the HST and there is an aerial structure pier between HST and the railroad, the concrete barrier shall transition to protect the pier. Refer to AREMA Pier Protection requirements. The minimum height of the barrier protecting the pier shall be 10 feet. The transition of the concrete barrier from inside HST right-of-way to the pier shall be at 3:1 slope or flatter.
- Berm shall not be used as an intrusion protection measure for below grade sections of HST.
- Refer to Standard and Directive Drawings for typical sections of various intrusion protection measures.

- HST Elevated Guideway
    - Where the side clearance from the closest conventional rail TCL is less than 25 feet to the face of a HST structure, such as a pier or a retaining wall (with the exception of a trench wall), a 6-foot high reinforced concrete barrier shall be constructed at a minimum distance of 1 foot from the face of the HST supporting structure. Where the side clearance is 12 feet or less, the height of the reinforced concrete barrier shall be 12 feet. The reinforced concrete barriers shall be designed to protect HST supporting structures from a direct impact by a derailed conventional railroad locomotive.
- For HST Elevated Guideway supported on MSE retaining walls, intrusion protection measures shall be identical to intrusion protection for at-grade section.
    - Refer to Standard and Directive Drawings for typical sections of various intrusion protection measures. These guidelines are for physical separation and do not include right-of-way considerations that may require additional separation. Additionally, separation requirements of freight railroad owners and operators shall be considered in establishing required separation.

6.3.2 Protection Against Intrusion of Roadway Vehicles

Protection against highway/roadway vehicles from intruding into the HST operating infrastructure shall be provided through sufficient lateral separation between state highway systems or local roadways and the HST system, or the installation of barriers. For highway vehicle collision loads, refer to the *Structures* chapter.

6.3.2.1 Protection against Intrusion of Roadway Vehicles into the HST Operating Infrastructure

For state highway systems, protection against errant roadway vehicles from intruding into HST operating infrastructure shall be provided. Caltrans requires protection for errant roadway vehicles when HST fixed objects are located within the highway Clear Recovery Zone (CRZ). Caltrans Highway Design Manual establishes 52 feet as the CRZ for the high-speed rail project. Therefore, when a high-speed rail corridor is constructed longitudinal to a freeway, expressway, or a conventional highway with posted speeds over 40 mph, the nearest fixed object or feature associated with the operation of the rail facility shall be located at a minimum of 52 feet horizontally from the planned ultimate edge of the traveled way. When the HST alignment is not longitudinal to a Caltrans freeway, expressway, or highway, the standard Caltrans 30 feet requirement for CRZ shall apply.

If these clearances cannot be provided, a design exception shall be obtained from Caltrans and the Authority along with appropriate roadside protection mitigation measures, such as installation of a metal beam guard rail or concrete barrier.

For protection of HST operating infrastructure, appropriate required type of roadside protection shall be site specific, based on site specific hazard analysis, and shall consider factors such as traffic volumes, speed, highway geometry, side slopes, accident history, and others. For instance, in locations where high volumes of cargo and tanker trucks are present with high probability of intrusion into HST operating infrastructure, a more stringent intrusion protection is required and shall be provided, such as a concrete wall up to 7.5-foot high meeting design force requirements specified for AASHTO TL-6 with a Caltrans type 60D barrier or metal beam guard rail installed for protection. However in most cases, a 56-inch high concrete barrier meeting design force requirements specified for AASHTO TL-5 is required to protect HST operating infrastructure from intrusion by errant vehicles.

For local roadways, protection against adjacent roadway vehicles from intruding into HST operating infrastructure shall be provided based on site specific hazard analysis and per the requirements of the local jurisdictions.

For both the state highway system and local roadway systems, the intrusion protection shall be designed to mitigate the risk of errant vehicles from an adjacent roadway intruding into the HST operating infrastructure. Refer to Standard and Directive Drawings for various conditions where intrusion protection measures are required along the HST alignment.

6.3.2.2 Protection Against Intrusion of Roadway Vehicles over the HST Operating Infrastructure

Protection against intrusion of roadway vehicles on grade separated structures onto the HST operating infrastructure below the structure shall be provided. The overhead structure shall be designed to include vehicular railing with sufficient strength to withstand collision loads defined in the *Structures* chapter. The vehicular railing shall extend to the nearest intersection or 100 feet beyond the end of the overhead structure with appropriate taper to redirect vehicles that may travel down the roadway embankment and into the Authority's right-of-way. In conjunction with keeping the roadway vehicle from intruding into the HST operating infrastructure, a protective screening and barrier shall be provided to prevent contact with the OCS, to prevent pedestrians from falling onto, and to reduce the risk of objects being dropped onto the HST operating infrastructure. Refer to the *Overhead Contact System and Traction Power Return System* chapter for minimum requirements of this protective screening and barrier. Refer to Standard and Directive Drawings for typical section of intrusion protection measures on and along a roadway structure over the HST.

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Supplemental protection shall be achieved through the use of intrusion detection technology in the fencing around HST operations. When the intrusion detection system is activated, HST operation is stopped, speed is reduced, or other appropriate actions will be taken by the signaling system and/or operations. Intrusion protection, if required, shall be designed in conjunction with the hazard analysis to determine the need for a physical barrier.

6.3.2.3 HST Pier and Wall Protection

Where HST piers, trench walls, and other structures are located within the Caltrans Clear Recovery Zone, install Caltrans-type concrete barriers to redirect errant vehicles from intrusion into the HST operating envelope.

6.4 Containment of HST Rolling Stock

HST rolling stock shall be contained within the operational envelope in order to reduce the potential for intrusion into an adjacent transportation system. Strategies to ensure containment include but are not limited to the following:

Use modern HST rolling stock, which have documented performance likely to minimize the risk of the train derailment and extending beyond its operating envelope. These protections may include use of bogie-mounted components and use of plate springs to prevent rails from large lateral movement.

Use of articulated trainsets to keep the HST vehicles upright and in line in the event of a derailment.

The raised track side cable trough walls in tunnels, trenches, and aerial structures provide derailment containment to protect cable troughs and OCS poles due to their close proximity to the track as compared with OCS poles and cable troughs located in the at-grade section. Refer to Standard and Directive Drawings for minimum side clearance requirements for grade separated structures.

Ensure the appropriate level of maintenance of infrastructure (per FRA standards) and rolling stock (per manufacturer requirements), to mitigate the risk of derailment.

On aerial structures, protection shall be provided by a derailment protection wall designed so that the HST remains within its operating infrastructure. Refer to the *Structures* chapter.

Chapter 8

Drainage

Revision	Date	Description
0	02 Mar 12	Initial Release, R0
1	Jun 2013	Revision 1
2	Feb 2014	Revision 2

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Tables

Figures

Acronyms

AREMA	American Railway Engineering and Maintenance-of-Way Association
Authority	California High-Speed Rail Authority
BMP	Best Management Practice
Caltrans	California Department of Transportation
FHWA	Federal Highway Administration
HDM	Highway Design Manual
HDS	Hydraulic Design Series
HDPE	High-Density Polyethylene
HEC	Hydraulic Engineering Center
HST	High-Speed Train
IDF	Intensity-Duration-Frequency
PVC	Polyvinyl Chloride
RCP	Reinforced Concrete Pipe
RWQCB	Regional Water Quality Control Board
SWDR	Storm Water Data Report
SWPPP	Storm Water Pollution Prevention Plan
USBR	United States Bureau of Reclamation

8 Drainage

8.1 Scope

This chapter provides design criteria for the hydrologic analysis and design of hydraulic facilities and provides guidelines for hydraulic facility implementation and for Best Management Practices (BMPs) for surface water quality treatment.

Unless otherwise noted, design guidance shall follow California Department of Transportation (Caltrans) Highway Design Manual (HDM) requirements for hydrologic analysis and hydraulics design. Regional criteria shall be used to determine surface water runoff data. Refer to the *General* chapter of this design manual for design life requirements for storm drain structures.

Locations where the California High-Speed Train (HST) alignment crosses existing drainage channels, drainage requirements for roadways and other structures located in or adjacent to the California High-Speed Rail Authority's (Authority's) right-of-way may be subject to regulations and additional requirements by other jurisdictions. Supplemental hydrologic and hydraulic requirements shall be considered for drainage facilities owned or operated by third-party rail operators/agencies, and property owners impacted by these improvements. Design requirements of local municipalities shall be considered for discharge within those jurisdictions. Where a drainage facility is required to connect to a third-party drainage facility, the Designer shall coordinate with the utility owner to determine if an upgrade to the existing facilities may be required.

8.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, standards, and guidelines. Regulations, codes, and standards, such as but not limited to the following, shall be the one applicable.

- California Public Utility Commission (CPUC) General Orders (GOs)
  – CPUC GO 95 - Rules Governing Overhead Electric Line Construction
  – CPUC GO 128 - Rules for Underground Electric Construction
- California Department of Transportation (Caltrans)
  - Caltrans Highway Design Manual (HDM), English Version
  - Caltrans Standard Plans and Standard Specifications
  - Caltrans Bridge Design Specifications (CBDS)
  - Caltrans Storm Water Quality Handbook: Project Planning and Design Guide

- Federal Highway Administration (FHWA), Hydraulic Design Series (HDS)
- U.S. Army Corps of Engineers
- U.S. Bureau of Reclamation (USBR)
- U.S. Bureau of Land Management
- Federal Emergency Management Agency (FEMA)
- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering
- American Association of State Highway and Transportation Officials (AASHTO)
  - Highway Drainage Guidelines
  - Model Drainage Manual
- Applicable Local Ordinances

Applicable local building, planning, and zoning codes and laws shall be reviewed for facilities, particularly those located within multiple municipal jurisdictions, state rights-of-way, and/or unincorporated jurisdictions.

8.3 Goals and Objectives

The following goals and objectives shall be considered in development of drainage design:

- The CHSTP shall not adversely impact the existing floodplain of the area adjacent to the HST corridor.
- Ensure critical HST structures/facilities are protected against 100- and 500-year flood events. Critical HST structures/facilities, in this chapter, refers to HST structures/facilities that are critical to safe operation of HST system, refer to Section 8.6.7.
- Comply with regulatory requirements.
- Contain drainage within the Authority's right-of-way.
- Keep runoff from outside the Authority's right-of-way from entering into the Authority's right-of-way.
- To the extent that is reasonable and practical, avoid placement of third-party drainage access points from within the Authority's access controlled right-of-way.

8.4 Hydrological Analysis

Hydrologic design and analysis shall conform to industry standards, codes, guidelines, and utilize applicable software. The criterion for each factor involved in hydrologic analysis to obtain optimum runoff calculations are outlined in this section. For criteria not included in this section, references shall be used as follows:

- Caltrans HDM for rainfall hydrological analyses
- FHWA HDS-02 for criteria not found in Caltrans and for snowmelt analyses

8.4.1 Time of Concentration

The time of concentration (T_c) shall be used to determine approximate rainfall intensity. The time of concentration is the sum of 2 travel times, including sheet flow/overland flow and shallow concentrated flow, usually in a gutter, swale or channel.

The minimum time of concentration recommended in urban areas is 5 minutes and in rural areas is 10 minutes. For gutter, pipe and channel flow, Manning's equation shall be used, per Section 8.5.2.

The Caltrans HDM Hydrology chapter shall be referenced for detailed methodologies on T_c calculation.

8.4.2 Intensity

Intensity is defined as the time rate of rainfall depth and is commonly given in inches per hour. The time of concentration depends on an initial estimate of an intensity value found from Intensity-Duration-Frequency (IDF) curves or the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server (PFDS) using the following link: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

IDF curves shall be obtained from local agencies or Caltrans for the most current and accurate information.

8.4.3 Design Storm Frequency/Recurrence Interval

Frequency establishes the frame of reference for how often precipitation with given characteristics is likely to occur. The design storm frequencies for the design of various storm facilities shall be as shown in Table 8-1.

Table 8-1: Design Storm Frequencies

Storm Facility	Rural	Urban
Drainage facilities crossing the track (e.g., culverts)	2% (50-yr) [(1)]	1% (100-yr) [(1)]
Drainage facilities not crossing the track (e.g., parking lots, access roads, station drainage facilities)	10% (10-yr) [(1)]	2% (50-yr) [(1)]
Ditches/storm drainage systems adjacent to the track	4% (25-yr) [(1)]	2% (50-yr) [(1)]
Freeways – Minor Ramps and Frontage Roads	10% (10-yr) [(2)]	
Conventional Highways – High volume, multilane or urban with speeds 45 mph and under		
Freeways – Through traffic lanes, branch connections, and other major ramp connections	4% (25-yr) [(2)]	
Conventional Highways – High volume, multilane or low volume, rural with speeds over 45 mph		
All State Highways	2% (50-yr) [(2)]	
Drainage systems crossing under bridge structure and on the right-of-way	2% (50-yr) [(1)]	1% (100-yr) [(1)]
Critical HST Structures/Facilities [(3)]	Min 0.2% (500-yr) [(3)]	Min 0.2% (500-yr) [(3)]

Notes:
(1) Based on Standard Engineering practices employed by other railroad operators within California.
(2) Caltrans HDM, Table 831.3 shall be referred to for Roadway Drainage Guidelines.
(3) For critical HST Facilities, refer to Section 8.6.7.

8.4.4 Snowmelt

For runoff calculations in areas where snowmelt may occur, refer to the FHWA HDS-02 Hydrology report.

8.4.5 Storm Runoff

Storm runoff shall be calculated in accordance with criteria and methodologies specified in the Caltrans HDM's Hydrology chapter and the applicable local procedures.

8.4.6 Floodplain Information

The proposed elevation of the track subballast (bottom) shall be a minimum of 2 feet higher than the 100-year Base Flood Elevation. Drainage facilities located within a floodplain shall be designed so that the proposed improvements will not result in the following:

- Increase the flood flow rate or inundation hazard to adjacent upstream or downstream property
- Raise the flood level of drainage way
- Reduce the flood storage capacity or obstruct the movement of floodwater within a drainage way

Refer to the Caltrans HDM General Aspects chapter for FEMA guidelines where encroachment on floodplains is anticipated.

8.5 Hydraulic Design

8.5.1 Basic Parameters

8.5.1.1 Discharge of Storm Drains into Local Drainage System

Discharge of stormwater from trackway sections, station sites, parking lots, and wayside facilities into local drainage system shall comply with environmental and regulatory permit requirements. Appropriate mitigation measures to prevent pollutants shall be taken as required before site drainage is discharged into local drainage system.

8.5.1.2 Debris Control

Debris may consist of trash, natural streambed material such as boulders, silts, sands, clays, sticks, tree limbs and other vegetation. Buoyant material will float during a storm event and other materials will roll or skip along channel bed. The frequency of the storm event will affect the quantity of debris that is carried along the channel; the more discharge in the channel will result in more debris carried.

Debris control shall be a significant consideration during the design of hydraulic structures such as catch basins, culverts, storm drain systems, and outlet structure from detention basins. Depending on the type of debris and location where the debris is controlled, there are several options for debris control structures such as debris racks, debris risers, debris cribs, and debris fans. The type and quantity of debris shall be determined and an appropriate debris control measure shall be implemented.

8.5.1.3 Access Control

For drainage structure access control requirements, refer to the *Civil* chapter.

8.5.1.4 Grounding and Corrosion Control

All metallic pipes and appurtenances shall be grounded and protected against corrosion. Refer to the *Grounding and Bonding Requirements* chapter and the *Corrosion Control* chapter for requirements.

8.5.2 Channel Hydraulics

8.5.2.1 Open Channel Hydraulics

This section presents the minimal criteria and design standards for the hydraulic evaluation and design of open channels. For criteria not included in this section, references shall be used as follows:

- AREMA's design criteria shall be followed for design of new open channels.
- Refer to Caltrans HDM for design criteria not available in AREMA, and for existing open channels.
- Local criteria shall be followed as required by the governing agency.

Transverse channels that pass through culverts shall join parallel ditches at an angle of approximately 30 degrees to minimize aggradations and deposition.

A. <u>Open Channel Flow</u>

The computation of uniform flow and normal depth are based on Manning's formula:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Where:

Q = Flow Rate (cfs)

n = Roughness Coefficient[1]

A = Flow Area ($feet^2$)

R = Hydraulic Radius, A/P, (feet)

P = Wetted Perimeter, (feet)

S = Slope of the Energy Grade Line, (feet/feet)

B. <u>Maximum Permissible Velocity</u>

Open channel flow velocities shall not erode nor cause deposition in the channel. For maximum permissible velocities of unlined channels refer to the Caltrans HDM. The maximum permissible velocities for lined/non-erosive channels are presented in Table 8-2

[1] The Caltrans HDM Physical Standards chapter shall be referred to for Manning's roughness coefficients.

Table 8-2: Maximum Permissible Velocities for Lined/Non-Erosive Channels

Type of Lining	Maximum Permissible Velocity (feet per second)
Unreinforced vegetation	5.0
Loose riprap	10.0
Grouted riprap	15.0
Gabions	15.0
Soil-Cement	15.0
Concrete	35.0

Source: Clark County Regional Flood Control District, Hydrological Criteria and Drainage Design Manual, August 1999.

C. Freeboard

Freeboard is the vertical distance between the design water surface elevation and the bottom of subballast or bottom of a bridge girder/soffit. The minimum freeboard required for track side ditches shall prevent saturation and infiltration of stormwater into the subballast and ballast sections of the track. The minimum recommended water depth in any channel section shall be 2 feet below the bottom of subballast or bottom of a bridge girder/soffit.

For superelevation requirement on curved open channel alignment for water surface elevation, refer to the Caltrans HDM.

D. Grade Control

If the ditch grade is steeper than the grade that results in maximum permissible velocities, drop structures shall be considered to maintain design velocities, and prevent erosion and scour to the channel bed and embankments. To mitigate sediment aggradation and degradation, a drop structure may be installed across a channel to create a vertical drop or a short sloping drop. Drop structures may be built with gabions, sheet piling, riprap, or concrete walls with footings.

E. Channel Section

Selection of a channel cross section may involve changing/improving the existing natural waterway (channel) or designing a new channel section. Where feasible, the channel section is preferred to have a flat bottom.

Changing a natural channel – Existing hydraulic conditions of a natural channel shall be assessed to confirm channel stability and evaluate the impact of proposed improvements. Natural waterways shall have adequate capacity to pass the flows from a design storm event. The stream mechanics shall be studied to analyze the need for erosion control structures.

Design of new channel section – The first estimation of a channel cross section is based on normal depth plus freeboard. The shape of the channel shall consider terrain, flow velocity, available right-of-way for the corridor, and quantity of flow to be conveyed. Channels shall be sized for the anticipated design runoff and to allow the subballast to drain. Open channels along the toes of trackway embankments shall be designed with sufficient depth to carry the peak flow with design water surface elevation below the bottom of subballast.

Hydraulic parameters of basic channel sections are provided in Table 8-3. For trackside ditch details, refer to Standard and Directive Drawings.

Table 8-3: Hydraulic Parameters for Channel Sections

Channel Section	Area (A)	Wetted Perimeter (P)	Hydraulic Radius (R)
Triangular V-ditch	zy^2	$2y\sqrt{1+z^2}$	$\dfrac{zy}{2\sqrt{1+z^2}}$
Trapezoidal	$(b+zy)y$	$b+2y\sqrt{1+z^2}$	$\dfrac{(b+zy)\,y}{b+2y\sqrt{1+z^2}}$
Rectangular	by	$b+2y$	$\dfrac{(by)}{(b+2y)}$

<u>Notes:</u>
b = base of rectangle or base of trapezoidal
z = side slope of trapezoid or V-ditch
y = depth of flow in channel

F. <u>Channel Lining</u>

Channel lining is the key factor that determines the roughness coefficient of a channel. The most commonly used channel materials are the following:

Grass lined channel (vegetative lined) – Grass linings provide protection to the channel from erosion. Due to the frequent maintenance involved, grass lined channels are not recommended within the Authority's right-of-way. They may be used as a best management practice for stormwater quality control where pollution prevention devices are required. Grass lined channels shall not be designed with side slopes steeper than 3:1 (H:V).

Riprap lined channel – Riprap lining is suitable for a short but steep channel reach. The nature of high friction of rocks contributes to the effectiveness of energy dissipation. Riprap channels may be considered where right-of-way is constrained and/or erosion occurs.

The roughness coefficient, n, of a riprap channel is correlated to the intermediate riprap rock size, D_{50}, in feet.

$$n = 0.0395\, D_{50}^{1/6}$$

Where:

n = Roughness, (non-dimensional)

The criteria for sizing of D_{50} and thickness of riprap lining shall be as follows:

- D_{50} (maximum) = 24 inches
- Thickness of riprap lining = 2*D_{50}
- Thickness of riprap lining shall be 50 percent more for water deeper than 3 feet.

Concrete lined channel – Concrete lining shall be designed to withstand various forces due to high gradient. Criteria for design of concrete lined channels include the following:

- Thickness of concrete lining = 6 inches for V < 30 feet per second
- Thickness of concrete lining = 7 inches for V > 30 feet per second
- Channel section shall be adjusted for superelevation changes in water surface.
- Side slopes shall be a maximum of 2:1 (H:V), or a structurally reinforced wall if steeper.
- A concrete cutoff wall shall be provided at both the upstream and downstream termini.

Composite channel – Composite channels shall be considered where the open channel shall have the hydraulic capacity to handle a wide spectrum of storm events. Due to the maintenance level involved, composite channels are not permitted within the Authority's right-of-way but may be designed outside the Authority's right-of-way where floodplain mitigation and/or pollution prevention/mitigation measures are required. The channel cross section shall be designed to have 2 sections: the lower section or the main channel, usually shallow, narrow and has a hard bottom, with a side slope ranging from 0.5:1 (H:V), to 2:1 (H:V) and an overbank section usually wide, flat and grass lined, with mild slopes of 10:1 (H:V).

Composite channels may be designed for more specific purposes such as low flow channels, trickle channels and wetland channels.

8.5.2.2 Overside Drains

Overside drains are used to prevent erosion of embankments and other steep-sloped surfaces by collecting surface runoff and conveying it to a stable or less erosive drainage facility. Water conveyance at cut slopes or fill slopes are generally erosive and are more likely to need an overside drain. The spacing of overside drains shall consider the quantity of flow and capacity limitation of the gutter or ditch and the ground configuration.

For cut slopes, the angle of the slope shall be considered for overside drain slopes. The overside drain shall be sized to convey a larger storm, so that the drain is not washed out following a major storm event. Rock riprap shall be considered to minimize the velocity, but may need a wire mesh to prevent rock slippage.

Fill slopes may require the use of overside drains. For roadways, curb openings will generally allow flow to discharge down an embankment into an open channel. Ditch openings or small spillways may be used to alleviate water from trackside drainage ditches. Riprap material and an aesthetic design shall be considered.

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Water from overside drains shall not be diverted to watersheds that originally did not contain the water, or negatively affect downstream properties.

Overside drains may be designed as pipe downdrains, flume downdrains and spillways as follows:

- **Pipe downdrains** – Usually made of plastic or metal material, pipe downdrains are recommended for slopes of 4:1 (H:V) or steeper and a minimum diameter of 8 inches. When overside drains are designed in areas where sediment debris is likely and the drain is expected to be longer than 50 feet, a larger pipe size shall be considered to minimize clogging in the pipe. Overside drain pipes shall be buried along the corridor, or designed to blend with the existing natural landscape.
- **Flume downdrains** – Generally rectangular in shape, flume downdrains are open channel chutes that can discharge water at steeper grades, 2:1 (H:V) or steeper. The flume invert shall be below surface grade so that the flume is even with the surface slope. Sharp bends in the flume are not permitted.
- **Spillways** – Asphalt concrete is typically used to create a spillway on slopes flatter than 4:1 (H:V). Spillways are most commonly V-shaped and shall be placed on compacted soil to prevent further erosion. Sharp turns are not permitted to prevent splash over of discharge.

Grate inlets may be used in cases where a depression in not feasible. Outlet velocities shall be mitigated by use of energy dissipators.

8.5.2.3 Underdrain System

Underdrains shall be located in areas where it is anticipated that groundwater may interfere with the stability of tracks, roadbeds and side slopes. The underdrain system helps to draw the water table down, preventing softening of sub grade soils, sloughing, or instability of slopes. The utilization of underdrain pipes shall consider subsurface conditions and geotechnical studies focusing on infiltration and percolation recommendation. Underdrain pipes shall be bedded in clean, granular or crushed aggregate material enclosed in an envelope of non-woven geotextile fabric. For criteria not included in this section, reference shall be made to Caltrans HDM design criteria for underdrain systems within roadways or highways.

A. Pipe Size

Underdrain pipe shall be a minimum of 6 inches in diameter for segment length less than 500 feet and a minimum of 8 inches for segment lengths over 500 feet. A minimum 6-inch PVC pipe shall be used to carry the water from underdrain system to an onsite drainage system or to the municipal stormwater system.

B. Location

Underdrains shall be used in the following locations:

- Under ballast in ballasted trackway which does not naturally drain towards the outside of the trackway, to intercept ground water and trackbed surface drainage infiltration through the ballast
- Along the toe of a cut slope to intercept seepage
- Along the toe of a fill on the side from which groundwater emanates
- Across the track or roadway at the downhill end of a cut
- Along the periphery of any paved area under which groundwater is likely to collect
- In retained cuts and on retained embankments
- Under the track slab at station platforms
- Between tracks at locations of outside station platforms and or where several sets of tracks are adjacent
- At low points in the profile, and 100 feet each side of a low point

C. Pipe Material

Underdrain pipes shall be perforated type and installed with perforations pointing down towards the bottom of the trench. Underdrain pipe shall be made of porous concrete, steel, aluminum, corrugated metal, rigid plastic, or polyethylene. For track drainage at stations and at-grade sections, perforated PVC or high-density polyethylene of schedule 80 shall be used.

D. Access Holes/Cleanouts and Risers

Access holes/cleanouts for underdrains shall have convenient access for maintenance crews and equipment. Cleanouts for underdrain systems shall be spaced at a maximum of 300 feet. Pipe materials for cleanouts to be used within trackway shall be plastic with metal risers. Risers shall be provided at beginning of underdrain runs and at a maximum of 300-foot intervals. For track drain/underdrain cleanout and riser details, refer to Standard and Directive Drawings.

E. Cover

Underdrain pipes for track drainage system shall have a minimum cover of 36 inches from the top of finished grade.

F. Depth and Spacing

Underdrain depth and spacing shall consider the permeability of the soil, the elevation of the water table and the amount of drawdown and time needed to ensure stability. Depth and spacing of underdrains shall be as follows:

Table 8-4: Suggested Depth and Spacing of Pipe Underdrains for Various Soil Types

Soil Class	Soil Composition			Drain Spacing (feet)			
	Percentage Sand	**Percentage Silt**	**Percentage Clay**	**3 feet Deep**	**4 feet Deep**	**5 feet Deep**	**6 feet Deep**
Clean Sand	80-100	0-20	0-20	110-150	150-200	-	-
Sandy Loam	50-80	0-50	0-20	50-100	100-150	-	-
Loam	30-50	30-50	0-20	30-60	40-80	50-100	60-120
Clay Loam	20-50	20-50	20-30	20-40	25-50	30-60	40-80
Sandy Clay	50-70	0-20	30-50	15-30	20-40	25-50	30-60
Silty Clay (1)	0-20	50-70	30-50	10-25	15-30	20-40	25-50
Clay (1)	0-50	0-50	30-100	15 (max)	20 (max)	25 (max)	40 (max)

Source: Caltrans HDM, Table 842.4.

Notes:

(1) Drainage blankets or stabilization trenches shall be considered.

G. Slope

Underdrain grades shall be not less than 0.5 percent. If 0.5 percent is not feasible, a slope that would provide a minimum velocity of 2 feet per second shall be provided in the full pipe condition.

H. Separation of Underdrain Systems

Design of underdrain systems shall consider separation of drainage flows:

- Pipes carrying surface water shall not discharge into underdrains. Surface water shall also be prevented from seeping into underdrains other than those provided to collect trackbed surface drainage.
- Where underdrains are located under trackbeds or paved areas, other than sidewalks, the filter material and filter fabric shall extend up to the top of prepared subgrade.
- Where underdrains other than those provided to collect trackbed surface drainage are located under unpaved areas or sidewalks, the filter material shall extend up to 6 inches below finished grade and the filter material shall be covered with impervious backfill material.

8.5.2.4 Energy Dissipators

Where the anticipated outlet velocity for a waterway exceeds the maximum permissible velocity for the bed material of the receiving channel, an acceptable means of energy dissipation shall be used to reduce the velocity to safe limits. Commonly used energy dissipators include natural scour holes, drop structures, internal dissipators, external dissipators, and stilling basins. These facilities decrease the chance of a hydraulic jump as well as erosion/scour.

This section presents the minimal criteria and design standards for the hydraulic evaluation and design of energy dissipators. For criteria not included in this section, the following references shall be used:

- Refer to Caltrans HDM for design criteria of energy dissipators on highways and freeways.
- Local criteria shall be followed as required by the governing agency.

To permit debris to be carried with the flow, dissipators employing obstructions shall be avoided unless it can be demonstrated that such obstructions will not collect debris.

A. Natural Scour Holes

This option consists of providing an area in which flows through the culvert will be allowed to form a natural scour hole. Scour holes shall be lined over an area sufficient to cover the potential scour hole with the class of riprap appropriate for the culvert exit velocity.

The following shall be considered in the design of natural scour holes:

- Undermining of the culvert outlet will not occur or it is impracticable to be checked by a cutoff wall.
- The expected scour hole will not cause costly property damage.
- Right-of-way or drainage easements at the site are sufficient to encompass the entire scour hole which may often be quite large.
- Environmental concerns due to sedimentation will not be a factor.
- There are no aesthetic concerns or other nuisance effects, such as insect breeding.

B. Drop Structures

Inclined or sloping drop structures – Where the difference in elevation from the upper channel bottom to the lower channel bottom is 10 feet or lower, these drop structures shall be used. The top of the crest wall shall be placed at a height above the upstream channel bottom. A downstream apron shall be provided to transition from the drop structure to the downstream channel.

Vertical drop structures – A vertical drop structure is designed to force the hydraulic jump to occur within a stilling basin next to a rectangular weir. When the flow line of the channel is too steep for the design condition, erosion and scour may occur to the channel bottom or the toe of the embankment. To mitigate sediment aggradation and degradation, a drop structure shall be considered. The drop structure shall have sufficient length and water cushions to prevent scouring of the downstream channel bed due to a nappe or hydraulic jump.

Material – The material used to construct the drop structure depends on the availability of materials, the height of drop required, and the width of the channel. Rock riprap and timber pile construction have been successful on channels having small drops and widths less than

100 feet. Sheet piles, gabions, and concrete structures are generally used for larger drops on channels with widths ranging up to 300 feet.

C. Internal Dissipators

Containing the hydraulic jump within the culvert is a form of internal energy dissipation. Internal dissipators shall be used where the scour hole at the culvert outlet is unacceptable, the right-of way is limited, debris is not a problem and moderate velocity reduction is needed. The 3 types of internal dissipators are tumbling flow, increased resistance, and broken back culverts.

Tumbling flow (roughness elements) – Tumbling flow may be applicable where culvert slopes are between 10 percent and 15 percent. Tumbling flow in culverts shall be the following:

- Use 5 rows of uniformly sized roughness elements in box culverts or open chutes
- Spacing (L) between the roughness element rows is set by choosing a ratio of L/h to be between 8.5 and 10, where h is the height of the element.

Increased resistance – Increasing resistance may cause a culvert to change from partial flow to full flow in the roughened zone. Velocity reduction is accomplished by increasing the wetted surfaces as well as by increasing drag and turbulence by the use of roughness elements. Increased resistance shall be used where a culvert is flowing partially full with inlet control and the requirement for outlet velocities is between critical and normal. The following criteria shall be considered for increased resistance design on culverts:

- Where slopes of culverts are less than 4 percent
- Five rows of roughness elements may be used.
- Where the height of element is 5 percent to 10 percent of the diameter

Broken-back culverts – Substituting a "broken-slope" flow line for a steep, continuous slope in a culvert may be used for controlling outlet velocity. The steep slope of the culvert is replaced by breaking the slope into a steeper portion near the inlet followed by a horizontal runout section. Broken-back culverts, at minimum, shall have the following:

- There shall be sufficient tailwater and sufficient friction and length in the runout section of the culvert
- Steep sections for which slope shall be less than or equal to 1:1.4 (H:V)
- Hydraulic jump may be completed within the culvert barrel

In situations where the runout section is too short and/or there is insufficient tailwater for a jump to be completed (or initiated) within the barrel, an outlet weir or a drop of the outlet followed by an outlet weir shall be designed. Sills are effective in forcing the hydraulic jump in broken-back culverts and in spreading the water back to the natural stream width.

D. External Dissipators

External dissipators shall be designed where the outlet scour hole is not acceptable, a moderate amount of debris is present, and the culvert outlet velocity is moderate, Fr < 3. Various types of external dissipators are discussed in the following sections.

Impact basin U.S. Bureau of Reclamation Type VI – The USBR Type VI basin was developed by the USBR and is contained in a relatively small box-like structure, with a vertical baffle. An opening is provided between the bottom of the baffle and the floor of the box.

The use of impact basin USBR Type VI is not recommended where debris or ice buildup may cause substantial clogging. The design of impact basin USBR Type VI shall achieve the guidelines presented below:

- Valid for discharges up to 400 cubic feet per second and velocities as high as 50 feet per second.
- In situations where the culvert entering the basin has a slope greater than 27 percent, the basin shall be constructed on a horizontal grade.
- The culvert shall provide a horizontal section at least 4 culvert widths in length immediately upstream of the dissipator.
- The end of the basin shall be provided with a low sill that, where feasible, shall be set at the same elevation as the downstream channel.
- Where needed to retain the roadway embankment, the end of the basin may be provided with an alternate end sill and 45 degree wingwalls.
- Where the velocities of flows exiting the basin exceed 5 feet per second, the channel downstream of the basin shall be provided with a riprap apron, per the guidelines presented in the section on ‘Riprap Aprons’.
- A moderate depth of tailwater will improve its performance. However, the tailwater depth shall not be above half of the height of the baffle.

Hook type impact basin energy dissipator – The hook energy dissipator is a type of impact basin that abates culvert outflow velocities by means of 3 hook-shaped blocks and an end sill in a uniform trapezoidal channel or a warped wingwall basin. The minimum criteria for hook type basins are presented below:

- Hook type Basin with Uniform Trapezoidal Channel
  - The side slopes of the basin shall be between 1.5:1 (H:V) and 2:1 (H:V), and the bottom width of the basin shall be 1 to 2 times the effective opening width of the culvert.
  - Where scour may occur, a riprap apron shall be provided downstream of the basin, per the guidelines presented in the section on ‘Riprap Aprons’.
  - A cutoff wall shall be provided at the end of the basin.

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  - These basins may be used where the Froude number of the culvert outflow is between 1.8 and 3.0.

- Hook type basin with warped wingwalls
  - Wingwalls warped from vertical at the culvert outlet to side slopes of 1.5:1 (H:V) at the end sill are recommended.
  - The recommended ratio of hook width/culvert width is 0.16.
  - The spacing between hooks shall be within the range of 1.5 to 2.5 times the hook width.
  - The height of wingwalls shall be at least twice the flow depth at the culvert exit.
  - A flare angle of 5.7 degrees per side is the optimum value for Fr > 2.45.
  - Where scour may occur, a riprap apron shall be provided downstream of the basin, per the guidelines presented in the section on 'Riprap Aprons'.

Riprap aprons – Riprap aprons for culverts shall be designed in accordance with outlet protection criteria mentioned in Section 8.5.3.4 of this chapter.

Riprap aprons for energy dissipators shall be designed per the equation:

$$D_{50} = \frac{0.692}{(S-1)}\left(\frac{V^2}{2g}\right)$$

Where:

D_{50} = median rock size (feet)

S = rock specific gravity (pound per cubic foot)

V= velocity at the end of energy dissipator (feet per second)

E. Stilling Basins

Stilling basins shall be used where the outlet scour hole is not acceptable, debris is present, and the culvert outlet velocity (V_o) is high, Fr > 3.

Riprap basin – The riprap basin shall be considered where the standard riprap apron or other energy dissipators are inadequate. A riprap basin is a depressed area of riprap placed at the outlet of a high velocity culvert, storm drain outlet, or open channel. Recommended minimum criteria for riprap basin are as follows:

- The basin shall be pre-shaped and lined with riprap that is at least 2 x D_{50} thick.
- The ratio of depth of scour (ds) to rock size (D_{50}) shall be greater than 2.
- The length of the energy dissipating pool shall be 10 x (ds), and the length of the apron 5 x (ds).

- A riprap cutoff wall or sloping apron shall be constructed if downstream channel degradation is anticipated.

Saint Anthony Falls stilling basin – The Saint Anthony Falls stilling basin uses a forced hydraulic jump to dissipate energy. The design consists of a sloping chute with chute blocks at its base, followed by blocks on the floor of the basin. The basin floor also has a sill located at the downstream end. The basin sidewalls may be parallel for a rectangular stilling basin or may diverge, beginning at the downstream toe of the chute to create a flared stilling basin. A cut-off wall and wingwalls shall be provided at the end of the stilling basin. Minimum design criteria for a Saint Anthony Falls stilling basin is provided below:

- Recommended where Fr = 1.7 to 17
- Requires a sufficient tailwater for efficient operation
- Sidewall flare shall not be greater than 0.5:1, 0.33:1, or flatter
- Height of the baffles is set equal to the entering flow depth
- Wingwalls shall be equal in height and length to the stilling basin sidewalls. Top of the wingwall shall have a 1:1 (H:V) slope.

8.5.2.5 Siphons

Inverted siphons (sometimes called sag culverts or sag lines), although not desirable, may be used to convey water by gravity under roads, railroads, other structures, various types of drainage channels and depressions. An inverted siphon structure shall operate without excess head when flowing at design capacity and shall not be used for drainage or irrigation where freezing may block the siphon's waterway.

Inverted siphons shall be used as follows:

- To carry flow under obstructions such as sanitary sewers, water mains, or any other structure or utility that may be in the path of the storm drain line
- Where avoidance or adjustment of the utility is not practical

This section presents the minimal criteria and design standards for the hydraulic evaluation and design of inverted siphons. For criteria not included in this section, the following reference(s) shall be used:

- Caltrans HDM design criteria shall be followed for design of siphons or sag culverts within roadways or highways.

A. <u>Pipe Material and Size</u>

Several pipe materials may be used for siphon construction:

- Welded smooth steel pipe with internal ceramic coating

- Precast reinforced concrete pressure pipe
- Reinforced plastic mortar pressure pipe

The conduit size through the inverted siphon used as a storm drain system shall be the same size as either the approaching or exiting conduit. In no case shall the conduit size be smaller than the smallest of the approaching or exiting conduit.

B. Transitions, Head Losses, Cover and Slope

Transitions are defined as the inlet and outlet portion of an inverted siphon and shall be used to reduce head losses and prevent channel erosion in unlined channels. Siphon transitions shall be located outside the Authority's right-of-way. Concrete inlet and outlet transitions shall be used for the following:

- Siphons crossing tracks and paved roadways
- 36-inch diameter and larger siphons crossing narrow (< 30 feet), unpaved, off-system roads
- Siphons in unlined channels with water velocities in excess of 3.5 feet per second in the siphon

Because an inverted siphon includes slopes of zero and adverse values, head losses through the structure due to friction, bends, junctions, and transitions shall be accounted. Sound engineering judgment shall be used to determine the maximum limits of head losses and if determined unacceptable, an alternative for the siphon design shall be considered. The total computed head loss shall be increased by 10 percent as a safety factor to ensure the siphon is not causing unexpected backwater.

The siphon profiles shall satisfy requirements of cover, siphon slopes, bend angles, and submergence of inlet and outlet.

- At siphons crossing tracks and roadways or highways, the minimum cover shall be based on the structural requirements of the siphon material.
- At siphons crossing under natural drainage channels, a minimum of 3 feet of compacted earth cover shall be provided.
- At siphons crossing under an earth channel, a minimum of 3 feet of compacted earth cover shall be provided.
- At siphons crossing under a lined channel, a minimum of 3 feet of compacted earth cover shall be provided between the bottom of the channel lining and the top of the siphon.

Siphon slopes shall not be steeper than 2:1 (H:V) and shall not be flatter than a slope of 0.005 feet/feet.

C. Velocity

The following velocity criteria are to be used in determining the length of the siphon:

- 3.5 feet per second or less for a short siphon not located under a trackway or roadway with only earth transitions provided at entrance and exit
- 2.5 feet per second or less for a short siphon located under tracks or roadways with either a concrete transition or control structure provided at both inlet and outlet
- 10 feet per second or less for a long (> 200-foot) siphon with either a concrete transition or control structure provided at the inlet and a concrete transition provided at the outlet

D. Collars and Blowoff Structures

Collars are placed at intervals along the siphon to reduce the velocity of any water moving along the outside of the siphon or through the surrounding earth, thereby preventing removal of soil particles (piping) at the point of emergence. Siphon collars shall not be used unless piping computations or observations of burrowing animals indicate they are needed.

Blowoff structures are provided at or near the low point of inverted siphons to permit draining of the siphon for inspection and maintenance or shutdown. Siphons greater than 18 inches in diameter shall be equipped with a blowoff structure. An access hole or similar access shall be included with a blowoff on long siphons 36 inches and larger in diameter to provide an access point for inspection and maintenance. To facilitate removal of any accumulated sediments and to expedite the draining process, an 8-inch minimum gate valve shall be used. The drain pipe shall outfall where drain water will not cause any damage.

E. Freeboard

Upstream channel freeboard is commonly provided to accommodate intercepted storm runoff, improper operation or drift blockage. Freeboard criteria for siphons are as follows:

- The channel bank freeboard upstream from siphons shall be increased 50 percent or 1.0 foot maximum to prevent washouts at these locations.
- The increased freeboard shall extend upstream a distance from the structure such that damage caused by overtopping the channel banks would be minimal.
- If the freeboard extends upstream from the transition inlet, a minimum distance as determined by dividing the freeboard height by the channel slope shall be used. And, for freeboard downstream from the outlet transition, a minimum distance of 50 feet or to the Authority's right-of-way, whichever is less, shall be used.

F. Wasteways

Wasteways are often placed upstream from a siphon transition to divert the channel flow in case of an emergency. A wasteway, either separate or integral with the inlet transition, shall be provided where significant damage would occur due to escaping channel waters. Escaping waters shall be conveyed to a point and released in a manner to avoid trackway, roadway or property damage. Wasteways are not permitted within the Authority's right-of-way.

G. Safety Devices

Safety measures (e.g., fences, grates) shall be provided near siphons to protect persons and animals from injury and loss of life. A hazard can occur both when the siphon is operational or dry. Inlet and outlet transitions shall be hydraulically efficient and have removable grates to minimize the hazard associated with human or animal ingress or debris blockage.

8.5.2.6 Pump Stations

The use of pump stations at sag or sump points shall be avoided, when practical. Long-term operation and maintenance costs shall be identified and considered prior to implementation of pumps. The use of a gravity system shall be fully evaluated through the use of long pipelines or adjustments to the grade or track profiles, before pumps can be considered for the project. Where a gravity system cannot be provided and pump stations are unavoidable, pump station design within the corridor shall conform to the following:

- FHWA, Hydraulic Engineering Center (HEC)-24 on Highway Stormwater Pump Station Design
- *Mechanical, Supervisory Control and Data Acquisition Subsystems,* and *Facility Power and Lighting Systems* chapters of this Design Criteria

For pump stations installed for the project, but owned, operated, and maintained by local agencies or a third party, the criteria to use shall be in accordance with the local governing agency.

Pump stations shall be designed to accommodate the inflow from a storm event according to Section 8.4.3. All possible flow shall bypass or pass-through downstream of the pump station, to reduce pumping requirements. Hydrological analyses of the watersheds that may discharge to the pump stations shall be carefully evaluated to avoid potential off-site drainage diversions from adjacent watersheds. The pump station design shall also address future build-out of the tributary watershed to verify that the pump station can handle increases in flow.

Pump stations shall be designed to accommodate space for equipment cabinets and conduit and cabling to provide for SCADA control.

8.5.3 Culvert Hydraulics

Existing drainage facilities within the corridor shall not be negatively impacted due to the proposed design. Where a transverse undercrossing is required to convey surface runoff, flood waters, and/or existing streams across the Authority's right-of-way, the crossing shall be provided within a culvert. When runoff is increased, existing culverts shall be upsized to allow for increase in flow.

The following sections outline minimal culvert criteria. For criteria not addressed in this section, the following references shall be used:

- AREMA shall be followed for design of culverts along the corridor.
- Caltrans HDM shall be followed for design of culverts along roadways and highways impacted by the improvements.
- Local criteria shall be followed if AREMA and Caltrans criteria do not specify.

8.5.3.1 Design Elements

Numerous cross sectional shapes are available to be used as culverts. The shape selection shall be based on the cost of construction, the limitation on the upstream water surface elevation, track and roadway embankment, available cover, and hydraulic performance.

The following culvert criteria shall be applied:

- Minimum culvert diameter/rise for trunk drains and culverts crossing under the track shall be 36 inches.
- Minimum culvert diameter/rise for lateral drains shall be 18 inches.
- Culverts and drains under platforms and station areas shall be a minimum of 18 inches.
- Culverts crossing under trackway shall not be placed within the limits of prepared subgrade unless the backfill material for the culvert and its compaction meet the requirements of prepared subgrade, refer to *Geotechnical* chapter for limits of prepared subgrade, material and compaction requirements. Culverts crossing under trackway shall be a minimum of 6 feet below top of rail and 3 feet below the flow line of ditch along the trackway.
- Culverts, within 45 feet of track centerline, shall have a minimum cover of 4 feet; culverts elsewhere shall have a minimum cover of 3 feet.
- In locations where the above criteria are not practical, reduced clearance may be provided with approval of the governing agency.

The selection of a culvert material shall consider structural strength, hydraulic performance and roughness, durability, corrosion and abrasion resistance. Common culvert materials are concrete, corrugated aluminum and corrugated steel. Culverts and storm drains passing beneath tracks or maintenance roadways shall be reinforced concrete pipe (RCP). Culverts and drains under platforms or in station areas that are not under tracks shall be RCP, polyvinyl chloride (PVC), high-density polyethylene (HDPE), or corrugated steel.

8.5.3.2 Location, Skew, and Slope

Culverts shall be placed to allow for cross-passage of surface runoff, flood waters, and where existing streams may exist. This may reduce embankment erosion, minimize debris buildup and, if placed often enough, limit carryover of drainage from one watershed to another.

Culverts shall be in alignment and on the same gradient with existing streambeds. Curvatures in the alignment of the culvert and angle points shall be avoided.

The slope of the culvert shall be the same gradient of the existing streambed unless the topography is generally flat, in which case the invert of the inlet and outlet structures shall be designed to avoid sedimentation in the culvert. A minimum self cleaning velocity of 2.5 feet per second shall be used for culvert design.

8.5.3.3 Inlet Structure

For inlet control, a maximum allowable headwater of 1.5 times the culvert diameter/rise shall be used. For the 100-year storm event, a minimum freeboard between the water surface elevation and the subballast shall be 2 feet.

Upstream properties shall be protected from ponding and backwater effects from an undersized culvert. Ponding at the inlet structure shall be prevented. A larger culvert size shall be implemented if ponding and discharge backup are anticipated at the culvert entrance. Overtopping of the tracks is not permitted.

Sound engineering judgment shall be applied to inlet structure design.

A. Headwalls

Headwalls shall be used in locations where right-of-way is constrained, in areas where vegetation growth on slopes is limited, to improve the culverts appearance and increase the hydraulic efficiency. The headwall shall be designed to have adequate strength and proportion to prevent sliding or overturning from excessive soil pressures and to prevent settlement.

B. Wingwalls

Wingwalls shall be considered to prevent erosion and scour and provide soil stability around the proposed culvert. Perpendicular, oblique, or parallel wingwalls, or a combination thereof, shall be used, depending on the physical and hydraulic conditions involved. Wingwalls shall be carefully designed not to alter the historical flow patterns of the existing stream and prevent turbulence during peak storm events. An entrance apron and cutoff wall shall also be considered to prevent erosion and scour and increase hydraulic efficiency at the inlet.

C. Flared End Sections

Flared end sections improve hydraulic performance of the culvert by allowing a smooth transition between the natural channel and culvert and are aesthetically appealing. Due to these reasons, flared end sections are preferred to headwalls and wingwalls within the corridor.

8.5.3.4 Outlet Structure

Due to the unnatural constriction and material of culverts, the outlet velocity is generally higher than natural stream velocities. Energy dissipators or outlet embankment protection, such as slope paving, riprap, headwalls and wingwalls, end sections, cutoff walls and toe walls, shall be provided at culverts along the corridor to minimize downstream erosion and reduce drainage velocities. Natural flow patterns of the existing stream shall be restored downstream of the culvert, with special consideration of the channel transition for the 100-year flood.

Refer to Section 8.5.2.4 for design of energy dissipators.

8.6 Trackway and Facility Drainage Systems

8.6.1 Track Drainage Systems

Standing water along rail tracks may shunt the signal circuits causing signal failures. Hence, standing water along rail tracks is not permitted and shall drain away from the tracks. Track drainage shall be provided to drain stormwater from tracks including cable troughs and right-of-way. Drainage criteria for cable troughs in at-grade sections are provided in the *Civil* chapter. A minimum 4-foot flat bottom ditch shall be used for drainage along the trackway toe of embankments or bottom of cut sections. Refer to Standard and Directive Drawings. Where a minimum 4-foot flat bottom ditch cannot be accommodated due to constraints, a V-ditch may be used. Underdrain track drainage pipe systems shall be used where right-of-way constraints make the standard flat bottom ditch or V-ditch unfeasible. Track drainage systems shall be used as a longitudinal drainage system to capture the onsite runoff generated from the trackway. The collected runoff may be conveyed to a local storm drain system, based on the hydraulic capacity or, conveyed to the nearest natural water body, after applying appropriate treatment measures as defined in the BMP section of this chapter. Refer to Standard and Directive Drawings for detailed track drainage configuration. Pipe size, material, slope, and access hole requirements for a closed track drainage system shall be consistent with the criteria in Section 8.5.2.3.

In a shared corridor, drainage shall be kept separate from the drainage system of the third party operator. If any upgrades to the third party's drainage system are anticipated, they shall be performed based on the third party's design requirements.

For criteria not included in this section, references shall be used as follows:

- AREMA shall be followed for design of new track drainage facilities.

8.6.2 Embankments and Cut Slopes

For embankments that support the HST trackway and for cut slopes, refer to the *Geotechnical* chapter for drainage and slope protection requirements.

8.6.3 Bridges/Aerial Structures

Design of HST bridges and aerial structures over waterways and associated drainage facilities shall be coordinated with local agencies or the U.S. Army Corps of Engineers. The 2 basic designs involved in this section are the HST aerial structures and HST structures over waterways (HST bridges).

This section presents the minimal criteria and design standards for the hydraulic evaluation and design associated with bridge drainage within the corridor. For criteria not included in this section, the following references shall be used:

- AREMA's "Roadway and Ballast" chapter for detailed information on the magnitude and level of scour created at piers and abutments and countermeasures
- HDS-01, Hydraulics of Bridge Waterways, FHWA for bridge hydraulic analyses
- HEC-21, Design of Bridge Deck Drainage, FHWA for bridge deck drainage design
- HEC-09, Debris Control Structures Evaluations and Countermeasures, FHWA for mitigating debris impacts to bridge structures
- Refer to local agency manuals for local criteria within each jurisdiction.
- The *Structures* chapter shall be referenced to ensure coordination with structural design of bridges and aerial structures.

8.6.3.1 Freeboard Protection

For the hydraulic design of bridges and aerial structures, a minimum of 2 feet of freeboard above the design frequency water surface elevation shall be provided.

8.6.3.2 Erosion Control and Scour Protection

Protection measures shall be taken to protect bridges or aerial structures, piers, and embankments from erosion and scour. Measures include providing riprap and/or vegetation on the slopes and streambed and increasing the distance between embankments. Where applicable, energy dissipators may be provided in accordance with criteria provided in Section 8.5.2.4.

8.6.3.3 Pier Design and Location

For structures over waterways, the spacing and location of the structural piers can significantly affect the hydraulic characteristics of the existing waterways. In locations where pier columns and protection walls interfere with drainage, an alternative drainage facility shall be provided to collect and carry water to a drainage system.

Piers shall be located outside of drainage channels and natural washes, where possible, to minimize negative impacts associated with scour and erosion at the pier. Where piers are located within channels, a streamlined design at the pier nose shall be considered. This shall be obtained by providing circular or rounded shapes at the upstream and downstream faces of piers in order to reduce flow separation, aligning bents with the direction of flow and increasing the length of the bridge to decrease velocities.

Debris buildup may occur at piers which can reduce the hydraulic capacity of the channel, increase the local scour, and potentially cause the pier to fail. The design shall consider the type of debris that could impact the pier. Depending on the debris type, protective devices such as steel plates, debris deflectors, wingwalls, and upstream debris catchment structures shall be used.

8.6.3.4 Deck Drainage System

Stormwater on a bridge or aerial structure surface can affect the water spread on the structure into the trackway, cause complications with maintenance, and negatively impact the aesthetics of the structure, by corrosion or debris. The deck drainage system includes the bridge or aerial structure deck, gutters, inlets, pipes, downspouts and end collectors, which are discussed in the following subsections.

A. Bridge/Aerial Structure Deck

A longitudinal drainage system shall be provided along the deck to minimize standing water on the bridge or aerial structure. Criteria for bridge or aerial structure deck design are as follows:

- For bridges and aerial structures with ballasted track, the minimum half circle perforated corrugated galvanized drain pipe, embedded in the ballast, shall be 8 inches minimum.
- For bridges and aerial structures with non-ballasted track, a drainage trough shall be designed to convey the deck drainage.
- The cross slope of the bridge/aerial structure deck shall be 2 percent.
- Standing water on the bridge or aerial structure shall not be permitted.

B. Inlets and End Collectors

The bridge and aerial structure end collectors are drainage inlets that collect flow before it reaches the structure and prevent flow from leaving the bridge or aerial structure. End collectors are typically drop inlets which convey a higher capacity; slotted drains may be used. Stormwater upstream of a bridge or aerial structure shall be fully collected prior to reaching the structure. To avoid flooding on the bridge or aerial structure and backup in the pipes, inlets and drains on the bridge or aerial structure shall account for a 50 percent clogging factor. Water accumulating on structure decks with ballasted tracks shall be drained by a semicircular or U-shaped channel formed in the concrete invert between track centerlines or at the low point of a single track aerial structure. Inlets shall be provided at intervals to collect the flow into the storm drainage system.

C. Pipes and Downspouts

The minimum longitudinal slope of drain pipes inside the box girder shall be 1 percent or generate a minimum velocity of 2 feet per second. Downspouts shall be considered in the aesthetics of the bridge or aerial structure. Pipes and downspouts located within the concrete of the structure provide more challenges for access and maintenance. Cleanouts shall be provided at convenient and accessible locations along the pipe. Cleanouts shall be located such that these can be reached from the ground for easy access for personnel, and at places where the pipes bend and debris build-up may occur. Cleanout locations shall be identified.

Outfalls from downspouts may discharge directly into storm drains, or nearby receiving water, considering the water is treated before discharging offsite. If the downspout discharge is directly to surface drainage, the free-falling water shall not come into contact with the structure members to avoid corrosion and deterioration. Stormwater from the bridge or aerial structure shall also not negatively impact the surface below; erosion control devices may be necessary at the outfall location and the surface channel shall be designed to carry and transfer the increase in flow. Downspouts that discharge directly to a storm drain shall connect to a manhole for easy access. The outfall invert shall be a minimum of 0.25 feet higher than the manhole invert to avoid debris clogging the storm drain.

Refer to Standard and Directive Drawings for drainage of aerial structures.

8.6.4 Tunnels

For drainage requirements in tunnels, refer to Standard and Directive Drawings. Drainage from tunnel and cut-and-cover structures shall discharge to portals or to a low-point sump in pump station.

8.6.5 Retaining Walls

Provide a concrete lined gutter behind retaining walls to redirect storm runoff away from the walls, refer to Standard and Directive Drawings. For other drainage requirements at retaining walls, refer to the *Geotechnical* chapter.

8.6.6 Trenches

Where the ground water table is high and interferes with the stability of track bed and side slopes, the trackway shall be in a trench section. Trench drainage discharges to a low-point sump in pump station and is discharged to local drainage system. Refer to the *Structures* chapter for trench wall heights for flood protection. For drainage concept in trench sections, refer to Standard and Directive Drawings.

8.6.7 Critical HST Structures/Facilities

HST critical facility sites, such as traction electrification system, automatic train control, communications, vent structures, traction power supply sites, operation control centers, yards, etc., shall be designed to drain so that the finish floor elevation or top of slab foundation of the facility sites remain 6 inches above a 500-year flood elevation or 2 feet above 100-year flood elevation, whichever is greater.

8.6.8 Stations and Platforms

Refer to the *Stations* chapter for station area and station platform drainage requirements. Station area drainage should discharge to the nearest municipal drainage collection system after proper treatment as necessary. Refer to Standard and Directive Drawings for station trackway drainage system within the station area.

Station drainage shall accommodate deluge system for depressed and underground stations.

8.6.9 Roadways

The following sections outline minimal drainage criteria for roadways, street improvements, parking lots and storm drains. For criteria not addressed in this section, the following references shall be used:

- Caltrans HDM criteria shall be followed for design of roadway drainage, including subsurface drainage facilities within the corridor.
- For third party roadway drainage requirements, refer to third-party's drainage criteria.

8.6.9.1 Surface Drainage

Physical characteristics of surface drainage for roadways leading to stations, yards, and other wayside facilities shall accommodate the following:

- Cross flow shall not be more than 0.1 cubic feet per second.
- Ponding shall be prevented in parking lots where vehicle stalls are located.
- Sound engineering judgment shall be used with careful consideration to train, vehicular and pedestrian traffic.

8.6.9.2 Storm Drain Design

Storm drains shall be designed in coordination with roadway and trackway surface flow. Inlets, subsurface piping, and maintenance access shall be considered when designing storm drains. Refer to the *Utilities* chapter for clearance requirements of storm drain facilities with adjacent infrastructure facilities.

8.6.9.3 Inlets and Maintenance Access

Inlet types may include curb-opening inlets, grate inlets, slotted drain inlets and a combination of all these types. Inlet type shall consider the proposed location of the inlet and the required capture capacity the inlet must convey. The location and spacing of inlets shall be designed to prevent ponding and flooding in traveled lanes. Inlets are generally placed upstream of pedestrian ramps and street intersections. Grate inlets are not recommended for pedestrian pathways. Inlets shall be placed prior to a superelevation reversal to prevent cross flow on the roadway.

Inlet capacity depends on the size and shape of the opening, grate type and the roadway geometry upstream of the inlet. To mitigate back-up and ponding of water, a minimum clogging factor of 50 percent shall be used for all inlet designs.

Design of manholes shall conform to the following maintenance hole spacing requirements:

Table 8-5: Maximum Manhole Spacing Requirements

Pipe Diameter	Maximum Spacing Requirements
Any pipe that cannot reach a self-cleaning velocity of 2.5 feet/sec.	300 feet
Less than 24-inches	400 feet
24 inches to 48 inches	400 feet to 800 feet
Greater than 48 inches	1,200 feet

Engineering judgment shall be used in determining spacing requirements. Access logistics for pipe maintenance shall be considered. Manholes and associated storm drains shall be located out of the roadway traveled way and intersections to avoid disruptions to traffic. If the subsurface drain is proposed to cross the tracks, manholes shall be considered for use at the Authority's right-of-way limits.

8.6.9.4 Pipe Characteristics

Subsurface pipes shall be designed at full flow capacity. To determine the appropriate pipe size, shape and material, Manning's equation shall be applied as described in Section 8.5.2. Storm drains may operate under pressure, provided the pipe material will not be jeopardized due to pressurization; however, the hydraulic grade line shall not rise above the manhole or inlet structure, and shall be a minimum of 1 foot below the surface finished grade.

Generally in urban areas, storm drain pipes are RCP material to provide a longer life of the system and minimize maintenance over the life of the pipe. RCP pipes shall be designed for facilities crossing under the tracks. Other pipe materials, including corrugated steel, HDPE and PVC may be considered along the corridor and at train stations, based on a life cycle cost analysis to justify the use of pipe type. For corrosion protection of metallic and RCP pipes refer to the *Corrosion Control* chapter. Criteria for pipe size and cover are as follows:

- Minimum pipe size within roadways shall be 18 inches
- A 3-foot minimum cover shall be provided for pipes

8.7 Detention/Retention of Surface Water Runoff

The main purpose of a detention basin is to temporarily store runoff volume to reduce peak discharge by allowing flow to be discharged at a controlled rate. This section provides minimal design criteria for hydraulic evaluation and design of detention basins, for the purpose of flood control and stormwater management.

Caltrans identifies detention basins as a design pollution prevention BMP, which temporarily detains runoff to allow sediment and pollutant to settle. If the detention basins are designed to include the purpose of a BMP, reference shall be made to Section 8.8. Proper treatment of polluted runoff shall be included in the design of the detention/retention basins. Retention storage can be defined as a depression or low point where water accumulates with no possibility for escape as runoff. These facilities allow for infiltration into underlying soils, and are considered as "Infiltration Devices."

Several factors such as the peak outflows, spillway sizing, and sedimentation govern the design of detention facilities. For design criteria and detailed design methodologies of detention basins not included in this section, refer to HEC-22, Urban Drainage Design Manual, FHWA.

8.7.1 Peak Flow Reduction and Release Rate

The facility's outlet structure shall limit the maximum outflow to allowable release rates. The maximum release rate may be a function of existing or developed runoff rates, downstream channel capacity, potential flooding conditions, and/or local agency regulations.

The system shall be designed to release excess stormwater expeditiously to ensure that the entire storage volume is available for subsequent storms. The facilities may need a paved low flow channel to ensure complete removal of water and to aid in nuisance control.

8.7.2 Required Storage Estimate

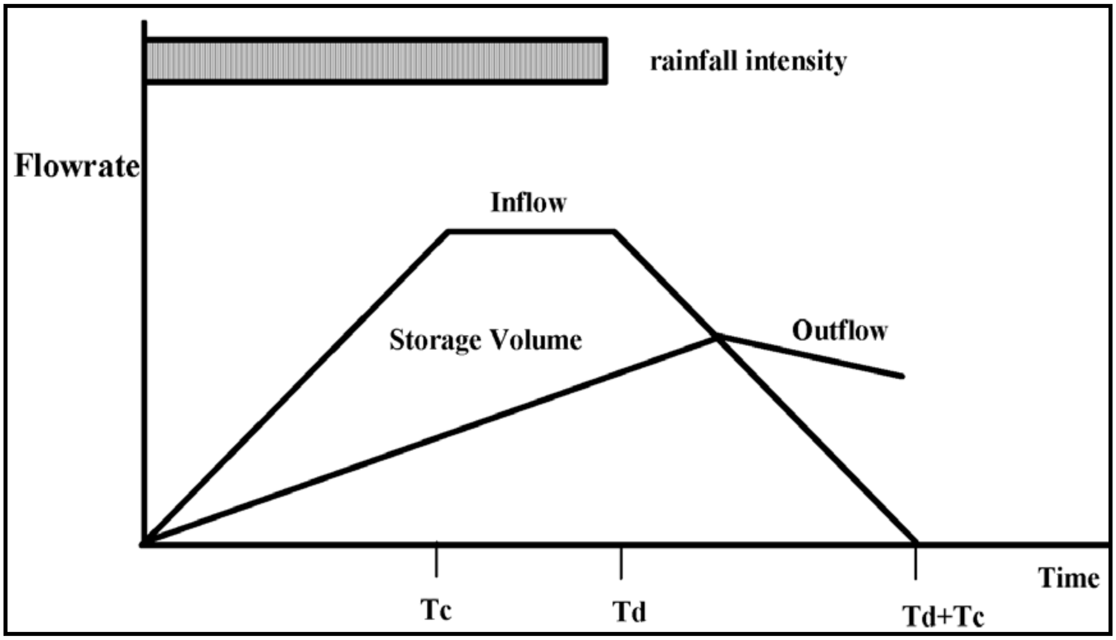
For watersheds greater than 150 acres, the required volume of storage for a detention basin that is necessary for peak flow reduction can be estimated as the difference between the inflow and the outflow hydrographs, using the Hydrograph Method.

The Volumetric Method shall be used to estimate the runoff detention/retention volume for watersheds less than 150 acres for the urban catchment. In this case, uniform rainfall is assumed and the required storage volume can be estimated by the volume difference between the rainfall volume coming to the basin and the runoff volume released from the basin. For simplicity, trapezoidal hydrograph depicted on Figure 8-1 can be considered for volume calculation. The inflow hydrograph has a linear rising limb over the time of concentration of the tributary watershed and the peaking portion of the inflow hydrograph is a plateau from the time of concentration, to the end of rainfall event. The effective rainfall volume is represented by the storage volume shown in the graph, where:

T_c = Time of Concentration (minutes)

T_d = Rainfall Duration (minutes)

Figure 8-1: Trapezoidal Hydrograph



Source: "Detention Basin Sizing for Small Urban Catchments", ASCE J. of Water Resources Planning and Management, Vol. 125, No. 6, November.

8.7.3 Basin Sizing

A stage-storage relationship defines the relationship between the depth of water and storage volume in a detention basin facility. The basin width to length ratio shall be greater than 2 so that the flood flows can sufficiently expand and diffuse into the water body to enhance the sedimentation process. Slopes on embankments shall maintain the bank slope stability. Slopes on earthen embankments shall not be steeper than 4:1 (H:V) and on riprap embankments shall not be steeper than 3:1 (H:V).

Detention basins may be designed in rectangular, triangular, trapezoidal or elliptical shapes, depending on the available right-of-way and required volume of storage. Once the storage volume is determined, the storage basin configuration is determined by multiple layers to accommodate the 10-, 25-, and 100-year storm event storage volumes.

8.7.4 Outlet Structures and Emergency Spillways

Outlet structures of a detention basin consist of low flow outlets and emergency spillway. The outlet structure is formed by risers, perforated plates, orifices, weirs, and culverts. The minimum size of a low flow outlet shall be 18 inches. The low flow outlets are designed based on the orifice principle, while the emergency spillway is designed based on weir equations. The invert of the emergency spillway shall be 2 feet above the major design stormwater surface elevation. It is recommended to have a minimum of 1 foot freeboard.

The obstruction to low flow conduits by debris can reduce outlet design release rate and cause the premature filling of the detention basin with stormwater, reducing the flood protection provided by the structure. All outlet works and low flow conduits shall be provided with a trash rack for debris control. The trash rack shall provide a maximum bar spacing not to exceed two-thirds of the outlet opening or diameter. The total area of the trash rack shall allow for passage of the design flow with 50 percent of the trash rack blocked.

8.8 Stormwater Quality Management

The Stormwater Quality Management program shall comply with requirements of the stormwater discharges from facilities regulated by National Pollutant Discharge Elimination System permit issued by the State Water Resources Control Board, Regional Water Quality Control Boards (RWQCBs) permit and various environmental permits issued by the appropriate local regulatory agencies.

Where a local storm drainage system is not available, such as in rural farm land areas, water treatment requirements shall follow the environmental permits issued by the appropriate regulatory agencies.

An effective stormwater management program involves incorporating stormwater BMPs during the planning and design phases (permanent BMPs), as well as during the construction phases (temporary BMPs) of a project. The Designer shall identify the pollutants of concern in the stormwater discharge because they can have numerous negative impacts such as the following:

- Reducing the storage capacity of hydraulic facilities due to the deposition of sediment and silt
- Increasing toxic release to aquatic life due to the metal dust and toxic fluids from train/vehicle leaks
- Contributing to non biodegradable pollutants such as street litter

Two major types of permanent BMPs are design pollution prevention BMPs and treatment BMPs. The Storm Water Data Report (SWDR) is a project specific document that documents the permanent BMPs, and shall be prepared for each phase of the project. The SWDR shall be prepared based on Caltrans procedures. The most common temporary BMPs are the construction site BMPs, usually documented using a Storm Water Pollution Prevention Plan (SWPPP). For guidance on evaluation, selection, and design criteria on permanent BMPs, SWDR, temporary BMPs, and SWPPP, the following references shall be used:

- Caltrans Storm Water Quality Handbooks, Project Planning and Design Guide, for design of BMPs
- Caltrans HDM design criteria

- Caltrans Storm Water Quality Handbooks, Construction Site Best Management Practices manual for implementation of stormwater BMPs during construction
- RWQCB shall be contacted and/or referenced for guidance, where necessary.

8.9 Application of Approved Software

The use of industry accepted hydrologic/hydraulic design programs is recommended. Where the drainage facilities impact or connect to facilities owned by others, local agency criteria shall be applied. The Caltrans HDM General Aspects chapter may be referred for approved software in use.

Chapter 10

Geotechnical

Revision	Date	Description
0	02 Mar 12	Initial Release, R0
0.1	07 Dec 12	Addendum 07 – CP01
1	June 2013	Revision 1
2	Feb 2014	Revision 2

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Appendix

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
Authority	California High-Speed Rail Authority
BDS	Bridge Design Specifications
Caltrans	California Department of Transportation
CBC	California Building Codes
CBR	California Bearing Ratio
CEG	Certified Engineering Geologist
CGS	California Geological Survey
CHSTP	California High-Speed Train Project
CPT	Cone Penetration Test
CPT_u	Cone Penetration Test with pore pressure measurements
FLH	Federal Lands Highway
FOS	Factor of Safety
FRA	Federal Railroad Administration
GBR	Geotechnical Baseline Report
GBR-B	Geotechnical Baseline Report for Bidding
GBR-C	Geotechnical Baseline Report for Construction
GDR	Geotechnical Data Report
GEDR	Geotechnical Engineering Design Report
GIP	Geotechnical Investigation Plan
GTGM	Geotechnical Technical Guidance Manual
HST	High-Speed Train
IGM	Intermediate Geomaterials
LOTB	Logs of Test Borings
LRFD	Load and Resistance Factor Design
MASW	Multichannel Array of Surface Wave
M-O	Mononobe-Okabe
MCE	Maximum Considered Earthquake
NEHRP	National Earthquake Hazards Reduction Program
OBE	Operating Basis Earthquake
OCS	Overhead Contact System
PDA	Pile Driving Analyzer
PDDM	Project Development Design Manual
PGA	Peak Ground Acceleration
RAM	Reliability, Availability, and Maintainability
SASW	Spectral Analysis of Surface Waves

SSI	Soil-Structure Interaction
USGS	United States Geological Survey

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

Elements of HST infrastructure, based on their importance to HST, shall be classified as Primary Type 1, Primary Type 2, Complex, 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, Primary Type 2, and Complex structures, while the Secondary structures shall be subject to the requirements of the governing local jurisdiction.

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards. Geotechnical design work for Primary Type 1, Primary Type 2, and Complex Structures shall be in accordance with AASHTO LRFD BDS with California Amendments, these geotechnical design criteria, and the requirements of the following standards and guidelines. Use of the LRFD methodologies in some earthquake engineering and geotechnical engineering areas requires careful examination for applicability. However, any variation to the LRFD methodologies is subject to the Design Variance Process as noted in the *General* chapter.

Standards

- American Association of State Highway and Transportation Officials (AASHTO)
  - AASHTO Standard Specifications for Highway Bridges
  - AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals
  - 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 Bridge Design Specification – AASHTO LRFD Bridge Design Specifications and California Amendments (to the AASHTO LRFD Bridge Design Specifications), hereafter referred to as “AASHTO LRFD BDS with California Amendments”
  - Caltrans Seismic Design Criteria (CSDC)
  - Soil and Rock Logging, Classification, and Presentation Manual, June 2010

- California Building Code (CBC)
- International Union of Railways (UIC) Code 719R Earthwork and Trackbed for Rail Lines (2008)

Guidelines

- 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), 2011
  - FHWA Geotechnical Technical Guidance Manual (GTGM), 2007
  - Geophysical Methods - Technical Manual (Application of Geophysical Methods to Highway Related Problems, cooperatively with Blackhawk Geosciences), DTFH68-02-P-00083, 2003
  - Soils and Foundations Workshop, NHI Course No. 132012, Volumes I and II FHWA-NHI-06-088, and FHWA-NHI-06-089, 2006
  - Subsurface Investigations – Geotechnical Site Characterization, NHI Course Manual No. 132031, FHWA-NHI-01-031, 2002
  - Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034, 2002
  - 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-10-034
  - FHWA Drilled Shafts: Construction and Procedures and Design Methods, FHWA-IF-99-025
  - FHWA Mechanically Stabilized Earth Walls and Reinforced Soil Slope Design and Construction Guidelines, FHWA-NHI-00-043
  - FHWA Earth Retaining Structures Manual, FHWA-NHI-99-025
  - FHWA Soil Slope and Embankment Designs, FHWA-NHI-01-026
  - FHWA Rock Slopes Reference Manual, FHWA-HI-99-007
  - FHWA Geosynthetics Design and Construction Guidelines, FHWA HI-95-038
  - FHWA Geotechnical Instrumentation, FHWA-HI-98-034
- National Cooperative Highway Research Program (NCHRP) Report 611; Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments, Transportation Research Board

10.3 General Requirements

The geotechnical criteria were developed from operational requirements outlined in the Federal Railroad Administration (FRA) Class 9 Track Safety Standards. While these HST criteria are developed prior to the development of final track and systems design, the design approach is intended to comply with FRA Class 9 standards to result in appropriate infrastructure facilities for the HST trackway construction.

Each 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. For specialized structures, such as mined tunnels and aerial structures, additional experience requirements apply as described below.

Geotechnical Designers for underground structures (tunnels and trenches) shall have served as the geotechnical engineer of record for the design of at least 3 similar structures that have been constructed, each exceeding 20 feet in width or diameter and 5,000 feet in length. Geotechnical Designers for aerial structures shall have served as the geotechnical engineer of record for the design of at least 3 rail or highway bridge projects that have been constructed, each exceeding 1,000 feet in length.

The Geotechnical Designers shall conduct work necessary to perform supplemental geotechnical investigation and complete the design for the California High-Speed Train Project (CHSTP). The Geotechnical Designers 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 final design and proposed means and method of construction.
- Perform additional field testing to characterize the in situ shear wave velocity (V_{s30}) profile and dynamic soil properties at Special Sites[1] along the project alignment. This information may be needed for Contractor to perform site specific seismic response analysis based on input ground motions to be provided by the California High-Speed Rail Authority (Authority). Shear wave velocity profiles and dynamic soil properties shall be obtained at river and creek crossings, at locations where soft or compressible soils or liquefiable

[1] Special Sites are defined as locations subject to liquefaction or strong nonlinear site effects such as at river crossings or sites underlain by NEHRP site categories E and F, and locations with underground structures such as tunnels, stations, and below-grade I-walls.

materials (i.e., soils under NEHRP site categories E and F) are encountered, at locations subject to nonlinear site effects, and at locations where the groundwater table occurs within 75 feet of the ground surface. Measurements of shear wave velocity, V_{s30}, shall be conducted at the special sites via seismic cones and/or downhole PS seismic suspension logging to a maximum of 500 feet or until reference rock material (minimum shear-wave velocity (V_{s30}) of 760 m/s) is encountered, whichever occurs first.

- 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 for 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 Designers 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 Designers. Geotechnical reports, calculations, and drawings shall be signed and stamped by the Geotechnical Designers. In addition, the Geotechnical Designers 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 or manuals (in addition to those listed). In situations where conflicts arise between these criteria and other applicable codes or manuals, the more stringent criteria will be used.
- Approving construction under their design control

Land subsidence is well documented in areas along portions of the proposed alignment. Consequently, the design and construction of the high-speed rail facilities shall consider the ongoing land subsidence conditions. Land subsidence shall be studied, analyzed, monitored, and mitigated to reduce its effect on the high-speed operations, passenger comfort, and long term serviceability. Groundwater pumping has been the primary factor responsible for land subsidence. Although halting or limiting the water pumping is an effective mitigation, it shall not be considered as a solution for this contract.

Refer to the scope of work in the contract documents regarding additional requirements for addressing land subsidence.

10.4 Subsurface Investigation and Data Analysis

The Geotechnical Designers 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 themselves as to the nature of the following:

- Soil, rock, groundwater, and subsurface conditions including gassy or potentially gassy ground, if applicable
- The geologic and seismic hazards (e.g., faults, landslides, rockfall, debris flows, liquefaction, soft ground, swelling or collapsible soil, or otherwise unstable soil) within and in the immediate vicinity of the project site
- Variations in the subsurface and groundwater conditions across the project site and adjacent areas that can potentially impact construction activities or train operations (e.g., ground movements or high-speed train induced ground vibration)

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 other site-specific factors (e.g., drainage, issues, proximity of the alignment and structures to adjacent structures) that may impact final design, construction and operations.

In addition, the Geotechnical Designers shall undertake investigations and data review to assess the potential for adverse conditions posed by areas of soil and/or groundwater contamination, chemically aggressive soil conditions (e.g., high sulfate content), corrosive ground, and regions that may be impacted by stray electrical currents.

For locations where structures containing steel and/or concrete are intended, a site specific corrosion study shall be performed to evaluate corrosive characteristics of soil and groundwater that have negative impact to concrete and steel. In addition to stray currents, 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 criteria are required:

- Soil resistivity readings shall be obtained to evaluate the electric conduction potential of soils at (1) each traction power facility site (supply/paralleling/switching station), which are to be spaced at approximately 5-mile intervals, (2) major structures, such as aerial structures and freeway overpass bridges, and (3) tunnel portal areas.
- Where there is an absence of major structures between traction power facilities, soil resistivity readings shall be obtained to evaluate the electric conduction potential of soils at approximately the midpoint between facilities.

- Where significant differences in soil resistivity values are identified at adjacent locations, additional readings shall be obtained so that an adequate basis is developed for the grounding design.
- 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.

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 methods and laboratory testing program utilized. In addition, the plan shall include a detailed description regarding the investigative methods which shall be optimized to make best use of cone penetration testing, soil/rock borings, monitoring wells and piezometers to efficiently characterize the subsurface conditions along the project alignment. This plan shall be submitted to the Authority for review and acceptance prior to commencing geotechnical investigations.

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

- Perform additional subsurface investigations to supplement existing geotechnical data for the design of elements along the proposed alignment. Specific guidance on subsurface investigation methods that shall be considered for this project is presented in *Appendix 10.A – Guidelines for Geotechnical Investigations* of this chapter.
- Supervision – Boring and in situ testing and inspection, and laboratory classification and testing, shall be performed by a trained geologist or geotechnical engineer under the supervision of a geotechnical engineer or an engineering geologist 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, stationing and offsets shall be referenced to the Project horizontal control system. Elevations shall be referenced to the CHSTP datum.
- 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 2 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, 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.). At a minimum, detailed geologic information shall be collected within 15 feet above the future tunnel crown and 10 feet below the future tunnel invert.
- 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 geologic or hydrogeologic data or seismic hazard evaluation results shall be released to a third party without the written 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. Preliminary documents such as GDR, GEDR, and the GBR-B have been provided to the Contractor to support the bidding process. The preliminary GDR presents the existing geotechnical data for the project. The preliminary GEDR, if available, presents the preliminary geotechnical design elements and analyses for the project and is based on the data included in the GDR. The GBR-B documents baseline subsurface conditions anticipated for the purpose of the bidding process (hence the suffix “B”). The Contractor will conduct additional subsurface investigations and develop the final design and construction documents. These final geotechnical reports include the Final GDR, containing all data collected for the project (preliminary data as well as that collected by the geotechnical designer); the Final

GEDR that documents the design assumptions, design process, geotechnical analyses and their results, and final design recommendations; and the GBR-C that will update the GBR-B based on new information obtained during the investigation period.

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 the preliminary design level investigation results along with the 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
- Vibration propagation characteristics of soils including surface waves such as Rayleigh waves
- Ground movement measurements from inclinometers and others such as Global Positioning System (GPS), Interferometric Synthetic Aperture Radar (InSAR) methods, etc.
- 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)[1]

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[1], 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 earthquakes (Operating Basis Earthquake [OBE] and Maximum Considered Earthquake [MCE]), magnitudes, and peak ground and bedrock accelerations, where applicable. Refer to the *Seismic* chapter for definitions of the design earthquakes
- Evaluation of seismic and geologic hazards including, but not limited to, liquefaction/lateral spreading, pre-historic landsliding and land 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
- Lateral earth pressures to be used in designing temporary and permanent excavation support structures
- Seismic earth pressure design considerations for embankments and structures

[1] GEDR is equivalent to design memoranda reference in Essex (2007) and the GBR framework

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- 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
- Impact of dynamic train loading on the ballasted tracks and/or non-ballasted tracks in terms of residual settlements on the foundation systems and subgrade soils
- Evaluate the impact of the ground vibration induced by high-speed train operations (i.e., Rayleigh wave) to proposed and existing infrastructures such as bridges, embankments, retaining walls, and underground structures, and carry out mitigations as necessary
- 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
- Discussion on studies to evaluate and assess the impact of land subsidence to the performance of the HSR systems
- 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; mitigation measures to reduce land subsidence; 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 and evaluation 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 summarize design assumptions and final design results developed in the GEDR, and also to document interpretations and baseline conditions anticipated for Construction. As part of the final design and construction planning process, the Geotechnical Designer shall interpret the various baselines expressed in the GBR-B, and consider those baselines in the development of the design and construction approaches. Based on the data collected and presented in the final GDR and the design process documented in the GEDR, Contractor will further develop and finalize the GBR-C by updating the GBR-B accordingly. An electronic version of the GBR-B shall be used to record modifications or clarifications in the “track changes” mode using a computerized word processing software program. In its completed form, the GBR-C will document 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 Bridge, Aerial Structure, and Grade Separation Foundations

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 AASHTO LRFD BDS with California Amendments 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 LRFD BDS with California Amendments, and as described below. Some Primary Type 2 structures may also be subject to design criteria of local jurisdictions (e.g., UPRR, Metrolink, Caltrans, etc.). For Primary Type 2 structures that are 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 applicable codes, except for deviations as provided for in the codes.

10.6.1 Geotechnical Data

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

Foundations to be constructed 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 California Amendments. Refer to the *Structures* chapter for load factors and load combinations. Load resistance factors for walls and shafts shall be in accordance with AASHTO LRFD BDS with California Amendments.

10.6.3 Allowable Foundation Settlements for Primary Type 1 Structures

Requirements for foundation settlement performance presented herein shall supplement to (or apply in addition to) the criteria indicated in AASHTO LRFD BDS with California Amendments. Foundation settlements shall be calculated from the Service 1 load combination plus any irreversible settlements resulting from the post-earthquake effects of Operating Basis Earthquake (OBE) such as those resulting from liquefaction induced down drag, seismic compaction, etc. The settlements include components of short-term and long-term settlements as well as elastic (reversible) and plastic deformation (irreversible) from dynamic train loading, and shall not exceed the values shown in Table 10-1. Transient and temperature loads in the Service 1 load combination shall be used to calculate the short- term settlements. Traction and braking forces need not be considered.

Compliance with the settlement limits in Table 10-1 shall be applicable to settlements that occur after completion of construction and installation of all superimposed dead loads including the trackwork. For approach embankments, the settlements shall be measured at the top of the embankment.

Differential settlement limits in Table 10-1 are required to control the long term changes of track geometry within track maintainable tolerances.

Table 10-1: Maximum Allowable Settlement Limits (4),(5) for Service 1 and OBE Load Cases

Settlement Criteria	Non-Ballasted Track	Ballasted Track
Differential Settlement Between Adjacent Structure Supports (1), (5)	≤ L/1500 (L = smaller span in inches), but no greater than 3/4 inch	≤L/900 (L=smaller span in inches), but no greater than 1-1/4 inch
Differential Settlement Between Abutment and Approach Embankment (2)	≤ 1/1000, but no greater than 3/8 inch	≤1/500, but no greater than 3/4 inch
Differential Settlement Between Abutment and Tunnel Portal	≤ 1/1000, but no greater than 3/8 inch	N/A (3)
Uniform Settlement at Structure Supports	≤ ¾ inch	≤ 1-1/4 inch

<u>Notes:</u>

(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 (refer to settlement effects (SE) in Structural chapter)

(2) Geostructures 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 (i.e., defined as settlements which are the sum of the remaining native foundation settlement and embankment consolidation settlement estimated to occur after 12 months of completion of embankment construction plus elastic and plastic deformations from dynamic train loading) by comparison of the monitored data and predicted settlement.

(3) Not applicable based on the assumption that ballasted track will not be used for tunnels.

(4) The settlements calculated from the Service 1 load combination plus any irreversible settlements resulting from the post effects of OBE (such as those resulting from post-liquefaction down drag, seismic compaction, etc.). For approach embankments and aerial structures, the Service 1 settlement limits and OBE load combinations are applicable to settlements that occur after completion of construction.

(5) For special conditions, such as a straddle bent adjacent to a single column bent, the allowable Differential Settlement between Adjacent Structure Supports as noted in this table will be reduced by the differential vertical deformations of substructures. Refer to the *Structures* chapter for information on these additional requirements.

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

No specific settlement limits are required for the Extreme Event Maximum Considered Earthquake (MCE) loading case, except that the structure shall not collapse and that foundation elements are capacity protected in accordance with the *Seismic* chapter.

10.6.4 Bridge, Aerial Structure, and Grade Separation Foundation Types

Bridge, aerial structure, and grade separation foundations shall be either shallow or deep foundations, depending upon the site specific conditions.

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 the allowable limits outlined in this chapter.

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Geotechnical design of abutment and bent/pier shallow foundations shall be carried out in accordance with AASHTO LRFD BDS with California Amendments Articles 10 and 11, and as supplemented in this chapter. Unless otherwise specified, refer to 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 10.8 of this chapter, AASHTO LRFD BDS with California Amendments Articles 10 and 11, and as supplemented in this chapter. Refer to the *Structures* chapter for LRFD load factors and load combinations and this chapter for additional 3 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 California 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 California 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 California Amendments.

B. Stability

1. Under normal loading[1] conditions, the location of the resultant of reaction forces shall be within the middle one-third of the foundation width.

2. Under exceptional loads[2], the location of the resultant of reaction forces shall be within the middle one-half of the foundation width.

3. Under ultimate loads[3], the location of the resultant of the reaction forces shall be within the middle two-thirds of the foundation width. Over-strength plastic hinge demands from MCE shall apply.

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[1] Normal Loads = $DC + DW + L + CF + E + WA + LF_2 + 0.6TU$

[2] Exceptional Loads = $DC + DW + L_1 + CF_1 + LF_1 + E + WA + WS + WL_1$

= $DC + DW + L_1 + CF_1 + LF_1 + E + WA + OBE$

[3] Ultimate Loads = $DC + DW + E + WA$ (buoyancy only) $+ MCE$

For loading definitions of cases 1, 2, and 3 noted above, refer to the *Structures* chapter and as summarized below:

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
L1	=	Single train of LLRR or LLV, whichever governs
LF1	=	Braking forces (apply braking to 1 train) for LLV loading
LF2	=	Acceleration and braking forces (apply braking to 1 train, and acceleration to the other train) for LLV loading
MCE	=	Maximum Considered Earthquake (refer to the *Seismic* chapter)
OBE	=	Operating Basis Earthquake (refer to the *Seismic* chapter)
TU	=	Uniform temperature effects
WA	=	Water loads, including stream flow and buoyancy,
WS	=	Wind load on structure
WL_1	=	Wind load on 1 train

For Primary Type 1 and Complex Structures bridge, aerial structures, or grade separations, a design strategy based on transient foundation uplift or foundation rocking as described in the *Seismic* chapter is not permitted.

C. Allowable Foundation Settlements

Settlements and differential settlements of shallow foundations under the service limit state shall not exceed those specified in Table 10-1. Refer to 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 10.9.4).

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 a stream or river current shall be founded such that the top of the footing has a minimum depth of 3 feet below the lowest adjacent finished grade. In areas susceptible to frost development, footings shall be placed on a firm foundation below the frost level, or on 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, or such that the top of the footing is at least 3 feet below the surface, whichever is deeper. In locations where expansive or collapsible soils are present, deleterious soils should be over excavated and replaced with suitable foundation material or footings shall be placed at a depth sufficient to eliminate impacts from swelling or collapsible soils.

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 shall be below the total scour depth determined for the 100-year flood, and the bottom of footings shall be below the total depth determined for the 500-year flood.
- **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 the following:
  - 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, or 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 while also meeting the required settlement criteria. Alternative deep foundation systems such as micropiles, rammed aggregate piers, augercast piles, and propriety systems shall not be allowed for support of bridges, aerial structures, or grade separations.

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. <u>Ultimate Pile Load Capacities</u>

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 bore 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 are not located in the piles or drilled shafts. If below-ground plastic hinging of the piles or drilled shafts is unavoidable, then a design variance shall be submitted per the *General* chapter. In cases where plastic hinges are a necessary part of the design, the design shall include an inspection protocol that does not require excavation to inspect the pile condition.

B. Settlements

Settlements of deep foundations shall not exceed those specified in Table 10-1. Design settlement values shall be verified with appropriate calculations in the design process. Piles/drilled shafts and connections to pile caps shall be checked for the estimated deflection from lateral loads.

For deep foundations where soils exhibiting potential liquefaction and lateral spreading in an earthquake are present, ground improvement may be considered to improve the foundation stability. Where ground improvement measures alone 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 groups 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 settlements and horizontal deformations, 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 times the pile diameters (or pile size). Piles or drilled shafts in any 1 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 often 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 can cause 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 often will have group efficiencies less than unity, depending on the pile spacing.

Accordingly, when the P-Y method of analysis is used to evaluate a laterally 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

Pile Center-to-Center Spacing (in direction of loading)	Pile Load Modifiers, P_m		
	Row 1	Row 2	Row 3 and Higher
3D	0.75	0.55	0.40
5D	1.0	0.85	0.7
7D	1.0	1.0	0.90

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 down drag and nominal loads shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to down drag (i.e., neutral plane[1]). The structure shall also be designed to meet settlement limits resulting from down drag and the applied loads and the structural limits resulting from the combination of down drag plus structure loads.

As noted by Fellenius (2004 and 2006), down drag increases the load developed in the pile at the neutral plane, and thus it is a structural capacity issue for pile design. For soil capacity calculations for pile design, the down drag load does not need to be included for most cases because when the pile is punching into the soil, all of the soil deposit resists the downward pile movement at the ultimate pile load. However, down drag loading should be considered when the soil below the neutral plane is subject to creep deformation or creep rupture.

[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.

In the case of soil liquefaction, the effects of soil liquefaction in soil above the neutral plane will be negligible if the dynamic loads do not raise the neutral plane significantly. The soil above the neutral plane has already loaded the pile downward under working loads. The loss of soil strength due to liquefaction in soils above the neutral plane does not change this loading, so the resulting effects are inconsequential. Observations following earthquakes indicate that pile foundations with their neutral plane well below liquefiable layers do not settle significantly (although one must always check that the dynamic loading does not push the neutral plane up into the liquefiable soils and that the bearing soil/rock materials below the neutral plane are sufficiently stiff and strong to resist dynamic loads). Methods proposed by Fellenius and Siegel (2008) should be used for evaluating down drag in deep foundations in liquefiable soils. In developing pile designs, care shall be taken to incorporate appropriate considerations for designs of drilled and/or driven pile installations. Driven piles develop residual stresses so that the neutral plane is located at depth under working loads. Drilled shafts transfer load to the soil from the top-down, so that under working loads the shaft may be providing all required resistance and the neutral plane is likely at the ground surface. For these cases, drilled shafts may settle significantly if the soil along its shaft softens significantly, such as due to soil liquefaction.

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 down drag 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 California 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, aerial structures, or grade separations, in addition to analyses for current site conditions, 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 as described below. 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 that 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.

As a minimum, indicator piles, test piles, and method test shafts shall be located according to the following criteria:

- One indicator pile and 1 test pile per 300 driven piles
- One indicator pile and 1 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 1 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 that will be encountered. These load tests shall be conducted on test piles, method test shafts, and production piles/drilled shafts.

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[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.

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 D1143, Test Method for Deep Foundations Under Static Axial Compressive Load
- ASTM D3966, Test Method for Deep Foundations Under Lateral Load
- ASTM D3689, Test Method for Deep Foundations Under Static Tensile 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 performed in the following sections. 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 that will be encountered.

The Geotechnical Designer shall perform a test shaft program consisting of method test shafts to (1) confirm adequacy of drilling methodology and equipment, and (2) load tests to verify compressive, lateral, and tensile load capacities per site as described below. 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 2 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.

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Driven Piles – An indicator pile program consisting of indicator piles, test piles, and load tests shall be conducted at each bridge, aerial structure, or grade separation 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 evaluate 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 5 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. <u>Integrity Testing</u>

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 addition, for foundations in and adjacent to rivers and creeks, the soil cover over the foundation top for deep foundations shall be at least 3 feet below the maximum estimated scour depth, and at least 10 feet below the river/creek bottom and a minimum of 3 feet below the maximum estimated scour depth for shallow foundations supported by soils.

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 Rocking

For Primary Type 2 and Secondary structures, if foundation rocking is the preferred design approach, then it will also be limited to cases where the subsoil is not susceptible to loss of strength under cyclic loading, and the footing can be considered to be supported on a rigid perfectly plastic soil with adequate, uniform compressive capacity, q_n which is defined as nominal bearing capacity of supporting soil or rock (refer to 10.6.4.1A).

10.6.5.3 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.

10.6.5.4 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

Rock Unconfined Compressive Strength (psi)	Embedded Depth (feet)
< 75	10
≥ 750	3

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.

10.7 Station and Secondary At-Grade Structures

10.7.1 Shallow Foundations

Per AASHTO LRFD BDS with California Amendments 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 site-specific information. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical Data Report; use of assumed values shall not be allowed. Designs of shallow foundations supporting rail structures or attached appurtenances shall be as required in AASHTO LRFD BDS with California Amendments 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 California Building Code (CBC), buildings not directly supported off the aerial trackway structure, shall be designed in conformance with the requirements of the California Building Code (CBC) – Footings and Foundations. Shallow foundations shall have a minimum ground cover of 3 feet as measured from the top of footing to finished grade.

As these structures are distinct from bridges, aerial structures, and grade separations addressed in Section 10.6.4, shallow foundations shall be designed to limit total settlement (defined as vertical downward deformation of the shallow foundations for their design life) to no more than 1-inch. Differential settlements shall not exceed either 1/2-inch between adjacent supports or the ratio of the amount of settlement between adjacent supports divided by the distance between the supports (in consistent units) shall be no greater than 1/500, whichever is less.

10.7.2 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 BDS with California Amendments. Differential settlements between adjacent supports and the total settlement shall be the same as those stated for shallow foundations in 10.7.1 above.

10.7.3 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. These miscellaneous structures shall be limited to those where settlements are not critical to their service performance.

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. Seismic issues related to foundation design such as down drag 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 down drag and lateral spread due to liquefaction shall be addressed as stated in this chapter.

10.8 Retaining Walls and Trenches

The criteria set forth in this section govern the static/seismic 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 AASHTO LRFD BDS with California Amendments Article 11, FHWA Earth Retaining Structures Manual, and the sections specified in this chapter. For permanent surcharge loads, refer to Section 10.12.5. 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 reinforced concrete 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).

10.8.1 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 AASHTO LRFD BDS with California Amendments Article 11 and additional seismic criteria specified in the *Structures* chapter.

Design of mechanically stabilized earth structures and reinforced soil slope embankments shall also be in accordance with the LRFD version of FHWA's manual FHWA-NHI-10-024/25 "Design and Construction of Mechanically Stabilized Earth (MSE) Walls and Reinforced Soil Slopes", Volumes 1 and 2. Embedded metallic strip reinforcing elements, if used, shall meet the requirements of corrosion protection as set forth in the *Corrosion Control* chapter. Design of retained fill shall accommodate future Overhead Contact System (OCS) pole foundations. Refer to the *Structures* chapter for design criteria of the OCS pole foundations over retaining walls.

For MSE walls with metallic strips or wire meshes, a minimum of five (5) retrieval test strips or wire meshes shall be installed and retrieved for corrosion evaluation. The test strips or wires shall be retrieved for inspection in 5, 10, 20, 30, and 50 years after completion of the wall. Details of these retrieval strips and wire meshes shall follow those called out by standard Caltrans drawings and guidelines or as recommended by the MSE wall suppliers.

10.8.2 Unacceptable Walls

Unacceptable retaining walls include mortar rubble gravity walls, timber or metal bin walls, "rockery" or masonry walls, and other wall types not specifically listed in Section 10.8.

10.8.3 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 hydrostatic pressure (water pressure), if submerged or below the groundwater table, 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 California 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.

10.8.3.1 Unrestrained retaining walls

Retaining walls with level backfill 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, other static loads and bearing as appropriate, and inertial forces of the wall itself; additional dynamic (seismic) earth pressures need not be considered.

The no-seismic-load options mentioned above shall be limited to internal and external seismic stability design of the retaining wall for level backfills. For sloping backfill, the no-seismic-load options shall be correspondingly 0.2g for 3H:1V and 0.1g for 2H:1V. All these no-seismic-load options shall be applicable to the condition that no liquefaction and no 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.

For walls with cohesionless backfill and located in areas where PGA values are expected to be greater than 0.30g, seismic active pressures shall be included in the stability analysis. Seismic earth pressures shall be estimated using the Generalized Limit Equilibrium (GLE) Method or Mononobe-Okabe (M-O) Method (Mononobe and Matsuo, 1929). Furthermore, the M-O Method should be used 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 or 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:

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

For wall geometry, seismic acceleration level, 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.

The horizontal acceleration coefficient (K_h) shall be the horizontal seismic coefficient as estimated by using the Bray et al. (2010) method assuming a wall movement of 1 inch for OBE case for retaining walls with level granular backfill. The total earth pressure (active and seismic) shall be of triangular distribution with its resultant acting at 0.33H from the bottom for routine walls (defined as walls that function independently of other systems or structures). For walls that have a critical function and act as part of an overall structure or system such as walls used as part of bridge abutments or part of tunnel portals, the earth pressures shall be separated into the incremental seismic pressures and the active earth pressures in the following manner:

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

Where:

ΔK_{AE} = Incremental seismic pressure coefficient

K_{AE} = Total seismic pressure coefficient

K_A = Active pressure coefficient

In addition, for walls that have a critical function and act as part of an overall structure or system such as walls used as part of bridge abutments or part of tunnel portals, the incremental dynamic (seismic) earth pressure shall be taken as a triangular distribution with the resultant acting at 0.67H from the bottom (i.e., an inverted triangle). This pressure shall be added to the active earth pressure for the design of the walls.

For walls that retain cohesive soils, the seismic demand may be reduced for retaining wall design (Anderson et al., 2008). For walls with sloping grades, the seismic demand on the wall generally increases and this increase should also be considered in those cases (Anderson et al., 2008). For design of retaining wall with cohesive retained soils or where native soils have a clear cohesive strength component, some effects of the cohesion in the determination of the seismic coefficient can be assumed. However, if the cohesion in the soil behind the wall results primarily from capillary stresses, especially in relatively low fines content soils, no cohesion can be allowed when estimating seismic earth pressure. Unless otherwise stated in this chapter, methods presented in Chapter 7 of the NCHRP Report 611 shall be used for backfill materials consisting of cohesive or cohesive and frictional (c-ϕ) material.

10.8.3.2 Restrained or Non-yielding Walls

For basement walls (i.e., non-yielding or walls restrained against rotation) with level backfill in locations where PGA values (for MCE ground motion) are less than or equal to 0.3g, walls shall be designed for only at-rest pressures, surcharge loads, other static loads and bearing as appropriate, 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.

As mentioned above, walls that retain cohesive soils reduce the seismic demand, while the sloping grades behind walls increase the seismic demand required for retaining wall design (Anderson et al., 2008). For design of retaining wall with cohesive retained soils, or where native soils have a clear cohesive strength component, refer to section 10.8.3.1 above for design of the walls.

The no-seismic-load options mentioned above shall be limited to internal and external seismic stability design of the retaining wall for level backfills. For sloping backfill, the no-seismic-load options shall be correspondingly 0.2g for 3H:1V and 0.1g for 2H:1V. All these no-seismic-load options shall be applicable to the condition that no liquefaction and no 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.

10.8.4 Base Pressure

Soil bearing pressures shall be determined based on the applicable backfilled or native 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 design load conditions.

10.8.5 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.

10.8.6 Settlements and Horizontal Deformations

Retaining walls directly supporting HSTs, Primary Type 1, shall be designed not to exceed those residual settlement limits shown in Table 10-1 and Table 10-5 and horizontal deformation of 1/2 inch for service 1 and OBE load cases. These settlement and horizontal deformation limits apply after the structure enters service. For Type 2 and Secondary walls, refer to AASHTO LRFD BDS with California Amendments.

To avoid long-term deflections in the track, track structures (ballasted and non-ballasted) shall not be constructed until the majority (i.e., 75 percent) 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. For loading associated with the MCE load, the settlement limits shall be evaluated and specified by the structural engineer (wall designer) who will ensure that no collapse criterion applies.

10.8.7 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).

10.8.8 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.

10.9 Embankments and At-Grade Earthen Structures for HST Trackway (Primary Type 1)

For roadway and site embankments, refer to the *Civil* chapter. For design loads, refer to the *Structures* chapter.

Embankments and at-grade earthen structures shall be engineered. Design of embankments and at-grade earthen structures shall focus on settlement of support ground and stability of embankment and at-grade earthen structures. Care shall be taken to avoid possible landslides within the embankment and at-grade earthen structure areas.

At each embankment or at-grade earthen structure, 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, adversely increasing pore-water pressures in the vicinity of the structure, 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 track, embankment fill, supporting subgrades, and adjacent structures
- 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.

10.9.1 Slope Inclination

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

Cut – 2H:1V or steeper if justified by slope stability analyses. Refer to Section 10.10.

10.9.2 Safety Factors

The stability of an embankment slope shall be evaluated 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

Loading Conditions	Factor of Safety
Normal (Permanent) (1)	≥1.50
Temporary (open less than 1 year)	≥1.30
Earthquake (OBE and MCE)	≥1.0 (2)

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_h depends on allowable slope deformation (Refer to Bray and Travasarou (2009) for estimation of K_h). Refer to Section 10.B.2.9.2 of *Appendix 10.B – Guidelines for Geotechnical Earthquake Engineering*.

$K_v = 0$

Where:

K_h = Horizontal seismic coefficient

K_v = Vertical seismic coefficient

10.9.3 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.

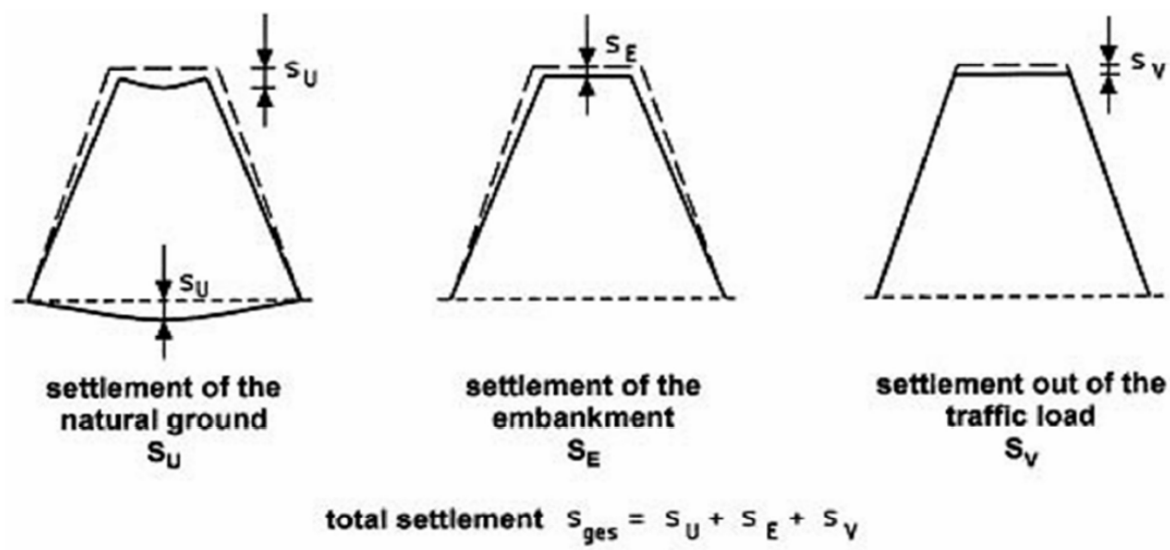
Settlement assessment shall be performed for new and existing embankments with particular emphasis on the following critical areas:

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

The vertical settlement of an embankment (which also affects overlying trackbed structure) is a combination of the permanent settlement of the foundation on which it is resting, plus permanent settlement of the embankment fill, and elastic and plastic deformations due to dynamic and repeated loading of the high-speed trains as depicted on Figure 10-1. Conventional settlement analyses shall consider 'immediate', 'consolidation', and 'secondary' components of settlement against the requirements of the 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
- Soils and Foundations Reference Manual, chapter 7, FHWA-NHI- 06-088 Volume I, 2006

Figure 10-1: Settlements of Embankments



Notes:

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

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 and constructed in the field, the 'residual' settlement estimates and differential settlements between locations along the length of the embankments shall be evaluated and estimated through track-earth- structure interaction analyses by the Geotechnical Designer.

Table 10-5: Maximum Residual Settlement Limits

Residual Settlement (1)	Non-Ballasted Track	Ballasted Track(4)
Differential Settlement (2),(3)	≤ 3/8 inch	≤ 3/4 inch
Uniform Settlement(3)	≤ 5/8 inch	≤ 1-1/8 inch

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.

(3) For Service 1 and OBE load cases.

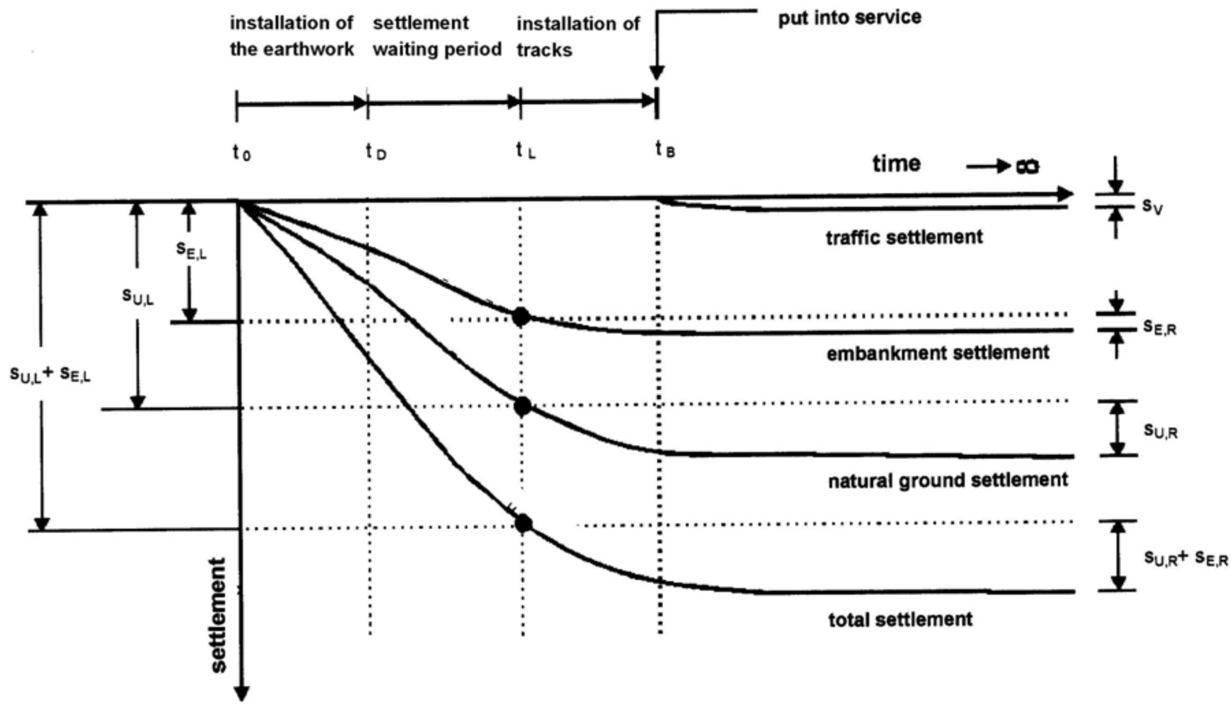
(4) For ballasted track, rail geometries will be maintained to meet FRA's guidelines as per normal maintenance.

Embankments shall be designed and constructed so as not to exceed the maximum residual settlement set forth in Table 10-5. "Residual" settlements occur over the design life after the track is laid and shall meet these criteria. Geostructures 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 (i.e., defined as settlements that are the sum of the remaining native foundation settlement and embankment consolidation settlement estimated to occur after 12 months of completion of embankment construction plus elastic and plastic deformations from dynamic train loading) by comparison of the monitored data and predicted settlement. These residual settlements are developed generally based on maintenance, passenger comfort, and track safety requirements. The residual settlements will be field verified by the Track Contractor.

If the predicted differential settlements 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 settlements and their duration are excessive, consideration shall be undertaken to change the design from an embankment to an aerial structure or other structure.

Settlement of earth structures is time-dependent and will vary by segment. The time duration of the "waiting (leaving) period" shall be evaluated and established. This period shall not be shorter than the 12 month monitoring period following initial fill embankment placement before re-leveling 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 depicted on 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 inches [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.

10.9.3.1 Track Subgrade Settlement Analysis

Track subgrade settlement analysis, using finite element methods such as ADINA, ABAQUS, ANSYS, PLAXIS, etc., shall be performed to estimate track-subgrade settlements as a result of dynamic loading of the high-speed trains. Limiting values are presented in Section 10.14.3.1 for ballasted and non-ballasted tracks over earthen structures such as embankments or retaining structures supporting high-speed trains.

10.9.3.2 Embankment Foundation Settlement Mitigation and Foundation Modification using Ground Improvement Methods

For track embankment segments or at-grade trackway, including features such as OCS poles, walkway, and ballasted and non-ballasted trackways 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 ground truth measurements, such as GPS measurements and traditional surveying and use of geotechnical instrumentation during and after construction.

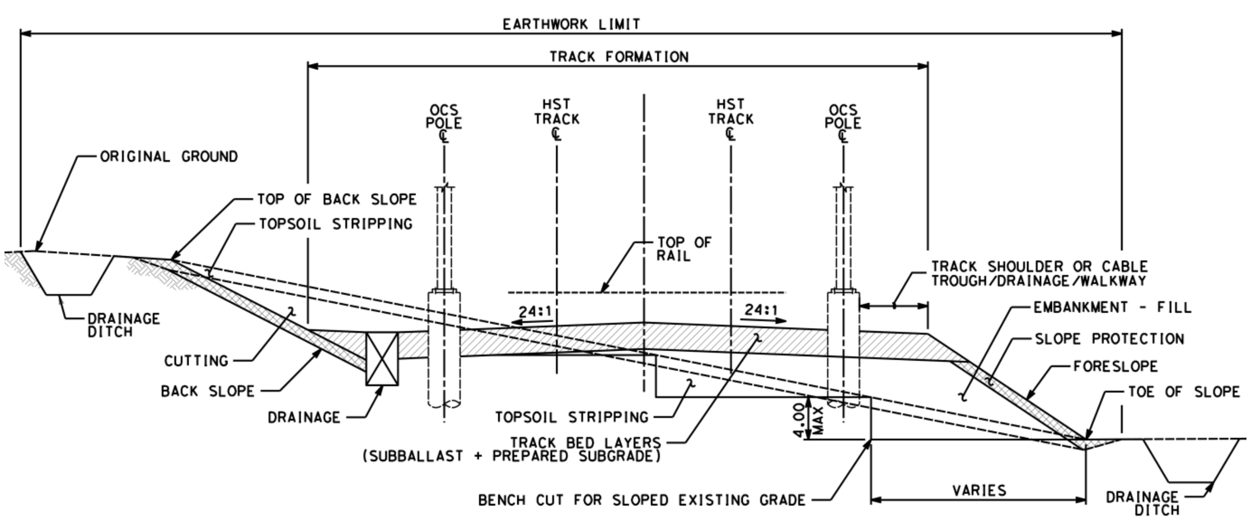
10.9.4 Benching of Slopes

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 (refer to 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 ensure the fill embankment is keyed into the existing slope (refer to 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



10.9.5 Particular Requirements

10.9.5.1 Foundation Support

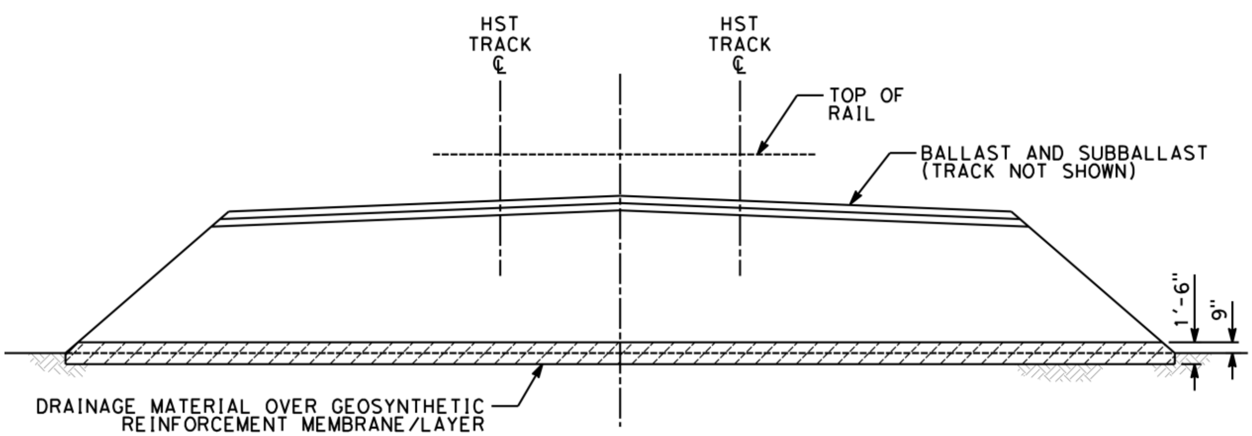
If the height of the embankment is less than or equal to 6.5 feet, as measured from the flat top of the subballast at the side edge of the embankment to the existing ground surface, and the foundation subgrades are loose and soft, compressible soils, they shall be removed and replaced with backfill and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

10.9.5.2 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.

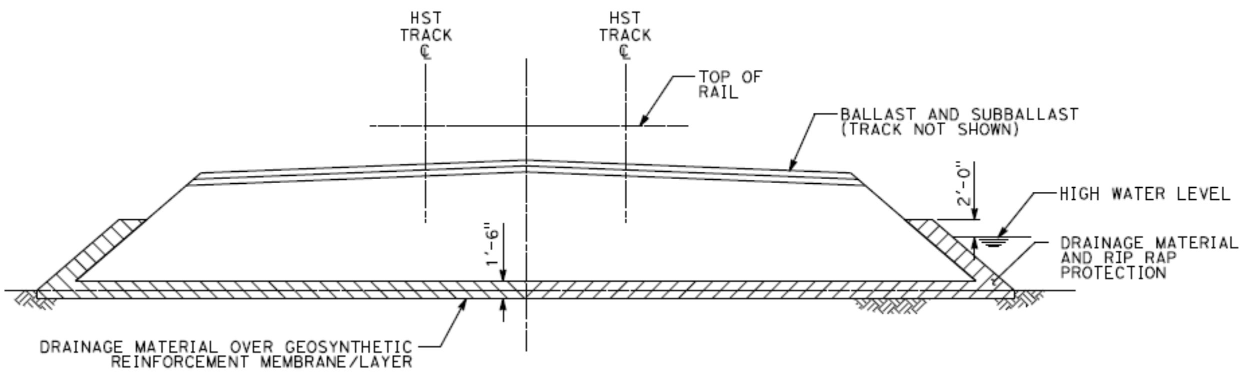
Figure 10-4: Earthwork Embankment in Wet Conditions



10.9.5.3 Embankment in Flood Plains

Where an embankment is located in a floodplain, the highest flood water level shall be evaluated from the 100-year flood. The embankment shall be, in addition to the drainage layer arrangement in Section 10.9.5.2, 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 (Sherard et al., 2004). 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



10.9.5.4 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 consideration of potential offsets for active fault crossings. Such consideration shall include both potential horizontal and vertical components of potential offset, as well as the relative orientation of this offset with respect to the track or embankment. Approaches to accommodate offset shall include making embankments wide enough and including designs with layers of geosynthetic cloth, geogrids drain rock at the bottom of embankments, and/or containment earthworks wide enough to accommodate the potential rupture offsets and subsequent re-alignment. Design of embankment over active fault locations shall consider life safety and preventive measures for ease of service restoration.

10.9.5.5 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.

10.9.5.6 High-Speed Train Induced Ground Vibration on Embankments and At-grade Segments

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, ballast or ballasted slab, subballast, prepared subgrade, embankments, at grade segments, 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 areas of soft, compressible, or loose soils, 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. Mitigation methods are available against excessive ground vibration from high-speed traffic. What method or combination of methods shall be used depends on factors such as (1) the frequency content of the generated ground vibration, (2) overall stiffness of the embankments or at grade segments, and (3) the type, consistency, and layering of the soils at the site. Mitigation measures may consist of replacing soft/loose soils with compacted fill, piled slabs, ground treatment such as dry deep mixing method (lime/cement columns) in soft clays or stone columns in loose sandy soils.

For design purposes, the following shall be required:

- Vibration induced stability of the embankment, at grade segments, and adjacent structures shall be verified.
- Tracks shall be supported by well compacted ballast/subballast, or non-ballasted track.
- Embankments or at grade segments supporting the track shall be adequately compacted.
- Subgrade underlying the embankment or at grade segments shall be competent and firm, and if soft compressible or loose soils are present, they shall be stabilized with ground treatment to increase its overall stiffness with undrained shear strength ≥ 15 psi or E_{v2} ≥ 6,500 psi. E_{v2} is the subgrade stiffness evaluated from the 2nd 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.

10.9.5.7 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 by field testing that E_{v2} of the existing subgrade is ≥ 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_{v2} ≥ 11,500 psi shall be provided.

10.9.5.8 Transition of Embankments to Structures

Embankments adjacent to the bridge, aerial structure, or grade separation 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

THICKNESS OF PREPARED SUBGRADE		
MATERIAL	TRACK TYPE	THICKNESS
WELL GRADED SOILS CONTAINING 5% TO 15% FINES	BALLASTED TRACK	14"
	NON-BALLASTED TRACK	6'-6"

GRADATION FOR PREPARED SUBGRADE MATERIAL	
GRAIN SIZE (mm)	PERCENTAGE PASSING [D=20 TO 125 (mm)]
P(2D)	100
P(DMAX)	100~99
P(D)	99~85
P(D/2)	84~55
P(D/5)	60~31
P(D/10)	49~23
P(D/20)	40~17
P(D/50)	30~11
P(D/100)	22~8
P(D/200)	16~6
P(D/500)	9~3
P(D/1000)	6~2

D = NOMINAL GRAIN SIZE
Dmax = 1.25D IF D $\geq$ 50 mm;
Dmax = 1.58D IF D < 50 mm

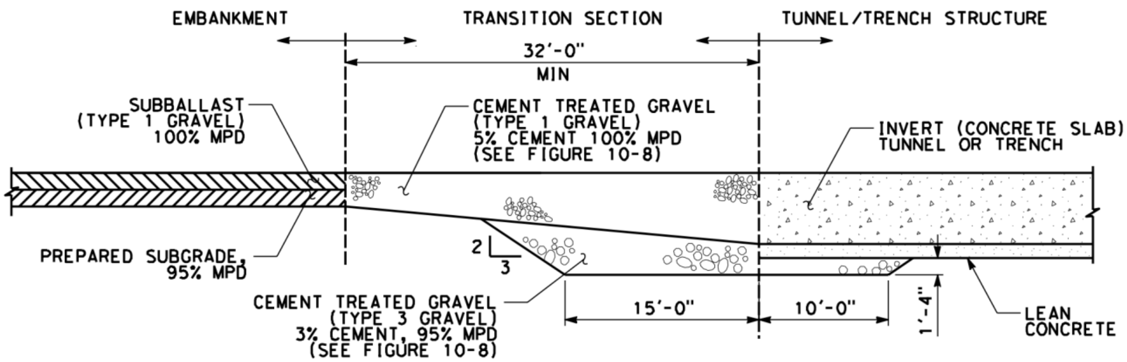
NOTES:

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 (SUPPORTING BALLASTED TRACK) THICKNESS SHALL BE 9".

LEGEND:

MPD MODIFIED PROCTOR DENSITY (AASHTO T180)

E_{v2} DEFORMATION MODULUS OF SECOND LOADING



1 **Figure 10-7: Transition from Cut to Embankment**

THICKNESS OF PREPARED SUBGRADE		
MATERIAL	TRACK TYPE	THICKNESS
WELL GRADED SOILS CONTAINING 5% TO 15% FINES	BALLASTED TRACK	14"
	NON-BALLASTED TRACK	6'-6"

GRADATION FOR PREPARED SUBGRADE MATERIAL	
GRAIN SIZE (mm)	PERCENTAGE PASSING [D=20 TO 125 (mm)]
P(2D)	100
P(DMAX)	100~99
P(D)	99~85
P(D/2)	84~55
P(D/5)	60~31
P(D/10)	49~23
P(D/20)	40~17
P(D/50)	30~11
P(D/100)	22~8
P(D/200)	16~6
P(D/500)	9~3
P(D/1000)	6~2

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

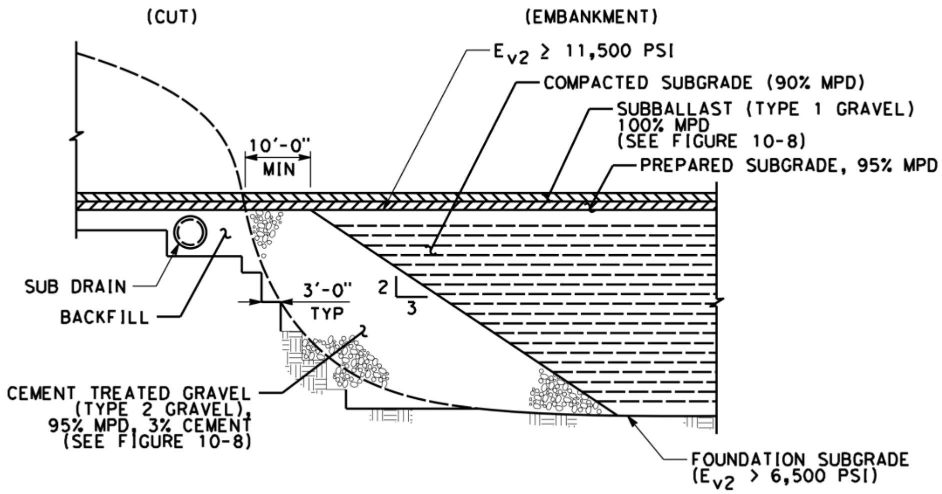
NOTES:

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



2
3

Figure 10-8: Transition from Bridge, Aerial Structure, or Grade Separation to Embankment

TYPE 1 GRAVEL NOTE:

CEMENT TREATED GRAVEL 0/31.5. COMPACTION SHALL BE GREATER THAN 100% OF MAX DRY DENSITY ACCORDING TO AASHTO T180 MPD. DEFORMATION MODULUS E_{v2} SHALL BE GREATER THAN 17,000 PSI.

TYPE 1 GRAVEL	
SIEVE	% PASSING
1.5 INCH	88~100
1.0 INCH	82~97
3/4 INCH	75~92
3/8 INCH	64~85
NO. 4	53~77
NO. 10	40~68
NO. 40	22~48
NO. 100	10~36
NO. 200	3~22

TYPE 2 GRAVEL NOTE:

CEMENT TREATED GRAVEL 0/20. COMPACTION SHALL BE GREATER THAN 95% OF MAX DRY DENSITY ACCORDING TO AASHTO T180 MPD $E_{v2} >$ 15,000 PSI.

TYPE 2 GRAVEL	
SIEVE	% PASSING
3/4 INCH	83~100
3/8 INCH	53~83
NO. 4	36~65
NO. 10	23~49
NO. 40	12~31
NO. 100	7~19
NO. 200	4~12

TYPE 3 GRAVEL NOTE:

GRAVEL 0/60. COMPACTION SHALL BE GREATER THAN 95% OF MAX DRY DENSITY ACCORDING TO AASHTO T180 MPD. DEFORMATION MODULUS E_{v2} SHALL BE GREATER THAN 80 11,500 PSI.

TYPE 3 GRAVEL	
SIEVE	% PASSING
2.0 INCH	80~100
3/4 INCH	43~72
NO. 4	21~46
NO. 10	14~35
NO. 40	7~19
NO. 100	3~10
NO. 200	2~6

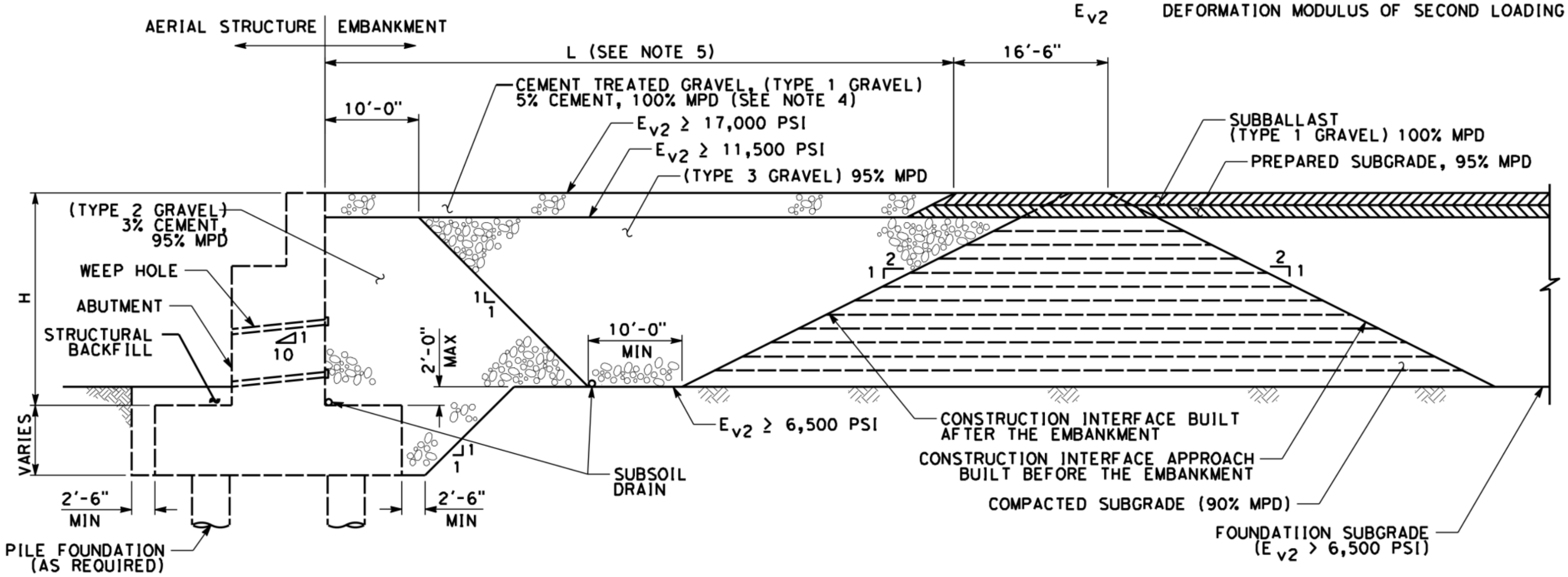
NOTES:

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)

E_{v2} DEFORMATION MODULUS OF SECOND LOADING



10.9.5.9 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 depicts a typical embankment in a cut section.

10.9.5.10 Drainage (Surface and Subsurface)

Control of surface and ground water is essential to avoid surface erosion and potential slope instability. In addition to the requirements set forth in the *Drainage* chapter, 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.

For secondary structures, the drainage design shall be made according to the requirements set forth in the jurisdiction of the local county, city, or third party such as Caltrans, UPRR, etc.

10.9.6 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

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

Notes:

Source: Per ASTM D3282 / AASHTO Subgrade Soil Group System

Refer to the *Trackwork* chapter and Standard Specifications for Trackbed layers of subballast and prepared subgrade.

* Rockfill is not acceptable for track embankment material.

(1) In addition to the AASHTO criteria, the maximum soil particle size is limited to 3 inches.

(2) Potential embankment fill source materials from groups A-2-7, A-5, A-6, and A-4 (with CBR <10) that can be shown by analysis and testing to meet 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.[11] 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.

10.10 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

[11] 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.

10.10.1 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*.

10.10.1.1 Design Requirements

Slope Inclination (Typical[12])

- **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

10.10.1.2 Safety Factors

For design criteria for stability of cut slopes, refer to Section 10.9.2.

[12] 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.

10.10.2 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 10.9.5.10 and Section 10.11.2. Impermeable coverings with drainage provisions (weeps and geocomposite mats) 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. For other drainage related design criteria, refer to the *Drainage* chapter for details.

10.10.3 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

10.11 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.

10.11.1 Protection of Existing Slopes

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

10.11.2 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.

For other drainage related design criteria, refer to the *Drainage* chapter for details.

10.12 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 prevent water buildup in the underground structures.

10.12.1 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.

10.12.2 Water Pressure (Buoyancy)

Refer to the *Structures* chapter for water loads (water pressure for design criteria for buoyancy) and requirements for different buoyancy resisting elements.

Refer to Section 10.8 for types of systems to be allowed to resist buoyancy.

10.12.3 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 structure to confirm seismic response of the buildings. In addition, the effect of stress redistribution onto existing adjacent structures due to the design of temporary support systems shall be considered and mitigated as necessary since it is determined by the means and methods selected by the Contractor.

10.12.4 Temporary Lateral Loading Conditions

Refer to the *Structures* chapter for construction loads and definition of temporary structures.

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 construction term water pressures that are not lower than the existing groundwater level or seepage pressures, 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 – Unless otherwise stated in this chapter, earthquake loads (i.e., seismic earth pressures) shall be considered.

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.

10.12.5 Permanent Lateral Loading Conditions

Soil Pressures – Permanent underground structures shall be designed for earth pressures as given in Section 10.8.3. 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 California Amendments Article 3.11.6.

10.12.6 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 1 inch and 1/2 inch, respectively. The Geotechnical Designer shall analyze the support of the excavation system taking into account the ground conditions, potential impacts to neighboring or adjacent structures or property, wall stiffness, requirements for wall bracing systems, global stability, and sequence of construction including timing of support installations to evaluate the lateral deformations and settlements for open cut excavation methods. In locations where adjacent structures or property impacts are not significant, more relaxed site-specific criteria may be considered through the design variance process provided that overall stability is maintained.

Ground settlement predictions due to cut-and-cover excavations shall utilize empirical recommendations given by Clough and O'Rourke (1990) or numerical modeling software such as Finite Element Analyses, Plaxis, or Finite Difference modeling software such as FLAC. The Geotechnical Designer shall consider the following:

- The installation and (where appropriate) extraction of the support systems
- Movements (settlement and lateral wall deformation) at all stages of excavation
- Consolidation settlements
- The effects of grouting, piling, soil improvement, dewatering, or any other measures required for the Works that could cause ground settlements
- Seepage analyses shall be carried out for all excavations, and the potential consolidation settlements shall be assessed
- Settlement contour plans associated with excavation of cut-and-cover excavations shall be prepared and shall include immediate and consolidation settlements

10.12.7 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.

10.13 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 they will behave elastically under the design OBE, and no collapse under the design MCE. Earth retaining structures shall be evaluated and designed for seismic stability internally, externally, and globally. Cut slopes in soil and rock, fill slopes, and embankments having impact on the operations of high-speed trains shall be evaluated for instability due to design seismic events and associated geologic hazards.

10.13.1 Design Earthquakes

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

10.13.2 Seismic Hazard and Ground Motions

For this project, preliminary design response spectra and ground motions (time histories) matching the site design response spectra has been provided to prospective design bidders for bid. Upon receipt of the site specific subsurface investigation data from the Contractor after the Notice to Proceed (NTP), the preliminary design response spectra and ground motions will be re-evaluated and updated, if necessary, by a seismic specialists team retained by the Authority for use during final design. The seismic hazard levels and new sets of input ground motions at half boundary will be developed by the seismic specialists team and provided to the Contractor for development of site-specific response analyses appropriate for structures to be constructed.

Design of site specific site response analyses and mitigation of the seismic hazard shall be the responsibility of the Contractor.

10.13.3 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, structures, and embankments, and develop mitigation measures to satisfy performance requirements.

Liquefaction-triggering evaluations shall be performed for sites that meet the following 2 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/foot, 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/embankments is not acceptable, the following remedial measures shall be considered:

- Liquefiable soils shall be removed; or
- Soil improvement techniques shall be used (refer to Section 10.9.5.5); 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.

10.13.3.1 Compositional Criteria for Liquefaction Susceptibility for Soils

A. Sandy Soils

Sandy soils with few amounts of fines that meet the above-mentioned 2 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 el 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 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.

10.13.4 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 3 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

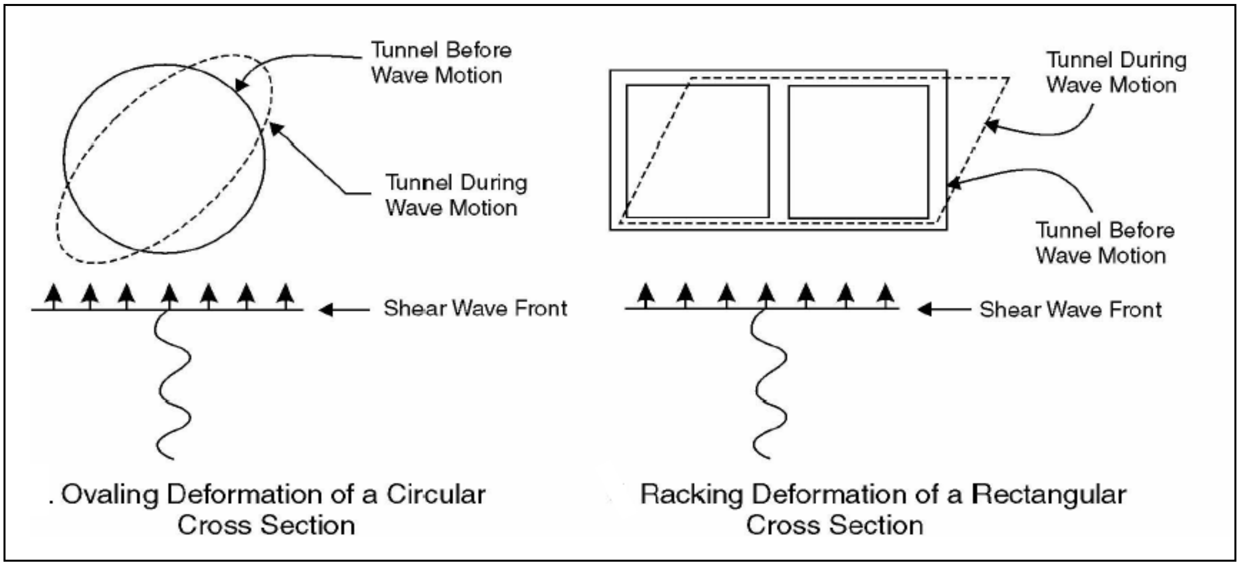
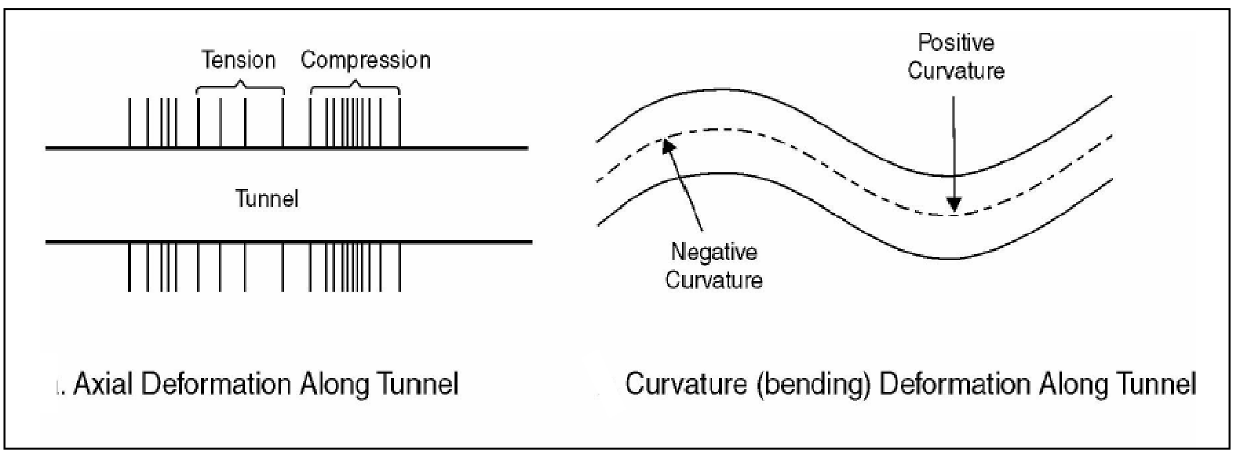


Figure 10-10: Tunnel Longitudinal Axial and Curvature Response to Traveling Waves



10.13.5 Effect of Ground Deformation

10.13.5.1 Transverse Ovaling Deformations

For bored circular tunnels, using either the precast concrete segmental lining or cast-in-place concrete lining, there are 2 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.

Refer to FHWA-NHI- Report, "Technical Manual for Design and Construction of Road Tunnels", Chapter 13 for general guidelines on transverse ovaling analysis for bored tunnels.

10.13.5.2 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 2 loading components:

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

There are 2 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 using 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.

10.13.5.3 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 evaluate 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 FHWA-NHI-09-010 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 FHWA-NHI-09-010. 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 areas 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 2 locations along the tunnel alignment. This time lag can be estimated by dividing the distance between the 2 locations by the horizontal wave travelling velocity (in the ground) VH = 2 km/seconds along the tunnel alignment.

The effect of local soil overburden is specified in Section 10.13.4.

10.13.5.4 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. Conventional numerical analysis software packages should be used for this process as applicable to the site specific requirements for the response analysis. Examples of commercially available software that may be appropriate include: SHAKE; PROSHAKE; SHAKE2000; DMOD; DEEPSOIL, and FLUSH.

Several analysis methods are available for evaluating the effect of local soil conditions on ground response during earthquakes. The following shall be used:

- The equivalent-linear one-dimensional total stress method
- The non-linear one-dimensional total and effective stress method
- The two-and three-dimensional equivalent-linear total stress methods
- The two- and three-dimensional non-linear total and effective stress methods

The one-dimensional site response analysis described above can be used for developing a ground displacement profile for the evaluation of 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.

10.13.6 Soil-Structure Interaction for Bridges, Aerial Structures, and Grade Separations

For bridges, aerial structures, and grade separations, the following primary soil-structure interaction (SSI) 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 springs (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.

10.13.6.1 Pile/Drilled Shaft Design Subject to Ground Displacements

Ground displacement loading can be divided into 2 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.

The LPILE computer program 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).

10.13.6.2 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), the use of 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.

10.14 Track Structure

A railway track structure is composed broadly of track-structure and formation. The track structure consists of rails, sleepers, and fastenings or non-ballasted track, while the formation of a track (ballasted track) is typically composed of ballast, subballast, filled/placed soil and the native ground or the subsoil. The filled/placed soil and the subsoil serve as a platform on which the track structure is constructed and are to provide a stable foundation for the subballast and ballast layers. Because of higher train speeds, dynamic forces and axle loads, design of HSR track structure requires higher and more stringent design standards than conventional railway track structure.

The track structure is subjected to cyclic loading due to high-speed train loads. Additionally, the high-speed train load also induces stress due to dynamic effect. The cyclic loading may result in progressive building or pore pressure causing large cumulative strains. The bearing capacity of the track structure has to consider not just a single load application but repeated loading as the allowable stress under repeated loading is much higher than under the static loading.

Design of track structures for trains at speeds greater than 160 mph shall consider the Rayleigh waves induced vibration due to high-speed trains (Section 10.14.3). When the speed of the high-speed train approaches the critical wave velocities in the track-ground (earth) system, large transient movements of the rail and ground will result, causing large rail deflections and formation instability as well as structural vibrations and associated noise in nearby buildings.

10.14.1 Formation Supporting Ballasted Tracks

Formation, for this project, defined as layers comprising subballast, prepared subgrade, and earth fill, provides the base for ballasted track 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 as stated in this chapter. The subballast and prepared subgrade provide support to the ballasted track and bear additional stresses due to static and dynamic effects of moving wheel loads. The load is transmitted through the subballast, prepared 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. 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 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.
- **Prepared Subgrade** – Below the subballast is the prepared subgrade layer, which in its most complete form, has a cross slope. It shall consist of imported or treated material depending of the quality of the upper part of the embankment or the bottom of the cut. In addition, it shall have a gradation as specified on Figure 10-6. Its deformation modulus, E_{v2}, from the 2nd loading in the plate load test shall not be less than 11,500 psi.
- **Earth Fill** – Underlying the prepared 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.

10.14.2 Determination of the Thickness of the Trackbed Layers

Trackbed layers are composed of ballast and subballast that are placed on top of the prepared subgrade overlying earthfill or existing 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 10.9.5.7.

10.14.3 Design of Formation for Dynamic Loading from HST Operations

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 undesirable 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 that 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.

10.14.3.1 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 speeds 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 1/12-inch and 1/6-inch for non-ballasted and ballasted trackways, respectively. These deflections are elastic and reversible after train passage. Plastic deformations induced by repeated high-speed (up to operating speed of 220 mph) train loads for non-ballasted and ballasted trackways shall be limited to 1/8 inch and 1/4 inch, respectively for its design life. These plastic deformations will be irreversible and remain after the train loads are removed. Deformation analyses using numerical modeling such as ADINA, ABAQUS, ANSYS, or PLAXIS, etc. 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.

10.14.3.2 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 and compressible ground shall be performed to evaluate the structural integrity of the formation supporting the trackways. As mentioned in Section 10.9.5.6, for high-speed railway, the running train produces compressive (P) waves, shear (S) waves, and Rayleigh (R) waves, of which Rayleigh waves – moving parallel to the surface, are the primary source of vibration energy carried away from the source and are less prone to geometric attenuation than P- and S-waves. The propagation of vibration is dependent on the source frequency and soil properties such as stiffness, depths of strata and damping. Stiff soils have high velocity, high frequency, shorter wave characteristics, while soft soils are the converse of the above. For embankment stability, the Rayleigh wave-induced vibration by the high-speed train is an important factor to be considered for design, especially for existing embankments over soft, compressible grounds.

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 (refer to Section 10.14.3.1).

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

Analytical methods such as a simple elastic beam model and modern numerical modeling using Finite Element Methods (FEM) such as ADINA, ABAQUS, ANSYS, or PLAXIS shall be used to model train-induced dynamic motion. Of these methods, the Winkler model can be used as a screening process as it is a very prevalent and simple numerical model. If this screening process confirms that the required critical velocity of the embankment or retaining wall meets the design value, then the numerical modeling can be waived. However, sophisticated FEM modeling shall be used for evaluation of high-speed train induced vibration on embankments over soft and compressible grounds.

In the Winkler model, 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) that is equal to:

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

Where k = Spring constant per unit length of the beam;
E = Modulus of elasticity of beam;

I = Moment of inertia of beam; and
ρ = Mass per unit length of beam

10.14.3.3 Drainage of Track and Formation

Drainage requirements for non-ballasted sections of track, as well as surface drainage in general, are described in the *Drainage* chapter.

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

10.15 Maintenance of Geo-Structures

For the CHSTP, a Reliability, Availability and Maintainability (RAM) Program has been established. One of the key components of this program is maintainability. The RAM program requires each contractor to establish a Contractor's RAM Program Plan for the Contractor's scope of work. The Contractor's RAM Program Plan goals shall include establishing provisions for safeguarding continual performance of geo-structures including, but not limited to, embankments, retaining walls, slopes, underground (cut-and-cover) structures, trenches, tunnels, culverts, etc. The design of geo-structures shall consider long term maintenance issues including, but not limited to (1) consideration of closing roads for maintenance of retaining walls, bridges, embankments, (2) difficult access for maintenance of elements in a cut-and-cover section, (3) lack of or limited access to remove rock debris from rock slopes, etc. These and similar issues shall be included in the Contractor's RAM Program Plan, considered and addressed in the design, and shall constitute an integral part of the final design of the geo-structures to mitigate aforementioned maintenance and maintenance access concerns.

10.16 References

1. Anderson, D. G., G. R. Martin, I. P. Lam, and J. N. Wang. (2008). *Seismic Analysis and Design of Retaining Walls, Slopes and Embankments, and Buried Structures*, NCHRP Report 611. National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC.

2. Bray, J. D., Travasarou, T., and Zupan, J. (2010). “Seismic Displacement Design of Earth Retaining Structures.” *Proceedings, Earth Retention Conference 3*, Bellevue, Washington, August 1-

3. Clough, G. W. and T. D. O’Rourke, 1990, Construction Induced Movements of In Situ Walls, Design and Performance of Earth Retaining Structures, Proceedings of a Specialty Conference at Cornell University, ASCE, New York, 439-470.

4. Fellenius, B. H. (2004). “Unified Design of Pile Foundations with Emphasis on Settlement Analysis.” Honoring George G. Goble – Current Practice and Future Trends in Deep Foundations, Proceedings, Geo-Institute Geo TRANS Conference, ASCE GSP 125, ASCE, Reston, 253-275.

5. Fellenius, B. H. (2006). “Results from Long-term Measurement in Piles of Dragload and Downdrag.” Canadian Geotechnical Journal, 43(4), 409-430.

6. Fellenius, B. H. and Siegel, T. C. (2008). “Pile Drag Load and Downdrag in a Liquefiable Event.” Journal of Geotechnical and Geoenviromental Engineerig, ASCE, September, 1412-1416.

7. Kenny, J. T. (1954). “Steady-State Vibrations of Beam on Elastic Foundation for Moving Load.” Journal of Applied Mechanics, December.

8. Li D., and Selig, E. T. (1998). “Method for Railtrack Foundation Design I: Development,” Journal of Geotechnical/Geoenvironmental Engineering, Vol. 124, No. 4, pg 316-322.

9. Sherard, J. L., Dunnigan, L. P., and Talbot, J. R. (1984). “Filters for Clays and Silts,” Journal of the Geotechnical Engineering Division, ASCE, June, pp. 701-718.

Appendix 10.A: Guidelines for Geotechnical Investigations

10.A.1 Purpose

These guidelines represent the 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:

- Summary and reference to separate geologic hazards report
- Description and discussion of the site exploration program, including specific goals and objectives
- 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) Office of FHWA. 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 as follows:

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 that may impact construction or operation of the project (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 evaluate 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 evaluate the viability of construction methods and potential impacts to adjacent facilities.

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

10. Develop design soil properties for engineering evaluations, including dynamic analysis to evaluate response associated with rail operations and seismic events.

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 the following:
   - 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 the available geologic and geotechnical data.
3. Initiate and prepare geotechnical investigation plans. Identify the anticipated required analyses and key engineering input for the final design and construction.
4. Perform field reconnaissance and geological mapping.
5. Finalize the Geotechnical Investigation Plan (GIP) and submit it to the Authority.

6. Obtain permits and rights-of-entry.
7. Perform exploration and laboratory testing for final design.
8. 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, as appropriate to efficiently and cost-effectively characterize the project site(s).

10.A.2.4.1 Desk Study

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 Preliminary 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 identifying proposed subsurface exploration points. The results of the geologic and seismic hazard evaluation shall be shared with the Geotechnical Designer to inform the final design. 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 evaluate 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 evaluated. 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 the overall geologic conditions along the project site. Profiles should be parallel to the rail alignment, but also perpendicular in locations of major structures where future project facilities (e.g., stations or ancillary facilities) or important geologic conditions (e.g., geologic hazards needing definition) extend perpendicular to the alignment. Evaluation of these areas will allow the Geotechnical Designer (in collaboration with the Engineering Geologist) to identify 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:

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

2. **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, CPT_u, and CPTs.
3. **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, seasonal fluctuations in groundwater levels, and ground movement in the field. Perform suspension PS logging or cross-hole seismic logging at deep boreholes (180 feet or deeper) where structures will be located over river crossings or unusual geologic conditions[13], and other boring locations selected by the Geotechnical Designer in collaboration with the engineering geologist.

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 orientation, spacing, and physical characteristics.

The objective of these observations and data collection efforts is to confirm the general types of soil and rock present, and topographic and slope features. For rock slopes, stability 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.

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[13] Unusual Geologic Conditions – Structures that are subject to and founded on the following geologic conditions:

- Soft, collapsible, or expansive soil
- High groundwater table (within 5 feet below ground surface)
- Soil having moderate to high liquefaction and other seismically induced ground deformation potential
- Soil of significantly varying type over the length of the structure
- Fault Zones
- Unusual geologic conditions shall be defined within the Geotechnical Reports.

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 investigation 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; 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 these geotechnical investigation guidelines.

- **Geophysical Methods** – Spectral Analysis of Surface Wave (SASW), Multi-channel Analysis of Surface 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 Technical Manual (2003). Secondary sources are NHI 132031 and USACE EM 1110-1-1802. 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 in providing a preliminary interpretation on a large spatial scale complementary to information from borings. The methods presented in FHWA (2003) shown as Exhibit 3.2-F of the GTGM are some of the most commonly used. 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, the range of wave velocities or other geophysical parameters within a particular stratum and the quality of the test and the experience of the testing team.

  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 characteristics of subsurface conditions, and assessment of this is considered important for high-speed train systems. The reference document for this testing is titled, "High-Speed Ground Transportation Noise and Vibration Impact Assessment," FRA Report No. 293630-1, December 1998.

  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 and can be used to supplement borings since boring samples are obtained for positive identification of soil types. 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.

- **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 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 T206. The use of automatic hammers for SPT is highly recommended, and drop height and hammer weight must deliver 60 percent energy so that an energy correction is not required. The SPT values obtained with non-automatic hammers are discouraged and could be 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 evaluate 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 is the responsibility of the Geotechnical Designer to obtain enough testable samples of rock and soil to complete the laboratory testing program detailed in the GIP accepted by the Authority. 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. Adequate subsurface exploration and sampling is necessary to obtain sufficient samples for adequate 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, whichever is more frequent. Continuous SPT samples with a gap of at least 6 inch between 2 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.

– **Silt or Clay Soils and Peat Sampling –** The SPT or 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 medium stiff to very soft clays. Take SPT and tube samples in same borings or take tube samples in separate undisturbed borings. Tube samples shall be sent to lab to allow consolidation testing (for settlement analysis) and strength testing (for slope and embankment 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 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 evaluated 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 1 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 the water table shall be evaluated where fluctuation will have significant impact on design or construction (e.g., borrow source, footing excavation, excavation at toe of landslide). Artesian pressures 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 aluminium 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 evaluate the electric conduction potential of soils at each traction power facility (supply/paralleling/switching station), which are spaced at approximately 5-mile intervals and at major structures, such as aerial structures and freeway overpass bridges, and at tunnel portal areas.
  - Where there is an absence of major structures between traction power facilities, soil resistivity readings shall be obtained to evaluate the electric conduction potential of soils at approximately the midpoint between facilities.
  - Where significant differences in soil resistivity values are identified at adjacent locations, additional readings shall be obtained so that an adequate basis is developed for the grounding design.
  - 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 2-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 other 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 properly labeled wooden core boxes indicating the run number and depth of each run with consistent orientation. The core boxes should be 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 digitally photographed, (at least 10 megapixels) taking at least 1 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 and photos 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, professional geologist, 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, person or persons logging the hole, 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, construction details shall be included (casing size, type of casing, depth of screen length of screen, screen opening, depth and type of filter material, sanitary seal and annular backfill material). Observation wells and piezometer should also be developed by bailing, surging or overpumping to enhance communication with the surrounding strata. For observation wells and piezometers, several measurements are usually necessary within a one-week timeframe following drilling to verify that measured groundwater levels or pressures have achieved equilibrium. Where seasonal fluctuations of groundwater levels are of concern, water level measurements shall be collected on a monthly or quarterly schedule, as appropriate to establish the nature and magnitude of variability. 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 AASHTO T206, 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.
  - **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.
  - **Flat-Plate Dilatometer Test –** This test uses pressure readings from an inserted plate at the base of a borehole to evaluate and assess 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.

- 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 NHI1 32031, USBR Geology Manual, and NAVFACDM-7.1.
    - **Pumping Test –** The pumping test requires not only a test well to pump from, but also 1 to 4 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; however, this test typically only examines a small volume of the permeable material around the instrument when compared to pumping tests. 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 water levels are monitored continuously while the natural flow of groundwater out of or into the well occurs until equilibrium in the water level is stabilized. 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 NHI1 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 NHI1 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 hyrdrogeologic 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 acceptance 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 work plans 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 acceptance 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 for acceptance 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: the type of soil or rock; presence of landslides or unstable slopes; the presence of rockfalls; rock rippability; fill suitability; presence of expansive or collapsible soils; presence of compressible soils; occurrence of groundwater and hydrogeologic features; potential for 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 for acceptance 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 acceptance 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
- Evaluate train induced vibrations and their impact on the embankment and adjacent structures

For cuts and fills, test borings and/or CPTs shall be advanced at least every 200 feet (for erratic or rapidly changing conditions) to 400 feet (for 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 3 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, aerial structures, and grade separations
- Stations
- Buildings
- Retaining walls
- Tunnels and portals
- Large culverts
- Mast-arm supports (OCS, signals, message signs)
- Landslides
- Faults

For bridges, 1 boring shall be drilled at the substructure unit under 100 feet in width and 2 borings per substructure unit over 100 feet in width, both drilled to a depth of 20 feet below pile/shaft tip elevation or 2 times maximum pile group dimension, whichever is greater or to a depth of a minimum of 10 feet into bedrock. In addition, at least 1 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, 1 boring shall generally be made at each corner and 1 in the center. This may be reduced for small buildings. For extremely large buildings and stations or highly variable site conditions, 1 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 1 boring or 1 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 both at the front of and 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 3 percent of the estimated tunnel cost. To characterize the rock in a proposed tunnel zone, rock borings should be advanced to depths such that they extend at least 1.5 to 2 times the tunnel diameter below the tunnel invert elevation. 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 1 boring shall be made at each designated location. CPT soundings 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 and 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 3 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 1 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 evaluate 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. Additional guidance will be provided when characterizing active faults that may produce surface rupture.

10.A.3.6 Construction Material Sources

Standards for investigations for construction material 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 1 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. For design of pavement, refer to *Civil* chapter for details.

10.A.4 References

1. American Association of State Highway and Transportation Officials (AASHTO)
   - Manual on Subsurface Investigations, MSI-1, 1988.
   - Standard Recommended Practice for Decommissioning Geotechnical Exploratory Boreholes, AASHTO R 22-97, standard Specifications, 2005.
   - Specification for Transportation Materials and Methods of Sampling and Testing, Part II: Tests, HM-28-M, 2008.
2. American Society of Civil Engineers (ASCE) reference titled “Geotechnical Baseline Reports for Construction – Suggested Guidelines”, ASCE 2007.
3. American Railway Engineering and Maintenance of Way Association (AREMA) – Manual for Railway Engineering, 2008 Edition.
4. ASTM, Annual Book of ASTM Standards, 2008 Edition.
5. Caltrans, Soil and Rock Logging, Classification, and Presentation Manual, June 2010.
6. Cornforth, D.H., Landslides in Practice: Investigations, Analysis, and Remedial/Preventive Options in Soils, Chapter 4, John Wiley & Sons 2005.
7. Federal Highway Administration (FHWA):
   - Geotechnical Technical Guidance Manual, May 2007.
   - Project Development and Design Manual (Draft) – Chapter 6 - Geotechnical, April 2011.

- Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications, FHWA-ED-88-053, 1988, revised February 2003.
- Road Tunnel Design Guidelines, FHWA-IF-05-023, 2004.
- Geophysical Methods - Technical Manual (Application of Geophysical Methods to Highway Related Problems, cooperatively with Blackhawk Geosciences), DTFH68-02-P-00083, 2003.
- Soils and Foundations Workshop, NHI Course No. 132012, Volumes I and II FHWA-NHI-06-088, and FHWA-NHI-06-089, 2006.
- Subsurface Investigations – Geotechnical Site Characterization, NHI Course Manual No. 132031, FHWA-NHI-01-031, 2002.
- Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034, 2002.

8. Federal Railroad Administration (FRA), High-Speed Ground Transportation Noise and Vibration Impact Assessment, FRA Report No. 293630-1, December 1998.
9. ISRM, Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses, 1981.
10. Kulhawy, F.H. and Mayne, P.W., Manual on Estimating Soil Properties for Foundation Design, EPRI Report EL-6800, 1990.
11. U. S. Army Corps of Engineers (USACE), Geotechnical Investigations, Engineering Manual, EM 1110-1-1804, Department of the Army, 2001.
12. U. S. Army Corps of Engineers (USACE), Soil Sampling, Engineering Manual, EM 1110-1-1906, Department of the Army, 1996.

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 2 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 2 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 down drag, 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 that could have significant impact on high-speed train 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 2 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.

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 penetration resistance. 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 2 design criteria established in the *Geotechnical* chapter:

CPT and/or CPT_u (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 3 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 3 methods are within 20 percent of each other, an average FOS shall be reported for that particular boring and/or CPT. If the FOS values from these 3 methods vary by more than 20 percent 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 1.1.

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 10.B.2.8. Liquefaction-induced total ground settlement of saturated granular deposits shall be estimated using Zhang et al. (2002) and at least 1 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 for fine-grained soils, 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 shear-induced 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 2 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 for fine-grained soils that can be sampled with minimal disturbance in lieu of empirical SPT- or CPT-based criteria. Even when laboratory based test results are obtained and used for design, 2 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 percent 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 10.B.2.9.1 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 2 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 1 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 10.B.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 I_c, 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 2 categories: ground improvement options and structural options.

10.B.2.5 Ground Improvement Options

Refer to Section 10.9.5.5.

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 2 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 10.B.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. However, the liquefaction susceptibility and triggering analyses performed as part of this procedure shall be based on Section 10.B.2.1and Section 10.B.2.2, respectively. Similarly, the lateral spread estimates shall be based on Section 10.B.2.3. The Geotechnical Designer shall compare the estimated lateral spread values with the allowable deformation values and develop mitigation plans described in Section 10.B.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 10.B.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 10.B.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 10.B.2.2.3shall be used. In addition, strength reduction due to cyclic degradation versus strength increase due to the effects of rate of loading shall be considered for normally consolidated clayey layers and non-liquefiable sandy layers. Chen et al. (2006) have discussed the effects of different factors on the dynamic strength of soils. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium FOS is less than 1.1, flow failure shall be considered likely. Liquefaction flow failure deformation is usually too large to be acceptable for design of structures, and some form of mitigation will likely be needed. However, structural mitigation may be acceptable if the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads.

If the FOS for this decoupled analysis is greater than 1.1 for liquefied conditions, k_y shall be estimated using pseudo-static slope stability analysis. The same strength parameters as used during the flow failure analysis shall be used. A new critical failure plane shall be searched assuming both circular and non-circular failure surfaces. Yield acceleration is defined as the minimum horizontal acceleration in a pseudo-static analysis for which FOS is 1.0. Using the estimated k_y values, deformations shall be estimated using simplified methods such as Makdisi and Seed (1978) and Bray and Travasarou (2007). Other methods such as Newmark time history method or more advanced methods involving numerical analysis may be used, but shall be checked against the simplified methods.

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

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 MCE case, the allowable deformation of 4 inches may be assumed. 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 (Drag Load) on Structures Due to Seismic Settlement

Downdrag loads on foundations shall be evaluated in accordance with AASHTO LRFD BDS with California Amendments Article 3.11.8, and as specified herein. AASHTO LRFD BDS with California Amendments 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 down drag 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”
2. American Association of State Highway and Transportation Officials (AASHTO), Manual on Subsurface Investigations, MSI-1, 1988.
3. American Association of State Highway and Transportation Officials (AASHTO) LFRD Bridge Design Specifications, 4th Edition, 2007.
4. American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318.
5. American Institute of Steel Construction, Steel Construction Manual, Thirteenth Edition.
6. ASTM International.
7. American Railway Engineering and Maintenance of Way Association (AREMA) Manual for Railway Engineering.
8. American Society of Civil Engineers, (ASCE) (2006) ASCE/SEI 7-05, “Minimum Design Load for Buildings and Other Structures Chapter 21.” ASCE, Reston, VA.
9. Bardet, J.-P., O'Rourke, T.D., and Hamada, M. (1999). "Large-scale modeling of liquefaction-induced ground deformation, Parts I: A four-parameter MLR model." Proceedings of the 7th U.S.-Japan Workshop on Earthquake Resistance Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, Technical Report MCEER-99-0019, Multidisciplinary Center for Earthquake Engineering Research, pp. 155-174.
10. Barton, N.R., Lien, R. and Lunde, J. (1974). “Engineering Classification of Rock Masses for the Design of Tunnel Support.” International Journal of Rock Mechanics and Mining Sciences, 6(4), 189-239.
11. Blake, T.F. et al. (2002). “Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California.” ASCE Los Angeles and SCEC (downloadable at: http://www.scec.org/resources/catalog/hazardmitigation.html#land)
12. Boulanger, R.W. and Idriss, I.M. (2011). “Cyclic Failure and Liquefaction: Current Issues.” Proceedings, 5th International Conference on Earthquake Geotechnical Engineering, Santiago, Chile, January 10-13.
13. Boulanger, R.E., Change, D., Brandenberg, S.J., Armstrong, R.J., Kutter, B.L. (2007). “Seismic Design of Pile Foundations for Liquefaction Effects”, Chapter 12 in Earthquake Geotechnical Engineering, Springer, The Netherlands.

14. Brandenberg, S. J., Boulanger, R. W., Kutter, B. L., and Chang, D., (2007). “Static Pushover Analyses of Pile Groups in Liquefied and Laterally Spreading Ground in Centrifuge Tests.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 9, pp. 1055-1066.

15. Bray, J. D., Travasarou, T., and Zupan, J. (2010). “Seismic Displacement Design of Earth Retaining Structures.” Proceedings, Earth Retention Conference 3, Bellevue, Washington, August 1-4.

16. Bray, J. D. and Dashti, S. (2010). “Liquefaction-Induced Movements of Buildings With Shallow Foundations.” Proceedings, Fifth International Conference on Recent Advances on Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor I.M. Idriss, San Diego, CA, May 24-29.

17. Bray, J.D. and Travasarou, T. (2009). "Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 9, p. 1336-1340.

18. Bray, J. D., and Travasarou, T. (2007). “Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 4, pp. 381-392.

19. Bray, J.D. and Sancio, R. B. (2006). “Assessment of Liquefaction Susceptibility of Fine-Grained Soils,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 9, pp. 1165-1177.

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

21. The California Building Code.

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

23. California Department of Transportation (Caltrans), “California Amendments to the AASHTO LRFD Bridge Design Specifications (Fourth Edition)”, September 2010.

24. California Department of Transportation (Caltrans), "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading," February 2011.
25. California Geological Survey (2003), Fault Rupture Hazard Zones, Special Publication 42, 47 pages, http://www.conservation.ca.gov/CGS/rghm/ap/Pages/Index.aspx
26. California Geological Survey (1996), Guidelines for evaluating the hazard of surface fault rupture, CGS Note 49, 4 pages, http://www.conservation.ca.gov/cgs/information/publications/cgs_notes/note_49/Documents/note_49.pdf
27. California High-Speed Train Project Program EIR/EIS, Task 1.11.a – Tunneling Issues Report, by Parsons Brinckerhoff, Inc., January, 2004.
28. Cetin, K.O., Bilge, H.T., Wu, J., Kammerer, A.M., Seed, R.B. (2009a). "Probabilistic Models for Cyclic Straining of Saturated Clean Sands." ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 3, p. 371-386.
29. Cetin, K.O., Bilge, H.T., Wu, J., Kammerer, A.M., Seed, R.B. (2009b). "Probabilistic Model for the Assessment of Cyclically Reconsolidation (Volumetric) Settlements." ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 3, p. 387-398.
30. Chen, W., Bray, J.D., and Seed, R.B (2006). "1-G Small Scale Shaking Table Experiments – Seismic Slope Responses and Deformations." Proceedings, 8th US National Conference on Earthquake Engineering, April 18-22, San Francisco, California, Paper No. 1322.
31. Coppersmith and Youngs (1992), Modeling Fault Rupture Hazard for the Proposed Repository at Yucca Mountain, Nevada
32. Dawson, E.M., W.H. Roth, A. Drescher (1999). "Slope stability analysis by strength reduction," Geotechnique, 49(6), 835-840.
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
34. European Standard EN 1991-2:2003 Traffic Loads on Bridges
35. European Standard EN 1990 annex A2: Application to Bridges
36. Faris, A.T., Seed, R.B., Kayen, R.E. and Wu, J. (2006). "A semi-empirical model for the estimation of maximum horizontal displacement due to liquefaction-induced lateral spreading," Proceedings, 8th U.S. International Conference on Earthquake Engineering; Paper No. 1323, San Francisco, CA; April 18-26; 10 p.
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.

38. Federal Highway Administration (FHWA) Publications:
    - FHWA and Oregon Department of Transportation (2001). "Rockfall Catchment Area Design Guide." Final Report SPR-3 (032).
    - FHWA-CFL/TD-06-002, "InSAR Applications For Highway Transportation Projects." April 2006.
    - FHWA-ED-88-053, "Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications." 1988, Revised Feb. 2003.
    - FHWA-IF-02-034, "Geotechnical Engineering Circular No. 5 (GEC5), Evaluation of Soil and Rock Properties." April 2002.
    - FHWA-NHI-05-123 "Soil Slope and Embankment Design Reference Manual." Sept. 2005.
    - FHWA-NHI-00-044, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes." Sept. 2009.
    - FHWA-NHI-05-052, "Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design Reference Manual." Feb. 2005.
    - FHWA-NHI-06-019/020, "Ground Improvement Methods Reference Manual, Volumes I and II." August 2006.
    - FHWA-NHI-06-088/089, "Soils and Foundations Reference Manual, Volumes I and II." December 2006.
    - FHWA-NHI-07-071, "Earth Retaining Structures Reference Manual." June 2008.
    - FHWA-HI-99-007, "Rock Slopes Reference Manual." 1999.
39. Idriss, I.M. and Boulanger, R.W. (2008). "Soil Liquefaction During Earthquakes." Earthquake Engineering Research Institute, MNO-12.
40. 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
41. Ishihara, K. (1985). "Stability of Natural Deposits During Earthquakes." Proc. 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Vol. 2. pp. 321-376.
42. Ishihara, K. and Yoshimine, M. (1992). "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes." Soils and Foundations, JSSMFE, Vol. 32, No. 1.
43. Kenny, J. T. (1954). "Steady-State Vibrations of Beam on Elastic Foundation for Moving Load." Journal of Applied Mechanics, December.
44. Kramer, S. L., and Wang, C. H. (2011). "A hybrid model for estimation of the residual strength of liquefied soil." Submitted for review.

45. Iai, S. Noda, S. and Tsuchida, H. (1988). "Basic considerations for designing the area of the ground compaction as a remedial measure against liquefaction," Proc. U.S.-Japan Joint Workshop of Remedial Measures for Liquefiable Soils, UJNR, Jackson, Wyoming, May 11-13.

46. Ledezma, C. and Bray, J.D. (2010). "Probabilistic Performance-Based Procedure to Evaluate Pile Foundations at Sites with Liquefaction-Induced Lateral Displacement," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 136, No. 3, p. 464-476.

47. Li D., and Selig, E. T. (1998). "Method for Railtrack Foundation Design I: Development," Journal of Geotechnical/Geoenvironmental Engineering, Vol. 124, No. 4, pg 316-322.

48. Mahin, Sakai, and Jeong, "Use of Partially Prestressed Reinforced Concrete Columns to Reduce Post-Earthquake Residual Displacements of Bridges", Paper No. B25, 5th National Seismic Conference on Bridges & Highways, San Francisco, CA, Sept 18-20, 2005.

49. Makdisi, F.I. and Seed, H.B. (1978). "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." ASCE Journal of the Geotechnical Engineering Division, Vol. 104, No. GT7, July, 1978, pp. 849-867.

50. Martin, G.R., Tsai, C-F, Arulmoli, K. (1991). "A Practical Assessment of Liquefaction Effects and Remediation Needs." Proceedings 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, March 11-15.

51. Martin, G.R. and Lew, M (1999). "Recommended Procedures for Implementation of DMG Special Publication 117 – Guidelines for Analyzing and Mitigating Liquefaction in California." Southern California Earthquake Center.

52. Matasovic, N., Athanasopoulos-Zekkos, A., and Zekkos, D. (2008). "Validation of generic municipal solid waste material properties for seismic design of landfills." 4th Geotechnical Earthquake Engineering and Soil Dynamics Conference, Sacramento, CA, May 18-22, 2008.

53. National Cooperative Highway Research Program (NCHRP),

   – Transportation Research Board (2004). "Load and Resistance Factor Design (LRFD) for Deep Foundations." NCHRP Report 507.

   – Transportation Research Board (2006). "Riprap Design Criteria, Recommended Specifications, and Quality Control," Report 568.

   – "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments," Report 611, Chapter 7.

54. Priestley, M.J.N. and Calvi, G.M., "Strategies for Repair and Seismic Upgrading of Bolu Viaduct 1, Turkey", Journal of Earthquake Engineering, Vol. 6, Special Issue 1, pp. 157-184, 2002.

55. Rathje, E.M. and Saygili, G. (2008). "Probabilistic Seismic Hazard Analysis for the Sliding Displacement of Slopes: Scalar and Vector Approaches," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 134(6), 804-814.

56. Rollins, K. M., Gerber, T. M., Lane, J. D., and Ashford, S. A. (2005). “Lateral Resistance of a Full-Scale Pile Group in Liquefied Sand.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 1, pp. 115-125.
57. Saygili, G., and Rathje, E. M. (2008). “Empirical Predictive Models for Earthquake-Induced Sliding Displacements of Slopes.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 134, No. 6, pp. 790-803.
58. Seed, R.B., and Harder, L.F., Jr. (1990). “SPT-based analysis of cyclic pore pressure generation and undrained residual strength.” Proceedings of the H. Bolton Seed Memorial Symposium, May Vol. 2.
59. Seed, R.B., K, O., Cetin, R.E.S., Moss, A., Kammerer, J., Wu, J.M., Pestana, M.F., Riemer, R.B., Sancio, J.D., Bray, R.E., Kayen, R.E., Faris, A. (2003). "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework." Keynote Address, 26th Annual Geotechnical Spring Seminar, Los Angeles Section of the GeoInstitute, American Society of Civil Engineers, H.M.S. Queen Mary, Long Beach, California, USA.
60. Shamoto, Y., Zhang, J., and Tokimatsu, K. (1998). “New Charts for Predicting Large Residual Post-Liquefaction Ground Deformation.” Soil Dynamics and Earthquake Engineering 17, p. 427-438.
61. Sherard, J. L., Dunnigan, L. P., and Talbot, J. R. (1984). “Filters for Clays and Silts,” Journal of the Geotechnical Engineering Division, ASCE, June, pp. 701-718.
62. Structural Welding Code, Steel, 1996 ANSI/AWS D1.1-96.
63. Taiwan High Speed Rail (THSR) Corporation: Volumes 1 and 3.
64. Taylor, D.W. (1937). “Stability of earth slopes.” Journal of the Boston Society of Civil Engineers, 24(3).
65. Tokimatsu, K. and Seed, H.B. (1987). “Evaluation of Settlements in Sand Due to Earthquake Shaking.” Journal of Geotechnical Engineering, ASCE Vol. 113, No. 8, pp. 861-878.
66. United States Army Corps of Engineers (USACE), EM-1110-2-1902, “Engineering and Design – Slope Stability.” Oct. 2003.
67. United States Army Corps of Engineers (USACE), EM-1110-2-2901, “Engineering and Design - Tunnels and Shafts in Rock.” May 1997.
68. Wells, D., and Coppersmith K., “New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement”, Bulletin of the Seismological Society of America, Vol. 84, pages 974-1002, 1994.
69. Wesnousky, Steven, 2008, "Displacement and Geometrical Characterization of Earthquake Surface Rupture: Issues and Implications for Seismic Hazard Analysis and the Process of Earthquake Rupture", Bull. Of the Seismological Society of America, vol. 98, No. 4, p. 1609.
70. Youd, T.L., and Garris, C.T. (1995). “Liquefaction-Induced Ground Surface Disruption.” Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11.

71. Youd, T.L.; Hansen, C.M. and Bartlett, S.F. (2002). “Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, pp. 1007-1017.
72. Youngs et al., (2003), A Methodology for Probabilistic Fault Displacement Hazard Analysis (PFDHA), Earthquake Spectra, Vol. 19, No. 1, p 191-219.
73. Zhang, G. Robertson P.K., Brachman, R.W.I. (2002). “Estimating Liquefaction-Induced Ground Settlements from CPT for Level Ground,” Canadian Geotechnical Journal, Vol. 39, pp 1168-1180.
74. Zhang, G., Robertson, P. K., Brachman, R. W. I. (2004). “Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 8, pp. 861-871.

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 and Mah, 2004; Hoek, 2010).

Where rock slopes exist 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 100 percent of fallen rocks. If right-of-way is not available to size catchment basins to achieve 100 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

- Seismic sensing
- 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 shall 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

Procedures for mapping shall follow those given in the Rock Slopes Reference Manual, FHWA-HI-99-007, 1998, “Appendix I, Geologic Mapping,” Parts 1, 2, and 3. At each mapping window, the CEG shall prepare a detailed section of the exposed cut.

Field observation data shall be recorded on approved forms similar to the 1 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 (at least 10 megapixels) 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 1 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 of the Rock Slope Reference Manual.

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. A pre-blast survey shall also be undertaken. 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 all 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.

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- 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

1. FHWA-HI-99-007, 1999, Rock Slopes Reference Manual.
2. Goodman, R.E. and Shi, G. (1985). *Block Theory and its Application to Rock Engineering,* Prentice-Hal, Englewood Cliffs, N.J.
3. Hoek, E. and Bray, J.W. (1981). “Rock Slope Engineering, Revised 3rd edition.” The Institution of Mining and Metallurgy, London.
4. Hoek, E. (2010). “Practical Rock Engineering.” in Rocscience 2010.
5. Hoek, E., Carranza-Torres, C.T., and Corkum, B. (2002). “Hoek-Brown failure criterion – 2002 edition.” *Proc. North American Rock Mechanics Society meeting in Toronto in July 2002.*
6. Markland, J.T. (1972). “A Useful Technique if Estimating the Stability of Rock Slopes When the Rigid Wedge Sliding Type of Failure is Expected.” *Imperial College Rock Mechanics Research Report No. 19*. London, Imperial College Press.

7. Wyllie, D.C and C.W. Mah (2004). “Rock Slope Engineering: Civil and Mining.” 4th Edition, Spon Press.

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Chapter 21

Overhead Contact System and Traction Power Return System

Revision	Date	Description
0	02 Mar 12	Initial Release, R0
0.1	Dec 2012	Execution Version
1	Sept 2013	Rolling Stock Procurement
2	Feb 2014	Revision 2

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Acronyms

ACSR	Aluminum Conductor Steel Reinforced
EMC	Electromagnetic Compatibility
HST	High-Speed Train
NESC	National Electrical Safety Code
OCS	Overhead Contact System
PS	Paralleling Station
SCADA	Supervisory Control and Data Acquisition
SS	Substation
SWS	Switching Station
TES	Traction Electrification System
TPF	Traction Power Facility

21 Overhead Contact System and Traction Power Return System

21.1 Scope

These criteria detail the overhead contact system (OCS), and the traction return system, including the parallel negative feeders.

The OCS is a system in which electrical conductors are supported aerially above the Authority's right-of-way, generally by means of insulators and appropriate mechanical support arms or brackets, and which supplies electrical energy from the traction power supply facilities to rail mounted, electrically-powered vehicles through onboard, roof-mounted current collection equipment (pantographs). The OCS comprises the following:

- All overhead wiring, including the messenger wires, stitch wires, and contact wires, mounted on OCS support structures or brackets
- The foundations, supporting structures, and any components supporting, registering, terminating or insulating the conductors
- Insulators, neutral-sections, auto-tensioning devices, and other overhead line hardware and fittings
- Equipment mounted on the supports for feeding, switching, detection or protection
- Overhead conductor rails and their insulated support arrangements (if used) in very restricted clearance locations

The traction return system is the means by which traction current is returned from the wheel-sets of traction units to the traction power facilities of the electrified railway track, comprising the negative feeders (due to the configuration of the autotransformer connections), the grounded running rails, aerial static wires (and buried ground conductors), together with all return current bonding and grounding interconnections. Grounding and bonding and lightning protection for the electrified railway is covered in the *Grounding and Bonding Requirements* chapter.

In an auto-transformer feed system, the feeder (often termed the negative feeder) is a paralleling conductor that is electrically separate from the catenary conductors over the tracks. This parallel (negative) feeder connects successive feeding points, and is connected via circuit breakers and/or disconnect switches to 1 terminal of a main power supply transformer or autotransformer in the traction power facilities. At these facilities, the other terminal of the transformers is connected to an OCS section (or sections) via circuit breakers or disconnect switches.

21.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards.

- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering, Chapter 33 Electrical Energy Utilization
- European Committee for Electrotechnical Standardization (CENELEC) Standards
  - EN 50119, 2001, Electric Traction Overhead Contact Lines
  - EN 50122-1 Part 1, 1998, Protective Provisions Relating to Electrical Safety and Earthing
  - EN 50124-1, 2001, Insulation Coordination: Part 1 – Basic Requirements
  - EN 50149, 2001, Electric Traction: Copper and Copper Alloy Grooved Contact Wires
  - EN 50206, 1999, Pantographs: Characteristics and Tests
  - EN 50317, 2002, Requirements for and Validation of Measurements of the Dynamic Interaction between Pantograph and Overhead Contact Line
  - EN 50318, 2002, Validation of Simulation of the Dynamic Interaction between Pantograph and Overhead Contact Line
  - EN 50367, 2006, Technical Criteria for the Interaction between Pantograph and Overhead Line
- California Code of Regulations (CCR) Title 8, Division 1, Chapter 4, Subchapter 5: Electrical Safety Orders
- Institute of Electrical and Electronics Engineers (IEEE)
  - IEEE C2, National Electrical Safety Code (NESC)
  - IEEE 80, Guide for Safety in AC Substation Grounding
  - IEEE 142, Recommended Practice for Grounding of Industrial and Commercial Power Systems
- California Public Utilities Commission (CPUC) General Orders (GOs)
  - CPUC GO 26-D, Regulations Governing Clearances on Railroads and Street Railroads with Reference to Side and Overhead Structure Parallel Tracks, Crossings of Public Roads, Highways and Streets
  - CPUC GO 95, Rules for Overhead Electric Line Construction
  - CPUC GO 143-B, Safety Rules and Regulations Governing Light Rail Transit
- Technical Specification for Interoperability (TSI) Energy, 2008, Technical Specifications for the Interoperability of Electrical Energy Subsystems

21.3 Definitions

Agency	The railroad or other jurisdictional entity that is responsible for the operation and maintenance of the railroad
Barrier	Equipment provided to prevent entry by an unauthorized person to a restricted area, structure or building, which also provides physical protection against direct contact with energized parts from non-normal directions of access
Bond	A bond is an electrical connection from 1 conductive element to another for the purpose of maintaining a common electrical potential (equi-potential).
Bonding Conductor	A conductor for ensuring equi-potential bonding
Collector Head	That part of the pantograph that runs under and in contact with, and collects current from, the overhead contact wire or conductor rail.
Cross Bond	An electrical bond that interconnects the running rails, which in signalized territory, must be connected through impedance bonds.
De-energized	Electrical apparatus, such as overhead wires, substation conductors, cables, switches and circuit breakers, which is disconnected from its electrical power source(s), but is not necessarily grounded. Note: This does not imply or ensure a safe state.
Direct Contact	Contact with energized parts
Direct Feed System	A traction power feeding system in which the transformers are fitted with a single secondary winding having 2 terminals. One terminal is connected to the running rails/ground and the other to the catenary conductors over the tracks.
Direct Traction System Grounding	The direct connection between conductive parts and the traction system ground Note: Grounding via impedance bonds, required by reason of signaling system track circuit considerations, is considered to be direct grounding.
Effectively Grounded	Intentionally connected to earth through a ground connection or connections of sufficiently low impedance and having sufficient current-carrying capacity to limit the build-up of voltages to levels below which undue hazards to persons or to connected equipment may result.

Electrical Section or Feed Section This is the entire section of the OCS, which during normal system operation, is powered from an SS circuit breaker. The SS feed section is demarcated by the phase breaks of the supplying SS and by the phase breaks at the adjacent SWS or line end. An electrical section maybe subdivided into smaller elementary electrical sections.

Elementary Electrical Section This is the smallest section of the OCS traction power distribution system that can be isolated from other sections or feeders to the system by means of disconnect switches and/or circuit breakers.

Electric Shock The effect of an electric current passing through the human body

Energized Electrical apparatus, such as overhead wires, substation conductors, cable, disconnect switches, and circuit breakers, that are connected to an electric power source.

Energized Part An energized part is a conductor or conductive part that is energized under normal service conditions, but does not include the running rails or parts connected to them. Energized parts include roof-mounted equipment on electric vehicles, such as pantographs, train line conductors, and resistor units. The full length of insulators connected to energized parts shall be classified as energized when considering electrical clearance requirements.

Fault Condition The presence of an unintended and undesirable conductive path in an electric power system

Feeder A current-carrying electrical connection, energized at high voltage (HV), between a traction power facility (substation, paralleling station or switching station) and the catenary conductors, which is energized at high voltage (HV) – 25 kV nominal, and is supported on the same structure as the catenary and static wire.

1. In an auto-transformer feed system, the feeder (often termed the negative feeder) is a paralleling conductor that is electrically separate from the catenary conductors over the tracks. This parallel (negative) feeder connects successive feeding points and is connected, via a circuit breaker(s) and/or disconnect switch(es), to 1 HV terminal of a main power supply transformer or an auto-transformer in the traction power facilities. At these facilities, the other HV terminal of the main power supply transformer or auto-transformer is connected to a catenary section (or sections) via circuit breakers or disconnect switches.

2. In a direct feed system, the feeder is a paralleling conductor that can be connected at frequent intervals to the OCS to provide localized electrical reinforcement of the circuit by increasing the effective cross-sectional area of the electrical system in that section.

Grounding Conductor	A conductor that is used to connect equipment or wiring systems to a ground electrode or ground grid
High Voltage (HV)	A nominal voltage of 600 Volts or more
Leakage Current	A current that flows to ground or to extraneous conductive parts, following a path or paths other than the normal intended path, but which is not of sufficient magnitude to create a fault.
Non-Current-Carrying Parts	Metallic parts within the Authority's right-of-way which do not normally carry load currents or return currents
Overhead Conductor Rail	A rigid metallic conductor, which substitutes for the contact wire and is mounted on insulators under a fixed overhead structure
Paved Areas	In selected areas of maintenance facilities, yards and shops, the trackway may be paved to the upper level of the running rails to provide for the crossing of maintenance vehicles over the tracks and under the overhead conductors. Note: Where railroads support high-speed operations, at-grade crossings of any description are not permitted.
Rail Joint Bond	A conductor that ensures the electrical continuity of a running rail at an uninsulated, bolted rail joint
Rail Potential	The voltage between running rails and ground occurring both under operating conditions when the running rails are utilized for carrying the traction return or under fault conditions
Rail to Ground Resistance	The electrical resistance between the running rails and the earth
Railroad or Railway Environment	The area adjacent to the running rails that is subject to the noise, vibration and air pressure of trains operating at high speed, and to the effects of the voltages, currents and electric fields associated with a 25 kV ac TES
Rake	A preset lean of an OCS pole from vertical
Regenerative Braking	A system in which the drive motors of the electric vehicles operate as generators and provide dynamic braking of the vehicle, while at the same time returning power to the OCS that can be used by receptive vehicles on the system or can be returned to the utility at the traction power substations.
Return Cable	A conductor that forms part of the TES return circuit, and which connects the rest of the return circuit to the substation

Right-of-Way classification	Right-of-Way (ROW) classifications are defined in CPUC GO No. 143-B Rule 9.04 1. Exclusive -- a railroad or railway right-of-way without at-grade crossings, which is grade-separated or protected by a fence or substantial barrier, as appropriate to the location (including subways and aerial structures). Note: This is the only type of right-of-way that is acceptable for the operation of trains at speeds in excess of 125 mph. 2. Semi-Exclusive -- exclusive right-of-way with at-grade crossings protected between crossings by a fence or substantial barrier, as appropriate to the location. Note: This type of right-of-way is not suitable for high-speed operations in excess of 125 mph, since at-grade crossings of the high-speed tracks cannot be permitted in accordance with FRA regulations.
Running Rails	The steel rails on which the rail vehicles run and which, in an electrified system, form part of the traction return circuit. The running rails may also be used for signal system track circuits, in which case special measures must be implemented to permit joint use with electrification.
Screen	A barrier that prevents unintentional direct contact with energized parts but will not totally prevent direct contact by deliberate action.
Short Circuit	A conductive path between energized and grounded components which may result in a high fault current. Note: Any such conductive path whether between conductors or between a conductor and ground is regarded as a short circuit.
Short Circuit Current or Fault Current	The electric current flowing through the short circuit or fault path.
Standing Surface	Any point on a surface where persons may stand or walk.
Step Potential or Voltage	The difference in surface potential experienced by a person bridging a distance of 3 feet-3 inches with their feet without contacting any grounded object.

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Stitch wire	The stitch wire is a supplementary tensioned conductor that is attached to the messenger wire and positioned at the supports with hangers supporting the contact wire. The spring effect of the stitch wire and hanger arrangement enhances the elasticity of the catenary at the support and provides for a better match with the mid-span elasticity of the catenary, thereby providing improvement in the quality of the current collection.
Stray Current	A current which follows a path or paths other than the intended electrical path (see Leakage Current).
Supports	The structural elements that support the conductors and their associated line hardware and insulators in an OCS.
Surge Arrester or Surge Suppressor	A protective device for limiting surge voltages on equipment by discharging or by-passing surge current; it limits the flow of power follow-on current to ground, and is capable of repeating these functions. Note: Sometimes referred to as a Lightning Arrester.
Touch Voltage:	The potential difference between the ground potential rise (GPR) and the surface potential at the point where a person is standing while at the same time having a hand in contact with a grounded structure (Per IEEE-80).
Traction Power Substation (SS)	An electrical installation where power is received at high voltage and transformed to the voltage and characteristics required at the catenary and negative feeders for the nominal 2x25 kV system, containing equipment such as transformers, circuit breakers and sectionalizing switches. It also includes the incoming HV lines from the power supply utility.
Traction Return Current	The sum of the currents returning to the supply source (i.e., the substation).
Traction System Ground	The traction system ground consists of the running rails, the aerial static wires and all conductive parts connected thereto and which are solidly connected to ground.
Traction System Grounding	Connection between non-energized metallic parts and the traction system ground.
Tunnel Ground	The electrical interconnection of the reinforcing steel in reinforced concrete tunnels and, in the case of other modes of construction, the conductive interconnection of the metallic parts of the tunnel. Note: In the case of single-phase ac traction systems, the tunnel ground is connected to the running rails and thus forms part of the traction system ground which may be supplemented by external ground connections to earth.

Voltage-limiting Device A protective device which operates to prevent the permanent existence of a dangerously high step or touch voltage.

21.4 Overhead Contact System Description and General Performance Requirements

In order to minimize the number of substations and Electromagnetic Compatibility (EMC) problems along the alignment, the line will be fed by a 2x25 kV, 60 Hz autotransformer power supply system, utilizing traction power substations, switching stations and paralleling stations.

The Traction Power Substations (SS) will be connected to HV utility supplies and spaced approximately every 30 miles, while the Switching Stations (SWS) will be spaced at approximately mid distance between SS, i.e., at about 15 miles from each SS, and the Paralleling Stations (PS) will be spaced at approximately 5 mile intervals. At the PS and SWS locations, the autotransformers will parallel the Track 1 and Track 2 power supplies and balance the 2 25 kV supplies (longitudinal parallel negative feeder and catenary) with respect to each other.

The OCS shall support voltage variations in accordance with IEC 60850 “Supply Voltages of Traction Systems”, as shown in Table 21-1.

Table 21-1: Traction Power System Voltages

Voltage Condition	Symbol	Voltage
Operating nominal system voltage		25.0 kV
Highest permanent voltage	U_{max1}	27.5 kV
Highest non-permanent voltage	U_{max2}	29.0 kV
Lowest permanent voltage	U_{min1}	19.0 kV
Lowest non-permanent voltage	U_{min2}	17.5 kV

In addition, the maximum short circuit current shall be 15 kA for protection measurement purpose and accordingly for specification of the electrical equipment.

At all traction power supply stations the center tap of the respective supply transformer or auto-transformer will be connected to and referenced to the running rails, which will nominally be at ground potential.

The OCS will provide electric traction power to the pantographs of the electric trains using the route and will, therefore, be configured as a 25 kV-0-25 kV arrangement with the catenary at a nominal voltage of 25 kV to ground and the longitudinal parallel negative feeder also at a nominal voltage of 25 kV to ground, but in phase opposition to the catenary. There is a 180 degree phase difference between the voltages of the parallel negative feeders and the OCS, giving a 50 kV phase-to-phase voltage difference between these conductors. The OCS shall transfer electric power from the Traction Power Substations to the trains under all operating conditions and shall provide for reliable operation under the environmental conditions detailed in Section 21.5.

Except at Phase Breaks, the OCS shall provide for uninterrupted traction power collection at the maximum operating speed of 220 mph.

To allow bi-directional working, enabling trains to continue operation under emergency conditions and to facilitate routine OCS maintenance, the OCS shall be divided into electrical sections and sub-sections. The OCS shall be sectionalized as indicated in Section 21.12.

To facilitate operations and maintenance activities, the OCS shall typically be equipped with non-load break motor operated disconnect switches at feeding points, which can be operated both locally on site and remotely through a supervisory control and data acquisition (SCADA) system. The switches shall be fitted with OCS voltage detection circuitry that will provide for remote monitoring of the system.

The OCS phase break arrangements shall be located at SWSs and, as required, at SS to electrically separate 2 successive catenary electrical sections fed from different 25 kV ac sources, i.e., not of the same phase. The electric trains shall pass through each phase break arrangement without establishing an electrical connection between the successive electrical sections which are fed from different phases. This shall be achieved at the designated maximum operating speed with the train pantographs raised and in contact with overhead contact wire, but with the pantograph breakers off.

Rail return shall primarily be through the running rails, but a static wire (ground wire) shall be provided that interconnects all OCS support structures (poles, portal structures, wall brackets, tunnel drop pipes, etc.), which shall be connected via impedance bonds to the running rails and to the ground grid at each traction power facility (TPF). Other cross-bonding connections may be required to minimize rail potential rise, and the frequency and location of these connections and of the impedance bonds shall be determined under the TP system design and coordinated with the ATC System design. Refer to the *Grounding and Bonding Requirements* chapter. In electrical sections remote from sections in which trains are operating, the parallel negative feeder effectively carries much of the return current and minimizes the amount that flows through the rails and static wires.

For a more comprehensive description of the traction power supply system and its associated facilities, refer to the *Traction Power Supply System* chapter.

21.5 Environmental Conditions and Climatic Loading Requirements

Information on climatic and environmental conditions in the corridor is given in the *General* chapter with data listed on a segment-by-segment basis. The OCS shall be designed on a system-wide basis to provide for reliable operation under the following environmental and climatic conditions.

21.5.1 Humidity

The OCS shall operate without failure or deterioration in all humidity conditions found in California. These include 100 percent humidity, including rain, heavy fog and salt-laden atmospheres in sections of the route near the ocean, and 100 percent humidity in tunnels.

21.5.2 Ice

Reference to Figure 7.1 ''Ground Snow Loads'' of the ASCE Standard "Minimum Design Loads for Building Structures" indicates limited snow falls and formation of ice along the alignment. In accordance with Table 250-1 and Figure 250-3(a) of the National Electrical Safety Code (NESC), the OCS design shall not consider ice loading.

21.5.3 Wind

The ASCE Standard "Minimum Design Loads for Building Structures" defines the basic wind speed corresponding to the wind load for wind force resisting structures as a 3 second gust speed at 33 feet above ground for open terrain, Exposure C, associated with an annual probability of 0.02 (50 year mean recurrence interval) of being equaled or exceeded. This basic wind speed, in accordance with Figure 6-1 of the ASCE Standard, is V_{bws} = 85 mph for the State of California. This 3 second gust speed corresponds to a mean maximum hourly wind speed of $V_{bws}/1.52$ = 56 mph approximately.

In accordance with Section 4.2.2 of Chapter 33 of the AREMA Manual, 2 different wind speeds, the operational wind speed and the design wind speed shall be used for OCS design:

- The operational wind speed shall be used to compute catenary support loading, catenary wire displacement for pantograph security, and permissible maximum span lengths, and will be taken as V_{op} = 60 mph.
- The design wind speed shall be used to determine the ultimate strength requirements of the OCS and shallwill be taken as V_{bws} = 85 mph corresponding to the ASCE and NESC basic wind speed for the route.

The wind velocity pressure q_z shall be calculated by the NESC formula:

$$q_z = 0.00256\ V^2\ K_z\ G_{RF}\ I\ Cf\ A \quad \text{in pounds/square foot}$$

Where:

- 0.00256 is the velocity pressure numerical coefficient reflecting the mass density of air for the standard atmosphere
- K_z is the velocity pressure exposure coefficient
- V is the basic wind speed = 3 second gust wind speed at 33 feet above ground for open terrain, Exposure C; i.e., V_{bws} in mph

- G_{RF} is gust response factor
- I is the importance factor (I being equal to 1.00 for OCS)
- C_f is the force coefficient shape factor
- A is the projected wind area

Note: K_z, V and G_{RF} are based on open terrain with scattered obstructions (Exposure Category C as defined by ASCE, and are used as the basis for the NESC extreme wind criteria). For very exposed areas, the wind velocity pressure shall be increased by the ASCE factor K_{zt}.

For OCS structural calculations, loads due to wind shall be multiplied by the load factors given in NESC Table 253-1. The effects of wind pressure on OCS poles, due to slipstream effects caused by the proximity to high-speed trains operating at speeds in excess of 125 mph shall also be considered.

21.5.4 Atmospheric Pollution

The OCS equipment shall be resistant to polluted atmospheres, such as may occur in highly industrialized areas, salt-laden marine atmospheres near the ocean, and persistent fog. In addition, the OCS equipment shall be resistant to the corrosive atmospheres that may be found in tunnels and cut-and-cover structures.

21.5.5 Ambient Temperatures Range

General: In developing the OCS arrangements, the designer shall take into consideration the typical and extreme ambient temperatures, as recorded on a segment-by-segment basis along the route.

Tunnels: For long tunnels, only the first 1300 feet of catenary from each portal shall be considered subject to external ambient temperature variations. For the balance of the tunnel length inside the 1300-foot limits, the Designer shall confirm the probable ambient temperature range, which may differ from the external range, and shall use the identified range for the tunnel OCS design.

21.5.6 Conductor Wire Temperature Range

General: In developing the settings for the auto-tensioning devices (balance weight anchor assemblies [BWAs]), the designer shall take into consideration the extreme operational OCS conductor temperatures. Based on the initial analyses, the messenger and contact wires and the parallel feeder conductors are likely to reach a maximum operating temperature of 176°F in above grade sections and for the first 1300 feet in tunnels.

21.5.7 Conductor Tensioning

General: The mechanical tensions in the messenger and contact wires shall be maintained automatically throughout the temperature ranges specified above.

The designer shall confirm the probable maximum conductor temperatures in the tunnels, which shall be used for OCS tensioning and support system design.

21.6 Overhead Contact Line Design

The OCS shall be of a proven design that is capable of sustaining satisfactory current collection for train operations at 220 mph.

The OCS System Designer shall be cognizant of and shall incorporate into the OCS design the fundamental design data and performance instructions, as defined in these Design Criteria, which include the following:

- Service and operations information
- Infrastructure characteristics
- Vehicle characteristics
- Pantograph characteristics
- Traction power system design
- Environmental conditions
- Safety requirements, including external limitations on contact wire height, uplift, system height, and/or clearances
- Life expectancy and desired maintenance/renewal philosophy for all components, plus allowable grooved contact wire wear
- Specification of EMC limitations

It is vital that the OCS design be coordinated with the pantograph and rolling stock designs because 1 of the most critical aspects in the development of the OCS design is the dynamic performance of the overhead contact line and the need to achieve good quality current collection at high operating speeds. Refer to the *Rolling Stock-Core Systems Interface* chapter. The principal measure of this aspect is loss of contact between the pantograph strip and the contact wire, which must be minimized. In order to minimize contact loss and the creation of arcs, the Designer shall comply with the requirements for dynamic behavior and quality of current collection as detailed in the following sections.

Other important aspects of the design are the provision of adequate clearances or provision of protective barriers and screens, together with effective grounding and bonding of the electrical system and wayside metallic objects, which are required to minimize safety hazards.

The recommended minimum clearances between energized parts and grounded parts are detailed in Section 21.14.8. The location of OCS equipment, including poles and downguys, shall be coordinated with clearances defined in the *Utilities* chapter.

21.6.1 Geometry of the Overhead Contact Line

The OCS shall consist of a simple, stitched auto-tensioned catenary system, using a bare hard-drawn copper, bronze or other copper alloy messenger wire supporting a nominally level (no pre-sag), solid copper alloy contact wire by means of copper alloy current carrying hangers.

In general, the catenary shall be supported by pole mounted cantilever frames, which shall be designed to provide the required system height and to register the correct stagger of the wires relative to the track centerline. The messenger wire shall be positioned vertically (plumb) above the contact wire. Back-to-back cantilevers, supported on single poles centered between tracks, shall not be used for the high-speed main tracks, except in station areas where their use will be permitted between a through-line and a station platform track.

An aerial static wire (ground wire), connected at regular intervals to the track via impedance bonds, shall be run alongside the catenary to interconnect each OCS support structure and bracket, such that all OCS non-live metallic supports are at the same ground (and track) reference potential.

The longitudinal negative feeder shall be supported near the top of the OCS poles, preferably on the track side, but may be positioned on the field side where the Authority’s right-of-way width or overhead structure configuration dictates.

The aerial parallel negative feeders, and the aerial static/ground wires that connect all OCS supporting structures, shall both be fixed termination bare ACSR (Aluminum Conductor Steel Reinforced) conductors, except where local site conditions (reduced clearances, etc.) dictate the need to use insulated cables for the negative feeders.

The method of auto-tensioning the messenger wire and contact wire shall be by balance weight and pulley tensioning devices. The tensions shall be applied to the contact and messenger wires individually, using separate balance weights, tensioning devices and anchoring positions.

The Designer shall evaluate overlap arrangements but the initial analyses indicate that the overlaps should comprise a 5-span configuration.

Maximum tension lengths from anchor to anchor shall not exceed 4,600 feet in open route sections and 4,000 feet in tunnels and adjacent to traction power substations and switching stations. Exceptions up to 5,000 feet may be allowed on a case-by-case basis. At approximately mid-distance between auto-tension terminations, mid-point anchor arrangements shall be installed, such that the maximum half tension lengths do not exceed 2,300 feet in open route sections and 2,000 feet in tunnels and at power supply stations and switching stations.

The maximum permissible span length between supports shall be determined using appropriate computer software programs, which shall take into consideration the permissible working range of the pantograph and allowable lateral displacement of the contact wire under the designated operating conditions, including dynamic movement of the vehicle and pantograph. The programs shall be fully validated and back-up hand calculations shall be furnished. The maximum permissible span differential (difference in adjacent span lengths) shall be no more than 33 feet with the proviso that, if the dynamic pantograph-OCS simulation results (detailed below) indicate other values may be more appropriate, these shall be adopted.

At overlap locations where sectionalizing is not required, uninsulated mechanical overlaps shall be installed that permit the pantographs to transition smoothly from 1 tension length to the next under power.

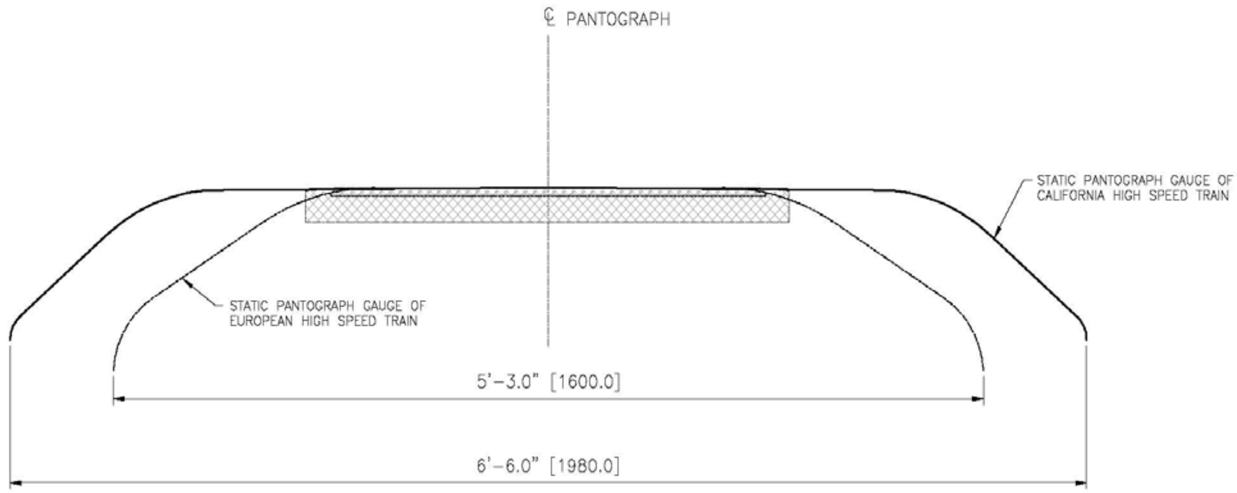
Wherever practicable, the OCS shall be free running under overhead bridges, i.e., no OCS or feeder support attachments under the structure. New bridges shall be designed to accommodate a free-running clearance height. Existing bridge clearances, if any, shall be reviewed to determine whether free-running OCS arrangements can be accommodated, which is the goal. The Designer shall secure permits for attachments to any third party owned bridges or structures where it is determined that OCS support, registration or termination attachments will be required.

In tunnels, the OCS, feeders and static wires shall, wherever possible, be supported by cantilevers attached to soffit-mounted drop pipes, or from wall-mounted support brackets. The OCS system height and cantilever geometry will be dictated by the available headroom. In limited clearance locations, particularly at low headroom bridges or in cut-and-cover tunnels, resilient arms, supporting and registering both messenger and contact wires, may have to be utilized. If any extremely restricted clearance locations are identified, it may be necessary to adopt the use of conductor rail arrangements, but these should be avoided wherever possible.

21.6.2 Geometry of the Pantographs

The Overhead Contact Line shall be designed to accommodate pantographs with a pantograph head profile, as depicted on Figure 21-1, which shall be based on the geometry detailed in EN 50367: 2006 Figure B.3, but having a maximum pantograph head width of 78 inches (1980 mm), and with horns made of insulating material. Based on the dynamic OCS-Pantograph simulations and other dynamic OCS analyses, the Designer may determine that a narrower width pantograph is acceptable. Regardless, it shall not be smaller than the 5 feet 3 inch (1600 mm) European standard profile as shown in Figure A.7 in EN 50367: 2006.

Figure 21-1: Combined Pantograph - Maximum Static Geometry (mm)



Pantograph heads fitted with contact strips, having independent suspensions, shall remain compliant to the overall profile with a static contact force of 15.75 pounds force (lbf) applied to the middle of the head.

The contact wire shall be installed and maintained at a nominal constant height of 17 feet 5 inches at the supports with a construction tolerance of ± 0.5 inches subject to the proviso that the contact wire height difference at adjacent structures shall be less than 1/2 inch to ensure the near-constant contact wire height that is required for satisfactory current collection by pantographs at high speed.

To satisfy the clearance requirements above paved areas in maintenance facilities, yards and workshops, as detailed in Clause 21.14.12, the contact wire shall be installed and maintained at a nominal constant height of 20 feet 6 inches at the supports with a construction tolerance of ± 1 inch subject to the proviso that the contact wire height difference at adjacent structures shall be not more than 1 inch. To make the transition from the mainline contact wire height of 17 feet 5 inches to the yard height of 20 feet 6 inches, the contact wire height shall be increased in accordance with the gradients specified in Table 21-2.

The maximum permissible contact wire gradients and the corresponding maximum gradient changes shall not exceed, according to the maximum speed, the following values:

Table 21-2: Maximum Permissible Contact Wire Gradient versus Operating Speed

Maximum Speed (mph)	Maximum Contact Wire Gradient	Maximum Contact Wire Gradient Change
> 125	0	0
125	2/1000	1/1000
100	3.3/1000	1.7/1000

75	4/1000	2/1000
60	6/1000	3/1000
45	8/1000	4/1000
30	13/1000	6.5/1000

On tangent track (straight track), the contact wire shall be staggered at each location to alternate sides of the pantograph center line, and the stagger shall normally be set at ±12 inches. On curved track, the staggers shall be calculated on a case by case basis taking into account the track superelevation, radius of curvature, and wind speed, but shall not exceed 15 inches. Registration elements (steady arms, contact wire clips, etc.) shall be as light as possible to minimize the possibility of creating a hard spot in the contact wire.

For pantograph security purposes, the permissible lateral deflection of the contact wire under the action of crosswind (defined as the maximum operational wind speed for which unrestricted train operations will be permitted) shall be the smaller of ≤ 15 3/4 inches (400 mm) or 55-L2 inches ([1.4–L2] m) for the 5 feet 3 inch wide pantograph, as specified in ENE 4.2.9.2, where L2 is the half-width of the dynamic envelope of the pantograph passage (as defined in Appendix A.3 of EN 50367: 2006). If a pantograph width other than 5 feet 3 inch is to be used, the 55 inch dimension shall be adjusted.

21.6.3 Compliance of the Overhead Contact Line System with the Infrastructure Gauge

The OCS design shall comply with the static and dynamic envelopes, as defined in the *Trackway Clearances* chapter, for vehicles that will run on the dedicated high-speed train tracks.

The design of civil structures shall take into account the space necessary for the passage of pantographs in contact with the overhead line equipment and for installation of the OCS itself, as detailed in Section 21.14.10 and as depicted in Figure 21-9, and the Standard and Directive Drawings. The dimensions of tunnels and other structures shall be mutually compatible with the geometry of OCS and the dynamic envelope of the pantograph, the static profile of which is depicted on Figure 21-1.

21.7 Conductor Tensions

The permissible tensile loading of the wires and ropes to be used shall consider the weighted parameters, as indicated in EN 50119: 2001, clauses 5.2.4, 5.2.5, and 5.2.6, which include maximum working temperature (excluding short circuit loading), allowable wear, wind and ice loads, tensioning accuracy and efficiency, termination fitting effects, welded or soldered joint effects, and creep, as applicable. In addition, the current heating effects of short circuit faults occurring during peak operations shall be assessed to ensure that the maximum permissible conductor temperatures are unlikely to be exceeded.

21.8 Catenary Conductors

Initial analyses have shown that the following conductors form a viable catenary. The Designer shall confirm conductor sizes and material selection as defined in these criteria. The same conductor and cable types and sizes shall be used throughout the entire system.

21.8.1 Contact Wire

The proposed contact wire is a 150 mm² (approximately 300 kcmil equivalent) grooved copper-magnesium alloy wire, designated CuMg 0.5 that shall comply with the requirements of EN 50149: 2001 Clauses 4.1, 4.2, 4.5, 4.6 and 4.7, regarding the material designation and composition, conductor appearance and condition, clamping groove, electrical characteristics (resistivity, resistance per mile [km]), tensile strength and percentage elongation after fracture, breaking load, and mass of the wire. Joins shall be permitted in drawing stock or intermediate rod stock, as detailed in EN 50149: 2001 Clause 4.8, but no joints shall be made in the completed wire.

21.8.2 Messenger Wire

The proposed messenger wire is a 300 kcmil, 37-strand, hard drawn copper conductor conforming to ASTM B-1 and ASTM B-8 requirements. Substitutes that can meet the electrical and mechanical requirements could be accepted, as detailed in Section 21.8.5 of these criteria.

21.8.3 Stitch Wire

The proposed stitch wire is a 76 kcmil, 7-strand, hard drawn bronze conductor, Alloy 55, conforming to ASTM B-8 and ASTM B-105 requirements.

21.8.4 Hanger Wire

The Designer shall select a suitable flexible conductor for the hanger wire, which together with the messenger wire and contact wire clips, shall provide an electrical connection between the messenger and contact wires.

21.8.5 Alternate Conductors

If the Designer opts to use conductors other than those indicated above, the Designer shall confirm conductor sizes and material selection as defined in these criteria. In addition, the Designer shall require the manufacturer to provide conformity verifications, as detailed in EN 50149, during the production phase of all catenary wires.

21.9 Other Overhead Conductors and Cables

Insulated cables and conductors required by other disciplines such as signal cable, signal-power cables, control wires and communications cables, will generally be installed underground but may occasionally have to be mounted aerially on the OCS poles.

These aerial conductors shall be mounted and spaced on the OCS support structures in accordance with the more stringent requirements of either the NESC or CPUC General Order rules, as they apply to each system classification. Mounting arrangements shall provide for the safety of maintenance personnel. These cables and conductors shall be mounted and profiled in such a manner as to avoid the Overhead Contact Line Zone and Pantograph Zone (Figure 21-5) to the greatest extent practicable. Loading calculations and structural designs for the support of these cables and conductors shall comply with these design criteria.

Insulated cables and bare conductors (other than the catenary conductors identified above) that are associated with the OCS may parallel or cross the Authority's right-of-way, including the parallel negative feeders, and static (ground) wires. The following selections have been made based on preliminary analyses. However, each shall be confirmed by the Designer.

21.9.1 Parallel Negative Feeder

In general, the parallel negative feeder shall be a bare stranded 556 kcmil ACSR "Eagle" conductor for use throughout the system. Since the mainline will be 2-track, with platform tracks at intermediate stations, 2 negative feeders are to be installed; 1 on each side of the Authority's right-of-way.

At locations where a bare conductor cannot be installed, appropriately sized insulated 25 kV cables with appropriate sealing ends shall be substituted and spliced into the bare conductor, which may or may not have to be terminated on a dead-end anchor pole.

21.9.2 Static (Ground) Wire

In general, the static wire shall be a bare stranded 4/0 ACSR "Penguin" for use throughout the system. Two static wires are to be installed, 1 on each side of the Authority's right-of-way, interconnecting all metallic OCS support structures, including OCS poles and bridge and tunnel drop pipes and wall brackets, to provide a continuous ground connection.

21.9.3 Insulated 25 kV Cable

Power feeder cables, where used, shall be insulated with a black, low-smoke, flame-retardant, ozone-resistant, ethylene-propylene compound jacket and the conductor shall be coated, soft-drawn stranded copper, covered with a double-wrapped separator tape or extruded semi-conducting ethylene propylene rubber (EPR) screen. Cables shall be sized to suit the identified ampacity requirements and installation location conditions.

21.9.4 Insulated Return Cable

Return cables, where used, shall be insulated with a black, low-smoke, flame-retardant, ozone-resistant, ethylene-propylene compound jacket and the conductor shall be coated, soft-drawn stranded copper, covered with a double-wrapped separator tape or extruded semi-conducting EPR screen. Cables shall be sized to suit the identified ampacity requirements and installation location conditions.

21.10 Dynamic Behavior and Quality of Current Collection

Good quality interactive dynamic performance with minimum wear can be assured by consideration of the quality of current collection, which has a fundamental impact on the life of the contact wire and pantograph components. Compliance with several measurable parameters, as detailed below, shall be achieved.

21.10.1 Requirements

The number of pantographs in service per train and the spacing between multiple pantographs is necessary to confirm the OCS phase break design arrangement. These factors have a significant impact on the quality of current collection, since each pantograph interacts dynamically through the OCS with the performance of other pantographs. This interaction is also affected by the wave propagation speed. The overhead contact line shall be designed for operation at the maximum line speed with 2 adjacent operating pantographs spaced at 656 feet apart, as indicated in Section 21.12.2. The 656 feet spacing shall be used in the OCS dynamic simulations, which shall be considered to be the conformity assessments for verifying compliance with the requirements for dynamic behavior and quality of current collection, as shown in Table 21-3.

It is possible that more than 1 type of pantograph and current collector head may be supplied, particularly if rolling stock is procured from more than 1 supplier. In all cases, the pantograph shall be of proven design for very high speed performance and shall be equipped with a fail-safe device that will detect any failures of the contact strips or collector head, which will trigger automatic lowering of the pantograph. The pantograph shall also be equipped with an uplift limiting device (pantograph stop) and with insulated horns. It is recommended that a carbon-based material be selected for the collector strips to minimize wear of the contact wire. The supplier shall be required to demonstrate the compatibility of the collector strip material with the contact wire.

To achieve good quality current collection, loss of contact between the pantograph strip and the contact wire shall be minimized, since loss of contact can generate electric arcs, which will cause rapid wear of both the contact wire and the pantograph head and collector strips, and may result in the creation of radio frequency interference and in the tripping of feeder circuit breakers (in the event of large arcs with excessive current draws for protracted durations due to significant contact loss). As indicated in EN 50367: 2006 Table 6, for sections dedicated to very high speed, the on-site measured arc percentage (NQ) shall be ≤ 0.2 percent at maximum line speed of 220 mph. For any given vehicle speed, the minimum arc duration that is to be considered shall be 5 milliseconds (ms); the arcing percentage characteristic (NQ), which is also known as the contact loss percentage, is given in percent by the EN 50367: 2006 Clause 3.16 formula:

$$NQ = \frac{\sum t_{arc}}{t_{total}} x100$$

Where:

t_{arc} is the duration of an arc lasting longer 5 ms;

t_{total} is the measuring time with a current greater than 30 percent of the nominal current.

The goal of assessing the interactive dynamic behavior and its impacts on current collection is to ensure there is a continuous and uninterrupted power supply to the electric vehicles with minimal disturbances.

At the design stage, the quality of the OCS-pantograph current collection shall be assessed at the maximum operational speeds for all proposed combinations of rolling stock and pantograph by means of computerized dynamic simulation models. The output from these simulations shall provide determinations of the dynamic effects on the OCS, including values of the simulated contact forces, mean contact force (F_m), standard deviation (σ), statistical value F_m–3σ, contact loss percentage (NQ), and vertical movement of the contact point (contact wire uplift - S_0). The permissible allowances for these factors at maximum line speed are detailed in Table 21-3.

Table 21-3: Requirements for Dynamic Behavior and Current Collection Quality

Operating Requirement	> 125 mph
Allowance for Steady Arm Uplift	2 S_0
Mean Contact Force F_m (N)	See target curve below
Standard Deviation σ_{max} (N)	≤ 0.3 F_m
Percentage of Arcing - NQ (minimum duration of arc 5 ms)	≤ 0.2 percent

21.10.2 Contact Wire Wave Propagation Speed

The speed of wave propagation in contact wires is a characteristic parameter that is used to assess the suitability of an overhead contact line for high-speed operation. This parameter depends upon the specific mass and the tensile stress in the contact wire.

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Based on the recommendations of Clause 4.2.12 of the Energy TSI, the maximum operational line speed shall be not greater than 70 percent of the wave propagation speed. Therefore, the wave propagation speed shall be greater than 314 mph for the 220 mph maximum operating speed.

21.10.3 Static Contact Force

The pantograph static contact force is the mean vertical force exerted upward on the contact wire by the pantograph collector head, and is caused by the pantograph raising mechanism when the pantograph is raised and the vehicle is at a standstill.

The pantograph static force shall be adjustable between 9 and 27 lbf (40 and 120 N). The nominal static force shall be 15.75 (+4.5/-2.25) lbf (70 +20/-10 N), and the OCS shall be designed to suit this permissible range in the static contact force from 13.5 to 20.2 lbf (60 to 90 N). Only pantographs designed and proven for very high speed performance shall be permitted.

21.10.4 Mean Contact Force

The mean contact force is the dynamically corrected statistical mean value of the forces due to static and aerodynamic effects, which depend on the design of the pantograph and the nature of the current collector strips on the pantograph head. It is equal to the sum of the static contact force and the aerodynamic force, which is caused by airflow on the pantograph elements at the considered speed. The mean uplift force is a characteristic of the given rolling stock/pantograph combination. In this context, F_m represents a target value that should be achieved to ensure current collection without undue arcing, but which should not be exceeded to limit wear and hazards to the contact wire and the current collection strips.

The design of the overhead contact line equipment must allow for the maximum and minimum contact forces that occur between the pantograph and the contact wire, at the maximum permissible speed of the vehicle, while taking into account the aerodynamic effects. The minimum contact force shall always be positive to ensure no loss of contact between the pantograph and the overhead contact line. Force values vary with different combinations of rolling stock/pantograph and OCS.

The overhead contact line shall be designed to be capable of sustaining this level of force for all pantographs on a train.

In the case of trains with multiple pantographs simultaneously in operation, the mean contact force F_m for any pantograph shall be not higher than the target value, since the current collection criteria shall be met by each individual pantograph. The target value for the mean contact force F_m as a function of the running speed for ac systems is depicted in Figure 21-2 and Figure 21-3.

Figure 21-2: Target Fm Values (lbf-mph)

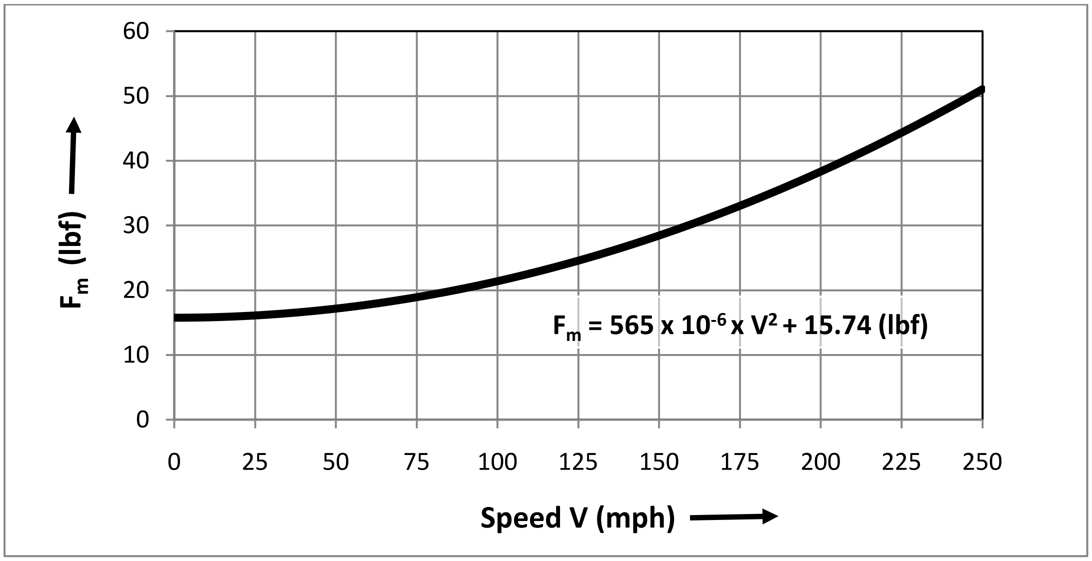
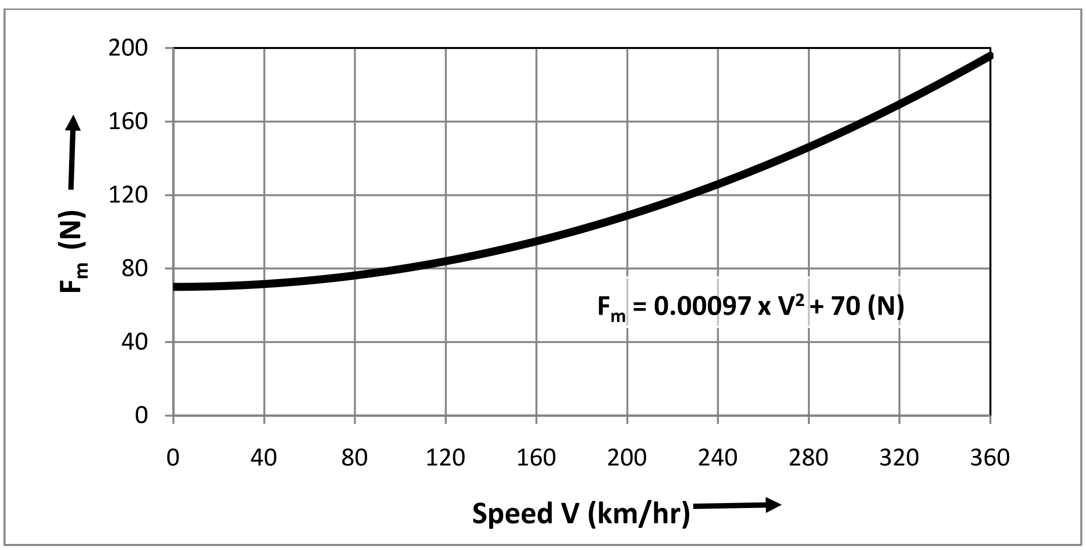


Figure 21-3: Target Fm Values (N-km/hr)



The maximum contact force (F_{max}) is normally within the range of $F_m+3\sigma$ (3 standard deviations) for level grade open route sections. Higher values may occur elsewhere, but shall not exceed 79 pounds (350 N) at speeds greater than 125 mph per EN 50119: 2001 Table 1.

The minimum permissible contact force is the force value at which loss of contact between the pantograph and contact wire is probable, and is represented by the statistical value $F_m–3\sigma$, which is a measure that permits the assessment of the consistency of contact between the pantograph and the OCS. The value $F_m–3\sigma$ must be positive to avoid contact loss.

21.10.5 Contact Wire Uplift or Vertical Movement of the Contact Point

The contact point is the point of mechanical contact between the pantograph contact strip and the contact wire.

The vertical height of the contact point above the track shall be as uniform as possible along the span length; this is essential for high-quality current collection. The maximum difference between the highest and the lowest dynamic contact point height within 1 span shall be less than 3.15 inches (80 mm) at the maximum operating speed of 220 mph. This value has been derived from the Energy TSI Clause 4.2.17 and Table 4.2.17 for Category I lines (as defined in the Energy TSI Clause 1.1).

During the design phase, the projected contact wire uplift (variation in dynamic contact point height) shall be verified by simulations in accordance with EN 50318: 2002. Uplift values shall be presented as a graph of the contact point vertical position against distance in the span to evaluate the extent of the vertical movement:

- For the maximum line speed of the overhead contact line
- For the longest span length
- Using the mean contact force F_m (as detailed above)

After installation, uplift shall be validated by measurements in accordance with EN 50317:2002.

The variation in contact point height need not be verified for uninsulated or insulated overlap spans or for spans above track turnouts or crossovers.

In order to maximize safety under all operating conditions (including strong wind conditions and slight mis-adjustments of the pantographs), the dynamic pantograph envelope at the maximum operating speed shall consider twice the value of the estimated or simulated uplift S_0 at the support point. The design of the OCS cantilever and registration shall allow the uplifted steady arm to clear the dynamic pantograph envelope. For initial design purposes, a minimum uplift of 10 inches shall be assumed.

Uplift values shall be confirmed by simulation results (as indicated below) and, if multiple pantograph-train consists are furnished, the designs shall accommodate, as a minimum, the greatest simulated uplift values.

21.10.6 Conformity Assessment

Conformance with the above criteria shall be confirmed by the OCS supplier by means of dynamic interactive OCS-pantograph simulations and through equivalent records of on-site testing results for speeds above 220 mph. Notwithstanding the above, the OCS shall be a proven system capable of sustaining satisfactory current collection for train operations at 220 mph. The simulation program shall meet the validation requirements detailed in EN 50317: 2002 and EN 50318: 2002.

The final designs and specifications shall require that measurements (in accordance with EN 50317: 2002) of the interaction between the pantograph and the OCS shall be performed on the high-speed line during the testing and commissioning phase to check for correct installation and to prove the safety and the quality of the current collection system. These measurements shall be carried out with an approved pantograph, exhibiting the mean contact force characteristics for the envisaged design speed, installed on approved rolling stock. The installed overhead contact line shall be accepted if the measurement results comply with the requirements stipulated in Table 21-3.

To check the performance capability of the current collection system, the following data, as a minimum, shall be measured:

- The contact force
- The contact wire uplift at the support as the pantograph passes
- The percentage of arcing and duration of arcs longer than 5 ms

In addition to the measured values, the operating conditions (train speed, location, etc.) shall be recorded continuously, and the environmental conditions (rain, temperature, wind, tunnel, etc.) and details of the test configuration (parameters and arrangement of pantographs, type of OCS, etc.) during the measurement tests shall be recorded in the test report.

When or if changes from the accepted and approved equipment are proposed, such as the use of a pantograph of proven design that is to be installed on a new type of rolling stock, or a new OCS design for additions to, or substitution of, existing sections of the system, or a new pantograph design that is to be installed on the approved rolling stock, conformity assessment testing shall be carried out in accordance with EN 50317: 2002 and/or EN 50206-1: 1999, with particular emphasis on the mean contact force and loss of contact requirements. If the tests are passed successfully, the new OCS design, or the specific proposed pantograph/rolling stock combination, will be approved for use on the high-speed line.

21.11 Current Capacity of Overhead Contact System

As a minimum, the current-carrying capacity of the OCS shall comply with the current-draw requirements specified for the trains.

The OCS, including parallel feeders, return circuit conductors and feeder connections, shall be designed to cater to the electrical current loading under steady state peak period operating and fault conditions, as defined by the system design, under the environmental and climatic conditions defined in Section 21.5, with reference to the advisories contained in Annex A to EN 50119: 2001. In addition, the current heating effects of short circuit faults and durations resulting from automated circuit breaker closure sequences (if adopted) occurring during peak operations shall be assessed.

The maximum temperature rise in the conductors caused by the load currents shall not lead to conductor temperatures to the point at which the mechanical properties are impaired. The maximum permissible temperatures for bare conductors are shown in Table 21-4.

Table 21-4: Maximum Permissible Bare Conductor Temperatures

Conductor Material	Max. Temperature (˚F)	Max. Temperature (˚C)
Normal and high strength, high conductivity Copper	176 ˚F	80 ˚C
Silver Copper alloys	212 ˚F	100 ˚C
Cadmium Copper alloys	176 ˚F	80 ˚C
ACSR	212 ˚F	100 ˚C

The melting point of any grease used in the strands of the conductors shall be higher than the temperature limits specified above.

The Designer shall undertake a design study to confirm the OCS complies with the specified requirements. Conformity assessment shall be carried out by design review.

21.12 Sectionalizing and Switching, and Pole-Mounted Equipment

To allow bi-directional working, enabling trains to continue operation under emergency conditions and to facilitate routine OCS maintenance, the OCS shall be divided into electrical sections and sub-sections. On the main tracks, only phase breaks (utilizing insulated overlaps) and insulated overlaps shall be used for power supply sectionalizing purposes. Mechanical section insulators will not be permitted except when used in the OCS above slow speed track turnouts and in the yard and shop areas.

To form the insulated overlaps, insulation shall be cut into the out-of-running sections of the messenger wire and contact wires of the 2 overlapping catenaries, having between them a limited air gap electrical clearance. The insulated overlap thus provides a sectionalizing point in the OCS as required for operational and maintenance reasons, but allows pantographs to transition smoothly from 1 energized electrical sub-section to the next under power.

21.12.1 Pantograph Spacing for Design of the Overhead Contact Line

The overhead contact line design shall be based on rolling stock operating with 2 raised pantographs spaced at 656 feet apart for dynamic simulation assessments

21.12.2 Phase Breaks

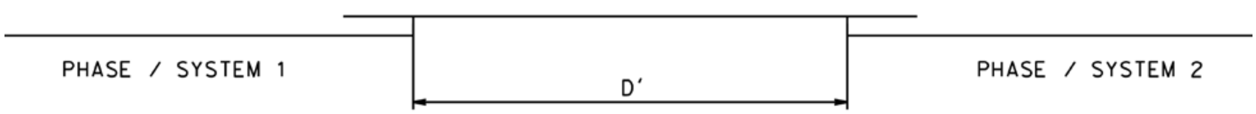
The design of the OCS phase breaks shall permit approved trains to move at all speeds up to the designated maximum operating speeds from 1 electrical section to an adjacent electrical section without bridging between 2 electrical phases or 2 separate utility supply systems.

Trains shall traverse the entire phase break with pantographs raised and in contact with overhead contact wire, but with the pantograph breakers open. The main circuit breaker onboard each power unit shall be opened automatically by an input from the ATC system before the train enters a phase break, and shall be similarly closed automatically after the train clears the phase break section in the OCS. Adequate means shall be provided to allow a train that is stopped within the phase break neutral section to be restarted and bidirectional movements shall be supported. The neutral section shall be configured such that it can be connected to, and energized from, either of the adjacent electrical sections by remotely controlled disconnect switches with the provision of appropriate interlocks to ensure the different phases cannot be inter-connected under any circumstances.

The geometry of the phase break elements shall prevent pantographs short-circuiting or bridging between power systems. Provision shall be made in the rolling stock design to avoid bridging of adjacent power supply systems should opening of the onboard circuit breaker(s) fail. Train consists with multiple pantographs shall have no electrical connection between the pantographs that are in service.

For high speed sections over 65 mph, the Long Phase Break (Figure 21-4) has been selected. The long phase break design will allow all pantographs of the longest compliant trains to lie within the neutral section. The length of the neutral section shall be at least 1,320 feet.

Figure 21-4: The Long Phase Break



Conditions: D' > 1,320 feet

21.12.3 OCS Sectionalizing in Tunnels

The Designer shall coordinate the sectioning of the power supply system in each tunnel with the pertinent agency that will be responsible for development of the Tunnel Emergency Evacuation Plan. The sectioning shall be designed to support the overall strategy for evacuation from the tunnel.

For tunnels that are greater than 3 miles in length, in which the signaling system permits the simultaneous presence of more than 1 train on each track, the OCS shall be divided into sections that do not exceed 16,500 feet (as indicated in Section 4.2.3.1 of the Safety in Railway Tunnels TSI).

Grounding devices shall be provided at tunnel portals and at tunnel access points, and close to the separation points between electrical sections. These shall be in the form of 3-position disconnect switches, providing for Closed (inter-connection between adjacent electrical sections), Open (no electrical inter-connection), and Closed to Ground.

The disconnect switches shall be motorized and shall be both remotely and locally controlled fixed installations. Switch indication panels shall be provided at or adjacent to each switching location, indicating for the benefit of emergency response personnel the status of each switch and whether the OCS is energized / de-energized / grounded.

Procedures and responsibilities for grounding the OCS in tunnels by the Power Director or Power Dispatcher shall be defined in the emergency plan.

21.12.4 Disconnect Switches

To facilitate maintenance work and emergency operations, the OCS shall be equipped with disconnect switches at all primary feeding and by-pass feeding locations.

Where feasible, the OCS disconnect switches shall be pole-mounted at trackside and shall be single pole motorized switches capable of remote operation and also of local motorized or manual operation. The switches shall provide for isolation of discrete sections of the OCS (track), such that segments of the OCS can be de-energized for maintenance purposes. The disconnect switches shall also provide for by-pass feeding arrangements that can be implemented during emergency conditions to permit contingency modes of operation. Remote operation shall be performed from the Operations Control Center (OCC) and shall be accomplished using the SCADA system. OCS disconnect switches shall be of the no-load-break type and shall be rated for the system voltage and anticipated current loads, and shall be designed to carry the worst-case overload and short circuit currents without overheating. As a safety precaution, the switch operating mechanism shall be fitted with a locking bar that will permit the attachment of maintainer locks.

In general, the disconnect switches shall be of the 2-position type, providing for Closed (inter-connection between adjacent electrical sections), or Open (no electrical inter-connection). For locations where solid grounding of the OCS is required, the OCS disconnect switches shall be of the 3-position type, providing for Closed, Open, and Closed to Ground connections.

Where motorized disconnect switches are located in the vicinity of a traction power facility, the 125 V dc motor power shall be supplied from that facility. At remote locations, such as interlockings, the 125 V dc motor power shall be supplied from a wayside power control cubicle (WPC).

21.12.5 Auxiliary Step-down Transformers

To provide power to remote wayside power control cubicles (WPC), the traction power negative feeder can be tapped. At selected locations, an auxiliary step-down transformer will be mounted on the back (field-side) of an OCS pole on both sides of the right-of-way to step down the 25 V negative feeder power to 480 V ac to provide redundant feeds to the WPC. The feeder tap shall be equipped with a lightning arrester, and a drop-out fuse assembly to facilitate maintenance work on the transformer.

21.12.6 Protection against Lightning Strikes

The OCS Designer shall investigate the incidence of lightning storms on a project section-by-section basis and shall determine appropriate lightning protection measures, based upon the incidence of lightning strikes in each area. Refer to the *Grounding and Bonding Requirements* chapter. Regardless, all cabled connections to the OCS and Negative Feeders shall be equipped with lightning arresters, preferably installed at disconnect switch locations. In addition, between TP facilities, the OCS and Negative Feeders shall be equipped with lightning arresters at a spacing not to exceed 2 miles.

21.13 Insulation Coordination Requirements for OCS Installations

Insulation requirements for railroad electrification systems are covered by EN 50124-1: 2001 Railway Applications – Insulation Coordination – Part 1: Basic Requirements. Insulation coordination implies selection of the electrical insulation characteristic of the equipment with regard to its application and in relation to its surroundings. Insulation coordination can only be achieved if the design of the equipment is based on the stresses to which it is likely to be subjected during its anticipated lifetime. In accordance with EN 50124-1: 2001 Clause 2.2.2.1, the OCS falls into either the OV3 or OV4 overvoltage categories for circuits that are powered by or from the overhead contact line, which are not protected against external or internal overvoltages, and which may be endangered by lightning or switching overvoltages. For a 25 kV OCS, the rated insulation voltage is given as 27.5 kV in Table D.1, and for fixed installations, the rated impulse voltage is given as either 170 kV or 200 kV for the OV3 and OV4 overvoltage categories respectively. The Designer shall determine the category applicable to the OCS and shall furnish justification for the selection. Pollution categories and associated creepage distances are also covered in EN 50124-1; insulation for the system shall be designed accordingly.

21.14 OCS Clearances and Protection against Electric Shock

Protection against electric shock can be achieved by establishing adequate safety clearances that minimize the possibility of direct contact by persons with energized parts, and/or by erecting suitable barriers or screens to prevent direct contact, and installing appropriate signs warning of the potential dangers, and by implementing comprehensive project-wide bonding and grounding systems. Refer to the *Grounding and Bonding Requirements* chapter of these criteria.

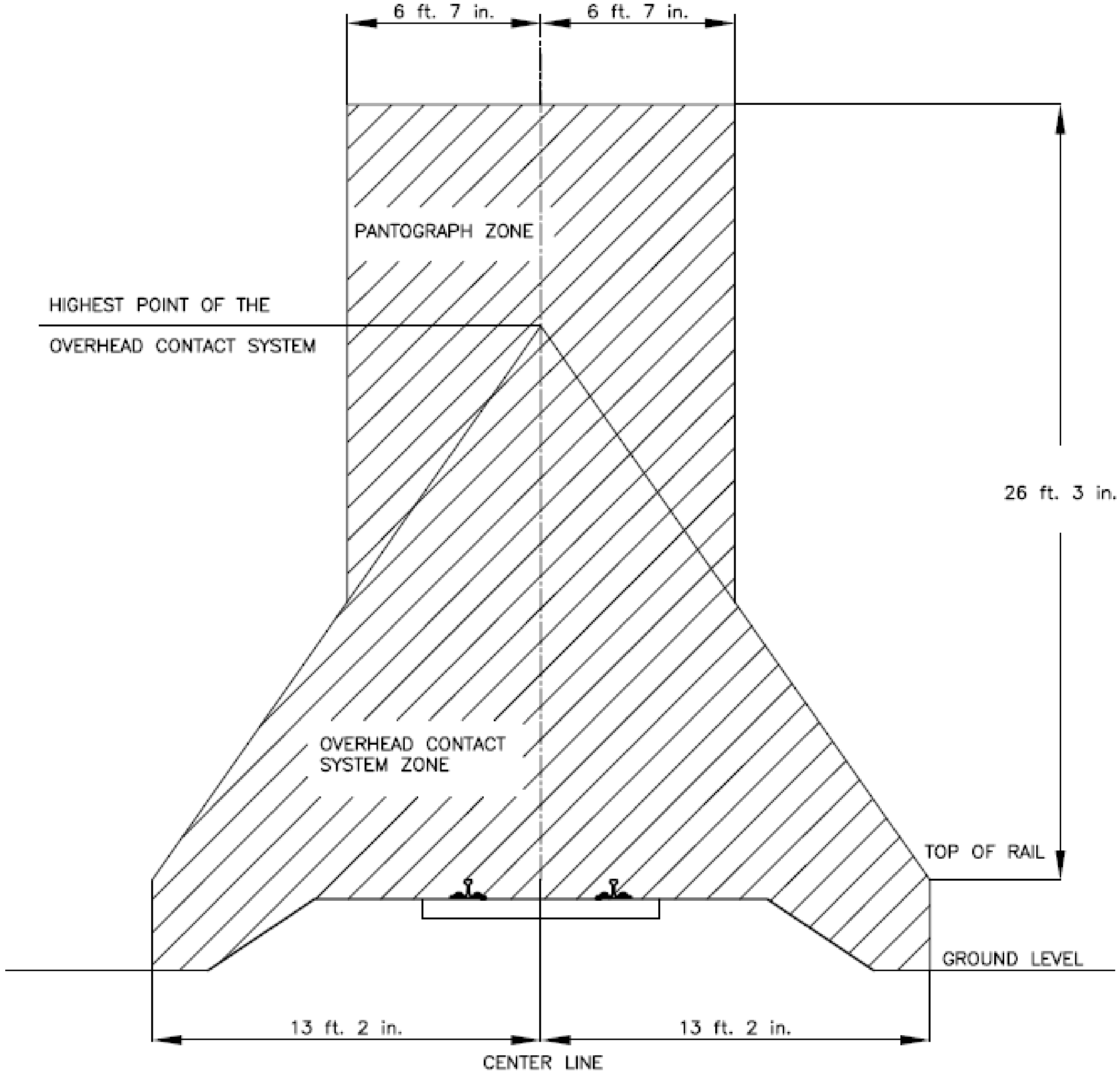
21.14.1 Overhead Contact Line Zone and Pantograph Zone

Structures and equipment may accidentally come into contact with a live broken contact line, or with the live parts of a broken or de-wired pantograph or energized fragments. Figure 21-5 has been derived from EN 50122-1: 1997 Figure 1 and defines the zone inside which such contact is considered probable but which limits are unlikely to be exceeded by a broken overhead contact line or damaged energized pantograph, or energized fragments thereof.

Note that the damaged pantograph may be live, even though it is not in contact with the overhead line, because it is inter-connected with other energized pantographs or because the train is in regenerative braking mode.

The limits of the overhead contact line zone below top of rail extend vertically down to the earth surface, except where the tracks are located on an aerial structure where they extend down to the deck of the aerial structure. In the case of out of running OCS conductors, the overhead contact line zone shall be extended accordingly.

Figure 21-5: Overhead Contact Line Zone and Pantograph Zone



Source: Derived from EN 50122-1: 1997 Figure 1

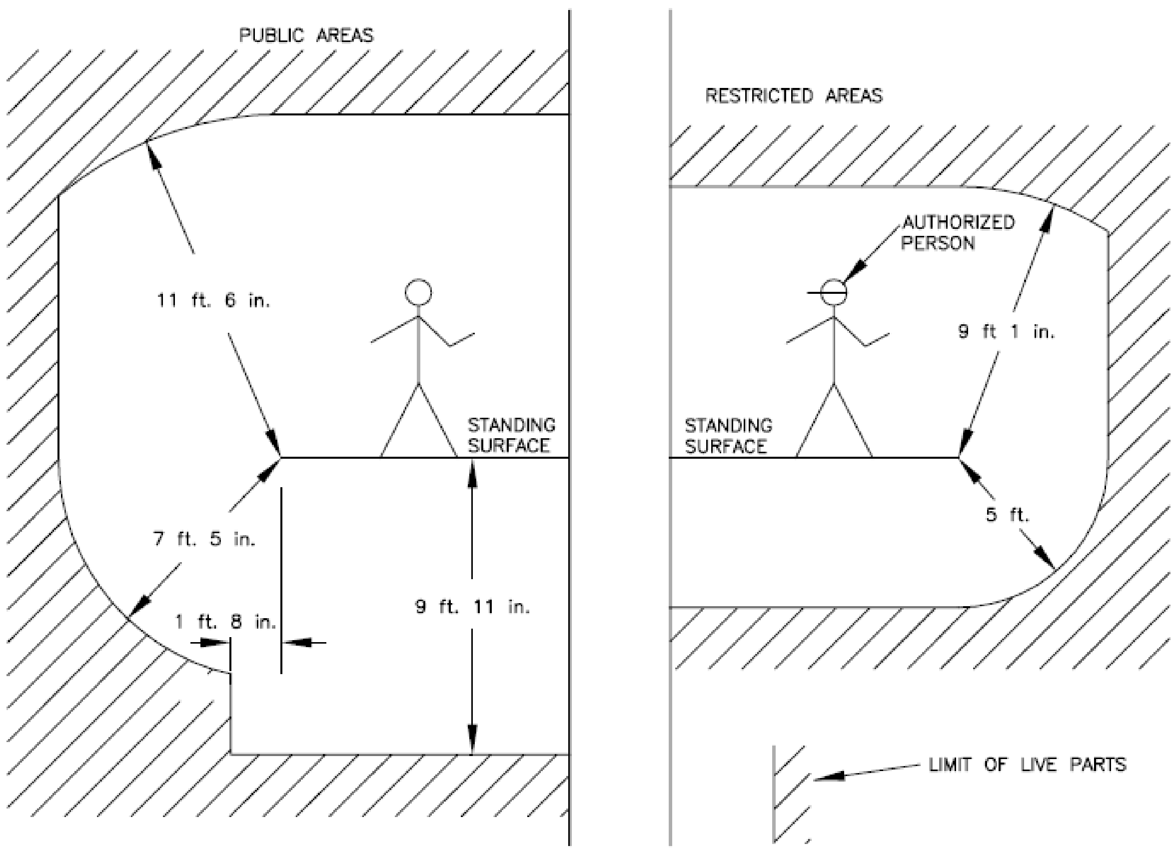
21.14.2 Protection by Clearances from Standing Surfaces

The minimum unconstrained clearances from energized parts to generally accessible areas (no barriers, screens or other physical restrictions to movement) for 25 kV systems have been derived from EN 50122-1: 1998 Figure 14 and values are depicted in Figure 21-6 for both public areas and restricted areas. The values shown are based on touching in a straight line without the use of tools or other objects, and shall be achieved under all climatic and loading conditions. These requirements apply to clearances from standing surfaces used by people to accessible live parts on the outside of vehicles as well as to live parts of the OCS.

Placing energized parts over walkways shall be avoided wherever practical.

Safe working clearances and approach distances for qualified employees shall be developed by the applicable agency for inclusion in the Safety Manual and in the appropriate work practices and procedures documents.

Figure 21-6: Minimum Required Safety Clearances – Unconstrained Access



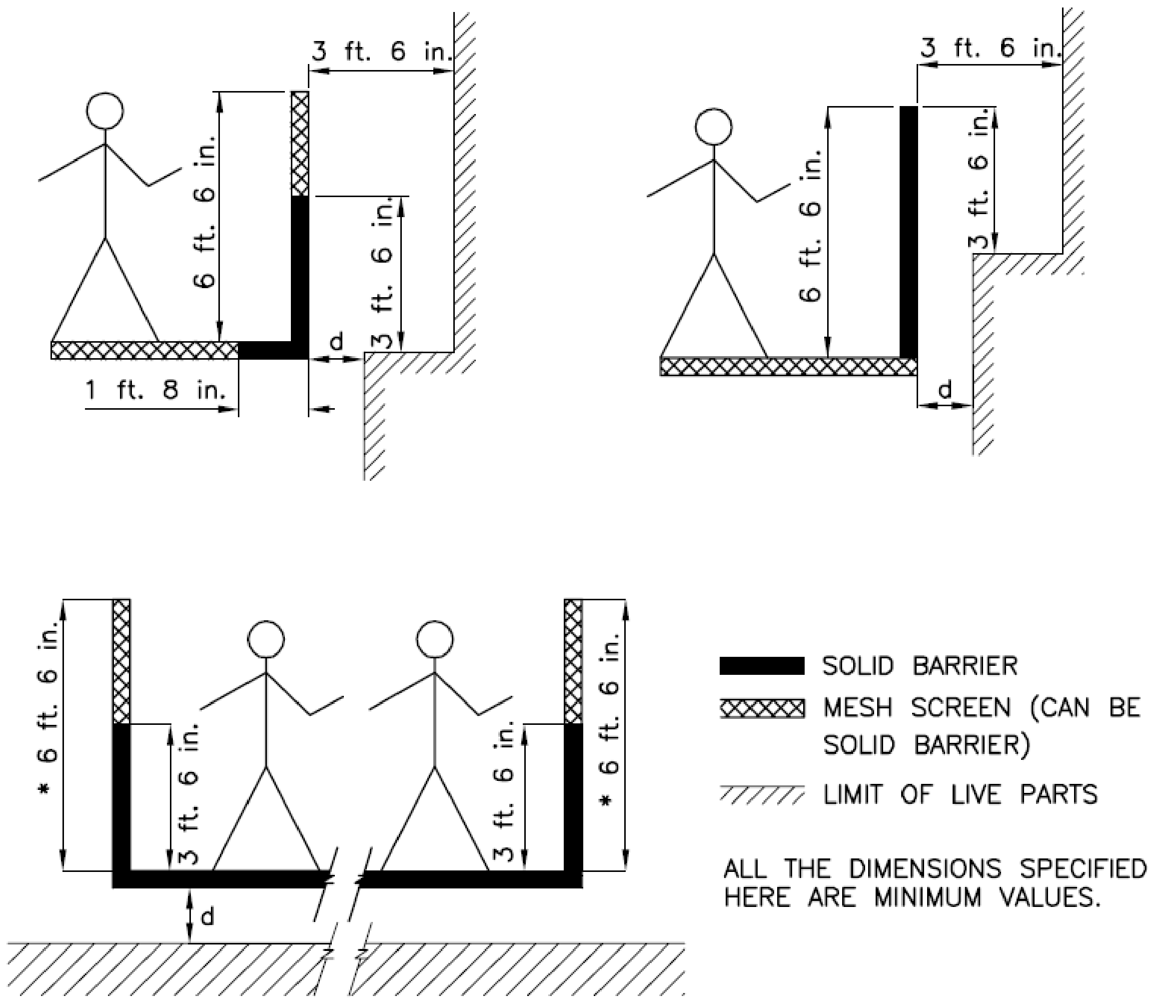
21.14.3 Protective Screening and Barriers for Standing Surfaces in Public Areas

The following criteria address OCS requirements only. For CHSTP fencing requirements, refer to the *Civil* chapter. The requirements for protective screening and barriers for use in public areas for protection against direct contact with adjacent live parts on the outside of vehicles or adjacent live parts of an overhead contact line system for normal voltages up to 25 kV ac to ground, where clearances are less than those shown in Figure 21-6, have been derived from EN 50122-1: 1997 Figure 18 (herein depicted on Figure 21-7) and are summarized as follows:

- Where the energized parts are located below the standing surface, protection of the standing surface shall be by means of a solid barrier.

- The minimum height of the protective barrier (solid barrier or a combination of solid barrier plus mesh screen, as shown) shall be not less than 6 feet-6 inches.
- Protective barriers of greater height may be required in areas where vandalism is prevalent.
- The value of Dimension “d”’ between the protective screen or barrier and live parts shall be determined from Table 21-5. Where mesh screens are used, 4 inches shall be added to the value of Dimension “d” and where buckling or warping of solid barriers is likely, 1.25 inches shall be added, in accordance with EN 50122-1 Clause 5.1.3.1.2.
- The length of the protective screening and/or barrier on structures that cross over the electrified railroad, which protect publicly accessible standing surfaces, shall be extended laterally 30 feet from the centerline of the outermost track (refer to Civil Standard and Directive Drawings) or a minimum of 10 feet beyond the outermost live parts of the overhead contact line (conductor or component). In the case of energized conductors not being used for current collection (e.g., line feeders, reinforcing feeders, out of running overhead contact lines), the barrier shall extend for a width of at least 10 feet on each side of the conductor, with the proviso that movements due to dynamic and thermal effects shall be taken into account.

Figure 21-7: Clearances from Protective Screens and Barriers for Standing Surfaces in Public Areas



Note: * for the height of Access Restriction (AR) fencing along public access areas, refer to the *Civil* chapter.

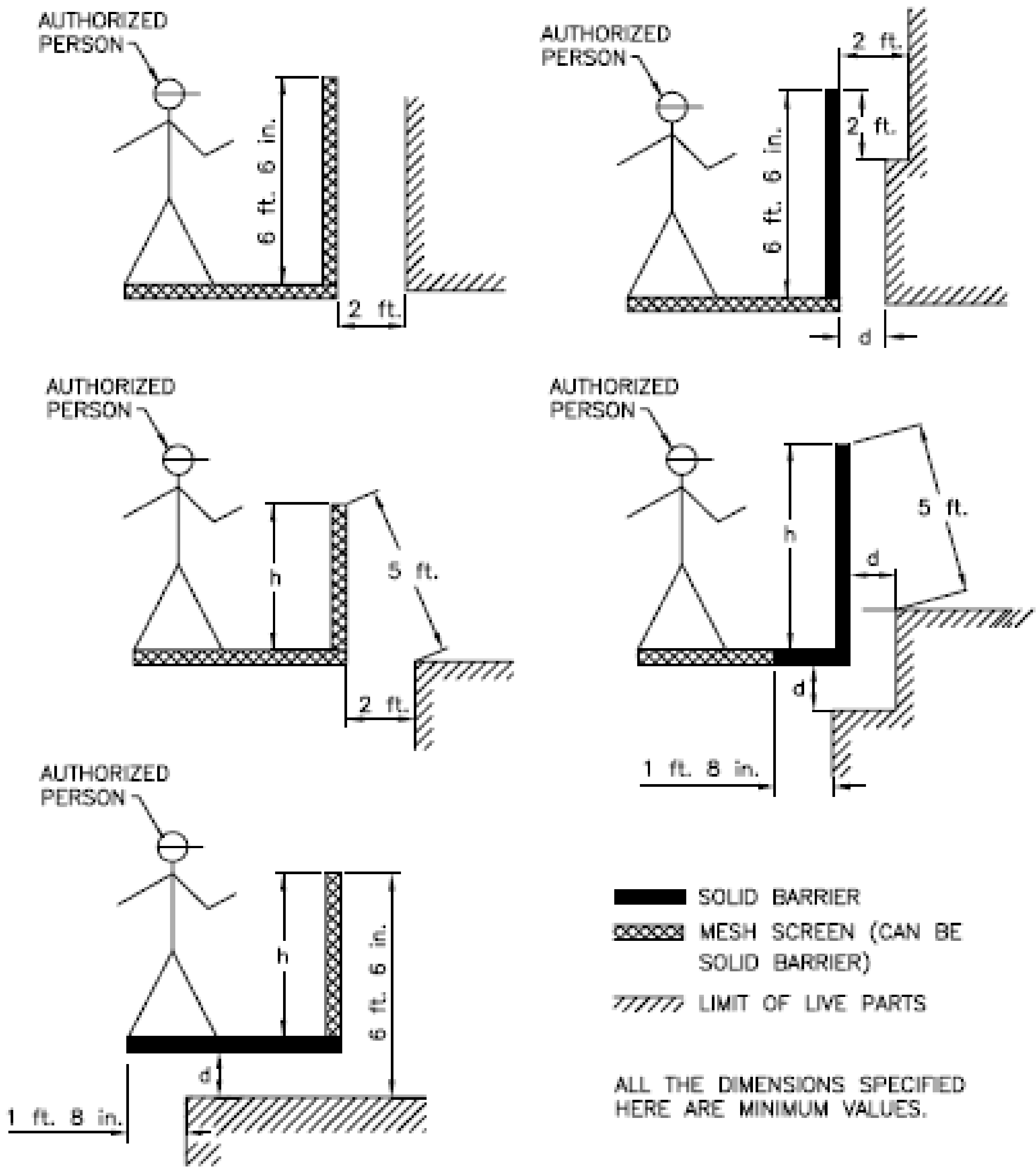
Source: Derived from EN 50122-1: 1997 Figure 18

21.14.4 Protective Screening and Barriers for Standing Surfaces in Restricted Areas

The requirements for clearances from protective screening and barriers for standing surfaces in restricted areas for protection against direct contact with adjacent live parts on the outside of vehicles or adjacent live parts of an overhead contact line system for normal voltages up to 25 kV ac to ground, where clearances are less than those shown in Figure 21-6, have been derived from EN 50122-1: 1997 Figure 16 and 17, and are shown in Figure 21-8 and are summarized as follows:

- For standing surfaces above live parts on the outside of vehicles or above live parts of an overhead contact line system, the protection shall be of solid barrier construction.
- The length of the solid barrier, protecting the standing surface, shall correspond to the pantograph zone and shall extend beyond the live parts of an overhead contact line by at least 1 foot 8 inches. In the case of energized conductors not being used for current collection (e.g., line feeders, reinforcing feeders, out of running overhead contact lines), the barrier shall extend for a width of at least 1 foot 8 inches on each side of the conductor, with the proviso that movements due to dynamic and thermal effects shall be taken into account.
- The height “h” of the protective screening and barrier shall be such that a clearance of 5 feet from the top of the protective screening and barrier shall be maintained (refer to Figure 21-8).
- The height of the side protective screenings and barriers shall correspond to the height of the required safety railing but should be a minimum of 3 feet 6 inches.
- The value of Dimension “d”’ between the protective screen or barrier and live parts shall be determined from Table 21-5. Where mesh screens are used, 4 inches shall be added to the value of Dimension “d” and where buckling or warping of solid barriers is likely, 1.25 inches (30 mm) shall be added, in accordance with EN 50122-1: 1998 Clause 5.1.3.1.2.

Figure 21-8: Clearances from Protective Screenings and Barriers for Standing Surfaces in Restricted Areas



Source: EN 50122-1 Figure 16 and 17

21.14.5 Additional Requirements for Protection Barriers and Screens

Protection barriers or screens shall be of sufficient strength and shall be supported rigidly and securely enough to prevent them from being displaced or dangerously deflected by a person slipping or falling against them.

Barriers and screens shall be permanently fixed, and shall be removable only with tools. Barriers in public areas shall employ non-removable, captive fasteners.

Barriers shall be of solid construction and fabricated from either conductive or non-conductive materials.

- Non-conductive barriers shall be surrounded by a grounded, bare conductor that is inter-connected with the traction system ground, preferably at not less than 2 locations.
- Conductive barriers shall be bonded and grounded by inter-connection with the traction system ground, preferably at not less than 2 locations.

Screens shall be of grounded, conductive, open mesh materials, and shall be grounded by inter-connection with the traction system ground, preferably at not less than 2 locations. Non-conductive mesh or plastic-coated metal mesh shall NOT be used.

Conductive mesh screens shall be constructed such that a cylinder, greater than 1/2 inch in diameter, cannot be pushed through the mesh. Mesh screen construction shall be such that required clearances to energized parts are maintained.

The style of barrier to be employed is dependent upon the type of standing surface and its proximity to the energized parts, and whether the surface provides for public or restricted access, as detailed above.

The size of the barrier or screen shall be such that energized parts cannot be touched in a straight line by persons on a standing surface.

The design of the protective screens and barriers shall minimize the loading on the existing structures and the adverse visual impact.

The metallic parts of overhead bridge screens and barriers shall be bonded to the static wires. Other metallic items under overhead bridges, within a lateral distance of 10 feet of any energized and uninsulated equipment below the structure, shall be directly or indirectly bonded to the static wires.

21.14.6 Protection Against Climbing

Where there is public access or trespass is likely, anti-climbing protection shall be provided at buildings and other structures supporting energized parts of the OCS. The anti-climbing protection shall include signs warning of the dangers of high voltage.

Access to fixed ladders, particularly at signal poles and signal gantries, and the means of access to any roof or other place, which could allow non-authorized persons to approach energized parts, shall be secured or otherwise protected.

21.14.7 Warning Signs

Permanent High Voltage Warning signs, as detailed in the *Civil* chapter, shall be installed as follows:

- Displayed in conspicuous places at all entrances to locations containing exposed current carrying parts
- Located on all enclosures that provide access to conductors, equipment, and apparatus that are energized at high voltage
- Displayed at all anti-climbing locations

The warning signs shall be posted in a consistent manner throughout the electrified route and shall be clearly visible to persons on or near the electrified lines.

21.14.8 Clearances for Utility Lines Crossing over the Electrified Railroad

The minimum clearance for overhead power, communications or other utility lines, which are not part of the Traction Electrification System (TES), shall be in accordance with CPUC General Order No. 95 Rule 38 Table 2 and shall be measured from the highest energized point on the TES.

For any crossing of the high-speed lines, the utility shall comply with the requirements of CPUC General Order No. 95 with regard to the conductor suspension arrangements and strength of the structures immediately adjacent to the crossing point.

For other utility requirements, refer to the *Utilities* chapter.

21.14.9 Electrical Clearances to Rail Vehicles and Structures

Clearances are classified as either Static or Passing.

Static Clearance is the physical air clearance between energized parts of a vehicle or OCS when not subjected to dynamic conditions or climatic influences or pantograph pressure, and an adjacent fixed structure or the grounded parts of a vehicle, while the vehicle is stationary.

Passing (or Dynamic) Clearance occurs under dynamic operating conditions that exist during the passage of a train, or when the OCS is affected by extreme climatic conditions, such as wind and/or ice loading. Passing (or Dynamic) Clearance is the physical air clearance between energized parts of either the vehicle or OCS and the grounded vehicle, or between energized parts of either the vehicle or OCS and an adjacent fixed structure.

Electrical clearances, shown in Table 21-5 and depicted in Figure 21-9, from energized parts to grounded parts of rail vehicles or structures are categorized as Normal and Minimum and are applicable, as noted, in Non-Polluted and Polluted atmospheric locations. Typical polluted conditions are detailed in Section 21.5.4 and the Designer shall determine their applicability. Polluted locations/areas shall be so noted in the designs, so all users are aware that increased clearances must be employed and maintained.

Table 21-5: 25 kV ac Electrical Clearances

Atmospheric Condition	Normal Clearance		Minimum Clearance	
	Static (C_A)	Passing/ Dynamic (P_A)	Static (C_A)	Passing/ Dynamic (P_A)
Non-Polluted	10.5 inches*	8 inches *	8 inches *	6 inches*
Polluted	12.5 inches **	10 inches**	10 inches **	8 inches **

* These clearance values are as stated in AREMA Table 33-2-4 (2010)

** For polluted atmospheres, 2 inches has been added as stated in AREMA Table 33-2-4 (2010)

The designated normal clearances shall be adopted at all locations, wherever practicable. Where it can be demonstrated that it is not practicable to provide normal clearances, adoption of the minimum clearances shall be permissible. However, prior to their adoption, the following factors shall be further evaluated:

- Fault current resulting from a breakdown of the electrical clearance
- Vulnerability of the OCS and railroad infrastructure to damage should a breakdown of the electrical clearance occur
- Consequences for the safety of persons should a breakdown of the electrical clearance occur
- Application and maintenance of tolerances of the OCS and railroad infrastructure
- Economic and technical considerations

The minimum clearance from bare energized ancillary conductors (the 25 kV negative feeders) to grounded structures under worst case conditions in non-polluted areas shall be 10.5 inches and 12.5 inches in polluted locations, as specified in the AREMA Manual Chapter 33 Table 33-2-2. These values shall be adopted for the project.

In a 2x25 kV ac system, there is a 180 degree phase difference between parts common to the energized negative feeder and parts common to the energized catenary system. The minimum clearance between these elements shall be 21.5 inches under static conditions or 12 inches under worst case dynamic conditions.

Enhanced clearances or other protective measures shall be provided at locations where there is a high probability of incidents due to birds, animals, icicles, or vandalism, or for particularly vulnerable structures. The maximum practicable value of electrical clearance shall be provided at all locations.

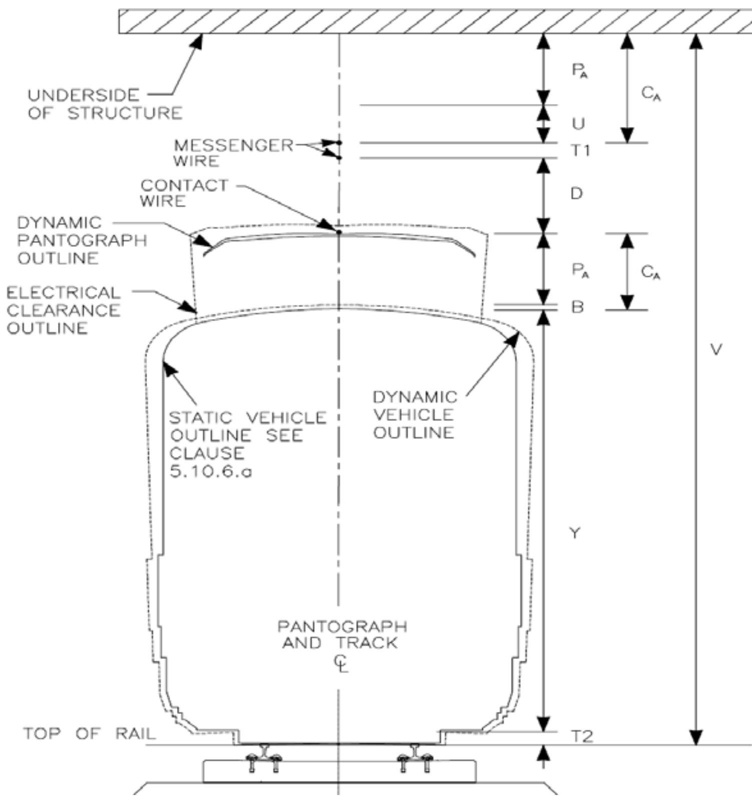
21.14.10 Clearance Envelope at Fixed Structures

In determining the minimum vertical clearance envelope at fixed structures, including OCS support structures and signal bridges, the following factors shall be assessed, as depicted on Figure 21-9:

- The static vehicle outline, which shall be based on the size of the high-speed rail vehicles.
- The dynamic vehicle outline, which shall take into consideration the dynamic swept envelope, track position and maintenance tolerances, including railhead side wear, and the effects of vertical and horizontal curvature, including track super-elevation. Refer to the *Trackway Clearances* chapter for further information regarding vehicle clearance requirements.
- The position of energized parts on the rail vehicles, including the dynamic pantograph envelope, allowing for pantograph carbon wear and dynamic movements and deflections of the pantograph frame, and vehicle construction and maintenance tolerances. The pantograph envelope shall include an allowance for chording effects, if the pantograph is offset longitudinally on the vehicle from a truck centerline.
- The position and size of energized parts of the OCS allowing for installation and maintenance tolerances, uplift and other dynamic movements, including those due to wind, temperature and loading conditions.
- Electrical clearance values as applicable for non-polluted or polluted areas.
- For minimum vertical clearances for new and existing structures, refer to the *Trackway Clearances* chapter and the Standard and Directive Drawings.
- In assessing the minimum vertical clearance of the overhead structure, the vertical clearance between the energized bare negative feeder cable shall also be considered.
- At locations where the bare negative feeder has to be mounted on the back or field side of the OCS poles, lateral clearances to adjacent structures shall take into consideration the position of this pole-mounted equipment and the conductor under worst case wind conditions.
- At phase break locations, positive feeders will be run from the phase break switching gantry to the OCS feeder disconnect locations outside the limits of the phase break. These bare positive feeders will be mounted on the back or field side of the OCS poles, and lateral clearances to adjacent structures shall take into consideration the position of this pole-mounted equipment and the conductor(s) under worst case wind conditions.
- To provide for safe clearances from OCS pole-mounted equipment, no structure that is more than 10 feet high above top of rail shall be constructed within 9 feet from the field-side of the

centerline of the OCS pole. This requirement shall also apply where OCS poles are mounted on top of retaining walls or trench walls. Where OCS poles are mounted inside trench structures or retaining walls, this spatial requirement shall be evaluated on a site specific basis.

Figure 21-9: Vertical Clearance Envelope at Fixed Structures



Source: AREMA Figure 33-2-3 and Figure 33-2-4

V	=	Total Vertical Clearance Required for Electrification
P_A	=	Passing (Dynamic) Electrical Clearance – see Note below
U	=	OCS Uplift
T1	=	OCS Construction Tolerance
D	=	OCS Depth
B	=	Vehicle Bounce
Y	=	Static Vehicle Load Height
T2	=	Track Maintenance Tolerance
C_A	=	Static Electrical Clearance

Notes: The diagram depicts the dynamic condition. For static situations, the Static Electrical Clearance (CA) shall be not less than PA+U or PA+B - refer to Table 21-5.

The minimum lateral clearance at fixed structures, including OCS poles and other OCS support structures and signal bridges, shall comply with the clearance requirements detailed in the *Trackway Clearances* chapter.

21.14.11 Applicable Pantograph and OCS Clearance Envelopes

In assessing clearances along the alignment to accommodate the OCS and Pantograph, 2 Clearance Envelopes have been developed, which incorporate electrical clearances based on the polluted area clearances shown in Table 21-5. These envelopes are detailed in the OCS Directive Drawings and shall be adopted.

For non-polluted areas, the diagrams can be reduced in size by incorporating the non-polluted electrical clearance values indicated in Table 21-5.

21.14.12 Paved Areas in Maintenance Facilities, Yards and Workshops

The normal clearance of the lowest energized part of the electrification system above paved areas in maintenance facilities, yards and workshops, is specified in Table 21-6. Warning signs shall be provided, as detailed in the *Civil* chapter.

At pedestrian and vehicle crossings in maintenance facilities or yards where vehicles over 8 feet in height are prohibited, a restricted clearance is permitted, as specified in Table 21-6.

Table 21-6: Minimum Clearances above Track Crossings in Paved Areas of Maintenance Facilities, Yards and Workshops

Normal Clearance	Restricted Clearance
20 feet 4 inches	18 feet 4 inches

21.14.13 Clearances to Vegetation

Based on the requirements stipulated in EN 50122-1: 1997 Clause 5.1.2.5, trackside vegetation shall be managed, such that there is no overhanging vegetation and that a minimum clearance of 8 feet-3 inches is maintained between the vegetation and energized parts of the OCS and/or negative feeders at all times and under all climatic conditions.

21.15 OCS Structural Requirements

21.15.1 General

In the above ground, at-grade sections, the OCS poles shall be either galvanized H-section wide flange beams or galvanized round, tapered tubular steel sections. In residential areas and at passenger stations, round, tapered tubular steel poles shall be used. All poles shall be of the bolted base type and shall be designed and manufactured to relevant U.S. steel standards.

Where multiple OCS equipments are to be supported in the above ground, at- grade sections, such as at overlaps and turnouts, multiple cantilevers may be attached to a single structure, which shall be of a heavier section such that the applied loads shall not cause twisting of the structure by more than 5 degrees.

For multi-track areas where independent poles cannot be installed between tracks, portal structures with bolted base support poles and with drop tubes to support the OCS equipment related to individual tracks shall be used, thereby providing for mechanical independence of the individual equipments.

In general, OCS poles in station areas shall be located between tracks. For situations where OCS poles must be located on station platforms, they shall be placed in a manner that minimizes the visual impact and obstruction to passengers, and shall be integrated with platform architecture design. The minimum distance from platform edge to face of poles shall be 7 feet. Counterpoise grounding shall be used within passenger stations and the aerial static wire shall be electrically isolated from the OCS structures and components connected thereto. Refer to the *Grounding and Bonding Requirements* chapter.

Wall brackets and drop pipe supports in tunnels, cut-and-cover box structures and trench structures shall be of galvanized steel, and shall be attached using either C-channels or anchor expansion bolts of the undercut type. Refer to the *Structures* chapter and Standard and Directive Drawings for details. Where bracket installation requires drilling of reinforced concrete, the specifications shall require that specialized equipment be used to locate the reinforcing bars before drilling commences. The minimum distance from a reinforcing bar to a drilled hole shall be 2 inches.

For OCS poles mounted on top of trench walls or retained fill walls, the anchor (hold-down) bolts shall be field drilled and anchored in place with epoxy resin grout (with appropriate loading tests being performed). Refer to the *Structures* chapter and the Standard and Directive Drawings for details. OCS poles shall not be placed on top of or attached to intrusion protection walls.

For OCS poles on aerial structures, refer to the *Structures* chapter and Standard and Directive Drawings for the embedded anchor bolt sleeve detail. Unused sleeves on aerial structures shall be filled in by the OCS Designer. For OCS poles on retained fill, mechanically stabilized earth (MSE) wall structures, provisions shall be made by the Structures Contractor for the future installation of the OCS foundations. Refer to the *Geotechnical* and *Structures* chapters and Standard and Directive Drawings for construction and space allocation details.

21.15.2 OCS Pole and Foundation Requirements

The pole and foundation locations shall be designed in a manner that avoids conflicts with existing or planned overhead or underground obstructions. For existing revenue service locations, the foundation shall be constructed in a manner that does not disturb the existing tracks under revenue service.

The loading assumptions and strength requirements shall meet or exceed the requirements of NESC rules. The general design loads include dead load, live loads such as wind and ice, and earthquake load. However, as noted in NESC Rule 250A4, the structural capacity provided by meeting the loading and strength requirements of NESC Sections 25 and 26 will provide sufficient capability to resist earthquake ground motions.

In addition to the load conditions indicated in NESC, a 100 mph wind plus 10 percent gust allowance shall be evaluated to prove no failure. The Designer shall also evaluate the local extreme climatic conditions and adjust the load combinations for worst case loads, including the effects of wind pressure on OCS poles due to slipstream effects per the *Structures* chapter.

All structures, poles, brackets, foundations and anchors shall be capable of handling construction loads imposed during erection and during catenary assembly and wire installation, and of withstanding a broken-wire failure, including breakage of both the static wire and parallel feeder conductor in any 1 span, without exhibiting major, catastrophic damage. These support structures shall also be capable of handling the loads due to breakage of other parts of the OCS. Pole and foundation loadings and structural designs shall be developed in accordance with the criteria defined herein. To facilitate aerial structure design, maximum loads for design of OCS pole foundation on aerial structures are specified in the *Structures* chapter.

All steel materials, related processes and manufacturing methods shall be specified in accordance with ASTM standards, wherever applicable and deemed appropriate, including requirements for hot-dip galvanizing of steelwork and hardware.

The design of bolted steelwork connections shall conform to AISC requirements and shall specify materials and methods in accordance with ASTM standards.

Anchor bolts (hold-down bolts) shall be galvanized.

OCS foundations and structures shall be designed so that their deflection under the loads imposed during normal operating conditions shall not cause a contact wire displacement that could prejudice acceptable tracking and performance of the pantograph current collector. To this end, the maximum allowable live-load operating deflection of the pole and foundation structure together shall be limited to 2 inches at the normal design contact wire height. For the purposes of structural design, this live loading shall be considered a dynamic operating condition, and the structure shall fully recover from its displacement due to the live loading.

For all non-operating loading conditions, excluding seismic conditions, the maximum total deflection of the pole and foundation together (measured at the pole top) shall not exceed 2.5 percent of the total pole length due to both static (dead) loads and live loads combined.

The foundation and steel pole, or vertical members of the support structure, shall be designed to enable the pole to be raked during installation. This rake shall allow for the static dead loads that are imposed on the structure by the cantilevers, equipment and along-track conductors. Rake installation shall provide for a visually plumb and vertical pole after application of the full static loading. This position shall serve as the design reference datum for the calculation of the live-load operating deflection. All OCS alignment and wire layout designs shall utilize this static, plumb, dead load position as the true pole-face reference datum.

The OCS foundations and poles shall be designed in a manner to minimize the number of types and sizes to simplify constructability, to avoid disturbing existing adjacent structures, to provide flexibility for pole rake adjustment, and to minimize future maintenance inventory and costs.

Anchor bolt patterns shall be selected to provide coordinated relationships between poles and foundations. The coordination shall be based on matching strengths and minimizing the number of required configurations.

Particular attention shall be given to the provision of a high level of protection against atmospheric pollution and contamination to maintain the design life without frequent maintenance cycles.

OCS support locations shall be individually numbered for ease of identification on site. Structure number plates shall be fitted to the structure at a height of 6 feet 6 inches approximately above rail level. For supports located in tunnels, the number plate shall be attached to the wall using suitable fixings.

21.15.3 OCS Poles

Poles shall be designed as free-standing structures, except for poles carrying wire terminations, which shall be down-guyed, typically in the along track direction. The lateral offset from centerline of tangent track to centerline of pole shall be 10 feet-8 inches. Offsets shall be increased as needed to satisfy curved track situations and/or signal sighting requirements.

Aerial structures will be designed in a manner such that OCS poles can be located at any position along the structure. Alternatively, working in close coordination with the OCS Designer, aerial structures can be designed to provide site-specific locations for OCS pole installations.

The OCS supporting structures shall be calculated in accordance with relevant American standards (ACI, AISC, ANSI, ASCE, NESC). The allowance for a one-third increase in allowable stress for wind combined loading shall be waived.

The design of structural and fabrication welding shall conform to the AWS, Standard D.1.1, “Structural Welding Code”.

Painted poles shall not be precluded from use in passenger stations, within any urban design area, or in other special circumstances. Painting shall be specified to conform to the Steel Structure Painting Council, “Steel Structure Painting Manual,” Volumes 1 and 2.

21.15.4 OCS Foundations

The OCS foundations shall be capable of meeting the structural loading requirements, and shall be designed for each individual location. The structural dimensions will be dependent on the following:

- Loads on the poles due to the OCS conductors, feeder cables, tensioning equipment, insulators, mid-point anchor ties, and all other necessary equipment
- Wind loads on the poles and associated OCS conductors and equipment
- Soil conditions
- Earthquake loads
- Operational requirements the applicable speed

OCS foundation designs shall be in accordance with ACI, AISC, and ASTM standards, other applicable codes, and proven foundation engineering and anchoring methods. Foundation designs shall consider buoyancy effects where applicable. For corrosion control requirements, refer to the *Corrosion Control* chapter for details.

Augered, cast-in-place concrete foundations with a nominal diameter of 3 feet shall be adopted for all normal situations. Site-specific conditions or unusual loading combinations may dictate the adoption of other types or sizes of foundations. The permissible increase in soil resistance values, as defined in the CBC as being applicable to free-standing structures, shall be taken into consideration in the design of OCS foundations, in accordance with the CBC formulae.

The OCS foundations shall be designed to exceed the maximum design capability of the pole or structure being supported by the foundation by not less than 25 percent to ensure the foundation will not experience failure under the specified operating and non-operating conditions. The overturning moment shall not exceed 85 percent of the stability moment.

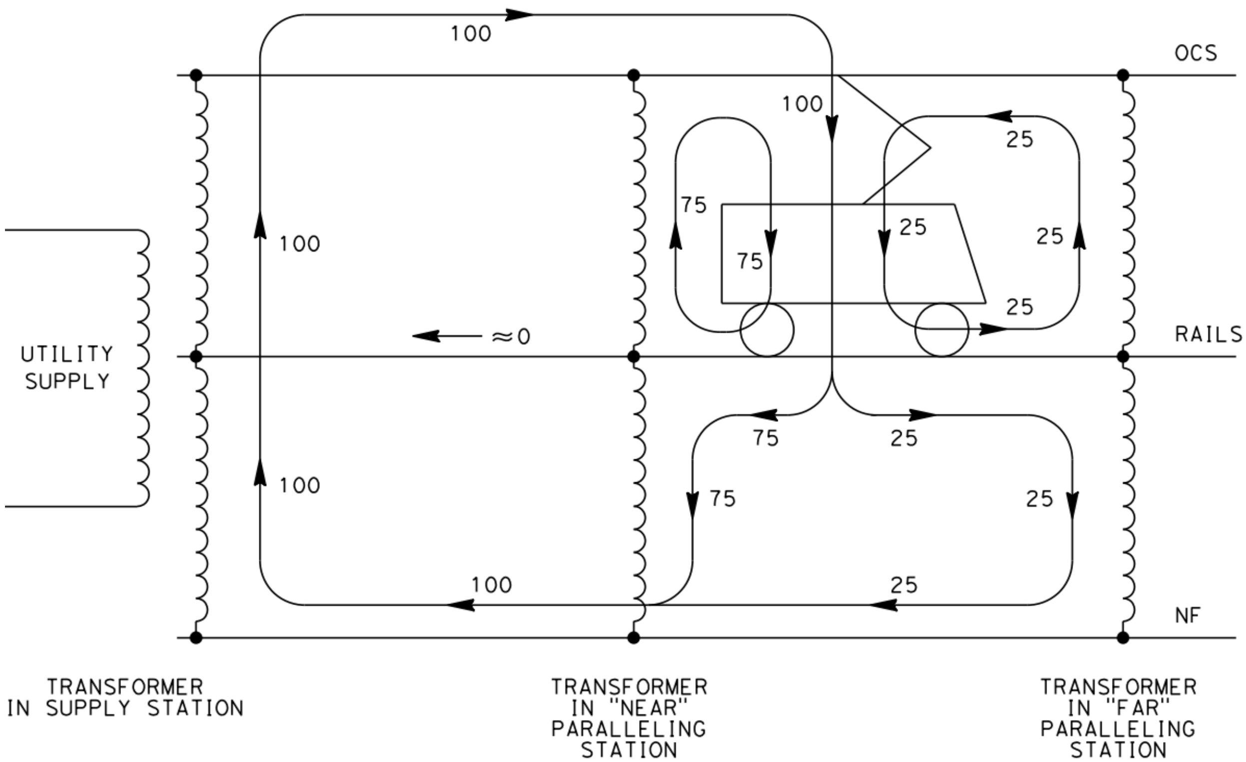
Where fragmented rock is encountered, excavation shall be required for the installation of standard foundations. Where solid rock is encountered below grade (i.e., with soil cover), epoxy-grouted dowels shall be anchored to the rock (with appropriate pull-out tests being performed), and the upper portion of a standard anchor-bolt foundation cast into the soil. Where solid rock is encountered at-grade (no soil cover), the anchor bolts shall be epoxy-grouted into the rock (with appropriate pull-out tests being performed) and with a small foundation-top cast around the bolts, primarily for aesthetic effect.

21.16 Traction Power Return System

The Rail Return System comprises the running rails, impedance bonds, static or ground wires, return cables, and the earth, each of which provides a part of the electrically continuous return path for the traction currents (refer to Figure 21-10). The Traction Power Return System comprises the Rail Return System together with the Parallel Negative Feeders, through which the whole traction current is returned from the wheel sets of the traction units to the substations.

The whole traction return current of a train operating between any 2 adjacent autotransformers flows through the rail return system within the bounds of these 2 autotransformers. These autotransformers, however, "force" a major portion of the traction return current to flow into the negative feeders, thereby minimizing the flow of return current in the rails in sections away from the train operating section. This is a safety related benefit of the autotransformer feed system, the other benefit being reduced electromagnetic interference produced in this system as compared to the direct feed system.

Figure 21-10: Typical Proportional Current Distribution in a 2x25 kV Autotransformer System for a Train Current of 200 Amps



It is recognized that Figure 21-10 is a simplified diagram and is not an accurate representation of all current flows, since portions of the return current flow from the train location back to the substation via the static wires and earth. Further, a portion also remains in the track rails.

Refer to the *Communications* chapter for conductor routing and cabling segregation for core systems equipment.

21.17 OCS Interfaces with Other Disciplines

To achieve satisfactory performance of the OCS and current collection by the electrically-powered HSR vehicles, it is essential that the OCS Designer work closely with other disciplines. The following sections highlight some of the major issues that shall be addressed during the final design process. It is not a comprehensive list, but provides general guidance to the OCS Designer.

21.17.1 Traction Power Supply System

- Confirmation of OCS and ancillary conductor sizes based on the traction power load flow studies
- Confirmation of insulated cable sizes for both aerial and underground applications based on the load flow studies
- Confirmation of traction power facility locations, and particularly of the SS and SWS where phase breaks are required
- Recommended frequency of static wire to rail connections based on the rail potential rise calculations
- Confirmation of proposed OCS sectionalizing scheme based on coordination with Operations and Maintenance requirements

21.17.2 Rolling Stock

- Confirmation of the selected vehicle and pantograph operating characteristics for input into the OCS-Pantograph dynamic simulation program analyses
- Confirmation of pantograph spacing, on train consists using multiple pantographs, for input into the OCS-Pantograph dynamic simulation program analyses and for development of OCS phase break designs
- Confirmation that trains with multiple in service pantographs have no electrical inter-connection between the pantographs

21.17.3 Train Control System

- Coordination of locations of impedance bonds
- Coordination of signal sighting requirements
- Coordination of wayside power cubicle requirements and locations

21.17.4 Communications System

- Coordination of wayside power cubicle requirements and locations
- Coordination of Broadband Radio System access point locations and clearance
- Coordination of OCS disconnect switch RTU (Remote Terminal Unit) and interface requirements

21.17.5 Bridge and Aerial Structure/Earth Retained Structure Design

- Coordination of location of OCS poles and pole loadings for any conflicts, coordinate with infrastructure and restore any associated components.

Refer to the *Structures* chapter for the allowable loading and space allocation requirements.

21.17.6 Trackwork

- Coordination of locations of impedance bonds.
- Confirmation of space requirements below rails for the installation of return cables and connections to impedance bonds.