California High-Speed Train Project

Request for Proposal for Design-Build Services

RFP No.: HSR 11-16
Structures Report
Clinton Ave to South of E. American Ave

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CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

DRAFT 30% DESIGN SUBMISSION

Fresno to Bakersfield

Sierra Subdivision
Procurement Package 1
Structures Report

May 2012

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CALIFORNIA
High-Speed Rail Authority

California High-Speed Train Project

Structures Report

06/29/2012 ADDENDUM 3 - RFP HSR 11-16

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URS/HMM/Arup Joint Venture

May 2012

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1.0 Introduction

The report identifies the key features of each of the structures in Package 1A,1B and 1C of the Fresno to Bakersfield Section within the Fresno to Palmdale Region of the California High-Speed Train Project (CHSTP).

This report covers only the HST structures considered nonstandard or complex. CHSR Design Criteria provides the definition of *nonstandard* and *complex* structures.

The design criteria divides structures into a classification hierarchy as follows:

- Primary structures (structures that directly support the HST tracks)
- Secondary structures (all other structures)

Primary structures are subdivided by importance into the following:

- Important structures (structures designated by the Authority to be important)
- Ordinary structures (all other structures)

Primary structures are also classified by technical complexity as follows:

- Complex structures: Structures that have complex response during seismic events through
    - Irregular geometry
    - Unusual framing
    - Long spans
    - Unusual geologic conditions
    - Close proximity to hazardous faults
    - Regions of severe ground motion
- Standard Structures. Structures that are not complex structures and comply with the pending CHSTP Design Guidelines for Standard Aerial Structures are the responsibility of the Engineering Management Team (EMT).
- Nonstandard Structures. Structures that do not meet the requirements for either standard or complex structures

Table 1.0-1 lists the structures in Package 1 of the Fresno to Bakersfield section of the HST and indicates their classification under the above system.

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Table 1.0-1
Structures and Components

Package Reference	Primary Structure Name	Structural Component	Location or Start Station on Alignment S	End Station on Alignment S
1A	Fresno Grade Separation	Reinforced concrete (RC) U-trough Primary - Nonstandard	10885+00	10909+30
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10909+30	10920+20
1A	Fresno Grade Separation	Covered trench at SJVR Northern Spur Crossing Primary - Nonstandard	10920+20	10940+05
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10940+05	10933+80
1A	Fresno Grade Separation	Covered trench at Dry Creek Canal Crossing Primary - Nonstandard	10933+80	10934+20
1A	Fresno Grade Separation	Covered trench at SJVR Southern Spur Crossing Primary - Nonstandard	10934+20	10935+20
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10935+20	10935+95
1A	Fresno Grade Separation	Jacked box beneath SR 180 Primary - Nonstandard	10935+95	10939+40
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10939+40	10941+90
1A	Fresno Grade Separation	RC U-trough Primary - Nonstandard	10941+90	10970+00
1B	Fresno Street Overpass	Two Span In-situ Box Underpass –Not in Contract	10991+70	10992+50
1B	Tulare Street Overpass	Two Span In-situ Box Underpass Primary - Standard	11001+53	11002+05
1B	Ventura Street Overpass	Two Span In-situ Box Underpass Primary - Standard	11020+50	11021+40
1B	Ventura Street UPR Bridge	Two span steel beam bridge Secondary – Non standard	11020+50	11021+40

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1C	Jensen Trench	RC U-trough Primary – Non-standard	11069+40	11140+00
1C	Fresno Viaduct Approach Ramp	MSE retained Embankment Primary - Standard	11141+00	11152+20
1C	Fresno Viaduct Golden State Boulevard	Steel Truss Primary - Complex	11152+20	11155+36
1C	Fresno Viaduct	Post tensioned spans Primary - Standard	11155+36	11191+47
1C	Fresno Viaduct South Cedar Avenue	Steel Truss Primary - Complex	11191+47	11195+02
1C	Fresno Viaduct SR 99 Undercrossing	Steel Truss Primary – Non standard	11195+02	11199+97
1C	Fresno Viaduct	Post tensioned spans Primary - Standard	11199+97	11216+02
1C	Fresno Viaduct Approach Ramps	MSE retained Embankment Primary - Standard	11216+02	11230+00
1C	Facility Access Structure	RC Box Underpass Primary - Standard	11218+00	11218+30
1C	Central Canal	RC Box Culvert Primary – Standard	11237+00	11237+40
1C	Viau Canal	RC Box Culvert Primary – Standard	11279+90	11280+10

1.1 Overall Design Assumptions for 30% Design

The amount of design undertaken at the 30% stage is limited to confirming that the design build (DB) contractor can continue to develop and design the concept proposed at 15% into a full detailed design suitable for construction.

In carrying out the 30% design, the designers have concentrated on the key aspects of the design stated in the 30% design scope. These aspects are determined in many cases by satisfying the requirements of the relevant design criteria.

For the U-troughs, the requirements are concerned with the following:

- Structural adequacy
- Constructability and consideration of adjacent constraints
- Technical feasibility

For the bridge structures, the requirements include the following:

- Structural adequacy
- Seismic performance as specified in the Seismic Design Criteria

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- Interaction between track and structure to ensure that adequate provision is made for relative and absolute displacements between track and structure
- Constructability and assumed construction method

1.1.1 Structural Adequacy

For the U-trough, the designers performed preliminary calculations on a number of cross sections to demonstrate that the assumptions about section wall thickness, shoring wall thickness, and excavation sequence were reasonable.

The designers performed similar calculations for key components of the U-trough — specifically the jacked box and Dry Creek Canal culvert. The Dry Creek culvert itself is not an HST structure, but preliminary design was necessary to demonstrate that there was clearance for the U-trough to pass under the creek and for the San Joaquin Valley Railroad (SJVR) tracks to pass over the structure without compromising its hydraulic performance. These calculations are attached at Appendices B (Fresno Grade Separation), C (Fresno Street Bridge).

1.1.2 Seismic Performance

The seismic design criteria gives the requirements for assessment of the seismic performance of structures. In terms of acceptability of the design, the requirements relating to seismic performance are Operability Performance Level (OPL) under the action of the Operating Basis Earthquake (OBE) and No-Collapse Level (NCL) of performance under the action of the Maximum Considered Earthquake (MCE):

- NCL at MCE:
- No collapse
- Significant yielding of reinforcing steel
- Extensive cracking and spalling of concrete but minimal loss of vertical load carrying capacity in columns
- Large permanent deflections
- OPL at OBE:
- Minimal impacts to HST operations
- No superstructure spalling onto tracks
- Minimal permanent deformations

The seismic design criteria define the two design-level earthquakes as follows:

- Maximum Considered Earthquake (MCE) - Ground motions corresponding to greater of:
- (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years with 5% damping) and
- (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to Mw) of any fault in the vicinity of the structure
- Operating Basis Earthquake (OBE) - Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years with 5% damping)

Response spectra for design have been the subject of separate studies (see also Appendix A Geotechnical Design Memorandum). The engineering management team (EMT) has provided spectra from these studies for use in the 30% design. These spectra are reproduced in Figure 1.1-1.

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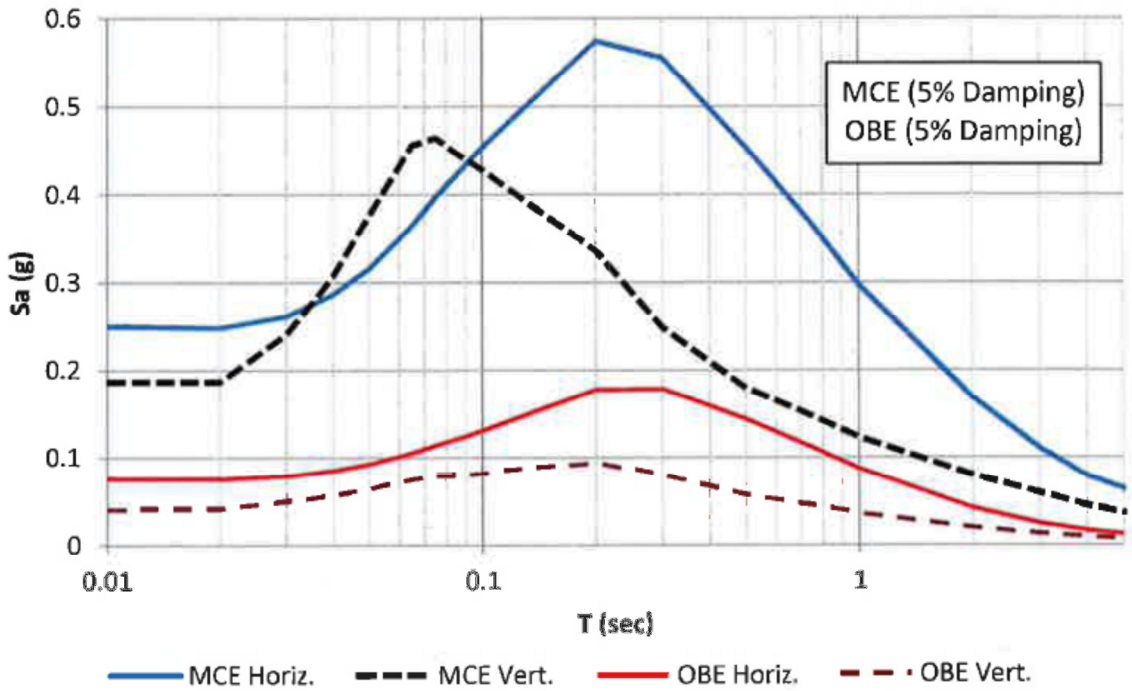

Figure 1.1-1
Design Response Spectra (Zone 4)

The seismicity in the Fresno area is categorized as Zone 4, which is the lowest category encountered on the Fresno–Bakersfield Section of the HST.

The peak ground acceleration (PGA) has been taken as the acceleration that corresponds to a period of 0.01 seconds — that is 0.0761g at OBE (red curve) and 0.2498g at MCE (blue curve). As these accelerations are less than 0.35g, in accordance with TM 2.9.10 clause 6.10.13, additional earthquake pressures can be disregarded for the design of buried structures such as the U-trough and the jacked box, provided that design for at-rest pressures is undertaken.

1.1.3 Dynamic Performance

Fundamental frequency checks have been carried out for the HST bridge structures in compliance with the requirements of TM 2.10.4 Seismic Criteria. Details of this analysis are reported in a separate submission.

1.1.4 Track Structure Interaction

The HST bridges within Package 1A (Tulare Street and Ventura Street) are relatively short, approximately 98 feet (29.87 meters). No critical interactions are expected along the length of these standard structures.

1.1.5 Information Required for Further Development of the Design

1.1.6 Assumptions Made for 30% Stage Design

The recommendations of the Geotechnical Design Memorandum (Appendix A) have been followed, including the following:

- Soil parameters (γ_b, $\emptyset$, c_u)

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- Assumed groundwater levels
- The requirements of TM 2.3.2 (see also Section 2.2.1)

For the jacked box assessment and preliminary design, assumptions have been made concerning the following:

- Soil parameters for the State Route (SR) 180 embankment
- The percentage of ground loss (overbreak) that can be expected during excavation
- Groundwater levels

For the U-trough shoring wall, assumptions have been made concerning the following:

- The type of shoring that will be used
- Soil parameters
- Temporary construction surcharges
- Temporary brace positions, spacing, and stiffness
- The Kinder-Morgan hydrocarbon line has not been considered in the analysis undertaken; if not diverted, the permissible movements that it can tolerate (unknown at present) may influence the design and type of shoring that can be used in the vicinity

More detail concerning these assumptions is provided in the individual sections for each structure.

The DB contractor or jacking specialist should verify these assumptions based on the results of the ground investigation and any other investigation they may undertake.

1.1.7 Further Information Required to Develop the Design

It is expected that the DB contractor will wish to have more detailed information regarding key design issues. These issues include, but are not limited to:

- Borehole details along the length of the U-trough
- Results of soils testing (currently planned)
- Results of long-term monitoring of groundwater levels
- More detailed assessment of surcharge loading
- Detailed knowledge of access routes and timing of access to site
- Details of the location of overhead contact system (OCS) posts and wall mountings
- Detailed discussions with Fresno Irrigation District relating to timing and construction sequencing of the 96-inch storm drain diversion
- Approved schedule of road closures and durations for cross streets
- Detailed discussions with Caltrans about acceptable settlements of the SR 180 bridge in response to more detailed proposals regarding box jacking process and methodology
- Greater detail about utility crossings in order to plan the protection measures required
- Definite information from Union Pacific Railroad (UPRR) about acceptability of the Tulare Street undercrossing bridge and the methodology for installation of the new deck

The design has not allowed for temporary or permanent surcharges applied to land outside the right-of-way other than the known UPRR loadings described above. It is considered advisable that negotiations for the right-of-way should include conditions either for the permitted use of land adjacent to the U-trough that limit the loading that can be applied or that additional land is purchased so that its use can be controlled.

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2.0 Fresno Grade Separation

The Fresno Grade Separation is a reinforced concrete (RC) U-trough structure that varies in depth from approximately 0 to 50 feet (0 to 15.24 meters). The trench will be constructed at a part of the route where the right-of-way width is constrained by adjacent properties; this restricts the methods by which the structure can be built, effectively excluding open cut excavation.

The grade separation structure is composed of a number of subtypes:

- Reinforced concrete U-trough
- Reinforced concrete U-trough with permanent high-level bracing
- U-trough structure with RC slab cover
- A section of RC-covered trench that is to be constructed off-line and jacked into position through an embankment
- Utility crossing structures

2.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno Area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
- Importance factor hI = 1.05
- Ductility factor hD = 1.05 for strength calculations
- Redundancy factor η_R = 1.05 for nonredundant elements, 1.0 otherwise

2.2 Key Design Features and Site Constraints

The grade separation is a simple RC U-trough (where the depth is such that additional permanent bracing is not required). These sections will be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3 which means that an "at-rest" earth pressure coefficient will be used instead of an "active" pressure coefficient. Appropriate load factors from the AASHTO LRFD code will be applied to give the design forces. The typical cross section of this configuration is shown in Figure 2.2-1.

The typical section indicates a 10-foot-high wall to the left of the section (east of the route). This collision protection wall provides protection for the HST route from intrusion by derailed trains from the adjacent UPRR. This wall has been added to the structure of the U-trough because the constrained width of the right-of-way in Roeding Park and other areas precludes providing this protection on an independent foundation. This wall is not required where the separation between the UPRR right-of-way and the HST right-of-way exceeds 102 feet (31.09 meters).

To the right (west) side, the wall has been raised 3 feet (0.915 meters) above ground level to provide a nominal delineation of the edge of the trench. Additional fencing is required for fall prevention in most areas; this is not shown on the section. At the right-side boundary, access restriction fencing is provided on independent foundations to delineate the right-of-way.

Either concrete channels or swales provide drainage of the ground adjacent to the U-trough.

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The section also indicates OCS equipment, which is attached either to the top of the walls or to the side faces of the wall. As the U-trough deepens, it becomes more convenient to mount the OCS on the side faces of the walls. In areas where the height of the conductor or feeder cables is within 10 feet (3.048 meters) of the ground, there is potential for a touching hazard. In these areas, it is considered prudent to raise the height of the wall to 10 feet (3.048 meters) above ground level so that the OCS can be mounted on the wall face instead of the top.

The OCS equipment is not part of the civil engineering contract; however, knowledge of its location is required in order to finalize the design of the wall in these areas.

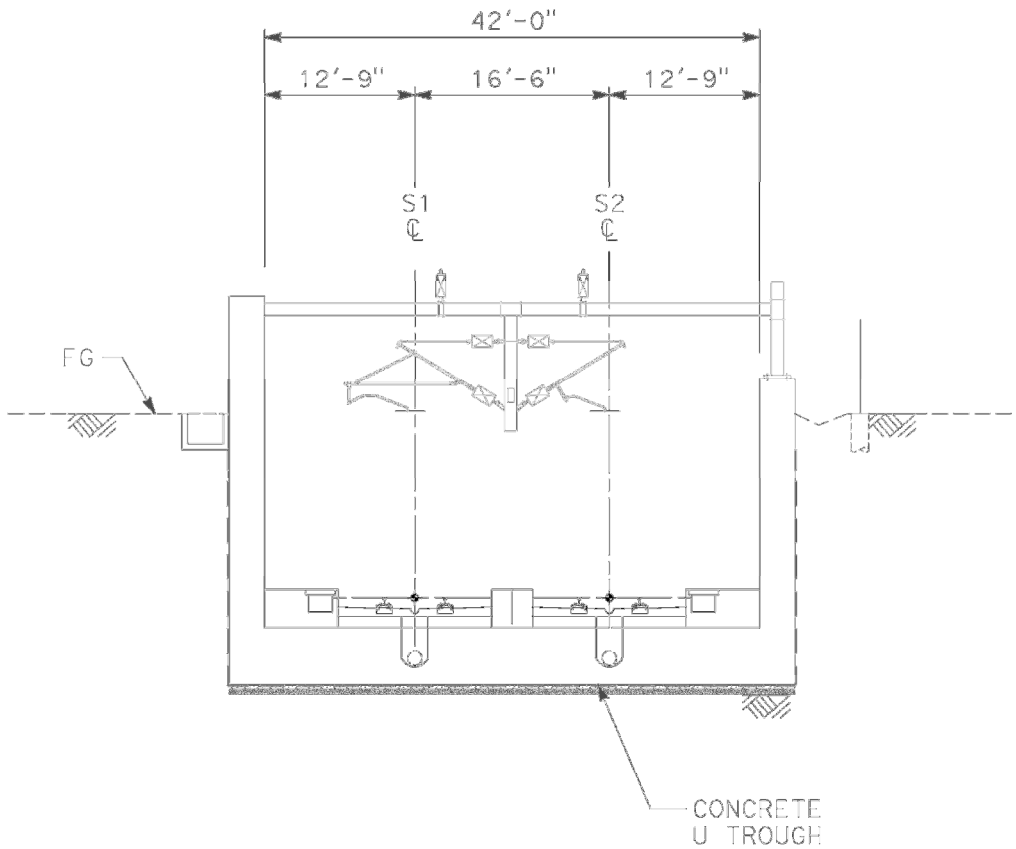

Figure 2.2-1
Typical Section of Unbraced U-Trough

At present, the ground investigation for this section of the route has not yielded any test results on which to base a design. Soil parameters used in the design have instead been based on historical geotechnical data along the HST Fresno subsection from State Routes 41, 43, and 99 as supplemented by City of Fresno residential development project records.

Where the depth of the trench exceeds approximately 30 feet (9.144 meters) from ground level to the top of rail, an unbraced section becomes difficult to achieve without excessively heavy reinforcement. Permanent bracing then becomes a more effective solution.

The minimum clearance requirements for the OCS system allow braces to be placed no lower than 27 feet (8.23 meters) above top of rail, which places the braces close to ground level at the start of the braced sections. As the trough continues to deepen, the braces maintain their clearance to the OCS. This also reduces the bending moments at the root of the wall.

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At Dry Creek Canal and some utility crossings, a reduced clearance of 24 feet (7.315 meters) has had to be provided. This is acceptable subject to approval provided that the OCS catenary supports can be arranged so that the catenary will fit under the obstacle.

Due to the additional stiffness provided by the brace, these sections must also be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3, using the "at-rest earth" pressure coefficient with appropriate load factors from the AASHTO LRFD code.

The typical section of this arrangement is shown in Figure 2.2-2.

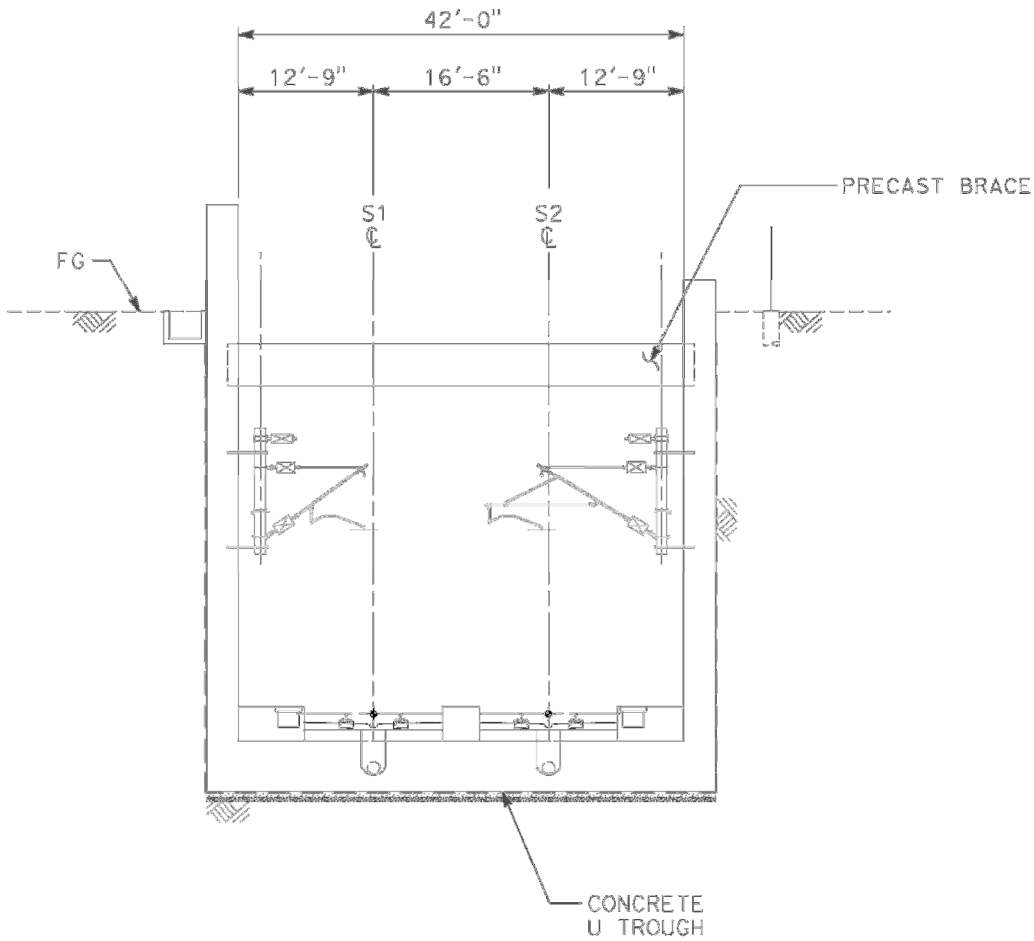

Figure 2.2-2
Typical Section of Braced U-Trough

2.2.1 Design Assumptions

2.2.1.1 Locked in Force from Shoring

In accordance with TM 2.3.4, the U-trough walls have been designed as rigid walls subject to at-rest earth pressures. In addition, where the walls will be restrained by permanent bracing, to allow for restraint forces that will be "locked-in" from the temporary bracing, the earth pressure calculated at the base of the wall has been assumed to act for the full height of the wall. This is similar to the pressures found in the design of temporary bracing to the excavation. The forces resulting from this assumption add approximately 25% to the forces that otherwise would be calculated.

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2.2.1.2 Groundwater Level

Groundwater levels have been assumed to be generally below the level of the excavation except in areas where there is a ready water supply. These are assumed to be at detention basin RR2 and at Dry Creek. In these places, the water level is assumed to be 10 feet (3.048 meters) below ground level as recommended by the Geotechnical Design Memorandum (Appendix A). Adjacent to these areas, it is assumed that water level gradually reduces.

2.2.1.3 Surcharge Pressure

For the majority of the length of the U-trough, the right-of-way has a width of only 60 feet (18.29 meters). To the east side, the right-of-way of UPRR abuts the HST right-of-way, and for approximately 1,000 feet (304.8 meters), Roeding Park abuts to the west. Consequently, in these areas it is not likely that the construction surcharges specified in TM 2.3.2 clause 6.4.4 will be possible. Nor is it likely that future developments will add to the surcharge. In areas where the route passes between G Street and H Street, surcharges are possible because the right-of-way width is greater and because a number of properties taken by the route may include saleable parcels of land.

The UPRR tracks are generally within 30 to 100 feet (9.144 to 30.48 meters) of the U-trough for much of its length, and the possibility of additional surcharge from derailments exists.

At its current location, the UPRR adds little to the force applied to the wall. The maximum contact pressure of the Cooper E80 loading (driving wheels) is 1,882psf at the underside of ties. This pressure was applied to the wall using the Boussinesq formula. The resulting moment effect at the base of the stem was back-calculated to an equivalent uniform surcharge. This procedure has demonstrated that a uniform surcharge of 420psf (3.86 feet [1.177 meters] of fill) would be adequate allowance for the Cooper E80 load and any short-term derailment surcharge, unless the offset to the nearest track centerline is less than 20 feet (6.096 meters).

Where the SJVR spur tracks cross the trough, a surcharge of 1,882psf has been applied.

Similarly, where adjacent land is available for potential development, a surcharge of 600psf as required by TM 2.3.2 clause 6.4.4 has been applied.

2.2.1.4 Collision Intrusion Barriers

Containment of derailed UPRR trains is provided by increasing the height of the U-trough wall to 10 feet (3.048 meters). Collision forces have been considered in the design of the upper sections of the wall where forces are concentrated. The design has allowed for two forces as specified by the UIC leaflet 777-2R. In practice, the upper force of 112.4 kips applied at a 9.84-foot height is only critical in the upper sections of the collision wall itself. The lower force of 449.6 kips at a 3.24-foot height is generally critical for the upper parts of the trough wall.

In order to reduce the risk of significant impact events affecting the body of the U-Trough wall it is recommended that the wall section immediately below the collision intrusion barrier should be designed to a higher capacity so that impact effects are localized to the area above ground level.

2.2.1.5 Methods of Counteracting Buoyancy

The concept for the trough is a development of the 15% stage design concept. The concept assumes that the temporary excavation for the trough is retained by shoring walls that are either removed or abandoned after the trough is constructed. For U-trough structures like this, rising groundwater levels are a threat because of the risk that the structural will float. This has happened in some rare cases.

A number of counterstrategies were considered in the development of the design, including the following:

- **Heels**
  The directive drawings indicate a heel detail, which means that in order to float, the buoyancy forces must overcome the weight of backfill over the heel in addition to the weight of the trough

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itself. This detail is designed for situations where the structure is constructed in open cut or at least with greater available space. It has not been considered suitable for this trough.

- **Thick bases**
  A second way to counteract buoyancy is to make the structure heavier. This is commonly achieved by thickening of base slabs. In the case of this structure, however, it would be necessary to have base slabs over 20 feet (6.096 meters) thick in some locations. This would be excessively costly, both in extra concrete and in extra excavation.
- **Attachments to walls**
  When a U-trough structure has a permanent shoring wall, it is common for the U-trough structure to be connected to the shoring walls using dowels or reinforcing bars drilled and post-fixed to shoring wall. The shoring wall then resists the uplift forces from buoyancy through skin friction with the ground. In the case of this trough, this option was discounted on the basis that the directive drawings require the trough to be "fully-tanked," i.e., to have continuous waterproofing membrane around its external surface. Dowel bars or reinforcement would have punctured this membrane, compromising the seal
- **Permanent walls**
  A development of the previous option is to combine the functions of the shoring walls with that of the permanent wall. This would limit the type of wall to either secant or diaphragm walling because of the need to maintain watertightness. The base slab of the trough would be constructed as the proposed U-trough but would be doweled to the diaphragm or secant pile wall at the edges. This option has not been pursued for the same reasons as above. However, a DB contractor may wish to develop this option further.
- **Micropiles**
  This option considers the construction of Micropiles of approximately 1-foot diameter at intervals along the length of the trough. Calculations suggest that one pile 35 feet (10.668 meters) long under each track at intervals of 5 feet (1.524 meters) would be sufficient to counteract the expected buoyancy forces. This method uses approximately 1/70th of the volume of concrete that would be required by thickening the base slabs.
- **Change watertightness requirement**
  There is a clear requirement that the trough should be watertight. This is expressed in the directive drawings that require waterproof membrane. However, if this requirement were to be relaxed to permit some water inflow, it could have the following effects on the design:

Benefits

- Open up the range of wall types to include contiguous (tangent piles)
- Remove all risk of buoyancy
- Reduce the need to design for water pressures
- Remove the need for waterproofing membrane

Drawbacks

- Need to increase the size of drainage pipes and sump storage capacity and pumping
- Increased pump running cost

Risks

- At some future date if groundwater rises to a level that the inflow cannot be carried by the drainage and sumps, it may be necessary to install a cut-off grout curtain to reduce inflow or to install a pumped groundwater abstraction system, if permitted
- Retrofitting the above works would be expensive and disruptive to operations

Of the options considered, Micropiles are thought to be the most economical and effective option for restraining the U-trough.

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Currently, information about actual groundwater levels is not available. Micropiles have been detailed on the drawings where water levels are expected to be highest. When groundwater information is available, this provision may be deleted or extended.

2.2.2 Key Constraints

Some constraints apply to the trench as a whole, while others are design and construction constraints that may apply to only one component of the structure. The key constraints on the trench are:

- The width of the right-of-way is generally less than 100 feet (30.48 meters) Adjacent to Roeding Park, it is approximately 60 feet (18.288 meters), and in the area of SR 180, 80 feet (24.384 meters). As the required minimum width for the track alignment and equipment is 42 feet (12.802 meters), this at worst leaves only 18 feet (5.486 meters) for the following:
    - All permanent retaining walls
    - Temporary shoring required for construction
    - Boundary controls required to delineate the right-of-way boundary (boundary fence, intrusion protection, intrusion detection, etc.)
    - Drainage (swales and channels)
    - The 96-inch storm drain diversion
    - Drainage sump access
    - Emergency egress stairs

  This width limitation is particularly critical in the Roeding Park area. Consequently, the method and sequence of construction of all parts of the trench should be developed in a carefully planned sequence to avoid the risk that parts of the site may become inaccessible for the completion of subsequent operations.

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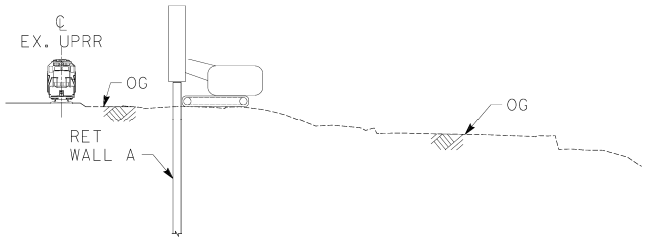
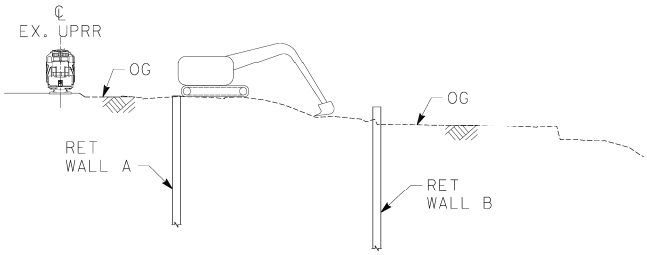
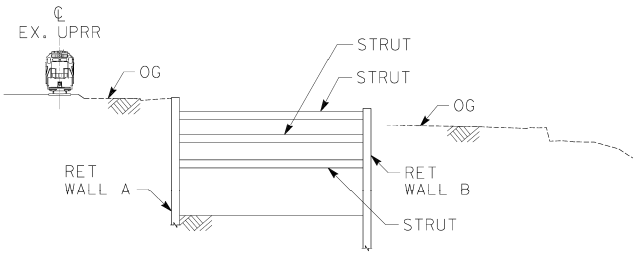
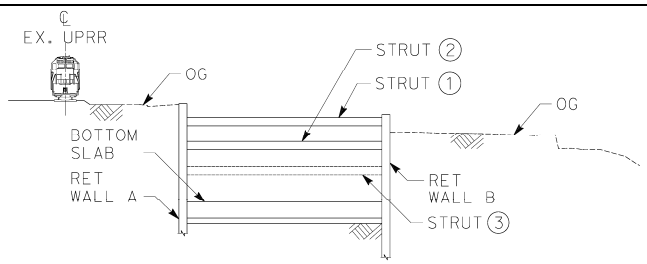
The assumed construction stages are shown in the Table 2.2-1.

Table 2.2-1
Assumed Construction Stages

Construction Stage	Stage Diagram
Stage 0 Install shoring walls from within HST right-of-way.	
Stage 1 Excavate to below first brace level.	
Stages 2, 3, etc. Excavate under previous stage bracing (using low-height excavators if required). Install bracing as needed.	
At required depth Place mudmat and waterproofing membrane. Fix base reinforcement. Cast U-trough base slab.	

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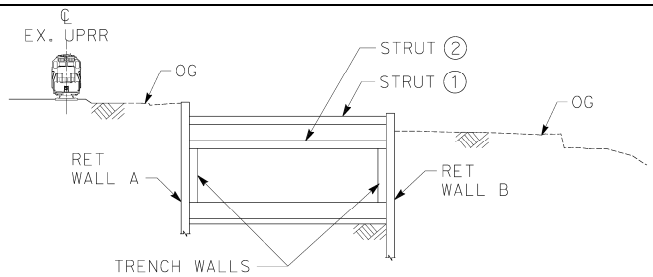
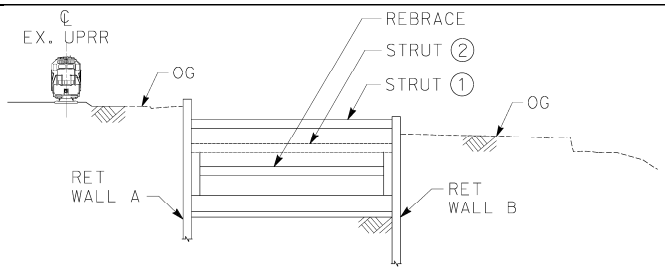
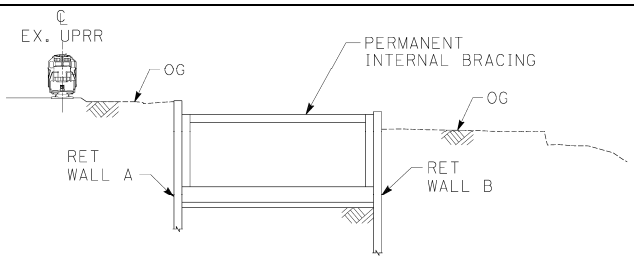
Stage 4 Remove lower brace (3). Fix waterproofing membrane. Fix wall reinforcement. Cast wall to part height.	
Stage 5 If required, reinstall lower brace. Repeat Stage 4 until full height achieved.	
Stage 6 Install permanent brace in deeper sections of U-trough. Cast collision wall if required.	

- The 30% design has developed this concept, and in order to confirm the feasibility of the proposed structure, calculations of the shoring wall requirements were carried out following the proposed construction sequence through to the permanent case. This work has confirmed that in principle a 3-foot wall thickness is feasible for the U-Trough, although in some locations it may need to be very heavily reinforced. In these areas, the contractor may wish propose a shoring system that can be considered as part of the permanent structure, which would allow a greater thickness of wall to be used for permanent design. However, this would require a design variance to be approved as the Design Manual prohibits this currently.
- The proximity of the UPRR tracks means that most of the length of the trench on the side adjacent to UPRR right-of-way requires the protection of a collision/intrusion barrier. The guidance provided by the project management team (PMT) was that this would be 10 feet (3.048 meters) high and 3 feet (0.915 meters) thick. (Note: this may be subject to change.)
  This barrier has been added to the top of the trench wall where possible because the trench provides a foundation that is sufficiently robust to carry the accidental forces and to provide a completely separate foundation in this area would be difficult because of limited space and difficult access post-construction.
- The trench will pass partially through the edge of drainage detention basin RR2 adjacent to W Belmont Avenue. The incursion of the HST route into the basin will result in a small reduction in the capacity of the basin.
  The PMT gave direction that in this area the shoring wall should be specified as a permanent structure of contiguous bored pile construction (known as Tangent Piles). This is required to act as a first line of defense, protecting the U-trough in the event of disturbance to the basin slopes and providing additional lateral restraint to the U-trough.

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- Also in basin RR2 near W Belmont Avenue, the existing storm drainage system outfalls into the basin via a 96-inch-diameter pipe that crosses the proposed HST route. The pipe must be diverted because its current invert is at a level that would conflict with the U-trough. The diversion route is shown on the utilities drawings and the structures drawings where it runs alongside the U-trough for approximately 500 feet (152.4 meters). The timing of its removal and construction of the diversion will be a critical aspect for coordination of construction and scheduling of the works in this area.
- The SJVR departs from the UPRR at two points. Both spur tracks cross the proposed route of the HST. To allow for this, short sections of covered trench have been designed. In order to maintain operational usage of the SJVR during construction, these sections of trench should be constructed at different times. Constructing the southern crossing first may ease the accessibility for construction of the northern crossing.
- Dry Creek Canal crosses the proposed route close to the location that the southern SJVR spur also crosses Dry Creek Canal. Consequently, the existing bridge that carries the SJVR over Dry Creek Canal must be removed. In this area, the trench has been designed as a covered U-trough. To provide clear separation of responsibility and ownership between the HST trough structure and the canal structure, a box culvert has been designed to cross over the HST U-trough. To ensure that the structures are separate, 1 foot of earth fill should be placed over the U-trough slab and below the base of the culvert.
  The culvert concept is a simple 2-cell RC box structure. A 2-cell structure has been selected to minimize the thickness of the top slab and therefore limit the amount of necessary vertical realignment to the SJVR while maintaining the existing soffit level.
  Initial discussions with the owners of the canal (Fresno Irrigation District) have confirmed that the concept would be broadly acceptable, subject to providing the ability to block off the cells for maintenance individually and with headwall details that match the profile of the existing canal on either end of the structure.
- The HST trench crosses under SR 180 at a point where it is on embankment. The alignment of the trench also conflicts in plan with the abutment of a bridge that takes the SR 180 over the UPRR and H Street. To avoid major disruption of SR 180, it is proposed that this section of the trench should be constructed using a box jacking technique.

2.3 Limits of Standard Bridge Design and Special Bridge Design

Standard bridge designs are not appropriate to this structure and the structure does not meet the criteria for a Special Bridge.

2.4 Construction Methods Assessment

2.4.1 Main Trench

The design team considered construction of the Fresno Grade Separation in the Constructability Memo at the 15% design stage. The 30% design has undertaken outline calculations based on assumed construction sequence to demonstrate the adequacy of the shoring system.

2.4.1.1 General Trough Excavation

The basic construction sequence described in section 2.2 and shown in Table 2.2-1 is extended slightly for the covered sections as follows:

- Construct temporary shoring walls
- Excavate to formation level, inserting temporary props as required
- Construct U-trough base slab
- Incrementally construct the side walls, removing temporary props as encountered and constructing permanent props if required by the design; where the section is covered, construct the roof lab using falsework supported from the base slab

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- Backfill over the covered sections

The above sequence could be applied over the full length of the trench where the U-trough is used, or it could be implemented in discontinuous sections if routes are available for removal of excavated materials. Because access to the excavation is difficult in the middle sections, it is likely that the contractor will choose to excavate the full length of the trench prior to constructing the U-trough, except at high-risk locations.

The high-risk locations are likely to be Belmont Basin, Dry Creek Canal, and SR 180. In these locations, the design team believes that a contractor would choose to do local excavations early in the construction period to overcome accessibility problems for the remainder of the trough construction.

2.4.1.2 Roeding Park Area

At the section of U-Trough adjacent to Roeding Park, the diversion of the 96" storm drain runs parallel to the trench for approximately 500 feet (152.4 meters). Over this length the invert level of the storm drain rises relative to the U-Trough such that the pipe will lie alongside the U-Trough. As the width available for construction of the U-Trough and the Storm Drain is restricted, it may be necessary to vary the expected construction sequence either to install the drain and U-Trough within a shoring wall on the boundary of Roeding Park or to construct the storm drain in advance of the U-Trough and then install the shoring wall alongside the storm drain.

2.4.1.3 E Belmont Avenue

The existing East Belmont Avenue will be closed temporarily in order to construct a new overcrossing bridge structure. Once the overcrossing is constructed, the road would be re-opened and the U-Trough structure constructed beneath it. However, when the bridge beams for the overcrossing are installed there may be insufficient vertical clearance for normal piling equipment, in which case low height equipment may be necessary. However, if the shoring wall can be constructed before the beams are installed this constraint can be avoided.

2.4.2 Jacked Box Concept and Constructability

Box and structure jacking has been used in many parts of the world over the last 50 years. It has become a well-established and successful technique in that time. In practice, there are many different forms and methods of jacking that can be used. Many of the techniques used are covered by patents, and as a result, it is likely that the successful DB contractor will employ a specialist subcontractor for this work who uses only one technique.

2.4.2.1 Description of the Structure

Where the jacked box is to be constructed, the proposed right-of-way has been increased to 80 feet (23.384 meters) because the excavation shoring walls would be constructed farther apart than in the other parts of the excavation to allow sufficient working space for construction of the box. As the excavation must be unbraced to allow space for constructing the box, it is likely that the shoring walls will also be more substantial in this area than in other parts of the U-trough. It is expected that the contractor will wish to extend the shoring to permit the construction of overhead braces that clear the top of the box (due to the topography of the area, the top of the box projects above ground level in the launch pit).

The box would be constructed on a base slab that is used to provide the reaction force against the jacks. This "jacking base slab" is also likely to be dowelled to the shoring wall to further distribute the jacking forces. Depending on the method used by the contractor, it is also possible that the jacking base slab will be extended part way up the sides of the jacked box as a way of providing lateral guidance to the box to ensure it stays properly aligned in the early (critical) stages of the jacking operation.

The box has been assumed to be a monolithic RC section, though it is also possible that the contractor may choose to divide the box into segments with "interstage" jacking between segments.

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The preliminary design has assumed the following key dimensions for the box:

- Length (excluding shield): 240 feet (73.152 meters)
- Thickness of the walls, roof, and base: 5 feet (1.524 meters)
- External width of the box: 53 feet (16.155 meters)
- External height of the box: 42 feet (12.802 meters)

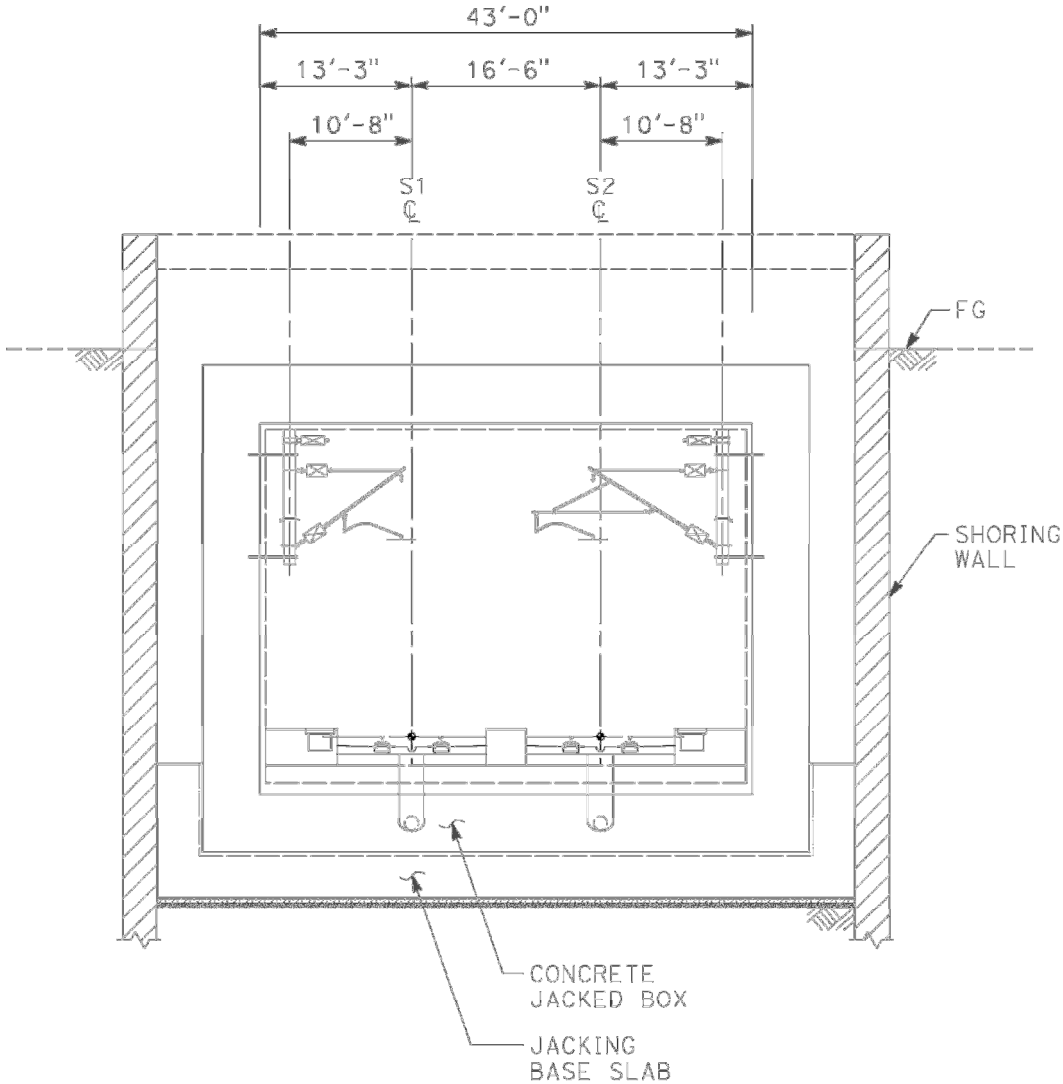

Figure 2.4-1
Cross Section of Launch Pit with Box in Position

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A view showing a similar-sized box structure under construction in this situation is shown in Figure 2.4-2.

Figure 2.4-2
Example of a Partially Constructed Box in Trench Prior to Jacking

At the leading edge of the box, a purpose-designed tunnel shield would be cast on to the normal wall of the box. This will incorporate a steel cutting edge that may also be adjustable as a method of steering the box during jacking. At the rear of the box, additional fixtures may be added to accommodate the thrust jacks. A typical cutting edge is shown in Figure 2.4-3.

Figure 2.4-3
View of Shield and Cutting Edge

Note: In this case, there is no roof slab as the excavation will be open-topped.

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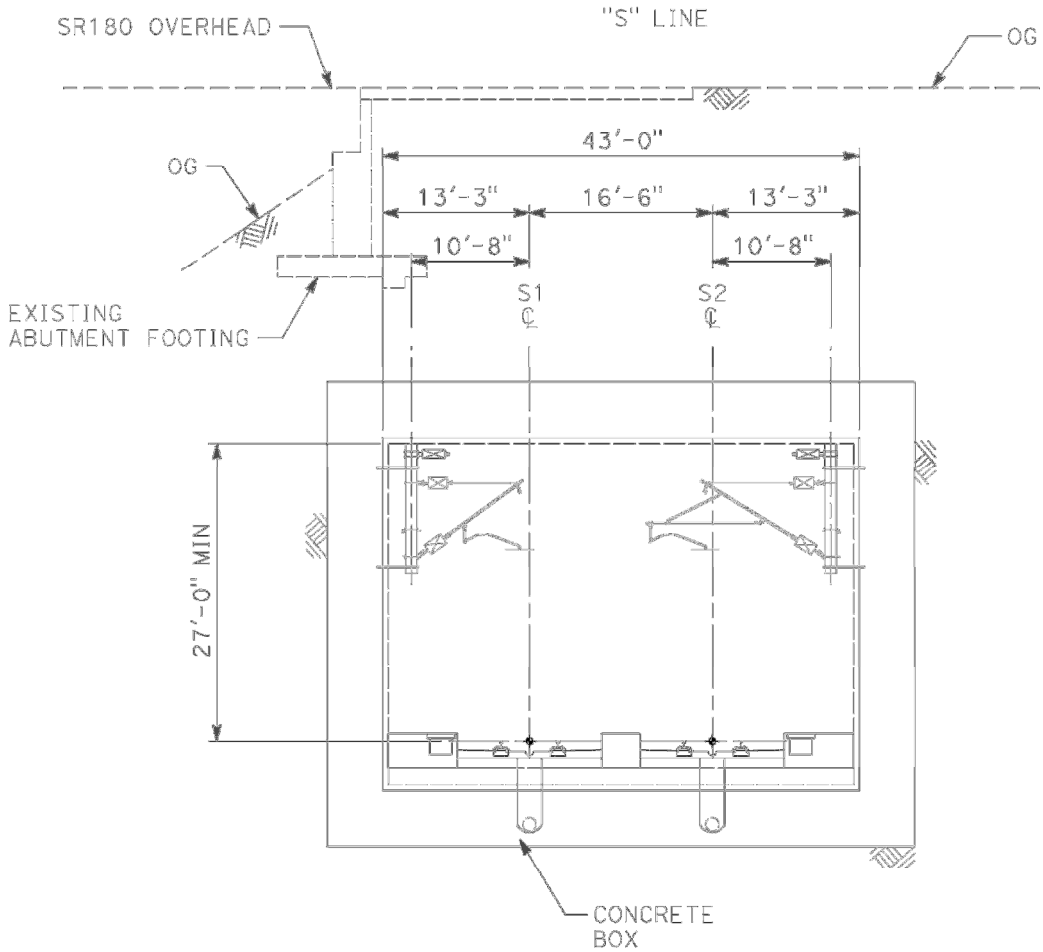

Figure 2.4-4
Cross Section and Key Dimensions of Jacked Box with Indicative Relationship to SR 180 Bridge Abutment

2.4.3 Anti-Drag System for Box Jacking

An essential component of the box jacking system is the method by which drag from the structure is reduced. This is required because as the box is jacked forward, there is a tendency for the box to drag the overlying ground along with it. In large embankments, there is some resistance to the drag force from the shear resistance of the embankment itself. However, this resistance may be insufficient to restrain the effect in the case of a wide box with low cover. If unrestrained, the ground on top of the box would be dragged forward, causing major disturbance and possible disruption to the overlying infrastructure.

The anti-drag system (ADS) is designed to prevent this — its use makes it feasible to consider box jacking where the depth of cover is as low as 6 feet (1.828 meters). The action of an ADS is illustrated in Figure 2.4-5.

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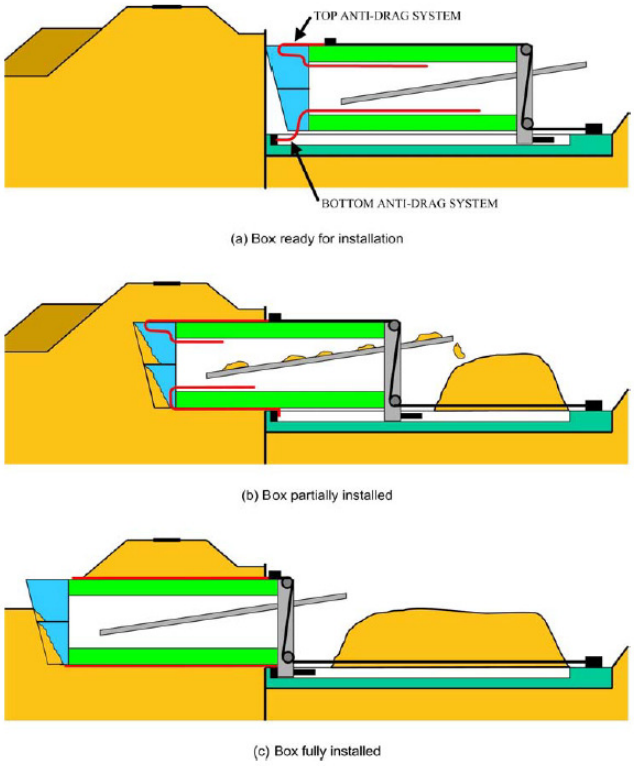

Figure 2.4-5
Illustration of Use of Anti-Drag System in Excavation

Figure 2.4-6
Anti-Drag Cables Laid Out Prior to Commencement of Jacking

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One proprietary ADS comprises arrays of closely spaced greased wire ropes. The lower ADS wires are anchored to the jacking base, with their free ends passed through guide holes in the shield and stored with their free ends inside the box (the lower red line in Figure 2.4-5). As the box advances, the ropes are progressively drawn out through the guide holes in the shield and form a stationary (anchored) layer between the moving box and the ground below. The jacking forces are absorbed by the ADS and transferred back into the jacking base by the wires.

The upper ADS wires are anchored to a frame above the box with their free ends passed through guide holes in the shield and stored inside the box (the upper red line in Figure 2.4-5). As the box advances, the wires are drawn out through the guide holes to form a stationary layer that is anchored to the frame and isolates the ground above the structure from the jacking force. The wires transmit the jacking force back to the anchor frame.

In this manner the ground above and below the box is isolated from the drag forces and remains largely undisturbed.

Other systems that provide anti-drag capability follow the same basic principle but may substitute steel strips for the wires described here or use scrap conveyor belting to fulfill the same function.

The ADS wires do not isolate the sides of the box from the jacking force, so it is necessary to provide a method of reducing the frictional resistance of the sides to ensure that the force transmitted to the ground at the sides is minimized. Ground drag on the sides of the box is usually reduced by arranging the cutting edge so that a slightly larger hole is excavated than the box dimensions. Typically, the excavation is oversized by about 1 inch. However, the amount of over excavation has an effect on the amount of settlement that is seen at the surface, so overdig should be kept to the minimum necessary. Ground drag can also be reduced by lubricating the ground/structure interface with bentonite slurry. Usually both these methods are used together.

To provide lubrication, slurry injection tubes would be cast into the walls of the box during construction. These tubes would be connected to a master valve linked to the bentonite supply pipe. Figure 2.4-7 shows a set of bentonite injectors arranged in a wall that is ready for concreting.

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Figure 2.4-7
Bentonite Slurry Injection Tubes

On completion of the jack, the bentonite injection tubes would be filled with cement grout to make a permanent seal.

The arrangement described above (with the exception of the upper ADS wires) is shown in Figure 2.4-8, which was taken at a recent railway project in the United Kingdom.

Figure 2.4-8
View of Jacking Area with Anti-Drag Restraints at Bottom

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2.4.4 Methodology for Jacking

The design team has developed a methodology to jack the box into place. This methodology has been discussed with a specialist jacking contractor, who has commented on the methodology and confirmed that it is feasible.

This methodology is as follows:

- Prior to construction of the jacked box, construct a structural base slab. This slab is designed to guide the box during jacking and provide a reaction base against which the jacking force can be applied.
- Lay a lubricated layer of sheeting on the jacking slab. This sheeting and lubricant could be a variety of materials, but the contractor consulted preferred steel plates as sheeting because less rigid materials have a tendency to ripple and jam the jacks. The concrete box section will be constructed on this layer.
- Construct the concrete box using normal RC techniques. Because of the need to construct the box in the bottom of the trench, it may be difficult to prop the shoring walls in this area. Surveyed ground levels indicate that the roof of the box will be above ground level in this location. Consequently, the design has assumed that a more substantial shoring wall section that requires no bracing within the height of the box would be used. It is also possible that the shoring wall could be extended to a higher level so that bracing could pass over the box.
- Prior to commencing jacking, it may be necessary to undertake ground improvement to the embankment fill that the box will pass through and beneath the bridge abutment to ensure that settlements of the SR 180 abutment remain within specified limits.
  Grouting may be necessary only to ensure that the excavation face is stable and provides enough support to the upper layers of the embankment.
  Grouting may need to be more extensive to limit the amount of settlement experienced at the surface if the embankment materials are particularly sensitive to disturbance. The amount of ground improvement needed depends on the settlement tolerance specified.
  It may also be necessary to use a multicellular face shield so that the excavation face is limited to smaller pockets that can be supported individually and excavated independently.
- Once constructed, jack the box against the shoring wall that closes the end of the trench; this is expected to be broken out from within the box. Apply the jacking force through jacks reacting against the jacking slab at the rear of the box. Concrete spacer blocks may be used to adjust the jack location as work proceeds. Additional lateral support to the shoring wall will be required to ensure stability after cutting off the lower part of the shoring wall within the box.
- Continue jacking the box and excavating the face from within the box. The jacking force may be reduced by the injection of bentonite or other lubricants between the outer face of the walls and the ground as work proceeds.
- On completion of the jacking, the cutting edge and face shield will be broken out to a point where they can be incorporated into the permanent trench walls.
- Decommission the jacking pit and complete trench construction by constructing a standard trench cross section within it.
- Backfill the space between the temporary shoring wall and the finished U-trough.

2.4.5 Support for the Excavated Face

Excavation of jacked boxes of this nature requires a balance between the rate of excavation of the material that the box is passing through and the rate at which the jacking force advances the box into the material. A secondary concern is that the excavation face will collapse in an uncontrolled way leading to over-break at the edges of the box. This may lead to the migration of material from outside the excavation zone into the excavation, which eventually results in excessive settlement at the surface. In the worst case, this might result in collapse of the overburden materials into the excavation (see Figure 2.4-9).

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The above sequence is more likely to occur in loose granular materials than in stiff cohesive materials. The SR 180 embankment is assumed to be constructed of well-compacted granular materials similar in nature to the in situ ground. This reduces the risk of collapse of the face.

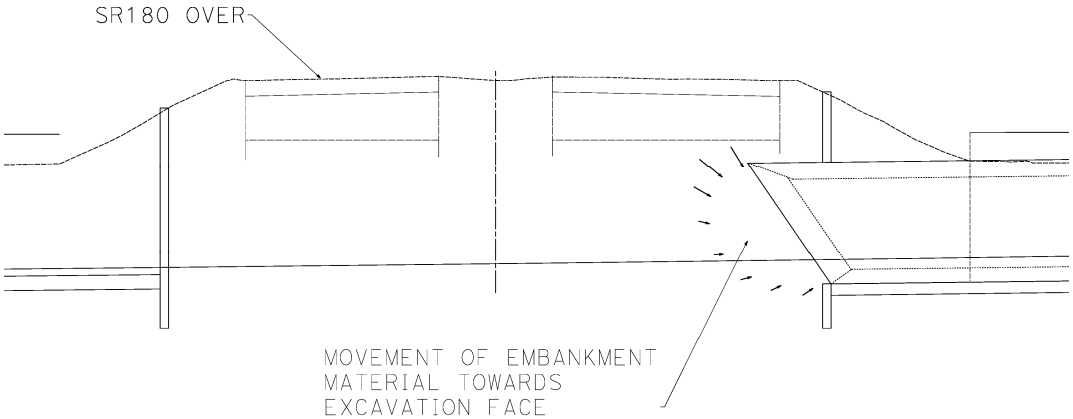

Figure 2.4-9
Excavation Process during Jacking

There are a number of ways to mitigate the risk of face collapse:

- **Pre-excavation grouting**
  The use of either chemical or cementitious grouts to increase the adhesion between the soil particles so that the excavated face behaves as a uniform, stiff, self-supporting mass during excavation. Grouting can be done either from the surface along the line of the jack or at intervals during excavation from the excavated face.
- **Compartmented excavation faces with support panels**
  The excavation size proposed for this box is approximately 53 feet (16.154 meters) wide by 42 feet (12.802 meters) high. This would present a large excavation face that may be difficult to control. In poor ground, it is common practice to subdivide the excavation face into several compartments that can be excavated by mini-excavator or by hand. This method gives the ability to control the excavation by selectively excavating certain compartments at different rates in order to steer the box and maintain directional control. Some contractors also use doors that retain the face when not being excavated. These may be hydraulically controlled and linked to the main jacks to ensure a constant pressure is exerted on the face.
- **Ground freezing**
  As an alternative to chemical or cementitious grouting, ground freezing increases the uniformity and cohesion of the excavated face by using the intergranular groundwater to bind the soil particles together for excavation. This technique is most commonly used where the excavation is below groundwater level so there is an abundant supply of water. However, because the freezing of water is an expansive process, this also means that there is a risk of heave at the surface. In extreme cases, the frozen mass can become marginally buoyant, leading to substantially increased heave.

Of the above techniques, ground freezing is considered inappropriate, as there is unlikely to be sufficient groundwater present for it to be effective.

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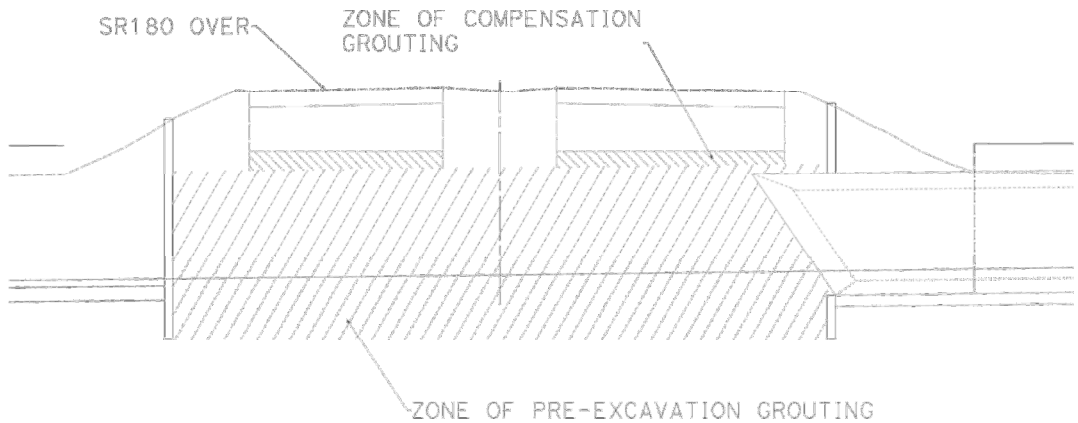

Figure 2.4-10
Zones Where Pre-Excavation Ground Treatment and Compensation Grouting May Be Used

The design team believes that the contractor will choose to use a combination of general pre-excavation grouting, compartmented excavation, and grouting from inside the box in advance of the excavated face. This combination works together quite conveniently — once excavation has started, it is possible for grouting to be done from one compartment while excavation is underway in other compartments. This also means that the excavation process can be regarded as a continuous operation. This is important as a major factor in maintaining the stability of the face comes from setting up a uniform "flow" of material through the box. If the process had to be stop/start with large time intervals between, it would be more likely to allow local collapse of weak areas, which would disrupt the uniformity of the "flow" with unpredictable results.

In cases where face collapse becomes a problem, the seemingly counterintuitive solution is often to increase the rate of excavation. This means that the calculation of required jacking force should be conservative to ensure substantial additional capacity is available if needed. In soft ground, a cellular shield configuration is normally adopted with the internal walls and decks buttressing the tunnel face. A cutting edge around the perimeter of the shield accurately cuts the hole through which the shield body and box structure pass. These cutting edges are sometimes adjustable to assist in the steering of the box. The shield provides safe access to the tunnel face for miners and machine operators, and egress for the ADSs.

The ground must have sufficient strength to arch safely across the open cells and must accept the incremental advance of the shield into it without distress. Sometimes it is necessary to improve the ground in advance of tunneling. In the ground conditions expected at the site, grouting ahead of the excavation is recommended if the water table is confirmed below the excavated profile.

Typically, 0.5 feet (0.152 meters) of soil would be trimmed from the face, and then the box would be jacked forward 0.5 feet (0.152 meters). This sequence is repeated until the tunneling operation is complete, thus maintaining the necessary support to the face.

2.4.6 Calculation of Jacking Load

The jacking load will consist of the following:

- Reaction on shield structure
- Friction due to the dead load of the concrete structure on the ground/concrete launch portal
- Friction on the top and side of the concrete box against the soil

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2.4.6.1 Reaction on Shield Structure

The reaction on the shield structure is assumed to be the passive pressure from the cutting edge of the shield. The thickness of the cutting edge is usually used to determine the reaction, and the resistance is calculated as the passive reaction on that area.

Based on experience on other projects, a 2-inch cutting edge around the perimeter has been assumed. Conservatively the outside perimeter is used.

Total Area: 2in x (2 x (516" + 624")) = 4,560 in^2

The passive reaction is calculated at the mid-level of the box (265-feet (80.772 meters) above datum):

$\sigma_P = K_P\ z\ g = 4.71 \times (320 - 265) \times 125 = 225$ psi

F = 225 x 4560 = 1,026 kips

A factor of safety should be applied. It is suggested to use 3.0.

F = 3,042 kips

2.4.6.2 Friction Due to Dead Load

The design of the component force for the weight of the structure is based on the dead load of the structure. This should be multiplied by a suitable coefficient of friction. The coefficient of friction between concrete and steel is conservatively assumed to be 0.3.

This is an upper bound value as both within the box and within the excavated profiles, the ADS formed by a series of greased wires at the interface between the top and the bottom surface (where the largest loads from gravity are expected) will drastically reduce this contribution.

Self-weight:

230ft x ((42 x 53) – (32 x 43)) ft^2 x 156pcf = 30,498 kips

Frictional force (FF) assuming a friction coefficient of 0.3:

FF = 30,498 kips x 0.3 = 9,149 kips

To include weight of shield and a factor of safety, the dead load should be multiplied by 1.2. Therefore the frictional forces would be 10,980 kips.

2.4.6.3 Friction from Soil and ADS

It is expected that the use of the ADS will impose additional forces in the jacking system. Based on references of similar projects, it is expected this load will be less than 4,500 kips.

The total load, including all the components defined above, would therefore be 18,500 kips. The maximum design single jack load is assumed to be 500 tons (1000 kips), and the number of jacks is expected to be less than 20 units. The jacking pit and the portal structure have been verified for this load.

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2.4.6.4 Ground Control

As discussed previously, the soft ground will most likely need to be pretreated to provide sufficient stand-up time during tunneling. In addition, the ground may need to be stabilized in advance to control surface settlement when tunnel jacking at such a shallow depth.

2.4.6.5 Monitoring

The jacked box tunneling operation must be carefully monitored and controlled to ensure the required performance and safety. Throughout the tunneling operation, movements at the ground surface over the area affected by the tunneling operation, jacking forces, and vertical and horizontal box alignment should be regularly monitored and compared to predicted or specified values.

2.4.6.6 Ground Settlement

The ground movements, including settlement due to the jacking of a box, are highly dependent on the method of construction, shield design, ADS's, and preparatory works. Most of the key parameters depend on the choice of temporary works, so the temporary works contractor would normally carry out the settlement assessment.

The settlement limits stated by Caltrans (see 2.4.9) relating to the abutment of the SR 180 Bridge are onerous. It is considered likely that the contractor would need to implement a compensation grouting system that will inject grout into the area below the foundation of the abutment in order to maintain or restore its original position.

In some compensation grouting schemes, the grout injection system may be linked to the movement monitoring system to automatically inject grout when the movement exceeds some defined threshold.

2.4.7 Alternative Methods of Constructing the HST Route Under SR 180

During the 15% stage of design development a number of alternative methods of constructing the HST trough in this location were studied.

These fell into two categories:

- Working under the SR 180 while in use
    - Using a jacked box
    - Propping the superstructure and using temporary bridges to carry traffic while excavating beneath to construct the U-trough
    - Extending the SR 180 bridge by adding a further span before excavating the U-trough below and through the extra span
- Closing the SR 180 in some way
    - Closing one travelway while running both directions on the other
    - Implementing full closure with diversion routes

Of these alternatives, the use of the box jacking technique was thought to be the least disruptive to Caltrans operations.

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2.4.8 Summary of Feasibility Design

The 15% design assumed that the box would be constructed in the U-trough to the south side of the SR 180 embankment. This site is a building to be demolished, and consequently there is an area of land with easy road access that may be used for temporary construction. There is no equivalent to the north of the SR 180.

The 30% design has developed the requirements for jacking a box and has confirmed the following:

- A structural design for the box can be achieved that also allows for the loads from the SR 180 bridge above
- There appears to be adequate clearance between the jacked box and the SR 180 bridge foundations (based on interpretation of the as-constructed drawings)
- The jacking force required to propel the box is achievable and in keeping with that required for similar structures on other contracts
- An experienced box jacking contractor considers the proposed method achievable
- There are ground treatment techniques that would render the embankment material suitable for the controlled excavation needed for the proposed technique

2.4.9 Discussions with Caltrans about the SR 180 Bridge

The design team met with Caltrans on October 23, 2011, to discuss the proposals for the box jacking and to determine their requirements for the following:

- Control of settlement of the SR 180 structure during the box jacking process
- Reinstatement of the bridge afterward should this be necessary

The team explained that the box would pass directly below the abutment foundation of the SR 180 bridge, and information was requested relating to permitted settlement of the structure.

Caltrans subsequently provided information that can be summarized as:

- The abutment movements must not exceed ¼ inches horizontally ½ inches vertically, whereas
- The vertical deck movement must not exceed 1 inch for continuous superstructures and 2 inches for simple spans
- All proposals relating to crossing of the SR 180 will be subject to Caltrans review and approval before work is permitted to commence

In order to comply with these movement limitations it is likely that the contractor will be required to undertake extensive grouting of the ground under the abutment. It may also be necessary to install compensation-grouting equipment linked to a settlement monitoring system to adjust the foundation of the bridge as jacking proceeds.

2.4.10 Conclusions

There are a variety of methods in use for the construction and installation of jacked structures.

It is considered highly likely that the main contractor will choose to use a specialist subcontractor that has its own preferred method of jacking. Some available methods may be unable to satisfy the movement tolerances specified and so contractors may investigate other options. The methodology described in this section is therefore considered "proof of concept" rather than a definitive statement of how this section of the grade separation should be accomplished.

The studies undertaken and summarized in this report have demonstrated that the methodology is feasible within the limits of the information available at this stage of scheme development.

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The constraints on movement required by Caltrans make the box jacking technique more challenging but not infeasible.

Therefore, we believe that the status of the design is that it is capable of being developed into a proposal that will be acceptable to the HST Authority and to Caltrans. However, the acceptance of Caltrans cannot be regarded as certain.

2.5 Temporary Construction Loadings Considered

During the construction of the U-trough, a number of temporary construction loads will be present for short or long periods. Refer to TM 2.3.2 clause 6.4.4.

The shoring design allows for the following:

- The effect of a Cooper E80 Train set on the Union Pacific tracks adjacent to the excavation. The peak pressure of 1882psf at underside of tie level has been converted into an equivalent uniform surcharge load of 420psf applied at ground level adjacent to the wall. (See 2.2.1.)
- A surcharge pressure of 600psf has been applied to areas where construction activity may use land adjacent to the U-trough. This is not additional to the train loading above and is also not applied in areas where construction access is not permitted.
- Variable groundwater levels in the section of the trench adjacent to the Belmont Basin and in the area of the Dry Creek Canal crossing.

2.6 Temporary Construction Easements

Temporary construction easements are required for the construction of the following:

- The diverted 96-inch storm drain outfall
- Dry Creek Canal structure
- SJVR connections
- Connections to the trench drainage sump
- Emergency egress stairwells and emergency access roads

The drainage sump is located between two spur tracks and will be connected to the local drainage system via a new detention basin. The basin will be constructed adjacent to the southern SJVR spur line.

2.7 Traffic or Pedestrian Diversion and Control

The construction of the trench requires the permanent closure of W Belmont Avenue Underpass, N Thorn Avenue, and part of Golden State Boulevard. Replacement overcrossing bridges are to be provided at W Olive Street and W Belmont Avenue.

Traffic management will be necessary to accomplish these changes. The contractor will be required to coordinate and plan works in these areas so that traffic disruption is minimized to the satisfaction of the City of Fresno.

For the construction of the U-trough, there will be a need for construction entry and egress points that connect to the road system. It is expected that the majority of excavated material from the U-trough will need to be taken offsite via these egress points, so it will be necessary to agree upon the amount, frequency, and operating hours for these entry/egress points with the City of Fresno.

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2.8 Drainage Concept

The track drainage within the trench will be carried in two longitudinal pipes cast into the base slab in accordance with the directive drawings. At the low point of the U-trough (STA 10926+00), the drainage flow will be collected at a sump adjacent to the west side of the trench structure where it will be pumped to a new detention basin located within the environmental footprint adjacent to the southern SJVR spur line. Outfall from the basin will be attenuated to discharge only at the rate of a 2-year storm as discussed and agreed with the Fresno Metropolitan Flood Control District (FMFCD).

For design, it has been assumed that the depth to the natural groundwater level is around 60 feet (18.288 meters) below ground level, except in areas where higher or perched water levels may be expected. This assumption is based on historic borehole data from Caltrans projects in the area in the absence of more recent information.

Higher groundwater levels have been assumed to exist at the Drainage Detention Basin (RR2) adjacent to W Belmont Avenue and at the point where Dry Creek Canal crosses the HST route. In both cases groundwater has been assumed to be 10 feet (3.048 meters) below ground level as recommended in the Geotechnical Design Basis Memorandum (Appendix A).

A ground investigation has been commissioned, but it will not be able to provide improved data before completion of the 30% design phase.

Based on the above assumption, it is not expected that cutoff walls will be required at the ends of the trench to limit groundwater inflow. Buoyancy checks have been carried out assuming groundwater levels as above. These checks show that either additional thickness of base slab or tension piles would be required to provide the necessary factor of safety against flotation.

2.9 Emergency Egress and Escape Provision

Although not strictly an elevated or underground facility, the team has agreed that it is appropriate to apply the requirements of NFPA130 for emergency escape/egress to the U-trough. This means that escape stairwells are to be provided at maximum 2,500-foot intervals through the box. Stairwells are provided as indicated in Table 2.9-1.

Table 2.9-1
Stairwell Provisions

STA	Locale	Egress features
10906+00	Adjacent to communication site, located in the abandoned connection of Golden State Boulevard to W Belmont Avenue	Stairwell is located close to the communication site and will share a common road access track. There is space for provision of a turning area for vehicles.
10925+00	Between the north and south SJVR spur connections	Emergency services access to the location of the stairwell will need to be agreed with the owner of the facility. There is space for provision of a turning area for vehicles.
10950+00	South of Divisadero Street and adjacent to G Street	Stairwell is located in an area currently used as a vehicle parking area with a frontage onto G Street. There is space for provision of a turning area for vehicles.

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Each stairwell is 10 feet (3.048 meters) wide by 25 feet (7.62 meters) long to allow for the later installation of a staircase.

The staircase is assumed to be 44 inches (1.18 meters) minimum width with 5-foot-wide (1.524 meter-wide) landings at 12-foot (3.658 meter) vertical intervals and 21 treads per flight.

2.10 Inspection, Service, and Maintenance Access

The trench structure itself will be a simple massive RC structure with a limited number of movement joints at intervals. There will be no specific provision for inspection or maintenance access other than the general maintenance access to the route.

The drainage sump will require pedestrian access at the surface and access for the installation and removal of pumps. Pedestrian access will also be provided by construction of an access door from the emergency walkway within the trench. Providing this door increases the risk that it may be dislodged by the passage of a train, so it is proposed that this door and the doors associated with the emergency escape stairs should be sliding doors. These may be fitted during a later contract.

Access for pump replacements will require a permanent easement and is likely to be via the area of land between the SJVR spur tracks.

Movement joints in the walls will be required to limit the effects of temperature and ground movement. These joints are intended to be no more complex than simple cast-in waterstop details.

2.11 Utilities Affected and Disposition

A number of existing utilities cross the route of the trench or are within the proposed right-of-way. Where these can be diverted, the proposed diversion route has been identified on the utilities and structures layout drawings. It has been a principle of this work to divert utilities into new infrastructure (such as road overcrossings) or into the fill over the covered parts of the trench where possible. Where there are specific crossing points that cannot be accommodated in this way, a utilities crossing structure is incorporated into the detail of the trench or the trench design has been modified to accommodate the utility.

Examples of where the trench design may be affected are as follows:

- **Kinder Morgan hydrocarbon line**
  This utility does not in fact enter the proposed right-of-way of the HST route. It runs along the UPRR right-of-way in an easement granted by UPRR. Its precise route varies along the right-of-way and in some places appears to be within 5 feet (1.524 meters) of the right-of-way. The location shown on the utilities plans is based on information provided by Kinder Morgan, but its accuracy has not been verified by excavation.
  For the construction of the trench, care must be taken to consider the effects that the trench construction methodology will have on this utility. At this time, all that is known about the line is that its diameter is 12in.
  Concern is based on the following:
  - The pipeline has been in service for around 30 years, and its current condition is unknown to the design team.
  - Given the above, it is unknown whether the pipeline is sensitive to the magnitude of ground movement that may be expected from construction of the U-trough.

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- The pressures at which hydrocarbon lines operate are usually very high in order to minimize the number of intermediate booster stations required. Consequently, a break in the line could occur explosively and be difficult to contain.
- The line is reported to be buried deep enough to pass under the depressed Fresno Street, which suggests it may be up to 20 feet (6.096 meters) deep. This depth is a further indication that the operating pressure of the line is high.
- The design team does not know whether the pipeline is currently leaking into the surrounding ground or has leaked in the past. The presence of hydrocarbons in the excavation would influence the choice of excavation methods that a contractor would use in the U-trough excavation.

In order to clarify these issues Kinder Morgan were contacted and provided an initial response by e-mail on December 5, 2011.
The main points are:

- *Location and Depth: Exact Location & depth and only be determined by potholing. The Alignment sheets will give the general location but no depth information.*
- *Type of pipe & diameter: this information is on the alignment sheets generally speaking, it is steel pipe 12.75" OD. Wall thickness varies.*
- *Foundation beneath the pipe not sure exactly what they are asking, usually the pipe is bedded in clean soil.*
- *Contents and pressure within the pipe: liquid petroleum products (motor and jet fuels). The maximum operating pressure (MOP) is around 1440 psig; however, the operating and control pressures will vary along the pipeline.*
- *Condition of the Pipe: the pipe meets or exceeds all regulatory requirements.*
- *Date of last inspection: KM has a robust inspection program; however, I do not see how this information is pertinent to your design team.*
- *General performance: overall good.*
- *Allowable movements: Lets discuss at our meeting, I need to know the context and purpose of movement.*
- *Design criteria: 49 CFR 195*
- *Support Methodology & serviceability criteria of the support: KM will determine the adequacy of any proposed supports.*
- *Local soil lithology: I don't believe we have the information for the hundreds of miles of pipelines that KM operates in the State.*

This information confirms that the working pressure of the pipeline is likely to be high.
No clarity is provided as yet regarding tolerance to movements of the ground or proximity to the shoring walls.
It is not clear how the pipe can be protected from ground movement but it is known that the pipe is placed in the earth of the trench. So that there are no additional elements that may stiffen the pipes response to movement.

Overall the pipeline's proximity to the excavation remains a concern. Therefore, it is recommended that this pipeline be diverted to the east side of the UPRR right-of-way prior to construction of the U-trough structure.

- **96-inch storm drain outfall crossing the HST route at STA 10897+30**
  This is a diversion of the existing outfall to drainage detention basin RR2 that is located adjacent to W Belmont Avenue. The diversion of the existing facility is essential to the construction of the U-trough in this area and the diversion route that is indicated on the drawings lies in close

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proximity to the trench.
After crossing under the HST route, the storm drain outfall runs parallel to the U-trough for over 500 feet (152.4 meters) until it reaches the detention basin. Its location, between Roeding Park and the U-trough, will be a substantial constraint on the working space available for construction of both the U-trough and the diversion. Both must therefore be considered together when developing the methodology for construction in this area.
The vertical position of the storm drain is also a constraint in the location of the emergency escape stairwell.

- **12-inch water line at STA 10915+60**
  This is a small-diameter water supply pipe whose route cannot avoid crossing the U-trough. The diverted utility passes around the edge of the detention basin before crossing the HST via a utility crossing bridge. The utility crossing bridge will be a concrete box that will totally enclose the sleeve through which the utility pipe is installed. Future maintenance of the utility will be carried out by withdrawing the pipe from its sleeve.
  At this location there are also a number of gas lines that cross the U-Trough so that the utility crossing structure is likely to be around 10-feet (3.048 meters) in width.
- **30-inch sewer line at STA 10933+30**
  This is an existing gravity sewer that currently passes under Dry Creek Canal. The diversion crosses the route at a vertical clearance of 24 feet (7.315 meters) because a pumped solution is considered unacceptable by its owners.
- **Dry Creek Canal culvert at STA 10934+05**
  Dry Creek Canal will be culverted to pass over the trench on its current alignment and invert level. In order to maintain separation of the culvert structure from the trench structure, a minimum thickness of 1 foot of fill is to be placed between the upper surface of the cover slab and the culvert foundation.
- **12-inch Gas Line**
  This pipeline is a diversion of an existing line and passes through the fill covering the HST adjacent to the Southern SJVR spur.
- **60-inch storm drain diversion at STA 10935+85**
  This is a new storm drain that is the diversion route for a drain that currently crosses the route of the HST at Divisadero Street. It is a gravity design and crosses the U-trough in a concrete sleeve structure at a minimum vertical clearance of 24 feet (7.315 meters).
- **SR 180 route crossing at STA 10937+00 to 10939+50**
  It is believed that any existing utilities along the route corridor are relatively shallow. The U-trough adjacent to the SR 180 is at considerable depth. The design concept in this location is for a large concrete box to be jacked through the embankment of SR 180, passing underneath any near surface utilities and the SR 180 bridge abutment at a depth of approximately 20 to 30 feet (6.096m to 9.144 meters) below road surface. The utility plans indicate an abandoned oil pipeline that, from its alignment, predates the construction of the SR 180 embankment. The utility information does not indicate that the pipeline was removed during construction of the embankment, so it is assumed still present. The depth of other oil pipelines in the area suggests that this line is at approximately 10 to 15 feet (3.048m to 4.572 meters) below ground level, which means that it would be encountered during the excavation of the jacked box. The contractor will need to be prepared to deal with the excavation of potentially contaminated ground on the route of the pipeline.
- **20-inch water pipe at STA 10940+15**
  Similar to the other crossing, this is a service line that cannot avoid crossing the route of the HST and for which there is no reasonable alternative route. The pipe will be carried by a concrete surround and will be sleeved through the structure to permit removal and replacement.
- **Flood overflow at STA 10942+80**
  This is not strictly a utility and the purpose of this structure is discussed under hydrological issues in the next section.

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2.12 Hydrological Issues

These issues are discussed in detail in the Floodplain Impact Assessment Report.

The main impact of the trench design is to ensure that the trench wall is substantially higher than the 100-year flood level in the Dry Creek Canal area. In this area, the 100-year flood level is approximately at ground level. Protection against flooding will be provided indirectly because the requirements for collision/intrusion protection require a wall 10 feet (3.048 meters) higher than ground level and at the west side the trench wall is 3 feet high.

During discussions with FMFCD, it was noted that the area to the south of SR 180, north of Divisadero Street, and to the east of the HST route would be cut off by the construction of the U-trough. The FMFCD has commented that in extreme flood events (50-year return period or more) this area can develop an overland flow toward the west that relieves flooding to the east. The FMFCD would like this "relief valve" to remain after construction of the U-trough. To provide for this, a closed box (similar to a utility crossing) has been added to the trench approximately at ground level. Under normal circumstances, this structure will be completely empty, but in the extreme cases described, it will allow water to flow across the HST route.

2.13 Noise Mitigation and Acoustic Treatment

No specific features have been included to mitigate the noise generated by the passage of trains. As the route is located in the trench, the trench itself will tend to direct noise generated by the passage of trains upward. This is likely to have little attenuation effect as the trench walls will be hard and reflective to sound. The presence of collision intrusion walls between the UPRR and the trench may provide some local attenuation to generated noise due to the increase in path length.

Discussions have taken place regarding the construction of a 15-foot-high sound wall or noise protection barrier between the HST corridor and Roeding Park. The details of this wall are not finalized at this time and its construction does not currently form part of the scope of the DB contract. However, should the wall be required at a later date, the details of the U-trough wall adjacent to Roeding Park as currently developed would have no difficulty in accommodating the additional loads imposed by such a barrier.

2.14 Compliance with Systemwide Bridge Aesthetics Features

No guidance has been provided on aesthetic considerations relating to this structure form.

2.15 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

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3.0 The Jensen Trench

The Jensen trench is an RC U-trough structure similar to the Fresno Grade Separation. It varies in depth from approximately 0 to 15 feet (0 to 4.572 meters). The primary reason for the trench is to allow the HST to pass under the E Jensen Avenue road bridge and to protect the HST from the effects of flooding from the FEMA-designated flood plain that lies adjacent to the route.

In contrast to the Fresno Grade Separation, the right-of-way at the Jensen trench is 130 feet (39.63 meters) wide, so it is likely that the trench structure would be constructed in open cut or with minimal use of sheet pile shoring. This also means that buoyancy effects could be counteracted by incorporating a heel to the base slab without needing to increase the right-of-way.

The depth below grade never reaches the point where bracing is necessary.

3.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they must be reinstated after an earthquake to allow resumption of train service. The structure is also noted as Non-Standard.

This classification implies the following:

- Design life is 100 years
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno Area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors — η_I, η_D and η_R — have been chosen as follows:
    - Importance factor η_I = 1.05
    - Ductility factor η_D = 1.05 for strength limit states
    - Redundancy factor η_R = 1.05 for non-redundant elements, 1.0 otherwise

3.2 Key Design Features and Site Constraints

The Jensen Trench is a simple RC U-trough. These sections will be designed as rigid walls in accordance with the design criteria, which means that an "at-rest" earth pressure coefficient will be used instead of an "active" pressure coefficient. Appropriate load factors from the AASHTO LRFD code will be applied to give the design forces. The typical cross section of this configuration is shown in Figure 3.2-1.

The entire length of the HST route in the trench is greater than the 102 feet (31.09 meters) separation distance, so no additional provisions for containment of derailed UPRR trains are necessary. It is possible that shaping of the grade profile between UPRR and the trench could provide derailment containment, should this be deemed necessary.

As the trench is partially in a defined FEMA flood plain area, the trench wall height above adjacent grade has been defined as the minimum of 3 feet or 1 foot above the adjacent 100-year flood level.

The FEMA-designated flood plain extends to both sides of the HST route; on the west side the wall height is defined in the same way as the east. Additional fencing is required for fall prevention in most areas; this is not shown on the section. At the right-side boundary, access restriction fencing using independent foundations is required, which will also typically delineate the right-of-way boundary.

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The right-of-way in the area of the trench has a greater width than at Fresno Grade Separation, and because the trench is relatively shallow, it is expected that the contractor will use a temporary open cutting or possibly a shallow retained cut. There is space within the right-of-way width to construct a small heel to the wall to provide resistance to buoyancy effects, should it be necessary. The relevant directive drawing DD-ST 010 limits the width of heel to a maximum of 5 feet (1.524 meters).

The drainage design assumes that swales will be provided adjacent to the U-trough.

The section also indicates OCS equipment, which would be mounted on the top of the walls using a standard portal framework.

The trench never becomes deep enough to require mounting the OCS on the sidewalls; however, if OCS supports are required where the right-of-way is constrained and adjacent to other structures, it may be prudent to provide a 10-foot-high (3.048-meter-high) wall to prevent a touching hazard. This situation occurs at the point where Jensen Avenue Bridge crosses over the HST.

The OCS equipment is not part of the civil engineering contract; however, knowledge of its location is required in order to finalize the design of the wall in these areas.

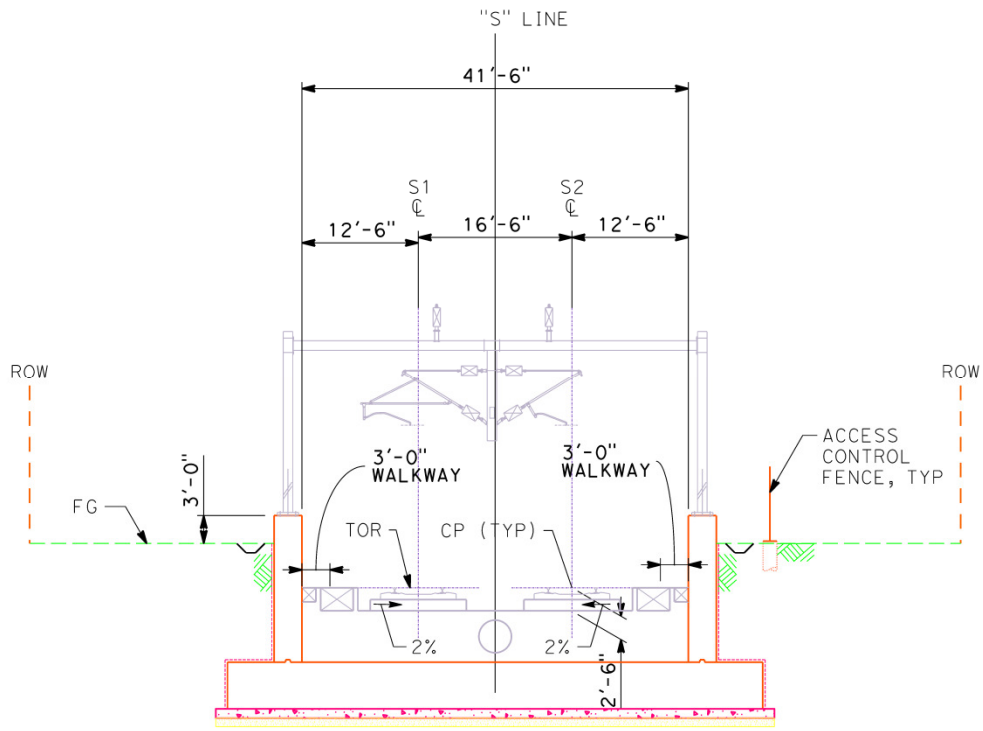

Figure 3.2-3
Typical Section of Un-braced U-Trough

3.2.1 Design Assumptions

3.2.1.1 Locked in Force

In accordance with the design criteria, the U-trough walls have been designed as rigid walls subject to at-rest earth pressures. It is assumed that temporary shoring will not be required, so no additional force would be locked in, apart from the pressures from compaction of the backfill to the structure.

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3.2.1.2 Groundwater Level

Groundwater levels have been assumed to be generally below the level of the excavation. This assumption is supported by the results from the ground investigation, which generally confirmed groundwater to be at a depth of over 60 feet (18.288 meters). In the area of the FEMA-designated flood plain, it is understood that Fresno Metropolitan Flood Control District (FMFCD) has made improvements to address the flood risk. However, the FEMA map remains unchanged and the structure design has consequently allowed for a flood water level of 1 foot (.305 meters) above ground level.

3.2.1.3 Surcharge Pressure

To the east side, BNSF's right-of-way abuts the HST right-of-way. This will preclude use of the land for any other purpose. In areas where the route passes between S Railroad Avenue and Golden State Boulevard, surcharges are possible because the right-of-way width is greater and because the properties taken by the route may include saleable parcels of land or only partial purchases.

The BNSF tracks are generally greater than 100 feet (30.48 meters) from the trench, so the risk of additional surcharge from derailment is low.

At its current location, the UPRR adds little to the force applied to the wall. The maximum contact pressure of the Cooper E80 loading (driving wheels) is 1,882psf at the underside of ties. This pressure was applied to the wall using the Boussinesq formula. The resulting moment effect at the base of the stem was back calculated to an equivalent uniform surcharge. This procedure has demonstrated that a uniform surcharge of 420psf (equivalent to a fill depth of 3.86 feet [1.177m] of fill) would be adequate allowance for the Cooper E80 load and any short-term derailment surcharge.

Where adjacent land is available for potential development, a surcharge of 600psf has been applied as required by TM 2.3.2 clause 6.4.4.

3.2.1.4 Methods of Counteracting Buoyancy

The concept for the Jensen Trench follows the same logic as discussed for the Fresno Grade Separation. However, as the right-of-way width is greater and the deepest trench depth is of the order of 6 feet (1.829 meters) from grade to top of rail, it is possible to construct the heel detail as per draft directive drawing DD-ST-010 (Dated 01-17-12). Using ground investigation data available for the route, it is possible to see that the water levels are very low and there is little chance that the trench will ever be exposed to buoyancy effects from groundwater directly. The presence of hardpan at a depth of 15 to 20 feet (4.572 to 6.096 meters), however, means that the trench may be exposed to local perched water tables. In a significant flood event it is possible that water will be standing in the flood plain area long enough to generate hydrostatic pressures on the base of the trench. The design has considered this possibility, and the maximum width of heel has been used to ensure adequate resistance to uplift. Where there is insufficient resistance, this may be enhanced by the thickening of the base slab, thickening of the walls, or backfilling with CLSM fill so that the full volume of backfill can be mobilized.

3.2.2 Key Constraints

Some constraints apply to the trench as a whole, while others are design and construction constraints that may apply to only one component of the structure. The key constraints on the trench include, but are not limited to:

- The horizontal and vertical clearances from the HST track to the soffit and foundations of the existing E Jensen Avenue bridge
- The horizontal and vertical clearances from the HST track to the soffit and columns of the SR41 bridge
- The provision of a suitable length of track at constant grade to allow for 2 crossovers located within 1 mile (1609.4 meters) of the station.
- Provision of features to exclude floodwater from the route (either walls or levees)

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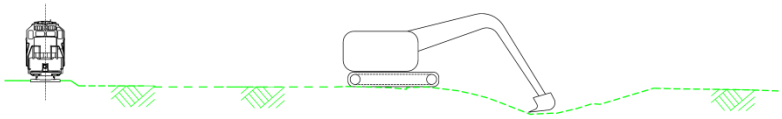
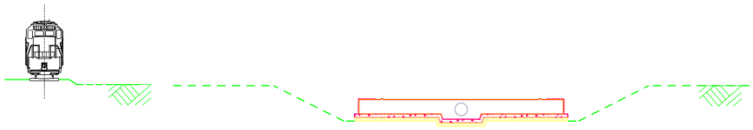
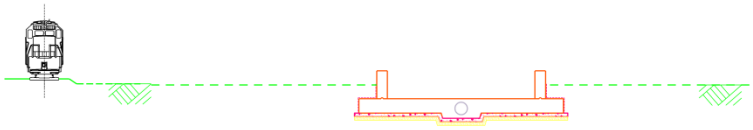
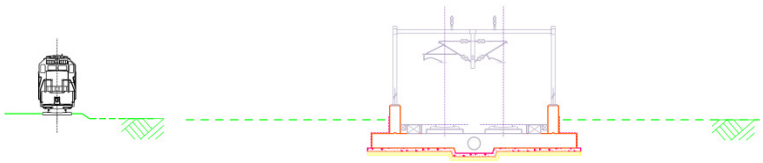
- The width of the right-of-way is generally 130 feet (39.63 meters) As the required minimum width for the track alignment and equipment is 41.5 feet (12.65 meters), this leaves at least 88.5 feet (26.975 meters) for the following:
    - All permanent retaining walls
    - Temporary excavation slopes and shoring required for construction
    - Boundary controls required to delineate the right-of-way boundary (boundary fence, intrusion protection, intrusion detection, etc.)
    - Drainage (swales and channels)
    - Drainage sump access
    - Access stairs

The assumed construction stages are shown in the Table 3.2-2.

Table 3.2-2
Assumed Construction Stages

Construction Stage	Stage Diagram
Stage 0 Excavate to approx. 1 foot (0.305 meters) above foundation level.	
Stage 1 Trim excavation and construct base slab.	
Stages 2 Construct walls and backfill.	
Stage 3 Complete track works.	

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3.3 Limits of Standard Bridge Design and Special Bridge Design

Standard bridge designs are not appropriate to this structure and the structure does not meet the criteria for a Special Bridge.

3.4 Construction Methods Assessment

3.4.1 Main Trench

The design team considered construction of the Jensen Trench in the Constructability Memo at the 15% design stage. The 30% design has undertaken outline calculations based on assumed construction sequence to confirm the adequacy of these assumptions.

3.4.2 Alternatives Considered

During the initial work at the 30% stage, the concept for the trench was reviewed and it was thought possible that an open cut solution may be possible. The potential advantages of an open cut would be simpler construction ad significant construction cost savings. It was agreed that the idea should be investigated further to determine whether there were any reasons not to use a cutting.

This study reported that there were many reasons for adopting a cutting, but also a number of major reasons for not adopting. These for and against were discussed with the EMT and it was agreed that the trench solution should be retained.

The summary of points for and against the cutting are listed in Table 3.2-3 below.

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Table 3.2-3

Comparison of Trench v Cut by Discipline

Discipline	Cutting	Trench	Comments
Alignment	Vertical and horizontal alignment is fixed by external constraints	Vertical and horizontal alignment is fixed by external constraints	Neutral
Structures	A cutting would remove the trench structure entirely. A retaining wall may still be required at Jensen Avenue. An intrusion protection wall may need to be added to protect the crest of the cutting slope.	A trench is a robust solution that satisfies the Authority's requirements but is a substantial structure to construct.	Moderately in favor of cutting option
Geotechnics	Floodwater infiltration requires significant and extensive works to prevent or accommodate inflow. Permanent and enhanced pumping capacity is required.	Novel design methodologies not required. The trench can be designed to accommodate hydrostatic forces using a number of simple and commonly used methods.	Strongly in favor of trench option
Drainage	Significantly increased storage and pumping requirement over baseline	Baseline storage and pumping requirement.	Strongly in favor of trench option
Utilities	Utilities can be accommodated by using pumps, siphons, or diversions.. Design variance required (utility within 8 feet of TOR).	Utilities can be accommodated by using pumps, siphons, or diversions. Design variance required (utility within 8 feet of TOR).	Neutral
Right of Way	Increased right-of-way required	Baseline right-of-way requirement	Moderately in favor of trench option
Operations	Equal protection provision compared to baseline. Increased risk from external derailment impacting HST operations	Baseline collision protection and risk to operations	Moderately in favor of trench option
Cost	Significantly reduced base cost compared to baseline. Additional costs from drainage infiltration works, collision protection wall, levee construction, additional detention basin	Baseline cost	Strongly in favor of cutting option

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	capacity. Additional lifetime costs from cost of pumping and maintenance.		

3.5 Temporary Construction Loadings Considered

During the construction of the Trench, a number of temporary construction loads will be present for short or long periods. As the trench is envisaged as being constructed in open cut, it is unlikely that any of these loads will affect the design of the structure. Should the right-of-way restrict the full excavation in open cut e.g. at Jensen Avenue Bridge it may be necessary to use localized sections of shoring wall. In such cases, the shoring should be designed to allow for the following additional loads where appropriate:

- A surcharge pressure of 600psf applied to areas where construction activity may use land adjacent to the shoring.

3.6 Temporary Construction Easements

Temporary construction easements are required for the construction of the following:

- Connections to the trench drainage sump
- Utilities diversions

The drainage sump is located close to the E Jensen Avenue Bridge and will be connected to the local drainage system via a new detention basin. The basin will be constructed adjacent to the southern end of the trench.

3.7 Traffic or Pedestrian Diversion and Control

The construction of the trench requires the permanent closure of S Railroad Avenue (part), E Florence Avenue, S Sarah Street, S Belgravia Avenue, S East Avenue and S Orange Avenue.

Traffic management will be necessary to accomplish these changes. The contractor will be required to coordinate and plan works in these areas so that traffic disruption is minimized to the satisfaction of the City of Fresno.

For the construction of the U-trough, there will be a need for construction entry and egress points that connect to the road system. It is expected that the majority of excavated material from the U-trough will need to be taken offsite via these egress points. It will be necessary to agree upon the amount, frequency, and operating hours for these entry/egress points with the City of Fresno. It is possible that some of the excavated material could be suitable for re-use as embankment fill for the Fresno Viaduct approaches.

3.8 Drainage Concept

The track drainage within the trench will be carried in a single longitudinal pipe as per the draft directive drawing No (DD-ST-010 dated 01/17/12).Subsequent discussions with PMT/EMT have permitted the drain to be cast into the structural base slab with an additional haunch where the pipe diameter exceeds the slab thickness.

A typical section of this arrangement is shown below:

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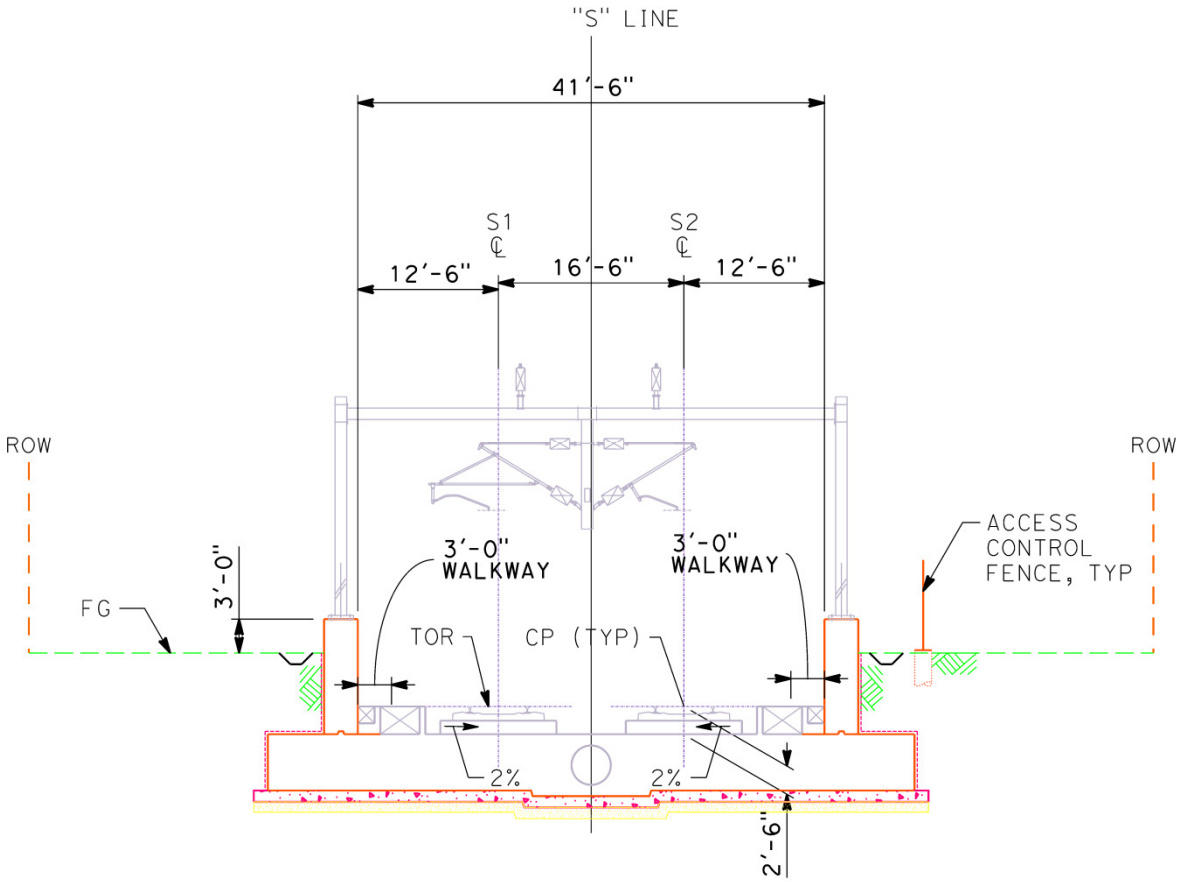

Figure 3.8-1
Typical Section of MSE Embankment

At the low point on the approach to the sump, the drainage pipe reaches a diameter of 3′ – 6″.

3.9 Emergency Egress and Escape Provision

Although not strictly an elevated or underground facility, the team has agreed that it is appropriate to apply the requirements of NFPA130 for emergency escape/egress to the U-trough. This means that escape stairwells are to be provided at maximum 2,500-foot intervals through the box. Stairwells are provided as indicated in Table 2.9-2.

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Table 2.9-2
Stairwell Provisions

STA	Locale	Egress features
111095+00	The south side of South East Avenue	Stairwell is located within the current property boundary of a Propane store.
11120+00	Located adjacent to the drainage sump. Vehicular access from Golden State Boulevard.	Emergency services access to the location of the stairwell will need to be agreed with the owner of the facility. There is space for provision of a turning area for vehicles.

Each stairwell is 10 feet (3.048 meters) wide by 25 feet (7.62 meters) long to allow for the later installation of a staircase.

The staircase is assumed to be 44 inches (1.117 meter) minimum width with 5-foot-wide (1.524 meter-wide) landings if required. As the trench is located in a flood plain, the stairs should rise to the top of wall level before descending back to grade.

3.10 Inspection, Service, and Maintenance Access

The trench structure itself will be a simple massive RC structure with a limited number of movement joints at intervals. There will be no specific provision for inspection or maintenance access other than the general maintenance access to the route.

The drainage sump will require pedestrian access at the surface and access for the installation and removal of pumps. Pedestrian access will also be provided by construction of an access door from the walkway within the trench. Providing this door increases the risk that it may be dislodged by the passage of a train, so it is proposed that this door and the doors associated with the emergency escape stairs should be sliding doors. These may be fitted during a later contract.

Access for pump maintenance and replacement will require a permanent easement and is likely to use the same access point as the emergency egress stairs near to Jensen Avenue.

Movement joints in the walls will be required to limit the effects of temperature and ground movement. These joints are intended to be no more complex than simple cast-in waterstop details.

3.11 Utilities Affected and Disposition

A number of existing utilities cross the route of the trench or are within the proposed right-of-way. Where these can be diverted, the proposed diversion route has been identified on the utilities and structures layout drawings. It has been a principle of this work to divert utilities around or away from the HST route where possible. Where there are specific crossing points that cannot be accommodated in this way, the utility has been diverted under the trench.

Examples of where the trench design may be affected are as follows:

- **86-inch Storm Drain at Church Ave (STA 11086+00)**
  This line is part of the FMFCD network of storm drains. It may be possible to divert the drain around the trench, but the length of diversion is long because of the restricted falls available. It has been assumed, that this utility will be diverted on line by the construction of an inverted siphon. The pipe will be sleeved under the Trench to permit future removal and replacement.
- **30-inch Sewer Line at Church Ave (STA 11086+00)**
  As with the storm drain, there appears to be no reasonable alternative route. It is proposed, that

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the pipe will be diverted on line to pass under the trench and a lift station will be provided to restore the outfall invert levels to match existing. The pipe will be sleeved under the Trench to permit future removal and replacement.

- **48-inch Sewer line at Jensen Avenue (STA 11122+00)**
  This sewer line cannot avoid crossing the route of the HST, there appears to be no reasonable alternative route. It is proposed, that the pipe will be diverted on line to pass under the trench and a lift station will be provided to restore the outfall invert levels to match existing. The pipe will be sleeved under the Trench to permit future removal and replacement.

3.12 Hydrological Issues

These issues are discussed in detail in the Floodplain Impact Assessment Report.

The main impact of the trench design is to ensure that the trench wall is higher than the 100-year flood level in the FEMA designated floodplain. In this area, the 100-year flood level is approximately 1 foot above ground level. Protection against flooding will be provided by the trench wall, which projects above grade level by a minimum of 3 feet (0.914 meters).

3.13 Noise Mitigation and Acoustic Treatment

No specific features have been included to mitigate the noise generated by the trains. The Jensen Trench will be relatively shallow so there will be little or no mitigation of noise from the passage of trains. The Jensen area is largely industrial and this means that there are few noise sensitive receptors in the locality.

3.14 Compliance with Systemwide Bridge Aesthetics Features

No guidance has been provided on aesthetic considerations relating to this structure form.

3.15 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

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4.0 The Fresno Viaduct

The Fresno Viaduct is composed mostly of standard 100- to 120-foot-span (30.48- to 36.576-meter-span) post-tensioned concrete box girders. However, at two locations, the standard spans are unable to provide a solution due to the large skew angle to the obstacle crossed. In these locations, steel truss spans have been detailed.

The right-of-way width for the viaduct is 60 feet (18.288 meters) wide.

4.1 Structure Importance Classification

The design criteria define all structures supporting the high-speed tracks to be primary structures because their reinstatement is necessary to permit resumption of train service after an earthquake.

The majority of the length of the viaduct requires columns that are less than 30 feet (9.144 meters) high. These spans are in accordance with the standard. However, during this design, the range of column heights has been extended to 40 feet (12.192 meters), so all spans are standard except for those supporting the trusses.

These classifications imply the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors — η_I, η_D, and η_R — have been chosen as follows:
    - Importance factor η_I = 1.05
    - Ductility factor η_D = 1.05 for strength limit states (1.0 for conventional designs)
    - Redundancy factor η_R = 1.05 for non-redundant elements, 1.0 otherwise

4.2 Key Design Features and Site Constraints

At the north end of the viaduct the HST is supported on MSE wall as indicated in Figure 4.2.-1 The north abutment terminates the retained section with an RC wall that is composite with two pilasters that support the truss of span 1 (Golden State Boulevard).

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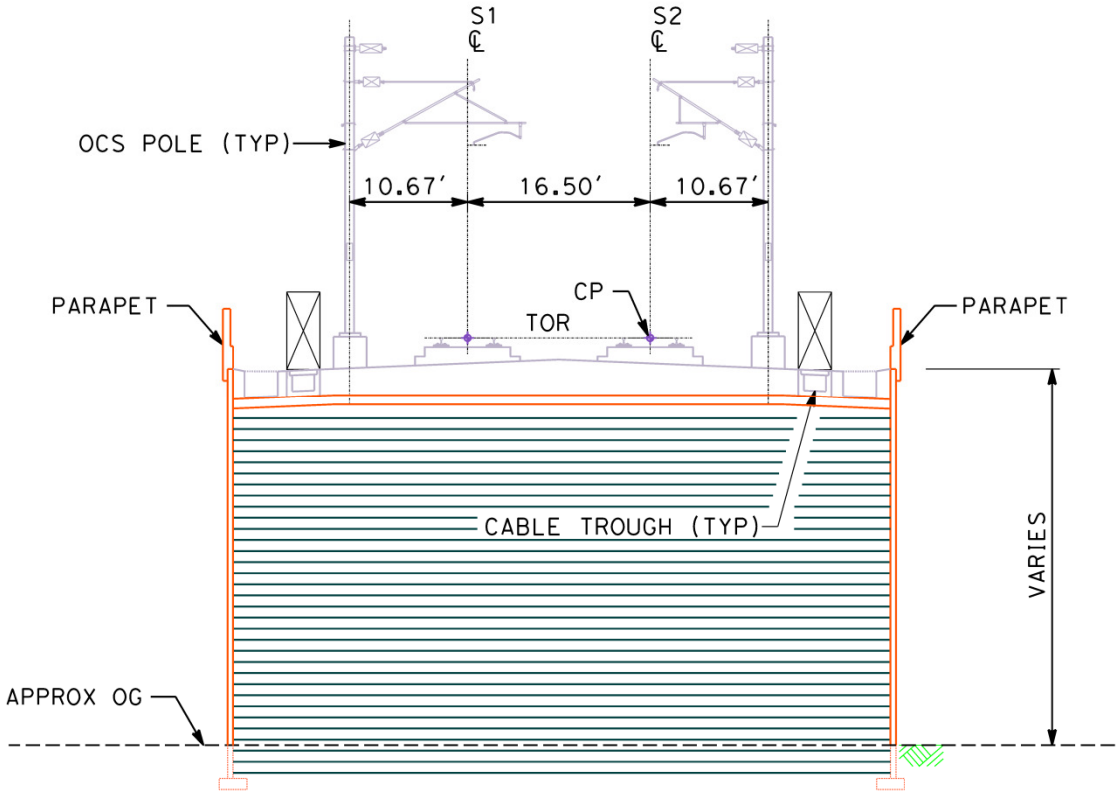

Figure 4.2-1
Typical Section of MSE Embankment

The truss has been configured so that the deck slab acts compositely with the lower truss chord members and is composite with the cross girders.

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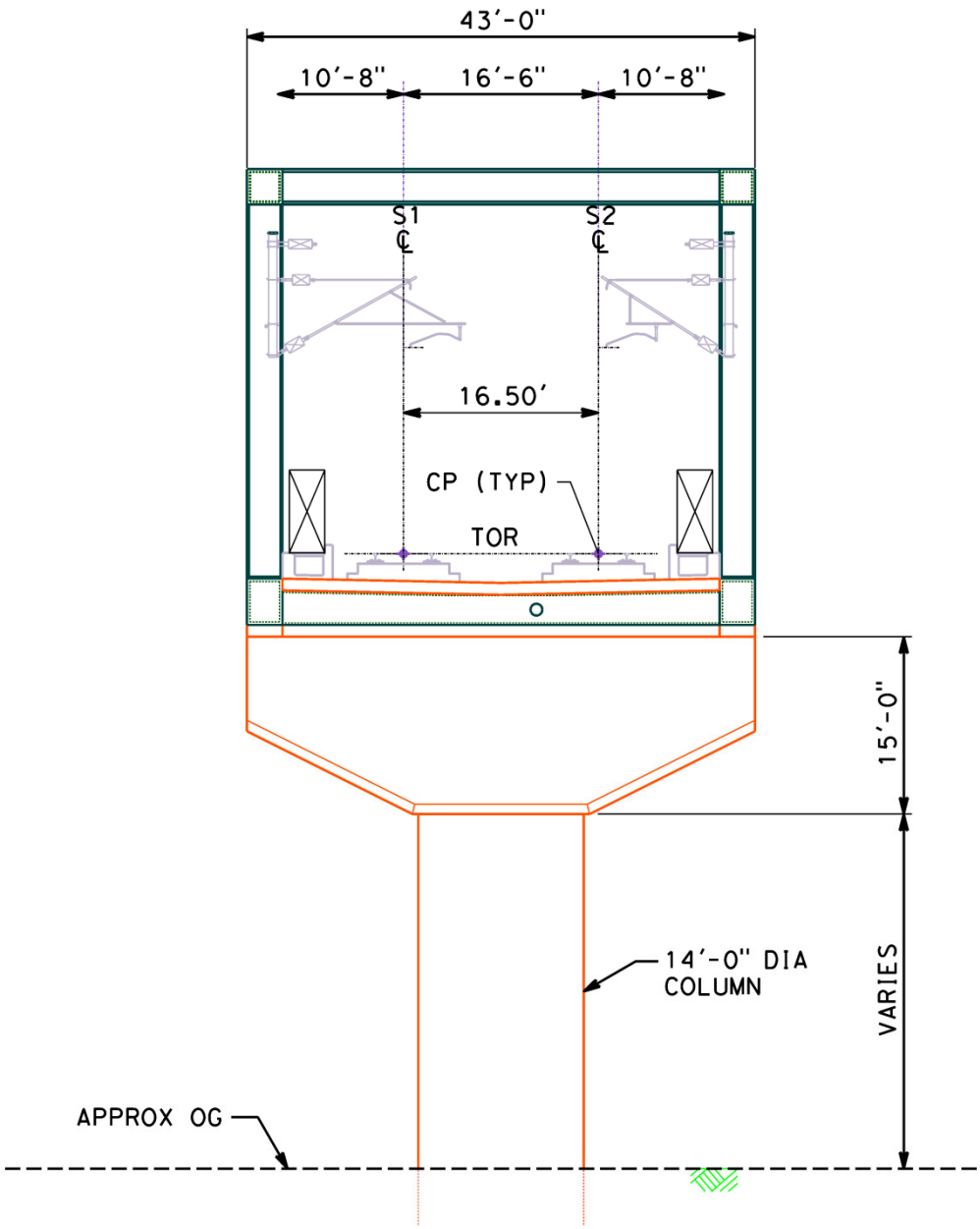

Figure 3.2-5
Typical Section of Truss Span

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4.2.1 Design Assumptions

For design, the requirements of the design criteria dictate that the truss spans at S Cedar and SR 99 cannot be analyzed in isolation. Therefore, a single model of the structure was constructed that combines the Truss spans with 10 standard spans to either side. The model also includes elements to represent each track with the rails modeled as single elements supported by nonlinear springs to represent the track clips.

The purpose of the model is to calculate structure forces and displacement, to model the effect of track structure interaction on the movement joints and on rail stresses, and to provide information to confirm the capacity of the foundations.

The analysis procedure to be followed has been described in the Seismic Analysis Plan.

4.3 Limits of Standard Bridge Design and Special Bridge Design

It is assumed that the standard bridge design is suitable for use on spans 2 to 32 and from spans 36 to 49. Span 1 and spans 33 to 35 are considered in this design.

4.4 Construction Methods Assessment

The three locations where truss structures are planned each have specific features that suggest a particular method of erection is most likely to be used by contractors. This does not rule out other methods of construction. It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here represents a subset of methods that could be used.

4.4.1 Golden State Boulevard

It is assumed that the area of land available to the north of Golden State Boulevard will be used for lay-down and assembly of structure components for the truss. The truss itself will be assembled on the line of the HST prior to the construction of the approach embankment and close to its final height supported on temporary trestles. Once fully assembled the truss will be lifted using a heavy lift vehicle and transported across Golden State Boulevard to be lowered onto its bearings on the far side.

The basic construction sequence is shown in Table 2.2-1 and is as follows:

- Construct temporary trestles as supports (Stage 1 and 2)
- Erect steelwork superstructure (Stage 3 to 5)
- Clear obstructions for heavy lift vehicle (Stage 6)
- Launch truss to contact with heavy lift vehicle
- Heavy lift vehicle transports the end of the bridge across GSB to prepared column bent No. 2 in Southern shoulder of GSB (Stages 7 & 8)
- Dismantle trestles and construct abutment wall and embankment (Stage 9)

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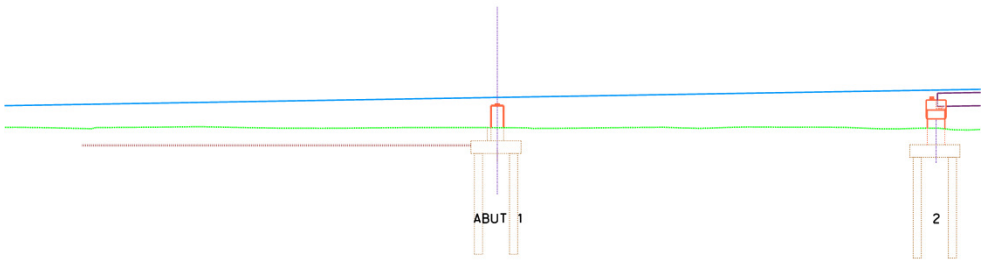
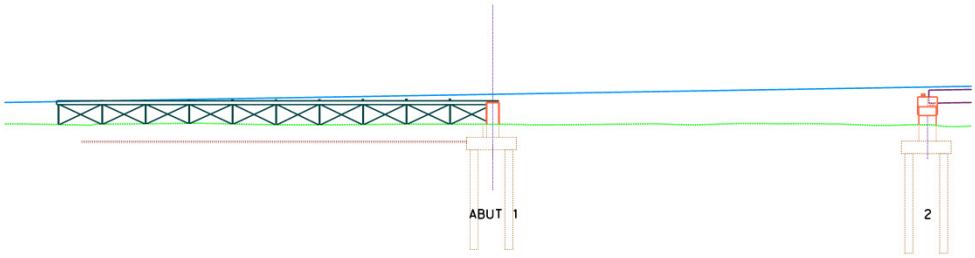
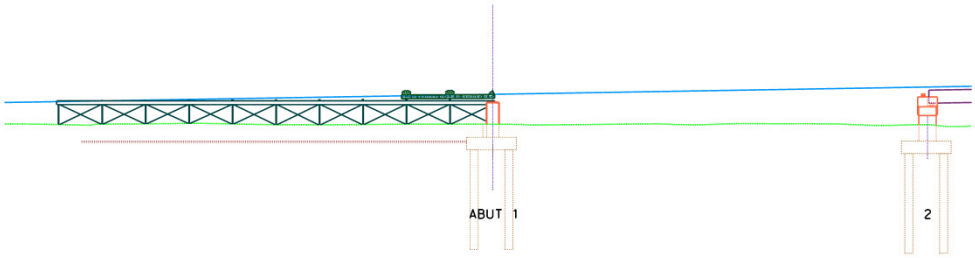
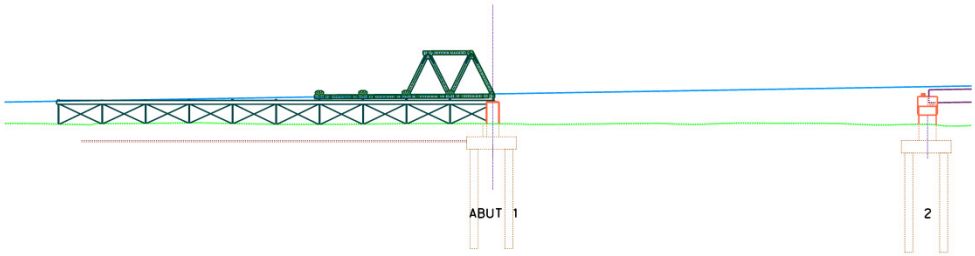
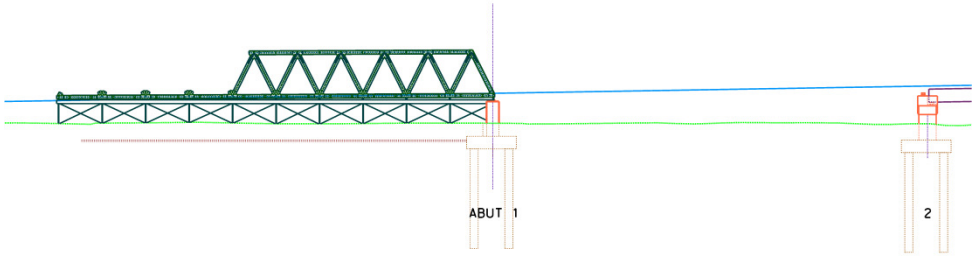
Table 4.4-1
Golden State Boulevard - Construction Sequence

Stage 1	
Stage 2	
Stage 3	
Stage 4	
Stage 5	

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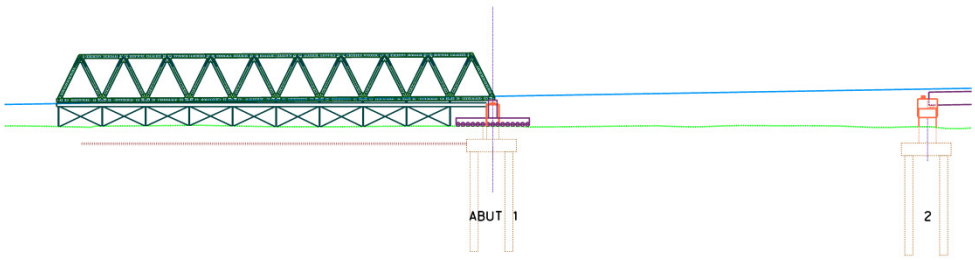
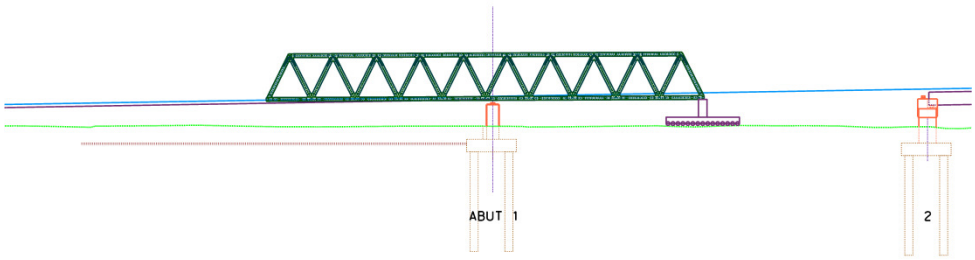
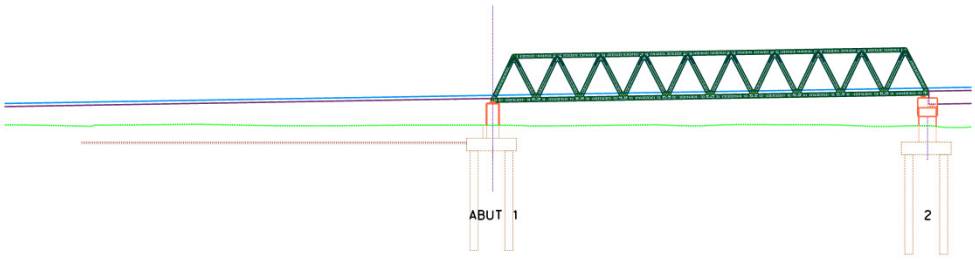
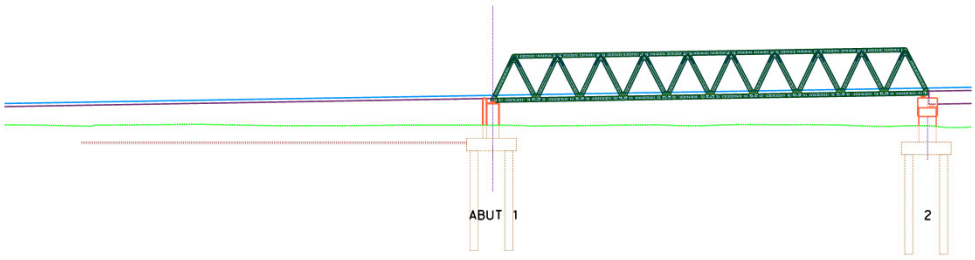
Stage 6	
Stage 7	
Stage 8	
Stage 9	

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4.4.1.1 South Cedar Avenue

The layout of the SR 99 junction and the HST suggests that the land to the east of S Cedar Avenue and within the junction could be used to erect the truss steelwork on trestles close to its final height. In parallel, a temporary slide track would be constructed to span S Cedar Avenue while still permitting traffic flow. On completion, the truss would be rotated or slid across S Cedar Avenue to land on its final bearing supports. This operation would be completed during a full closure of S Cedar Avenue.

Two alternatives for the slide process are indicated in Figures 4.4-1 and 4.4-2. The construction sequence is as follows:

- Construct temporary trestles as supports
- Erect steelwork superstructure
- Erect radial slide track across S Cedar Avenue
- Launch/rotate structure along slide track to land at pier Bent Nos. 33 and 34
- Dismantle slide track and trestles

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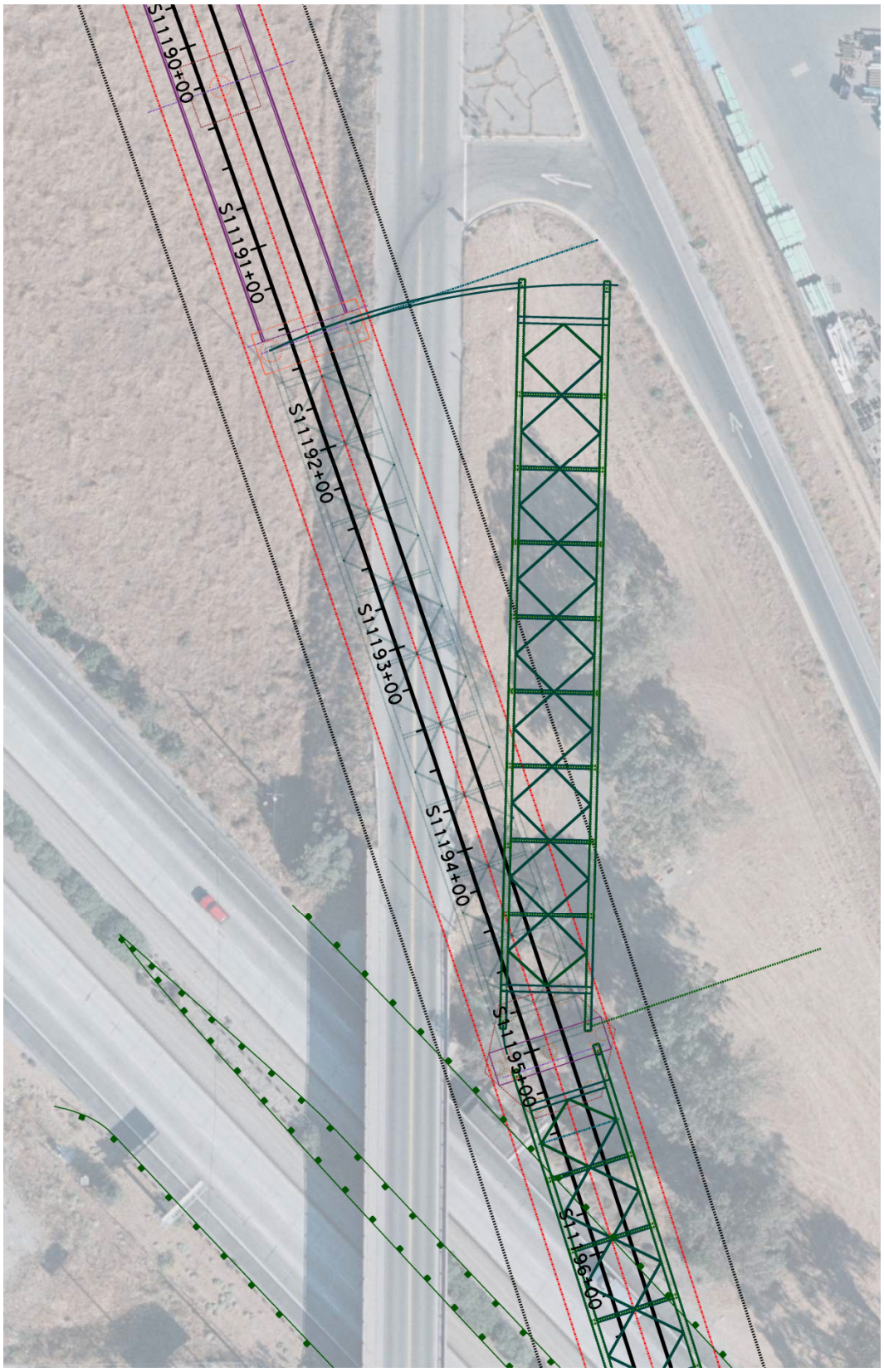

Figure 4.4-1 Erect Truss and Rotate into Position

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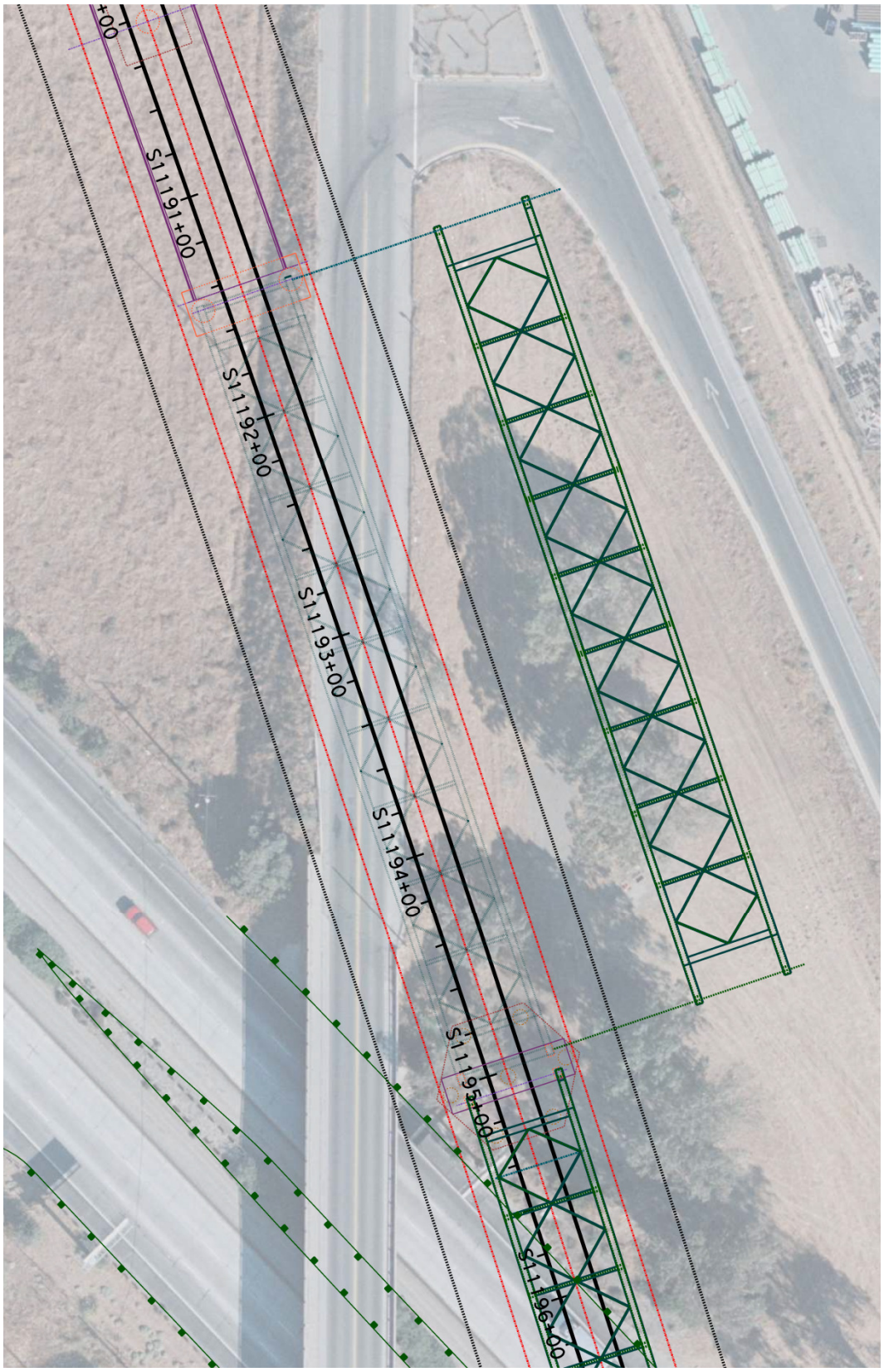

Figure 4.4-2 Erect Truss and Slide into position

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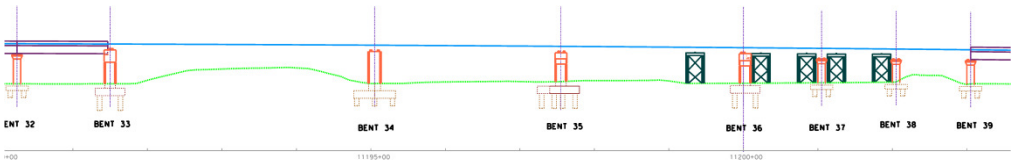
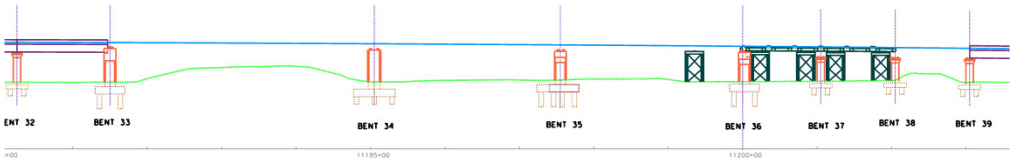
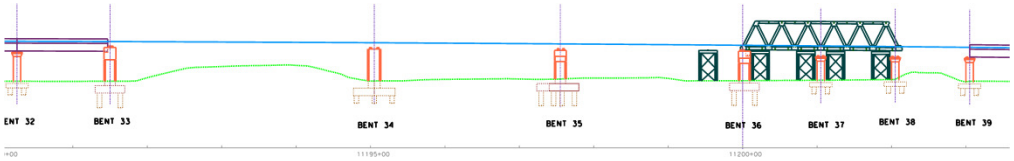
4.4.1.2 SR 99

The layout of the SR 99 junction to the south of the SR 99 permits space for construction between the main route and the southbound on ramp. This space is insufficient for the assembly of the full length of the truss, so incremental launching has been assumed. Discussions with Caltrans have indicated that movement of the structure will not be permitted under live traffic flow. Temporary closures will therefore be required during the launching stages of the program.

The basic construction sequence described below and shown in Table 4.4-2 is as follows:

- Construct temporary trestles as supports between on ramp and main route and also in shoulders and median of SR 99 (Stage 1)
- Erect steelwork superstructure for initial section of truss to the south of SR 99 (Stages 2 and 3)
- Launch steelwork onto temporary trestles (during a temporary closure of all or part of the SR 99) (Stage 4)
- Erect next stage of truss steelwork in southern cutting of SR99 (Stages 5 and 6)
- Repeat until structure complete
- Dismantle trestles (Stage 7)

Table 4.4-2
SR99 Construction Sequence

Stage 1	
Stage 2	
Stage 3	

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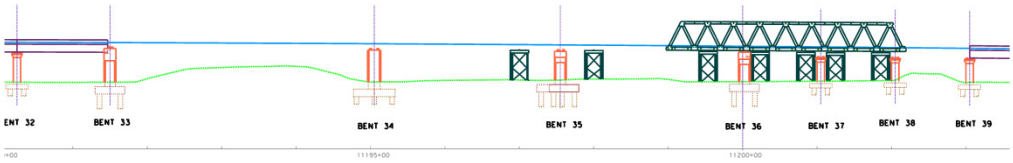
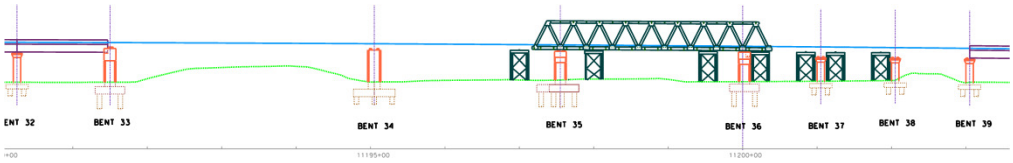
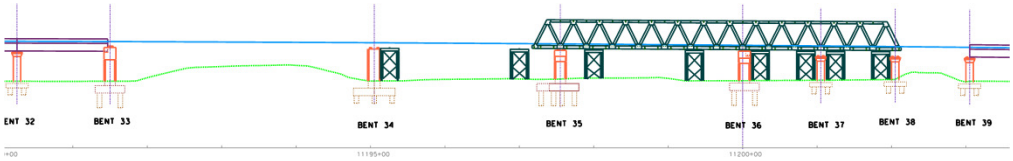
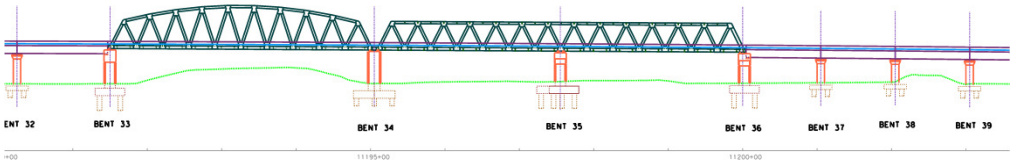
Stage 4	
Stage 5	
Stage 6	
Stage 7	

This method of construction has been used on other projects with similar constraints as can be seen in the following photos from Kuala Lumpur.

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Figure 4.4-3 Erection of Truss Bridge by Incremental Launching

Figure 4.4-4 Finished Structure

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4.5 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described. It is likely that additional temporary bracing may be required to prevent excessive distortion.

For incremental launching, the contractor will be required to provide calculations that support the proposed launching sequence and methodology.

4.6 Temporary Construction Easements

A general temporary construction easement of 100 feet (30.48 meters) width has been indicated for the full length of the viaduct. This should be sufficient for the foreseeable requirements for construction of such a structure.

Additional temporary construction easements may be required for the construction of the following:

- Storm drain diversions
- Colony Canal diversion
- Foundation construction adjacent to Caltrans facilities (SR 99 median and shoulders)
- Connections to the trench drainage sump
- Emergency egress stairwells and emergency access roads

4.7 Traffic or Pedestrian Diversion and Control

Closures of the SR 99 will not be permitted without a viable diversionary route. Caltrans will require the identified route to be clearly signposted for users.

4.8 Drainage Concept

The track drainage for the Fresno Viaduct will be carried from deck level through to a permanent drainpipe fitted within the void of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system. For the steel truss spans, provision will be made for collecting water at track level. This will be conveyed to the ends of the structure via a longitudinal carrier pipe that will be sleeved through the transverse girders of the trusses. At the ends of the truss structures the carrier pipe will discharge to the nearest available downpipe as per the standard spans.

4.9 Emergency Egress and Escape Provision

Provision for emergency escape will be made in accordance with NFPA130. This means that escape stairs are to be provided at maximum 2,500-foot (762 meter) intervals along the viaduct. In assessing locations, the retained embankment has been considered as part of the structure.

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Table 4.9-1
Escape Stair Locations

STA	Locale	Egress features
11152+00	Adjacent to the start of the Span 1	Staircase constructed as part of the MSE retaining wall adjacent to Abutment No 1
11177+00	Adjacent to the entrance to the Valley Wide Beverage facility	Staircase detail in accordance with the directive drawing.
11203+00	Adjacent to existing facility to be demiolished.	Staircase detail in accordance with the directive drawing. Vehicular access via Muscat Avenue.

4.10 Inspection, Service, and Maintenance Access

The standard viaduct will be a simple concrete section that is inspectable from both inside and outside.

The steel truss spans are envisioned to be be constructed using hollow fabricated steel sections for the chords and I-sections for the diagonals. These are unlikely to be inspectable and should be treated internally with corrosion inhibitors or sacrificial thickness of steel. Externally, the truss structures will be painted and inspectable with the use of hydraulic access platforms.

4.11 Utilities Affected and Disposition

The major utilities that cross the route of the viaduct are as follows:

- **Colony Canal at Golden State Boulevard (STA 11156+00)**
  This canal has been culverted, probably at the time of construction of Golden State Boulevard. It runs in twin pipes along the line of the southern shoulder of Golden State Boulevard before turning southward. The southward turn clashes with the location of the foundation for Bent No. 2 of Fresno Viaduct, so it is proposed to divert the canal along the boundary line of the right-of-way to remove the foundation clash.
- **Central Canal adjacent to BNSF spur (STA 11167+00)**
  The Colony Canal is culverted beneath a BNSF spur line and on exit clashes directly with the location of Bent No 12. Rather than divert the canal or move the column, it is proposed that the channel of the canal could be locally increased in width to allow the water to flow around the column. The column pile cap will be sufficiently large to prevent any flow scour issues at the column. This will require FID agreement.
- **Access structure for facility (STA 11218+00)**
  Box structure to be constructed in retained embankment to provide access to existing produce transport facility that would otherwise be cut off from road connection at S Cedar Avenue.

4.12 Hydrological Issues

Hydrological issues are discussed in detail in the Floodplain Impact Assessment Report.

4.13 Noise Mitigation and Acoustic Treatment

No specific features have been included to mitigate the noise generated by the passage of trains. The viaduct parapets are capable of mounting noise barriers if required.

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4.14 Compliance with Systemwide Bridge Aesthetics Features

TM 200.06 provides guidance on non-station structures. The scheme detailed on the drawings and analyzed represents the functional baseline case.

4.15 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

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California High-Speed Train Project

APPENDIX A

Fresno to Bakersfield Package 1A, 1B and 1C

Geotechnical Design Memorandum for Nonstandard and Complex Structures

Prepared by:

URS/HMM/Arup Joint Venture

May 2012

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ABBREVIATIONS / ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASD	Allowable Strength Design
BGL	below ground level
Caltrans	California Department of Transportation
CBC	California Building Code
CBDS	California Bridge Design Specifications
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
CIDH	cast-in-drilled-hole
HST	California High-Speed Train Project
deg	degrees
DWR	California Department of Water Resources
EL	elevation
EMT	California High-Speed Train Engineering Management Team
ESS	Excavation Support System
FOS	factor of safety
g	gravity
GPS	Global Positioning System
GWL	groundwater level
HMM	Hatch Mott MacDonald
HST	High-Speed Train
in	inches
JV	HMM/URS/ARUP Joint Venture
K	USDA Soil Erodibility Factor
ksf	Kips per Square Foot
LL	lower limit
LRFD	Load and Resistance Factor Design
MCL	Maximum Considered Earthquake
mi	miles
mm	millimeters
M_W	Moment Magnitude
$(N_1)_{60}$	Standard Penetration N-Values Corrected for Hammer Energy, Overburden Pressure, and Field Procedures
N_{60}	Standard Penetration N-Values Corrected for Hammer Energy
NAD27	1927 North American Datum
NAVD88	1988 North American Vertical Datum
NA	not applicable
NCL	Non-Collapse Performance Level
NRCS	Natural Resources Conservation Service
OBE	Operating Basis Earthquake
pcf	pounds per cubic foot
pci	pounds per cubic inch
PGA	peak ground acceleration
PMT	California High-Speed Train Project Management Team
SJV	San Joaquin Valley
SPT	Standard Penetration Test
SPT N	Standard Penetration Test Blow Count
Sta	Station
T	Period
TM	Technical Memorandum
UBC	Uniform Building Code

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UL	upper limit
USGS	United States Geological Survey
USDA	United States Department of Agriculture
$(V_S)_{30}$	Average Shear Wave Velocity in the upper 30 meters of soil
yr	year

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1.0 Scope

This geotechnical design memorandum addresses the following nonstandard and complex structures in Package 1A (Sta. 10806+00 to Sta. 10970+00), 1B (Sta. 10970+00 to Sta. 11030+00) and 1C (Sta. 11030+00 to Sta. 11300+00) in Fresno, California:

- Package 1A structures include the Fresno Grade Separation
- Package 1B structures include the Fresno Street Overpass
- Package 1C structures include the Jensen Trench and Fresno Viaduct

Refer to the Section 1.0 of the Trenches, Bridge, and Elevated Structures Report for definitions and locations of nonstandard and complex structures.

2.0 Physiography and Geologic Setting

2.1 Physiography

The topographic provinces included in this report are based on the USACE topographic map (1962) and the physical model GIS layer. These reference sources cover the Fresno-to-Bakersfield portion of the alignment in its entirety, but are out of date. Project-specific surveying data and light detection and ranging data, to be completed in the future, will be used to more fully describe the physiography and topography of this segment of the alignment.

The HST segment between Fresno and Bakersfield is located fully within the San Joaquin Valley (SJV) at an elevation between about 200 feet and 400 feet above sea level (ASL) and passes through gently undulating low relief terrain with shallow natural slopes through the urban areas of Wasco, Shafter and Bakersfield. Through Subsection FB-L (Bakersfield South) from the alignment's intersection with Highway 99 to the terminus of the study area east of Edison the topography gently rises from about elevation (EL) 400 feet to 680 feet ASL as it flanks the southern foothills of the Tehachapi Mountains. The general physiography and topography of the SJV within the study area is shown on Figure 2.1-1. Superimposed upon this large-scale, relatively flat topography is a localized topography of river systems caused by recent incisions. This localized topography is comprised of short steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) and narrow gully-type valleys, depending on their age and the amount of flow.

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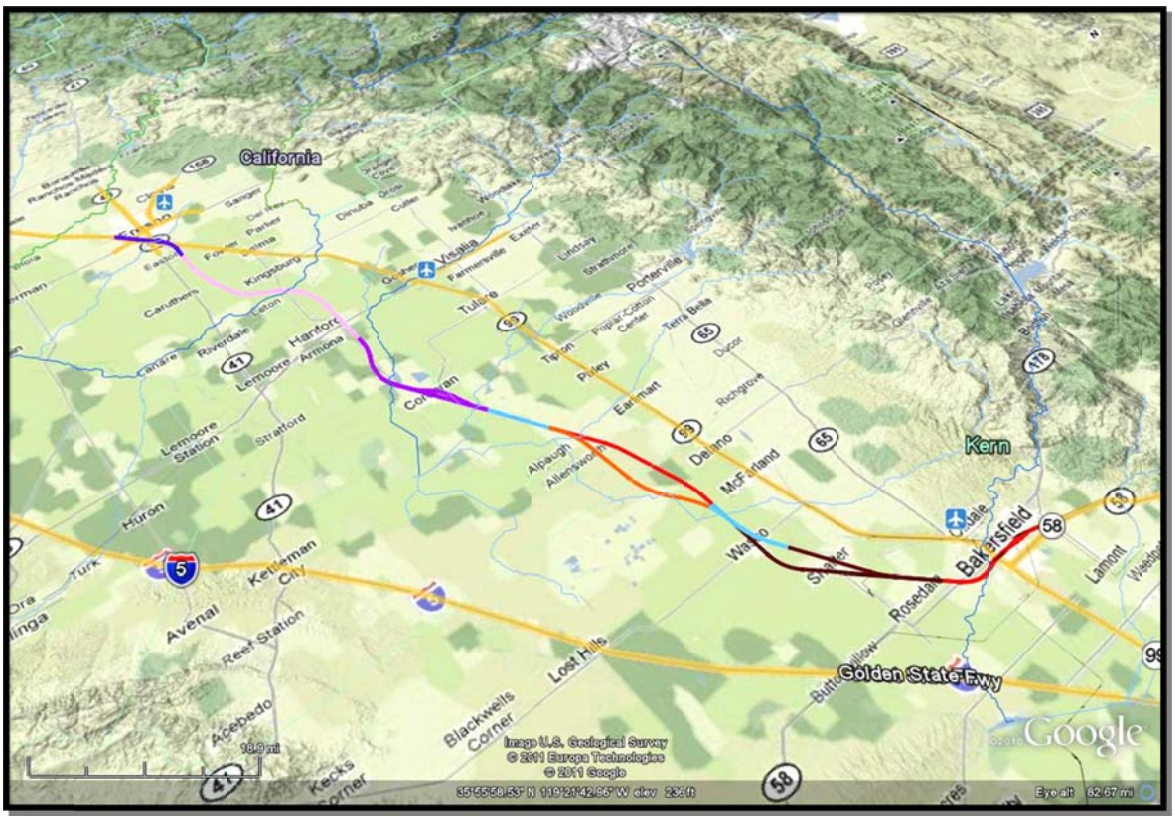

Figure 2.1-1
General Study Area Physiography and Topography (© 2011 Google Inc., 2011)

2.2 Geologic Setting

The San Joaquin Valley (SJV) comprises the southern part of the approximately 400-mile long Great Valley geomorphic province. The Great Valley geomorphic province is an asymmetric synclinal trough that is filled with sediments up to 30,000 feet thick. Infilling with sediments has occurred since the Jurassic period (> 145 million years), providing a large, flat-lying alluvial plain setting in which the FB alignment corridor will be constructed. Bordering the Great Valley are mountain ranges, principally the Sierra Nevada ranges that represent the Sierra Nevada geomorphic province to the east, and the Temblor and Diablo ranges associated with the Coast Ranges geomorphic province to the west (Figure 2.2-1). The Tehachapi Mountains and Klamath Mountains define the southern and northern limits of the Great Valley, respectively.

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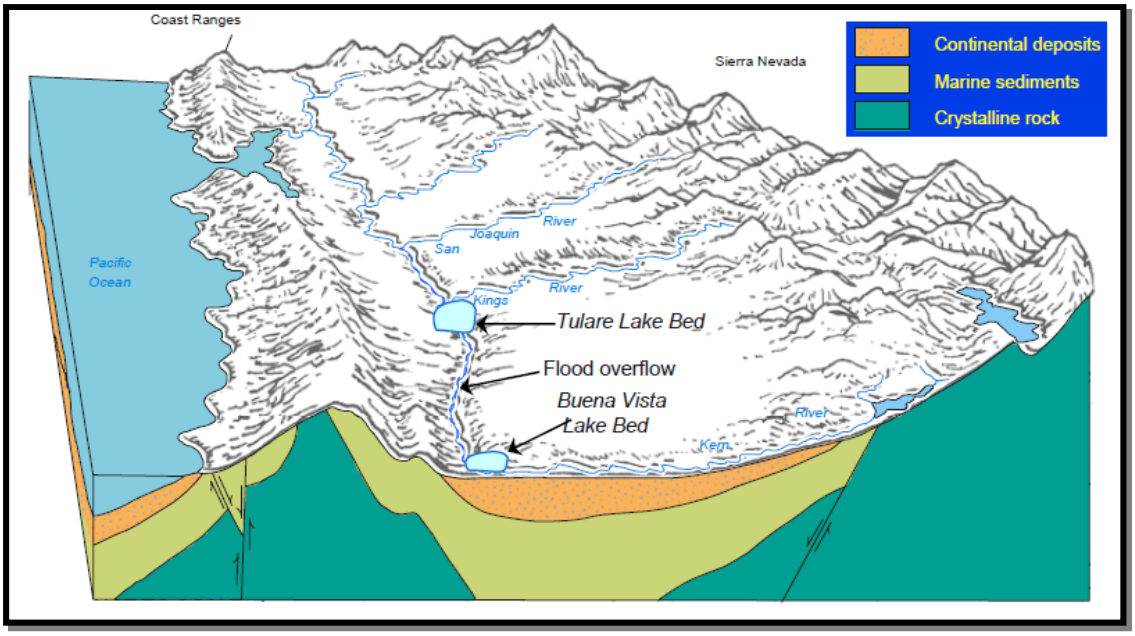

Figure 2.2-1
The Great Valley Geomorphic Province (Page, 1986)

The SJV is a large sedimentary basin, but it provides for a somewhat varied geological setting. Given the asymmetry of the synclinal trough with its axis off center to the west (Norris and Webb 1990), basin sediments are deeper on the western side of the SJV compared with the eastern side. Southwestward tilting of the trough has also contributed to greater thickness of sediments at the southern end of the SJV compared with the northern end. Bedrock geology also differs from the east to west:

To the east of the valley, the Sierra Nevada is composed primarily of pre-Tertiary granitic rocks and is separated from the valley by a foothill belt of Mesozoic and Paleozoic marine rocks and Mesozoic metavolcanic rocks along the northern one-third of the boundary. The Coast Ranges west of the valley have a core of Franciscan assemblage of late Jurassic to late Cretaceous or Paleocene age and Mesozoic ultramafic rocks. (Gronberg et al. 1998)

Such variability is testament to the tectonic environment in which the SJV is located, and the interplay that this tectonic environment has had with the formation of the SJV to the present-day.

3.0 Seismic Setting

The project area is bounded to the west by the seismically active central Coast Ranges. The Coast Ranges are traversed by faults of the San Andreas Fault system, including the San Andreas Fault itself, as well as several active reverse faults. To the south, the study area is bounded by the Wheeler Ridge and Pleito Thrust faults. To the east and southeast, the project area is bounded by the active White Wolf, Garlock and the Southern Sierra Nevada Faults. .

Beyond the Garlock and the Southern Sierra Nevada Faults to the east lie two broad zones of distributed shear: the Sierra Nevada Fault Zone (SNFZ) and the Eastern California Shear Zone (ECSZ). These zones accommodate the movement between the Pacific and North American plates, which has resulted in a number of large, damaging earthquakes during historic time.

More proximate to the Package 1, seismicity is more likely to be governed by faults such as the Great Valley blind thrust fault system as well as the Ortigalita and Nunez faults. The 1983 Coalinga Earthquake

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is attributed to the Nunez fault. To the east lie the Clovis fault (just 12 miles from the alignment) and the Owens Valley, Round Mountain and Hilton faults in the Sierra Nevada. The Owens Valley fault is responsible for one of the largest earthquakes in modern history – the 1872 Owens Valley Earthquake with an epicenter some 90 miles to the east of Fresno.

3.1 Faults and Seismicity

There are no known "capable" or seismically active faults crossing the alignment along the study area as defined by the AP Earthquake Zoning Act. The mapping of faults, however, is dependent on geological information. The depth of sediments in the Central Valley, in excess of 30,000 feet, has restricted the mapping of the structural geology of the bedrock, and therefore faulting is likely to be under-reported in this area. Active and potentially active faults have been mapped in the project vicinity by a number of government agencies and scientific entities. A more comprehensive list of known faults within the vicinity of the project area is presented in Table 3.1-1.

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Table 3.1-1
Capable Faults within the Study Area

Fault Name	Fault Type	Slip Rate (mm/yr)	Distance and Bearing to FB HST Alignment (mi)
San Andreas	Right-Lateral Strike-Slip	20-35	47 mi (or more) W of alignment
Great Valley (Segments 10-14)	Blind Thrust	1.5	25–35 mi (or more) W of alignment
Ortigalita	Right-Lateral Strike-Slip	0.5 to 1.5	64 mi W of Fresno
San Joaquin	Reverse	-	57 mi W of Fresno, slightly E of Ortigalita Fault
O'Neill	Reverse	-	58 mi W of Fresno, slightly E of Ortigalita Fault
Nunez		-	48 mi W of Corcoran
Foothills	Normal	0.1	90 mi NW of Fresno; 40 mi E of Stockton
Round Valley/Hilton Creek	Normal	1	80 mi NE of Fresno
Clovis Fault	-	-	12 mi E of near Clovis
Corcoran Clay Fault Zone	Normal	-	Spanning across the HST alignment from Hanford to the Kern/Tulare County line.
Owens Valley	Right-Lateral Strike-Slip	1.5	85 miles east of the HST alignment
Kern Canyon	Normal	-	66 miles east of alignment at Hanford
Kern Front	Normal	-	30 miles south east of Tule River Crossing
Kern Gorge	Normal	-	14 miles to the northeast of Bakersfield
Southern Sierra Nevada (Independence Section)	Normal	0.1	80 miles W of alignment
Oil Field Fault Zone (North)*	Normal	-	2.25 North of the Alignment
Oil Field Fault Zone (South)*	Normal	-	0.75 miles North
Garlock	Left-Lateral Strike-Slip	2-10	34 miles South East
White Wolf	Left-Lateral Reverse	3-8.5	13 miles South East
Breckenridge	Normal	-	18 miles East
Poso Creek	Normal	-	0
Wheeler/Pleito	Normal	1.4	30 miles South
Edison Fault	Normal	-	Edison Fault Crossing is in Bakersfield and is discussed in Bakersfield to Palmdale alignment. Listed here for information only
Southern Sierra Nevada (Haiwee Reservoir)	Normal	7-14	44 miles East

* These faults appear on the Caltrans 1996 Seismic Hazards Map but have apparently have been derated since they do not appear on the Caltrans 2007 Deterministic Peak Ground Acceleration Map.

Source: SCEC 1999, WGCEP 2007, Caltrans 2007, USGS, CGS

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3.2 Seismic Design Criteria

The system performance criteria approach uses design earthquakes to which HST facilities will be designed. As more devastating earthquakes have a lower probability of occurrence, a probabilistic approach to defining earthquake hazard is used in engineering design. A "return period" identifies the expected rate of exceedance of a given ground motion level. In certain cases, deterministic methods are used to evaluate specific earthquakes that are estimated to produce the most severe ground motion.

3.2.1 Design Earthquakes

For the Fresno portion of the HST alignment, the two design-level earthquakes are defined as follows, in accordance with Technical Memorandum (TM) 2.10.4:

- **Maximum Considered Earthquake (MCE)**: Ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years with 5% damping) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to M_w) of any fault in the vicinity of the structure.

- **Operating Basis Earthquake (OBE)**: Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years with 5% damping).

3.2.2 Performance Levels

At 30% Design, the MCE corresponds to the Non-Collapse Performance Level (NCL). The main objective is to limit structural damage to prevent collapse during and after a MCE. The OBE governs evaluation of the Operability Performance Level (OPL). The primary objective of the OPL limit state evaluations is to verify that the structures respond elastically to the effects of the OBE with no spalling.

3.2.3 Response Spectra and Peak Ground Acceleration

Procedures for defining the seismic design parameters for the HST are defined in TM 2.10.4. The project management team (PMT) and seconded staff from the Regional Consultants developed site-specific, spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield. The alignment was divided into eight zones based on shear wave velocities published by the United States Geological Survey (USGS) as well as the variations in the calculated ground motion parameters. This section of the alignment falls within Zone 4 of PMT's study area. Table 3.2-1 summarizes seismic design parameters provided by PMT for 30% Design.

Table 3.2-1
30% Design Seismic Parameters

Seismic Parameter	OBE	MCE
Peak Ground Acceleration (g)	0.08	0.25
Moment Magnitude (M_w)	6.7–7.9	7.1–7.9

The PMT also developed spectrally matched acceleration response spectra for 30% Design for Zone 4 (see Figure 3.2-1). Figure 3.2-1 shows design response spectra for both vertical and horizontal ground motions. Peak Ground Accelerations (PGAs) in Table 3.2-1 were taken as the spectral acceleration at the period (T) of 0.01 seconds.

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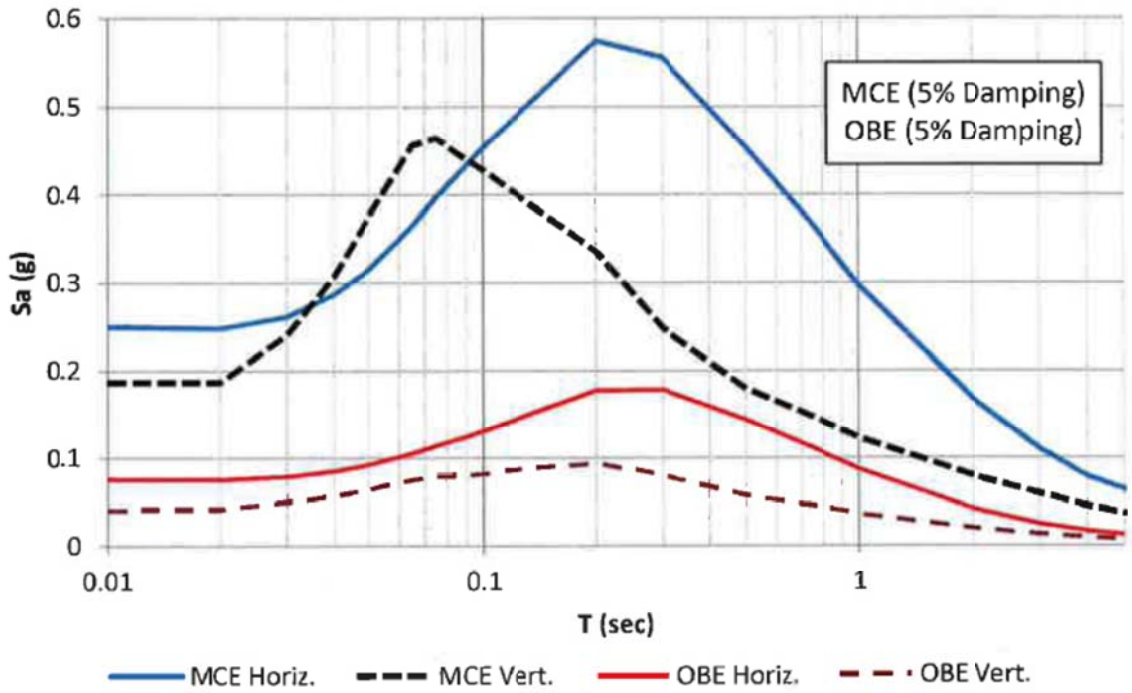

Figure 3.2-1
Design Response Spectra (SC Solutions, 2011)

4.0 Geologic and Seismic Hazards

Table 4.0-1 provides a summary of the geologic and seismic hazards pertinent to the study area. A more detailed summary of geologic and seismic hazards is provided in the Fresno to Bakersfield Geologic and Seismic Hazard Report (URS/HMM/Arup 2012).

Table 4.0-1
Summary of Geologic and Seismic Hazards for Fresno Section

Location	Surface Ground Rupture	Seismically Induced Ground Deformation	Seismically Induced Flooding	Volcanic Hazards	Land Subsidence	Areas of Difficult Excavation	Unstable Soils	Soil Corrosivity	Erodible Soils	Seasonal Flooding	Slope Instability	Hazardous Minerals	Abandoned Mines and Karst
Fresno	L	L	L	L	L	M	L-H	M-H	M	M-H	L	L	L

• N = Negligible L = Low
M = Moderate H = High
Table excerpted from the Geologic and Seismic Hazards Report (URS/HMM/Arup 2012)

The following discusses provided more detail on several of the geologic and seismic hazards with moderate risk levels.

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4.1 Difficult Excavation

Based on the studies of the near-surface (less than 5 feet deep) soils by the USDA and NRCS, hardpan soils should be expected in the Package 1 study area. The San Joaquin soil series has a hardpan present between 12 to 48 inches below the surface (USDA and NRCS, 2008) and is identified between W Clinton Avenue and Belmont Street. Hardpan in this area can vary from 4 to 17 inches thick. Similar hardpan may be encountered in the Greenfield series (W Belmont Avenue to Tuolumne Street), which makes up 8% and 4% of the Fresno section, respectively.

Based on the information from the GI, this hardpan will be encountered deeper than 5 feet during excavations for the proposed Fresno Grade Separation, Jacked Box, Jensen Trench, and during excavation for new foundations for the proposed Fresno Viaduct and other ancillary structures. Where encountered, it is difficult to excavate with conventional equipment such as track mounted excavators, and scrapers. Hardpan is also difficult to excavate with conventional solid flight augers.

Hardpan is difficult to excavate when dry but may lose its strength and become easily remolded when saturated leading to reduced bearing and lateral capacity. For this reason, hardpan within 5 feet of the ground surface should not be relied upon for support of permanent structures.

4.2 Expansion Potential

The 30% Design ground investigation studied soil expansion potential. Results of Atterberg limits tests indicate Alluvial Fan and Existing Fill have a low degree of shrink and swell potential associated with a mean Plasticity Index of less than 18 percent (Holtz 1959 and USBR 1974).

The shrink/swell potential can also be estimated using soil dry density correlated from CPT data and laboratory moisture content tests, and liquid limit results from laboratory tests. The results are shown in Figure 4.2-1.

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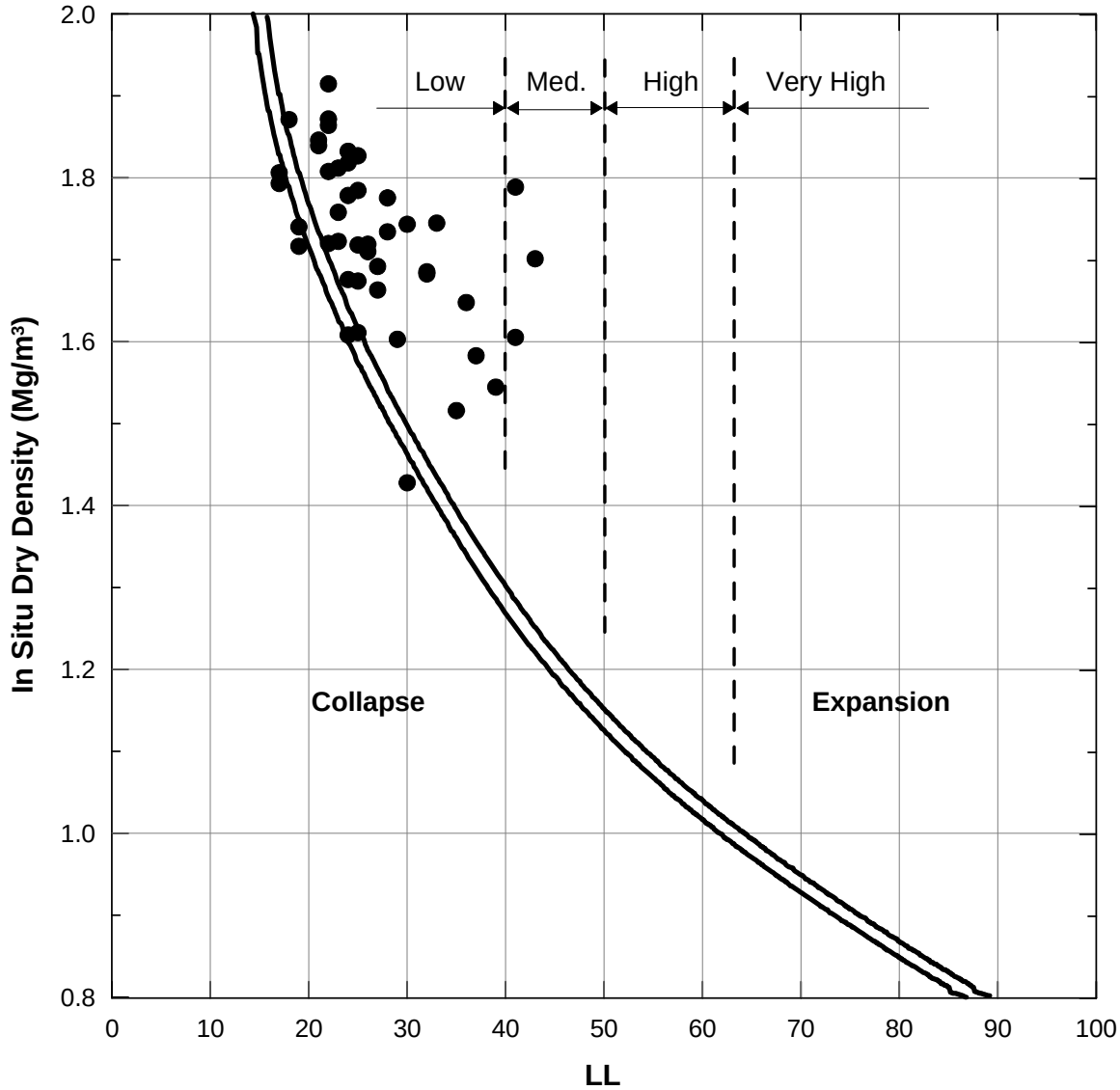

Figure 4.2-1
Guide to Collapsibility, Compressibility, and Expansion
(Mitchell and Gardner 1975, and Gibbs 1969)

4.3 Corrosion Potential

The soils along the study area have a moderate to high risk of corrosion for uncoated steel and a low to moderate risk of corrosion of concrete (USDA and NRCS, 2008). The San Joaquin soil series, which makes up approximately 30% of the study area and is generally found between W Clinton Avenue and W Belmont Avenue, has a high risk of corrosion for uncoated steel and a moderate risk of corrosion for concrete. This San Joaquin soil is generally composed of a combination of silty sand, silty clay, and lean clay and has a soil pH ranging from 5.6 to 6.5 near the surface and 6.1 to 7.3 to a depth of 5 feet, with zero soil salinity.

The Hanford soil series, which makes up approximately 24% of the study area and is generally found between W Franklin Avenue and Fresno Street, has a moderate risk of corrosion for uncoated steel and a

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low risk of corrosion for concrete. The Hanford soil series is composed of a combination of silty sand and silt, and has a soil pH from 6.1 to 7.3 and zero soil salinity.

Table 4.3-1 provides a description of the corrosivity risk level within the study area. This conclusion has been supported by the results of the 30% Design ground investigation which found low corrosion potential based on laboratory testing for corrosion by ASTM methods, as shown in Table 4.3-2.

Table 4.3-1
Risk of Corrosion for Uncoated Steel and Concrete based on NRCS and USDA Information

Location	Risk of Corrosion Uncoated Steel	Risk of Corrosion Concrete
W Clinton Ave to Ventura Ave	Moderate–High	Low–Moderate

Table 4.3-2
Baseline Corrosion Parameters

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation	Assumed Baseline
Minimum Resistivity (ohm-cm)	ASTM G 57	37	1,130 to 20,900	6,526	4,457	**2,000**
pH	ASTM D 4327	37	6.9 to 8.4	7.6	0.3	**6.9**
Chloride (ppm)	ASTM D 4327	37	6.2 to 124.0	15.0	20.5	**124**
Sulfate (ppm)	ASTM D4327	37	0.8 to 273.1	28.8	47.5	**273**

4.4 Hydrocompaction and Collapse Potential

Detailed evaluations of hydrocompaction of soils along the study area could not be found, likely because the HST alignment is located outside areas in the SJV that have historically experienced hydrocompaction. Moreover, outside of the urban corridors, the alignment traverses through heavily irrigated farmlands. Although none has been recorded, any hydrocompaction through irrigated farmlands has likely long since been exhausted. Figure 4.2-1 shows minimal potential for collapse for the soils in the Package 1 study area.

4.5 Subsidence

The SJV has experienced settlement due to groundwater and oil extraction.. The area may have experienced land subsidence in the early 1900s when subsidence was prevalent in the SJV. However, no significant land subsidence is known to have occurred in the last 50 years as a result of land development, water resources development, groundwater pumping, or oil drilling in the area of the structures covered in this report. A Global Positioning System (GPS) control network has been established throughout the Plan Area. This control network consists of more than 20 control points that are tied to the High Precision Grid Network using the 1988 North American Vertical Datum (ASL). This control network is utilized to survey existing local benchmarks to monitor subsidence (FID et. al., 2006).

A cursory evaluation of subsidence along the alignment was made by comparison of the current ground surface elevations along the alignment — taken from the Fresno to Bakersfield 30% Record Set Plan & Profile Sheets — to ground surface elevations from Google Earth.

Based on this evaluation, there does not appear to be any subsidence within the study area. Considering the magnitude of groundwater declines within the City of Fresno and the ongoing subsidence in

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neighboring communities, largely due to groundwater abstraction, the lack of documented subsidence is rather remarkable. However, communities in the SJV that are experiencing substantial groundwater-abstraction-induced subsidence are also underlain by substantial clay deposits including the Corcoran Clay, whereas Fresno is not. Based on past ground response performance, the risk of subsidence within the study area is considered low.

5.0 Geotechnical Data

This Appendix section presents geotechnical data and design conclusions in support of Package 1A and 1B structures based on historical geotechnical data. The JV compiled the historical data largely from limited Caltrans sources. The 30% Design level geotechnical investigation (GI) was completed after the design of Package 1A and 1B structures. An effort was made to verify that the conclusions made based on the Caltrans data were appropriate based on the additional data compiled from the GI and subsequent laboratory testing.

Section 5 and 6 of this Appendix present the historical information for Package 1A and 1B, with only minor modifications. For instance, where a previous edition indicated that site-specific geotechnical information is not available, the text is revised to reflect the existence of new data.

The JV has implemented the geotechnical parameters developed from the GI for use in Package 1C design. A thorough presentation and discussion of the 30% Design parameters developed from the GI is not included in this Appendix. For details relating to the data acquired, please refer to the Record Set 30% Design Geotechnical Data Report (JV 2012). Package 1 Design parameters were developed by the JV and presented in a baseline report to the PMT/EMT (JV 2012). Many of the baselines presented in that document are considered conservative, as they were selected as mean minus one sigma design parameters. For the design of the Package 1C structures, geotechnical design parameters were developed as prescribed methods in programmatic Technical Memoranda and, where appropriate, AASHTO and Caltrans standards.

5.1 Geotechnical Investigations

No site-specific geotechnical investigation was available for the preliminary design of the structures (i.e., trenches, bridges, and elevated structures) in Package 1A and 1B of the Fresno to Bakersfield Section of the HST alignment. The preliminary design for structures in Package 1A and 1B is based on historical geotechnical data along the HST study area in Fresno, mainly along State Routes 41, 43, and 99, as well as those of the City of Fresno residential development projects. However, the preliminary design of the Fresno Viaduct in Package 1C was performed based on data from the site-specific geotechnical investigation. Parameters used in design of Package 1A and 1B have been checked for consistency with the ground conditions encountered during the GI.

Caltrans has completed a large number of projects along the HST study area in Fresno, mainly along Routes 41, 43, and 99. The project dates range from 1953 to 1997. For each project, several boreholes were drilled, logged, and plotted on a cross section.

In total, data is available from about 350 boreholes within 2 miles of the alignment; however, many of the borehole logs offer little detail about the soils beyond the potential depth of the viaduct foundations. The boreholes extend to a maximum depth of 121.8 feet below ground level (BGL), with an average borehole depth of 42 feet BGL.

The City of Fresno provided 14 geotechnical data reports for residential development projects. These reports are dated between 1987 and 2007 and contain 135 borehole logs. The boreholes range in depth from 10 feet to 21.5 feet BGL.

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5.2 Stratigraphy

Generally, the stratigraphy within the study area consists of alternating layers of poorly graded sand (SP), silty sand (SM), silt (ML), and silty clay (CL). More specific information, based on historical ground investigations, is presented below.

Where Route 99 and W Clinton Avenue cross the alignment in northern Fresno, the ground surface at time of drilling was around elevation (EL) 295 feet (ASL) (Caltrans 1953a, 1953b, 1990). Standard Penetration Test (SPT) N-values (blow counts) in granular soils above EL 265 feet (between 0 to 30 feet deep) generally range from 4 to 30, which corresponds to loose to medium dense soil. Below EL 265 feet, N-values are generally greater than 30, corresponding to dense to very dense soil. However, several boreholes show high N-values near the surface, which may be due to the hardpan. The stratigraphy within the study area can be described as follows:

- Ground level to 15 feet BGL (EL: 285–270 feet ASL) – alternating beds of loose to very dense poorly graded sand, silty sand, and silt (SPT N 4–100) including possible hardpan at shallow depth
- 15–25 feet BGL (EL: 270–260 feet ASL) – beds of stiff to hard silt (SPT N 16–60)
- 25–45 feet BGL (EL: 260–240 feet ASL) – alternating beds of dense to very dense poorly graded sand, silty sand, silt and poorly graded sand with silt (SPT N 31–90)
- 45–70 feet BGL (EL: 240–225 feet ASL) – alternating beds of dense to very dense poorly graded silty sand, stiff to hard silt, and low plasticity silty clay (SPT N 27–170)
- Groundwater was not recorded

Figure 5-2.1 shows the locations of existing geotechnical data from Caltrans borings near where the proposed alignment intersects Route 180.

Figure 5.2-2 shows the variation of SPT blow counts for some of the boreholes at the intersection of W Nielsen Avenue and Route 99 about a half a mile east of the alignment, near the northern end of the Fresno Grade Separation. Figure 5.2-3 shows the variation of SPT blow counts for some of the boreholes between W Nielsen Avenue and Route 180 on Route 99.

Please note that SPT N-values in Figures 5.2-2 and 5.2-3 have been converted to N_{60}. Values greater than 100 have been omitted from the figures.

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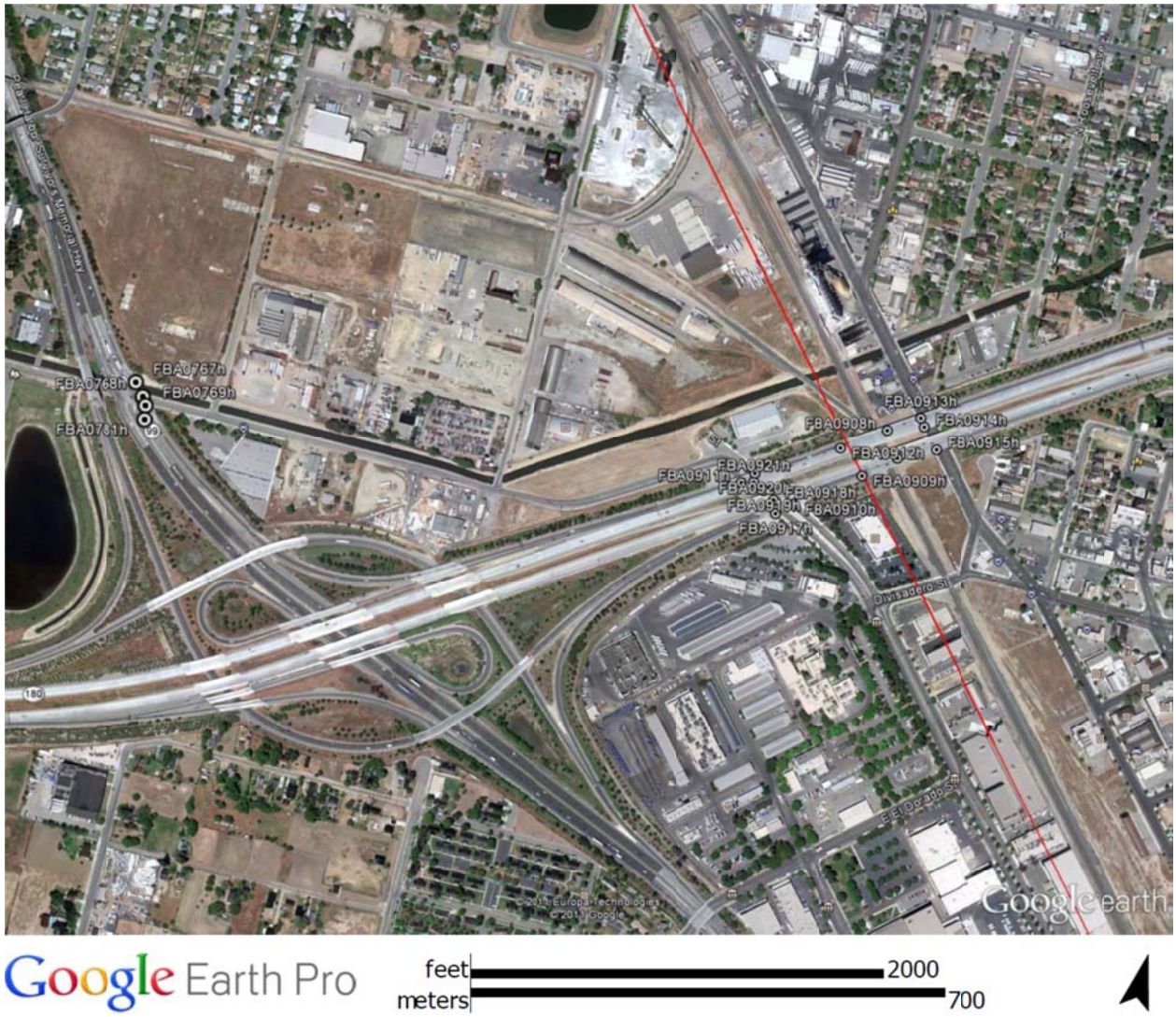

Figure 5.2-1
Map of Historical Geotechnical Data near Route 180 Intersection (© 2011 Google Inc., 2011)

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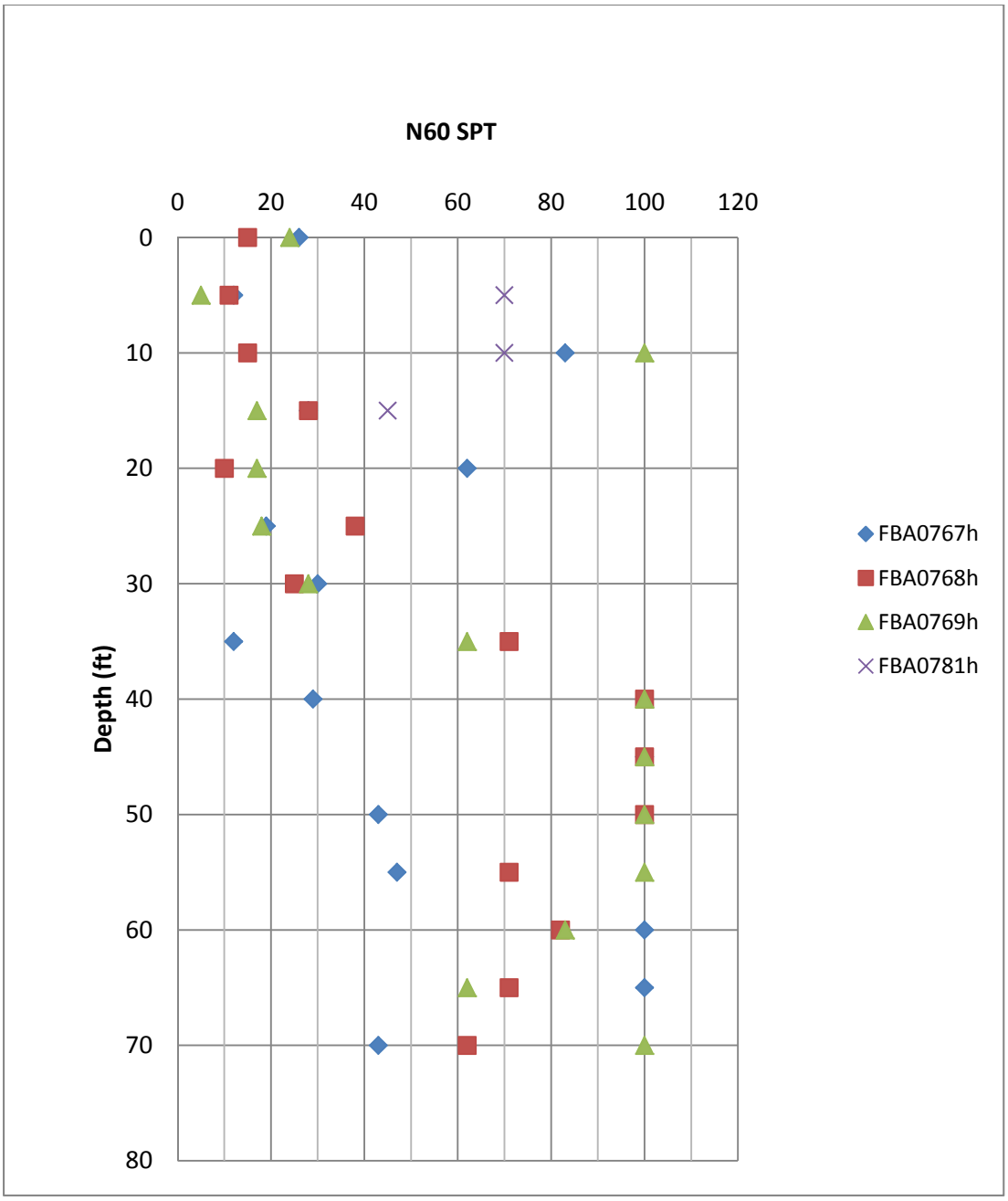

Figure 5.2-2
Historical N60 Values at Route 99 and W Nielsen Ave

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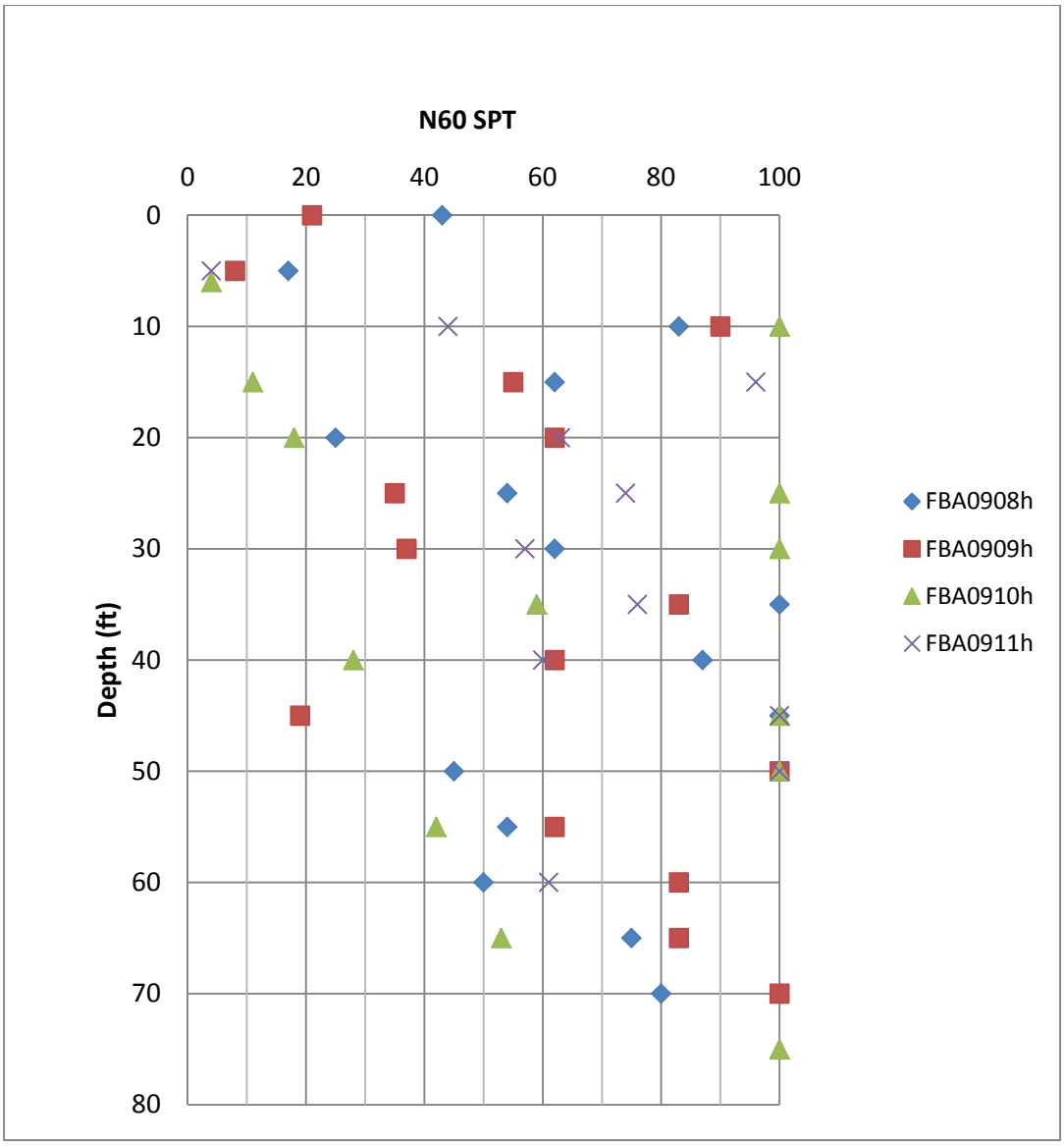

Figure 5.2-3
Historical N60 Values at Route 99 between W Nielsen Ave and Route 180

Near where Route 41 crosses the alignment (slightly south of the study area), the ground surface at time of drilling was around EL 285 feet (Caltrans, 1963). N-values vary greatly in this area, ranging from 4 to 100. In the top 10 feet, they tend to be between 4 and 20, which correspond to very loose to medium dense soil. Below 10 feet, density ranges from medium dense to very dense soil. Some boreholes show a clayey silt layer at approximately EL 245 feet (40 feet). The stratigraphy within this reach of the alignment can be described as follows:

- Ground level to 15 feet BGL (EL: 285–270 feet ASL) – alternating beds of loose to very dense poorly graded sand, silty sand, and silt (SPT N 4–100) including possible hardpan at shallow depth
- 15–25 feet BGL (EL: 270–260 feet ASL) – beds of medium dense to very dense silt (SPT N 16–150)

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- 25–45 feet BGL (EL: 260–240 feet ASL) – alternating beds of dense to very dense poorly graded sand, silty sand, silt, and poorly graded sand with silt (SPT N 31–90)
- 45–60 feet BGL (EL: 240–225 feet ASL) – alternating beds of dense to very dense poorly graded silty sand, silt, and low plasticity silty clay (SPT N 27–70)
- Groundwater was not recorded

Figure 5-2.4 shows the locations of existing geotechnical data from Caltrans borings near where the proposed alignment intersects Ventura Avenue. Please note that SPT N-values in Figure 5.2-4 have been converted to N_{60}. Values greater than 100 have been omitted from the figure.

Figure 5.2-5 shows the variation of SPT blow counts with depth for some of the historical boreholes at the intersection of Highway 99 and Ventura Avenue.

Figure 5.2-4
Map of Historical Geotechnical Data Near Proposed Alignment Intersection with Ventura Ave
(© 2011 Google Inc., 2011)

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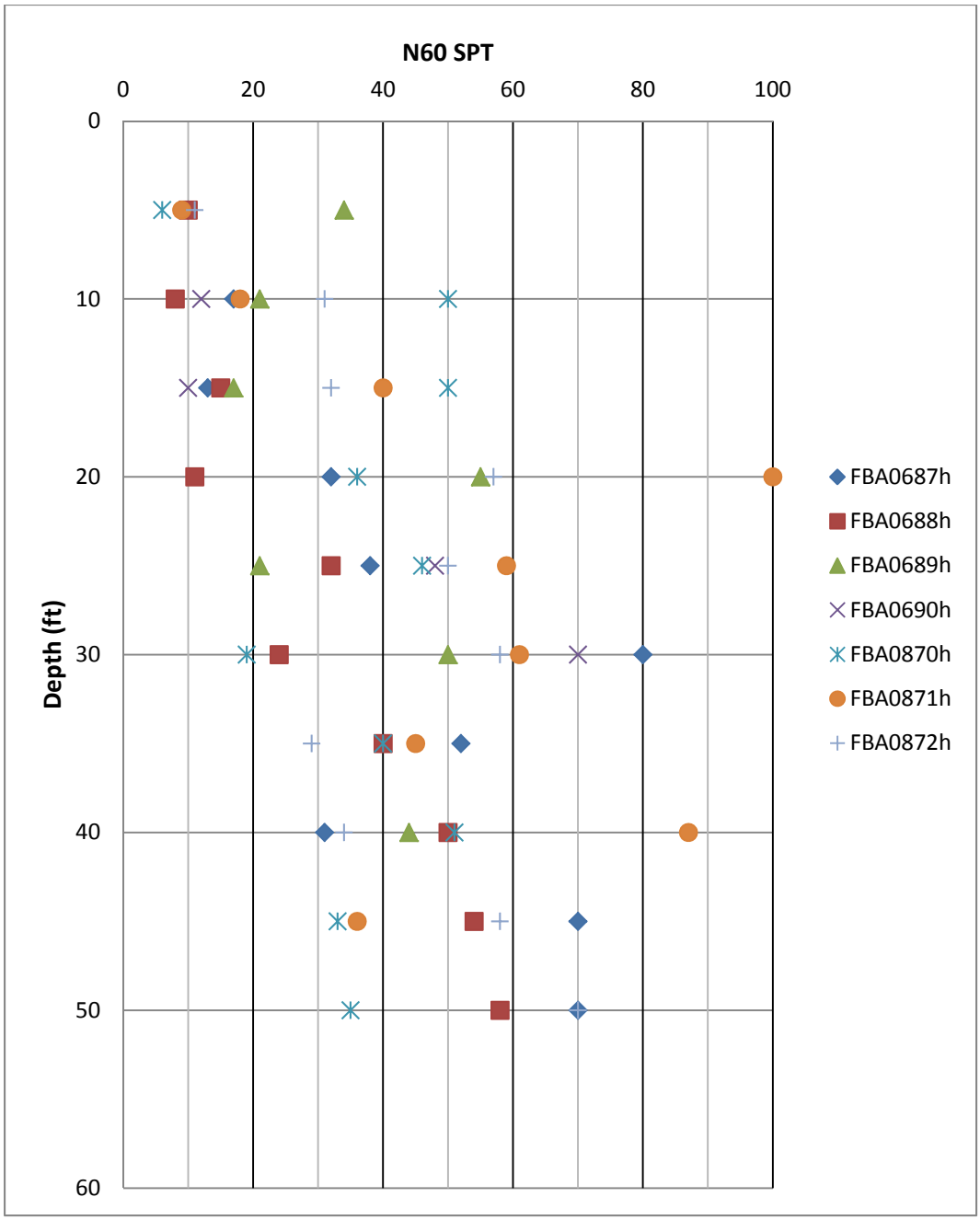

Figure 5.2-5
Historical N60 Values at Route 99 and Ventura Ave

5.3 Laboratory Testing

While no laboratory data associated with the Caltrans investigations are available, the City of Fresno geotechnical data reports contain a moderate amount of laboratory data. Since the City of Fresno boreholes only extend to a depth of approximately 20 feet BGL, all of the historical laboratory data is above this level. Table 5.3-1 summarizes the available historical laboratory test data.

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Table 5.3-1
Historical Laboratory Testing Data

Test	Minimum	Maximum	Mean	No. of Results
Moisture Content (%)	1.7	33.8	11.3	180
Dry Density (pcf)	77.8	132.2	112.0	154
Fines Content (%)	3.8	99.0	42.2	31
Cohesion (psf)	0	500	300	7
Friction Angle (deg)	26.3	40.0	33.4	9
Void Ratio	0.35	0.88	0.56	26
Peak Shear Strength (ksf)	0.3	2.4	1.4	15
Residual Shear Strength (ksf)	0.7	2.2	1.4	9
Coefficient of Virgin Compression, Cc	0.02	0.06	0.04	7
Coefficient of Recompression, Cr	0.01	0.01	0.01	7
pH	6.4	9.6	8.0	2

5.4 Groundwater Levels

Historically, the groundwater table elevation has fluctuated but has generally experienced a depletion of about 50 feet since the 1960s. Prior to urbanization and agricultural pumping, the groundwater table was within 20 to 30 feet of the ground surface. For the majority of the boreholes done by Caltrans, groundwater was not encountered. However, in October 1959, the groundwater in the vicinity of Route 99 at Central Avenue was between EL 245 feet and EL 264 feet (26.5 to 38 feet BGL). In June 1997, the groundwater in the vicinity of the intersection of Route 99 and Route 180 was between EL 196 feet and EL 203 feet (73.8 to 76.1 feet BGL).

Table 5.4-1 summarizes the historical groundwater levels along the alignment over the past 50 years.

Table 5.4-1
Groundwater Table Depths (feet BGL)

Location	1960–65	1986–88	1999–01	2005	2009–11
Clinton	70	88	98	110	-
Roeding Park	64	84	94	100	-
Ventura Ave	59	71	80	101	125

Figure 5.4-1 shows a hydrograph of historical water well levels in the city of Fresno over the past 80 years. This hydrograph is reasonably consistent with hydrographs of wells along the alignment presented in the HST Fresno-to-Bakersfield Geologic and Seismic Hazard Report (URS/HMM/ARUP, 2011) along the HST alignment, which show a general trend of groundwater depletion within the Fresno city limits.

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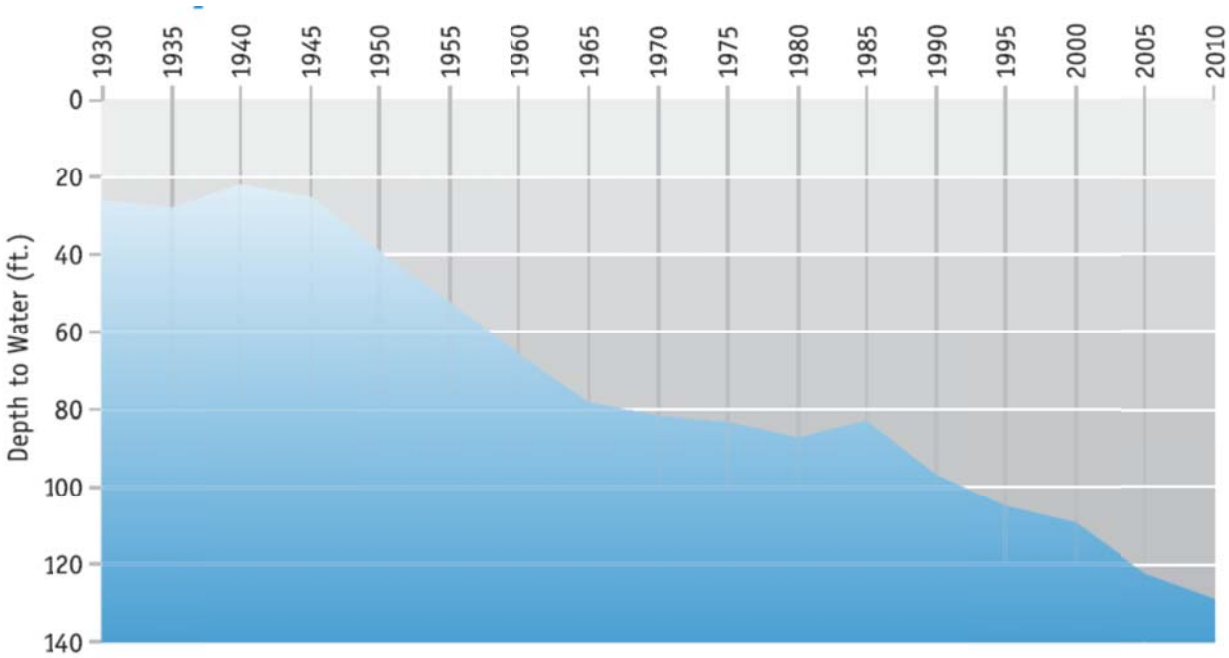

Figure 5.4-1
City of Fresno Historical Water Levels (City of Fresno, 2010)

Current groundwater in the unconfined aquifer is likely to be encountered approximately 120 feet BGL from Clinton to Ventura Avenue, with possible localized mounding in the vicinity of the Dry Creek Canal.

For preliminary design purposes, however, a groundwater table depth of 40 feet is presumed to be reasonable throughout the study area. Expectations of groundwater mounding or perched conditions associated with the Dry Creek Canal appeared unfounded based on the results of the 30% Design GI and conclusions. The explorations in the immediate vicinity of the canal did not encounter groundwater to depth, and piezometers installed in the downtown Fresno area show groundwater consistently deeper than 80 feet below grade. Using these data, a design groundwater level of 40 feet below site grades has been selected for the structures under consideration in this report.

5.5 Ground Model

For the interpretation of the historical geotechnical data and the geologic setting of the Fresno subsection, a "typical" ground model is assumed for the design of each complex and nonstandard structure. This ground model represents the worst credible geotechnical situation. It is anticipated that 70% of the geotechnical conditions are better than the typical ground model and about 30% of the foundation conditions are worse.

The design parameters used for Package 1A and 1B are based on the limited data available from historical geotechnical explorations, whereas the design parameters for Fresno Viaduct are based on the Geotechnical Baseline Report. The historical geotechnical explorations were not intended for final design of the structures necessary for the HST project. Since rock conditions are not indicated in the historical data reviewed, the developed ground model does not contain any rock properties.

The design soil profiles for the Fresno Street HST Overpass are shown in Table 5.5-1. Soil parameters used in analyses for Package 1C structures are discussed in Section 7.2.

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Table 5.5-1
Design Soil Profile for Fresno Street HST Overpass

Soil Design Parameters	Structural Fill Layer A	Silty Sand Layer B	Silty Sand Layer C
Thickness of layer (ft)	35	25	>70
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	30	18	50
Friction Angle, ϕ' (deg)	37	35	37
Total Unit Weight, γ (pcf)	125	125	125
Young's Modulus, E (ksf)	980	760	980

The design soil profiles for the north and south ends of the Fresno Grade Separation are shown in Tables 5.5-2 and 5.5-3.

Table 5.5-2
Design Soil Profile for Fresno Grade Separation – North of Station 10924+00

Soil Design Parameters	Silty Sand Layer A	Silty Sand Layer B
Thickness of layer (ft)	40	>60
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	20	70
Friction Angle, ϕ' (deg)	35	40
Total Unit Weight, γ (pcf)	125	125
Young's Modulus, E (ksf)	800	1540

Table 5.5-3
Design Soil Profile for Fresno Grade Separation – South of Station 10924+00

Soil Design Parameters	Silty Sand Layer A	Silty Sand Layer B	Silty Sand Layer C
Thickness of layer (ft)	10	10	>80
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	20	45	60
Friction Angle, ϕ' (deg)	37	40	39
Total Unit Weight, γ (pcf)	125	125	125
Young's Modulus, E (ksf)	940	1540	1420

6.0 Liquefaction

The liquefaction assessment for the Fresno Package 1 alignment was performed for both the OBE event (return period of 50 years) and the MCE event (return period of 950 years). These calculations were

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performed based on the historically available borehole Fresno to Bakersfield database. Additional calculations of liquefaction potential were completed with the 30% Design GI data. The conclusions presented in this section remain applicable to design for the structures

Soil liquefaction is the loss of shear strength in sandy soils due to an increase in pore pressure during dynamic loading. It is most commonly associated with shallow, loose, saturated deposits of cohesionless soils subjected to strong earthquake shaking, often causing significant damage. The vast majority of liquefaction hazard is associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, the hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths (CGS, 2008).

As per the Geotechnical Analysis and Design Guidelines (TM 2.9.10), the liquefaction assessment should be performed to a depth of 75 feet according to one of the following methods:

- Youd et al. (2001)
- Seed et al. (2003)
- Idriss and Boulanger (2008) (not used)

For the purpose of this study, the Youd et al. (2001) as well as the Seed et al. (2003) methodologies were implemented. The final results are reported in terms of Factor of Safety (FOS) as per TM 2.9.10 and conclusions are drawn accordingly. Since more than one approach was used in this study, if different methods resulted in values within 20% of each other, the presented FOS values were averaged. Otherwise, the more conservative of the results (lower FOS) was reported. This study provides an SPT-based liquefaction evaluation.

6.1 Methodology

For the 30% Seismic Design stage, two levels of design earthquakes are considered: OBE and MCE.

The liquefaction assessment was based on the historical SPT measurements, and the analysis followed the research and subsequent methodologies described by Youd et al. (2001) and Seed et al. (2003). For each methodology, a minimum required $(N_1)_{60}$ curve to resist liquefaction (or liquefaction triggering curve) was defined with depth. This triggering curve was determined based on the liquefaction curves and the appropriate fines content following the procedures and assumptions of each method. Additionally, for both methods, the measured SPT N-values were corrected to $(N_1)_{60}$ as recommended by Youd et al. and Seed et al. The hammer energy efficiency correction, overburden pressure correction, and other factors are listed below:

$(N_1)_{60} = N.C_N.C_R.C_S.C_B.C_E$
C_N = correction for overburden pressure
C_R = correction for short rod length
C_S = correction for nonstandardized sampler configuration (= 1)
C_B = correction for borehole diameter (= 1.0)
C_E = correction for hammer energy efficiency (= 1.25)

The corrected SPT N data $(N_1)_{60}$ were then plotted on the same graphs with liquefaction triggering curves for comparison.

6.2 Assumptions

The PGA values used for the analyses presented in Table 3.2-1 are based on the 30% Design Ground Motions Report for the HST project performed by SC Solutions (2011). The report assumed an average shallow shear wave velocity $(V_S)_{30}$ of 935 feet per second for the liquefaction analysis within the study

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area. For the Seed et al. (2003) method, an average shear wave velocity $(V_S)_{40}$ of 935 feet per second was also considered over the top 40 feet of soil.

The earthquake magnitudes for the design earthquakes, based on the 30% Design ground motions drawings provided by SC Solutions, are summarized in Table 3.2-1. A lower limit and an upper limit magnitude were assigned to each design earthquake.

Since the available geotechnical borehole data were sparse and not located along the alignment, soil layers with a uniform total unit weight of 125 pounds per cubic foot were assumed throughout Package 1. Review of the historical borehole data shows extensive existence of silty sand and lean clay in the region. However, since the available quantitative information about the fines content of the soils was very limited, a wide range of fines content was considered in the liquefaction assessment: 5%, 15%, and 35%. This sensitivity analysis demonstrates the effect of fines content in the liquefaction susceptibility.

The liquefaction analysis originally considered for Package 1A and 1B included assumed groundwater levels at 10 feet below grade and 40 feet below grade. Based on the data retrieved during the GI, the 10-foot case has been removed from the programmatic concern for Package 1. The data and discussion are preserved herein for informational purposes only.

6.3 Results

A liquefaction assessment was carried out for the Fresno area using two simplified methods based on the assumptions elaborated in the previous section.

The results of this assessment are presented in Figures 6.3-1 to 6.3-8, each based on different assumptions. Figures 6.3-1 and 6.3-2 present the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the OBE and a groundwater level of 10 feet. These figures reveal that there is no liquefaction hazard within the study area under this hazard level.

Figures 6.3-3 and 6.3-4 illustrate the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the OBE and a groundwater level of 40 feet. As expected from the previous cases, liquefaction is not triggered at any depth for either lower- or upper-limit magnitude. For the Seed et al. (2003) method for OBE with lower-limit magnitude (Figures 6.3-2 and 6.3-4), the triggering curves are not shown in the plots. This is because the calculated cyclic shear stresses are small and no corresponding data can be read from the liquefaction triggering curves. Although the liquefaction triggering curves were calculated for different fines content (5%, 15%, 35%), the results suggest that this parameter does not have any effect in the liquefaction susceptibility under the OBE shaking level because of the very small cyclic shear stress ratios induced in the OBE case.

Figures 6.3-5 and 6.3-6 present the results for the case of the MCE with a groundwater depth of 10 feet. Comparing the left (lower-limit magnitude) and the right (upper-limit magnitude) plots in each figure, it becomes apparent that the liquefaction susceptibility increases once larger earthquake magnitudes are applied. More specifically, for the lower-limit magnitude earthquake with groundwater modeled at a depth of 10 feet, a few data points suggest liquefaction potential for fines content of 5%.

These figures show that as the fines content increases, potential for liquefaction decreases. In summary, the liquefaction likelihood for the MCE with a lower-limit level and groundwater level of 10 feet is moderate. This is because most of data points suggesting the potential for liquefaction are single sample points in the boreholes, which does not suggest a continuous layer spatially.

For the upper-limit magnitude earthquake, there are more points showing the potential for liquefaction. The boreholes with potentially liquefiable material under this level of ground shaking are concentrated on the northern and southern part of the alignment within the study area. Again, as the fines content increases, the liquefaction potential decreases. Thus, for greater fines content of 15% or 35%, the liquefaction hazard for northern and southern parts is reduced.

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Figures 6.3-7 and 6.3-8 present the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the MCE and a groundwater level of 40 feet. According to these figures, there are very few data points showing the potential for liquefaction in the upper-limit magnitude case. Thus, there is no liquefaction potential for either MCE level when a groundwater depth of 40 feet is assumed.

The observations from these sensitivity analyses highlight the importance of an adequate geotechnical investigation to correctly monitor the level of groundwater. This should reflect the seasonal fluctuations and address the perched water existence. Furthermore, the effect of soil type (e.g., sandy or clayey) will have a remarkable impact because the fines content affects the cyclic behavior of the soils. Liquefaction potential shall be reevaluated in the future as site-specific geotechnical investigation data becomes available.

The liquefaction potential (FOS<1.2) for all boreholes is summarized in Table 6.3-1. The FOS shown corresponds to the average of FOS for data points below a FOS of 1.2. Liquefaction only occurs under the MCE shaking level with a groundwater level of 10 feet and is more severe for the upper-limit earthquake magnitude.

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Table 6.3-1
Fresno Liquefaction Evaluation Results

Design Earthquake	Groundwater Depth (ft)	Earthquake Magnitude (M_w)	Fines Content (%)	No. of boreholes with potential liquefaction	Average FOS	Liquefaction potential
OBE	10*	6.7 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No
	40	6.7 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No
MCE	10*	7.1 (LL)	5	7	0.89	Yes
			15	4	1.01	Yes
			35	1	0.91	Yes
		7.9 (UL)	5	10	0.92	Yes
			15	7	0.95	Yes
			35	4	0.95	Yes
	40	7.1 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No

* Note that this groundwater condition has been removed from the program.

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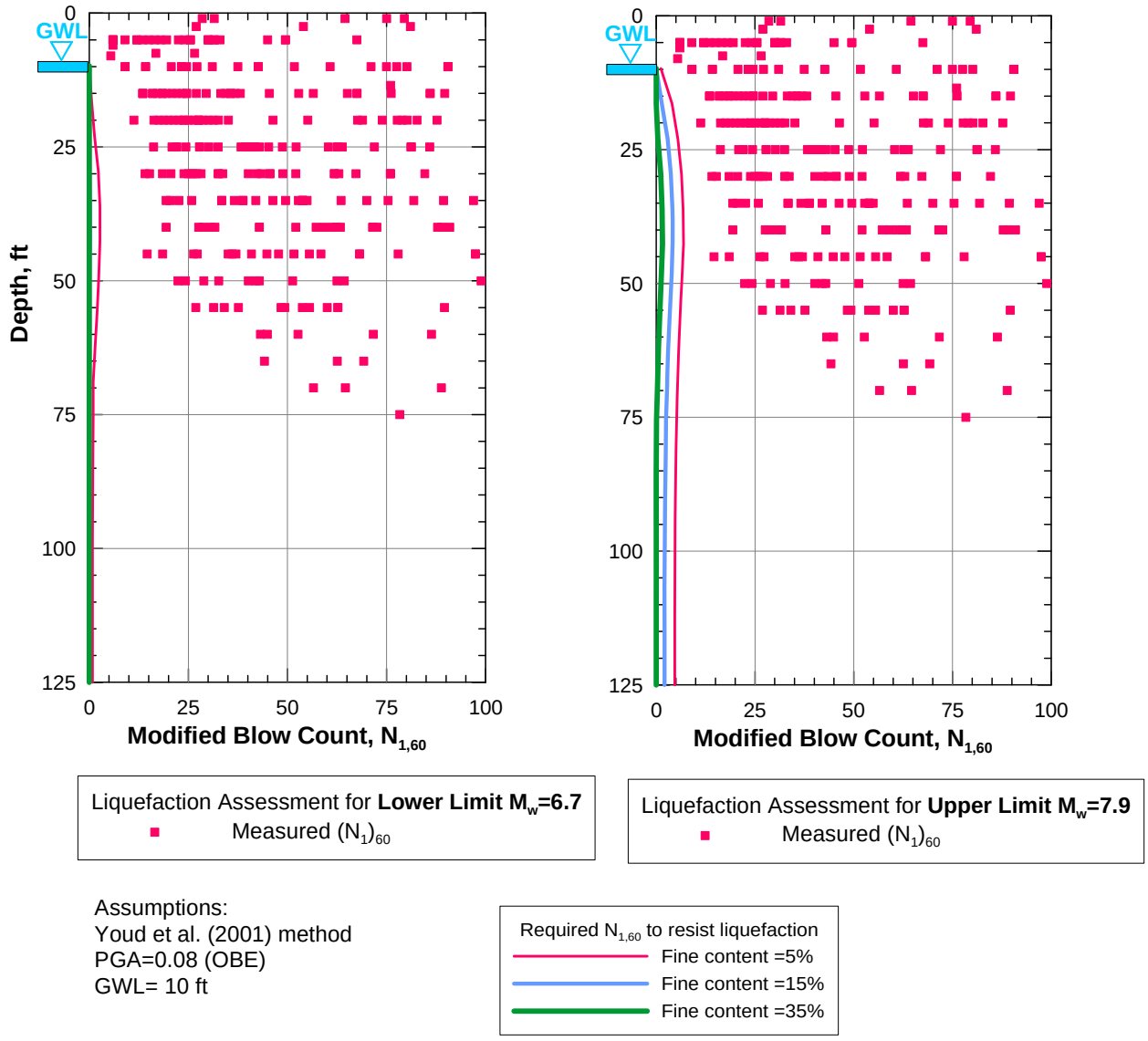

Figure 6.3-1
Liquefaction Assessment Results (Youd et al. [2001], OBE, GWL = 10 ft)

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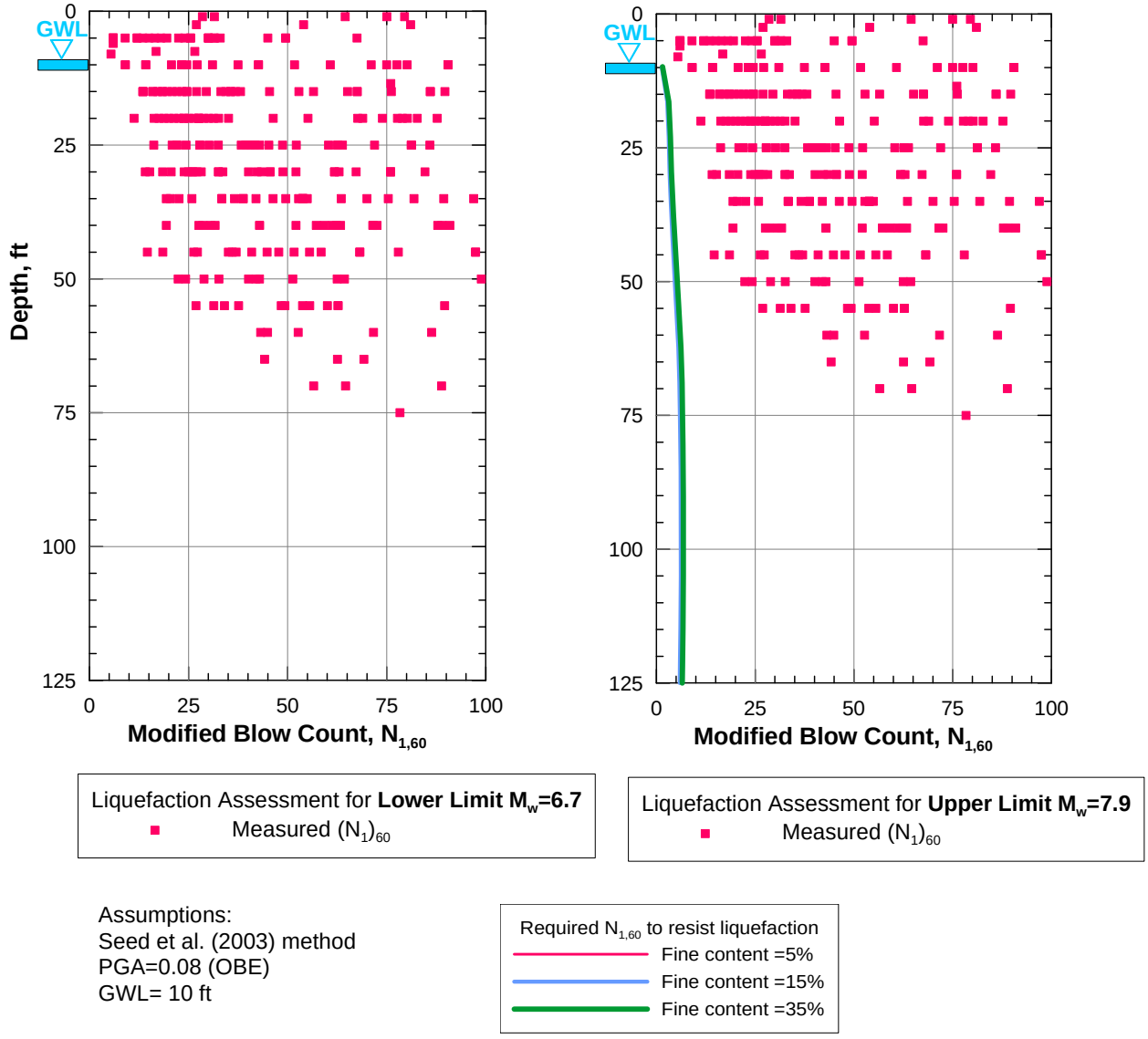

Assumptions:
Seed et al. (2003) method
PGA=0.08 (OBE)
GWL= 10 ft

Figure 6.3-2
Liquefaction Assessment Results (Seed et al. [2003], OBE, GWL = 10 ft)

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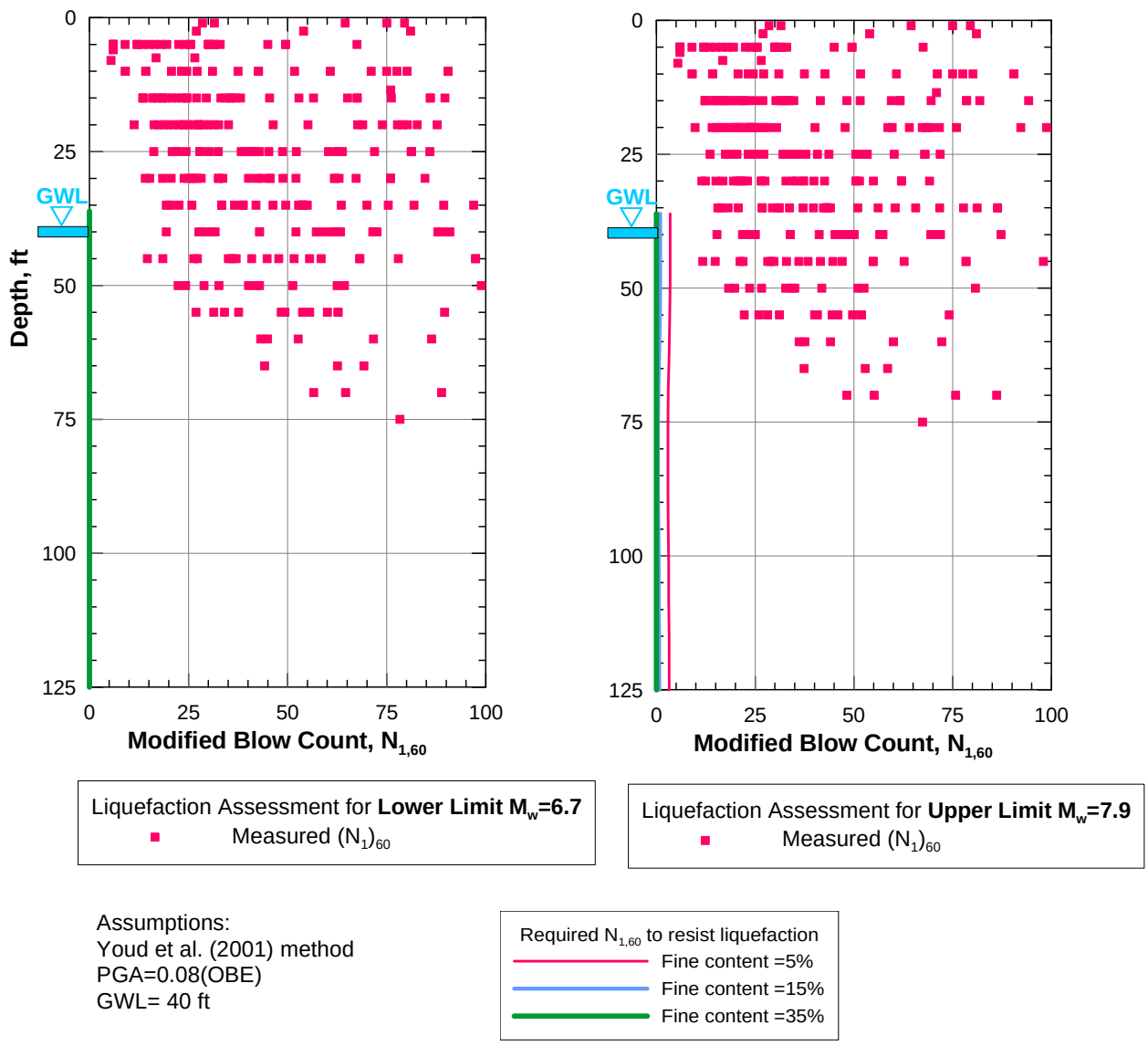

Figure 6.3-3
Liquefaction Assessment Results (Youd et al. [2001], OBE, GWL = 40 ft)

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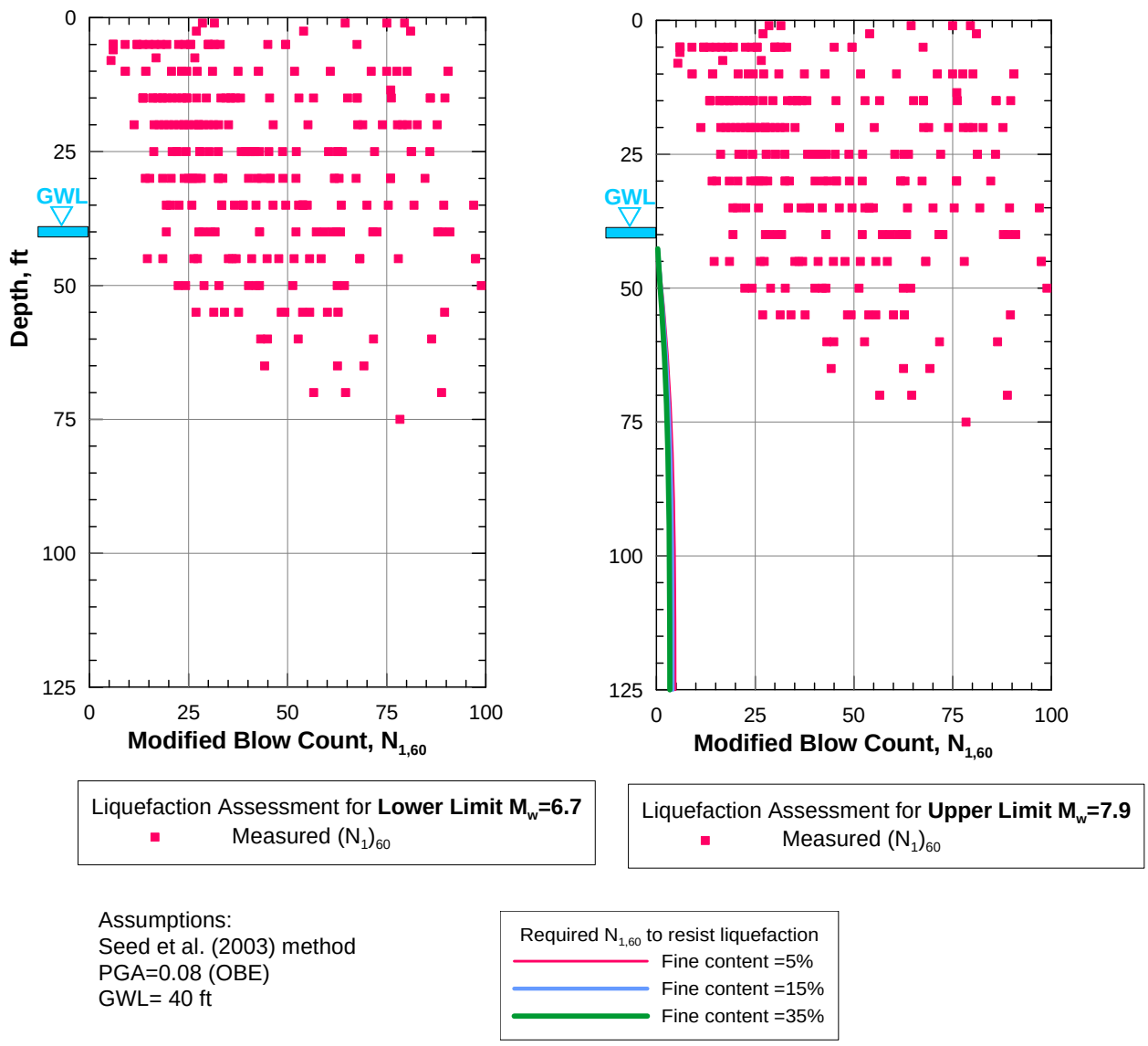

Figure 6.3-4
Liquefaction Assessment Results (Seed et al. [2003], OBE, GWL = 40 ft)

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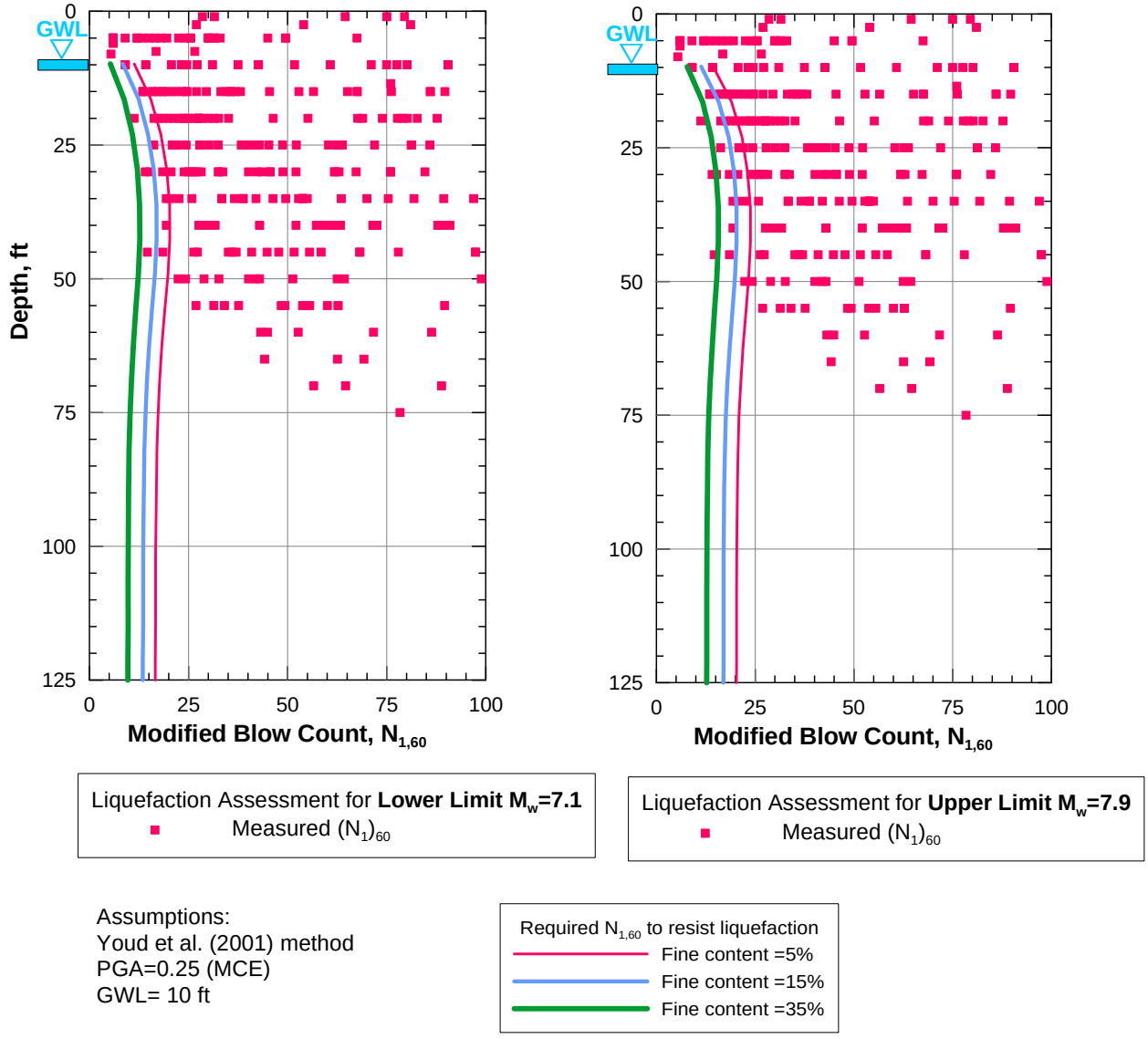

Figure 6.3-5
Liquefaction Assessment Results (Youd et al. [2001], MCE, GWL = 10 ft)

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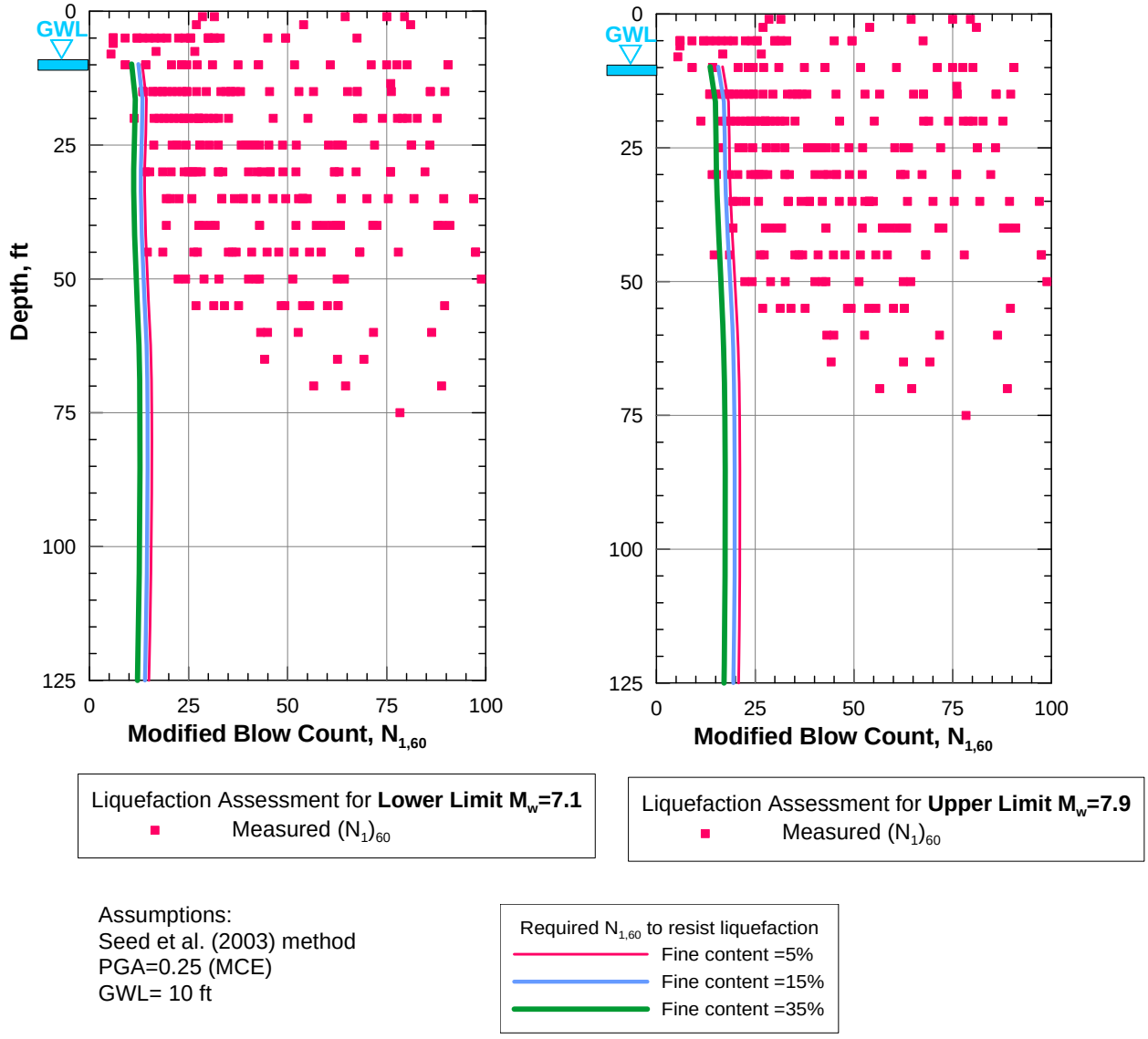

Figure 6.3-6
Liquefaction Assessment Results (Seed et al. [2003], MCE, GWL = 10 ft)

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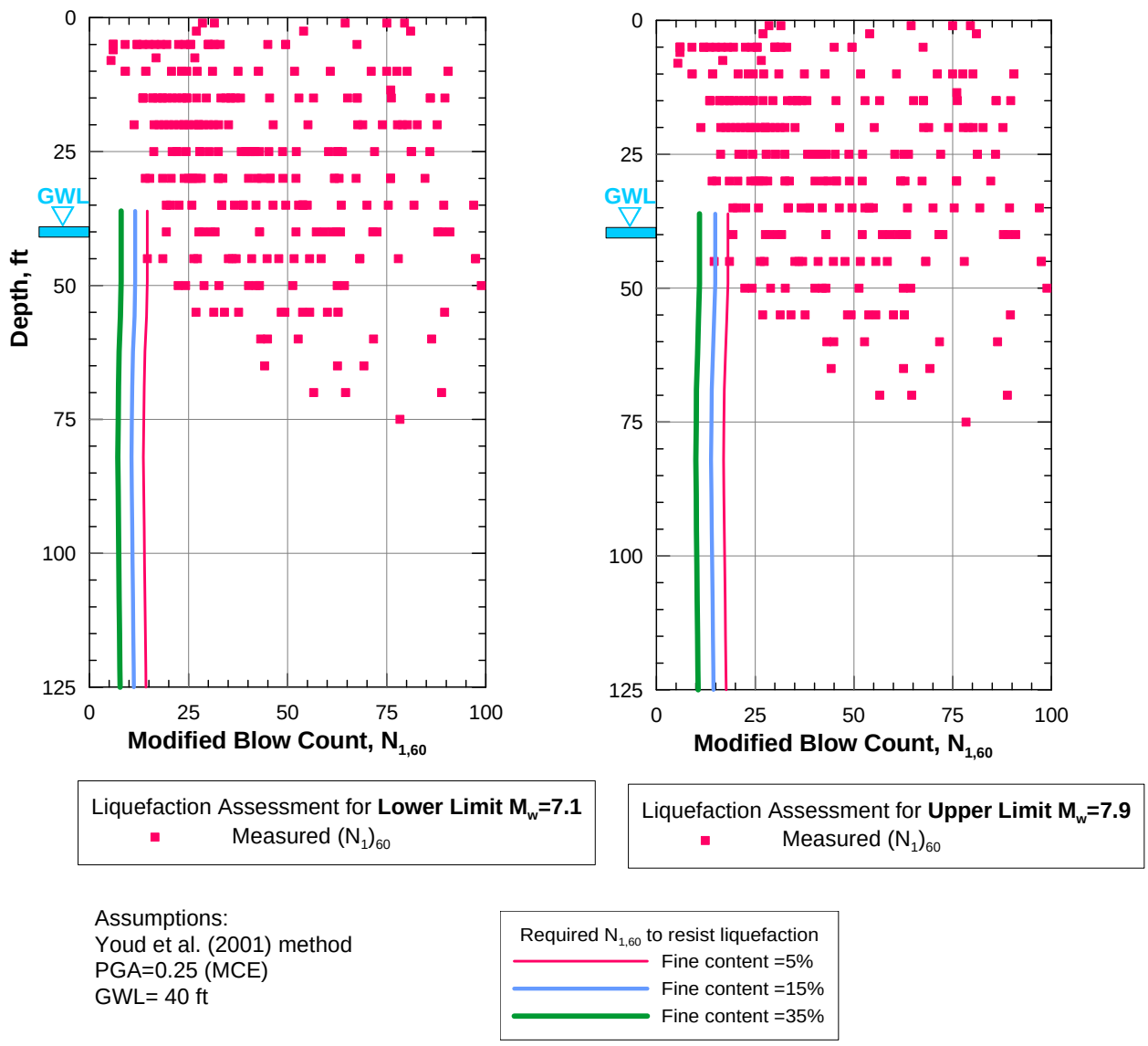

Figure 6.3-7
Liquefaction Assessment Results (Youd et al. [2001], MCE, GWL = 40 ft)

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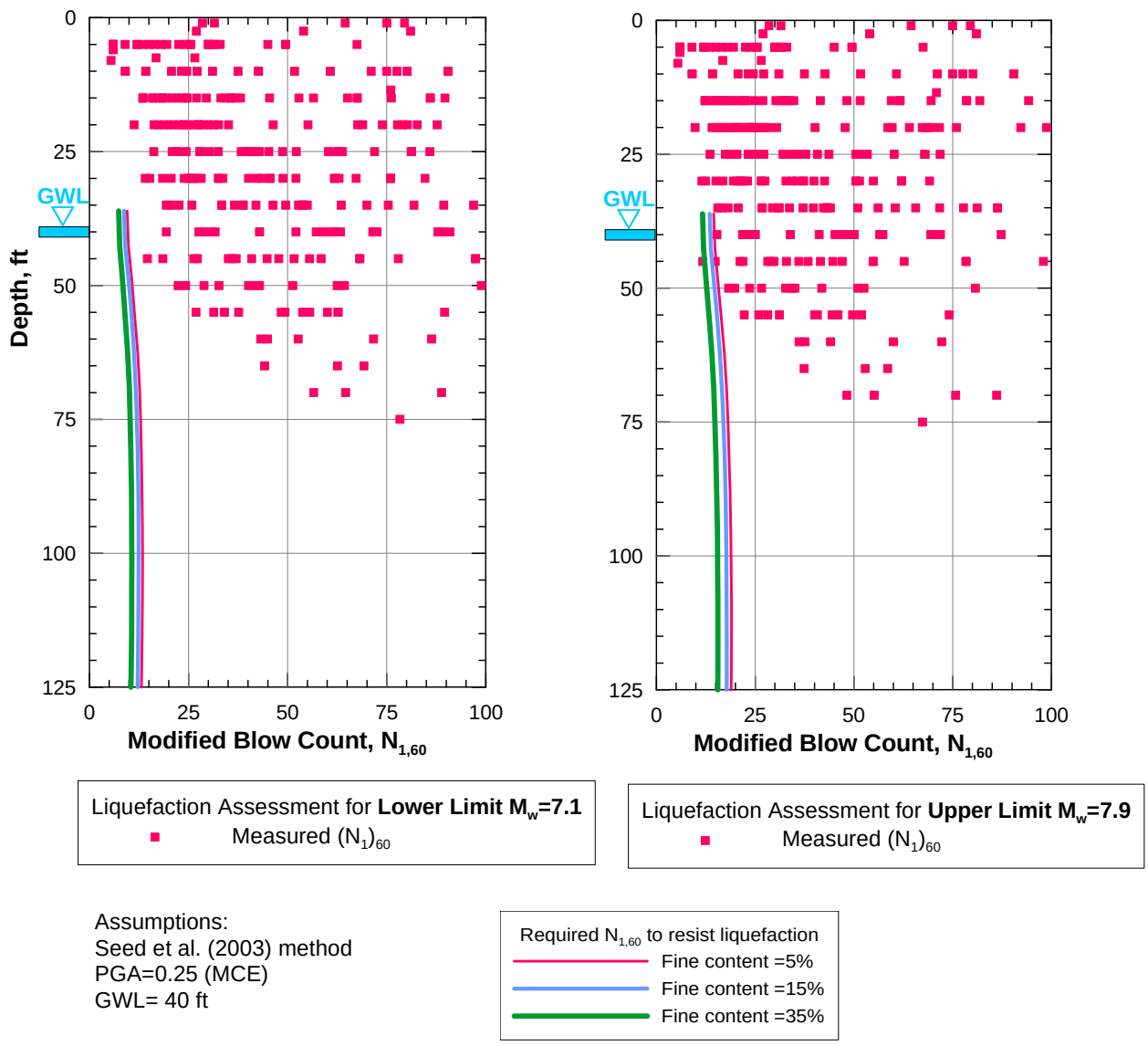

Figure 6.3-8
Liquefaction Assessment Results (Seed et al. [2003], MCE, GWL = 40 ft)

6.4 Conclusions

Using the available SPT data from the historical borehole database at the vicinity of the Fresno section (Package 1) of the alignment, the PMT assessed the liquefaction based on the simplified methods of Youd et al. (2001) and Seed et al. (2003). A summary of the average FOS resulting from liquefaction assessment is presented in Table 6.3-1.

This data suggests the moderate liquefaction potential for the case of the MCE with an upper-limit magnitude of 7.9 and groundwater at a depth of 10 feet. As shown, the liquefaction susceptibility decreases with increasing fines content.

Liquefaction is not considered a hazard for both lower- and upper-limit magnitudes of the OBE with groundwater at a depth of 10 and 40 feet or for both magnitudes of the MCE with groundwater depth at 40 feet.

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6.5 Seismic Deformations

TM 2.9.10 requires consideration of seismic deformations for only the OBE event. Based on the liquefaction assessment above, the study area is not considered particularly prone to seismically induced deformations, including liquefaction of unsaturated (dry) soils above the level of seasonal groundwater fluctuation.

7.0 Design

7.1 Fresno Street Overpass

The design of the Fresno Street overpass will be executed by Caltrans, and has been removed from the JV scope

7.2 Fresno Viaduct

Pile stiffness matrices were produced for the Fresno Viaduct structures for the structural model. Horizontal, vertical and rotational single pile stiffnesses were estimated for the Fresno Viaduct which crosses over the Golden State Boulevard, South Cedar Avenue and SR 99 Undercrossing.

Uncoupled analyses were performed to estimate the displacements and rotation at the top of a single pile subject to horizontal forces, vertical forces and bending moments applied at the top of the pile. Pile stiffness matrices were subsequently developed based on the results of these analyses. For more detail regarding the implementation of the stiffness matrices to the structural models, refer to the main body of this report. Pile bearing capacity estimation or force distribution in the pile groups is not included in the scope of this report.

Assumptions about the pile head constraint, soil stiffness and concrete elastic modulus were incorporated in the model. The cases considered for each loading condition are summarized below:

- Lateral Load case
    - Free and fixed pile head
    - Static load and cyclic load soil stiffness
    - Short term and long term concrete elastic modulus: Full stiffness and nonlinear behavior for concrete was considered for short term loading conditions, while a reduction factor of 0.5 was applied to the full stiffness to account for creep effect in concrete in the long term.
- Bending Moment case
    - Static load and cyclic load soil stiffness
    - Short term and long term concrete elastic modulus: Full stiffness and non linear behavior for concrete was considered for short term loading conditions, while a reduction factor of 0.5 was applied to the full stiffness to account for creep effect in concrete in the long term.
- Vertical Load case
    - Short term and long term concrete elastic modulus: A factor of 0.7 was applied to the full concrete stiffness to consider cracking in the short term, while a reduction factor of 0.5 was used to account for creep effect in concrete for long term loading conditions.

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The single pile analyses under horizontal load and bending moment were performed using LPILE6. The LPILE6 models are in accordance with the API and Matlock and Reese methods recommended in AASHTO Section A10.2. A nonlinear modulus for the pile was calculated based on the stress-strain curve of the concrete. The compressive strength of concrete is used in the software to derive the stress-strain curve of the material. For this reason, a full compressive strength of 5000 psi was used for the short term analyses, while a reduction factor of 0.5 was applied to the full compressive strength of concrete to account for creep effect under long term loading conditions.

The foundations of the structures in Fresno Viaduct consist of pile groups and thus, both fixed and free pile head conditions were modeled to take into account the existing pile cap. The fixed head rotational stiffness was calculated using an imposed bending moment from the loading conditions provided by the structural team. Rotational stiffness was not calculated for the free-head case for moment loading.

For the vertical single pile stiffness estimation two different approaches were implemented. The first method makes use of the software Pilset with its built-in formulae for T-z relationships by Mindlin. Pilset is produced by the software house Oasys. The second methodology uses that proposed by Fleming in the paper "A new method for single pile settlement prediction and analysis", 1992, Geotechnique 42, No.3 pp 411-425. Both methods model the settlement of a single pile under incremental vertical loading to develop a vertical stiffness. The lower of the two stiffness curves calculated has been provided.

The nominal side resistance in cohesionless soil used in Fleming (1992) method is estimated following the O'Neill and Reese (1999) formulation, specified in the AASHTO 2010 (Section 10.8.3.5.2b) as follows:

$$q_s = \beta\sigma'_v \leq 4.0 \quad \text{for } 0.25 \leq \beta \leq 1.2$$

In which

$\beta = 1.5 - 0.135\sqrt{z}$ for $N_{60} \geq 15$

$\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z})$ for $N_{60} < 15$

Where

σ'_v = vertical effective stress at soil layer mid-depth (ksf)
β = load transfer coefficient
z = depth below ground, at soil layer mid-depth (ft)
N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

The pile geometry, soil profiles and results of the analyses are presented in the following section.

7.2.1 Stiffness Matrices

The design soil profile is same throughout the Fresno Viaduct and thus, the following results are applicable to all structures. A 6.5 feet diameter, 100 feet long pile was considered in the analyses for the Fresno viaduct. The foundations of the structures in this part of the alignment consist of pile groups and pile caps approximately 10 feet deep. Therefore, the pile head was modeled at a depth of 10 feet below the existing ground level to allow the depth necessary for the excavation and pile cap construction. Both fixed and free pile head conditions were modeled in LPILE6 for the lateral loading case.

The stiffness parameters were developed using the design soil profile identified in the Geotechnical Baseline Report for the Fresno Viaduct, presented in Table 7.2-1. Groundwater level was modeled at a depth of 40 feet below ground surface.

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Table 7.2-1
Design Soil Profile for Fresno Viaduct

Soil Design Parameters	Layer A	Layer B
Depth of layer (ft)	0 - 60	> 60
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	35	55
Friction Angle, ϕ' (deg)	37	37
Total Unit Weight, γ (pcf)	126	132
Young's Modulus, E (ksf)	770	622
Poisson's Ratio, v	0.3	0.3
Modulus of Horizontal Static Subgrade Reaction, $K_{h,\ static}$ (pci)	80	80
Modulus of Horizontal Cyclic Subgrade Reaction, $K_{h,\ cyclic}$ (pci)	40	40

The results of the analyses are presented in the following Tables.

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Table 7.2-2
Fresno Viaduct - Stiffness Matrix for Horizontal Load, Free Pile Head and Short Term Concrete Stiffness

Horizontal Load (kips)	Static Load soil stiffness		Cyclic Load soil stiffness	
	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)
0	1679	-355847	1043	-257745
50	1679	-355847	1043	-257745
100	1678	-355467	1042	-257414
150	1677	-355063	1041	-257074
200	1676	-354667	1041	-256723
250	1675	-354258	1040	-256368
300	1673	-353849	897	-201396
350	1479	-288403	773	-156851
400	1294	-230091	703	-132617
450	1187	-198451	680	-125564
500	1118	-176739	662	-119899

Table 7.2-3
Fresno Viaduct - Stiffness Matrix for Horizontal Load, Fixed Pile Head and Short Term Concrete Stiffness

Horizontal Load (kips)	Static Load soil stiffness	Cyclic Load soil stiffness
	K_{p-y} (kips/in)	K_{p-y} (kips/in)
0	3781	2376
50	3781	2376
100	3779	2374
150	3776	2372
200	3774	2370
250	3771	2038
300	3258	1805
350	2948	1755
400	2761	1754
450	2757	1730
500	2756	1694

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Table 7.2-4
Fresno Viaduct - Stiffness Matrix for Bending Moment, Short Term Concrete Stiffness

Bending Moment (kips x in)	Static Load soil stiffness		Cyclic Load soil stiffness	
	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)
0	355810	-41918176	257779	-35727045
5000	355810	-41918176	257779	-35727045
10000	355450	-41839254	257515	-35660795
15000	355061	-41760628	257230	-35593100
20000	354661	-41679692	256938	-35522717
25000	354253	-41595954	256638	-35451941
30000	248587	-16759777	171522	-13624968
35000	200338	-13872649	143195	-11711757
40000	180204	-12859793	129878	-10930485
45000	171856	-12466963	121860	-10494746
50000	165071	-12161337	116902	-10236253

Table 7.2-5
Fresno Viaduct - Stiffness Matrix for Vertical Load, Short Term Concrete Stiffness

Vertical Load (kips)	K_{t-z} (kips/in)
0	4461
500	4461
1000	4448
1500	4426
2000	4402
2500	4374
3000	4336
3500	3541
4000	2203
4500	1524
5000	1141
5500	893

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Table 7.2-6
Fresno Viaduct - Stiffness Matrix for Horizontal Load, Free Pile Head and Long Term Concrete Stiffness

Horizontal Load (kips)	Static Load soil stiffness		Cyclic Load soil stiffness	
	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)
0	1525	-301405	945	-218055
50	1525	-301405	945	-218055
100	1523	-300851	944	-217576
150	1521	-300276	943	-217064
200	1520	-299679	942	-216546
250	1518	-299082	774	-156904
300	1245	-214758	688	-127579
350	1127	-179925	651	-115959
400	1075	-166477	637	-111401
450	1045	-158493	626	-107808
500	1027	-153201	623	-107394

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Table 7.2-7
Fresno Viaduct - Stiffness Matrix for Horizontal Load, Fixed Pile Head and Long Term Concrete Stiffness

Horizontal Load (kips)	Static Load soil stiffness	Cyclic Load soil stiffness
	$K_{p\text{-}y}$ (kips/in)	$K_{p\text{-}y}$ (kips/in)
0	3417	2142
50	3417	2142
100	3413	2139
150	3409	2136
200	3405	1825
250	2850	1658
300	2611	1657
350	2610	1604
400	2608	1602
450	2579	1554
500	2536	1513

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Table 7.2-8
Fresno Viaduct - Stiffness Matrix for Bending Moment, Long Term Concrete Stiffness

Bending Moment (kips x in)	Static Load soil stiffness		Cyclic Load soil stiffness	
	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)
0	301350	-32975005	218090	-28091466
5000	301350	-32975005	218090	-28091466
10000	300761	-32860147	217659	-27993953
15000	300128	-32740369	217197	-27890891
20000	281525	-22005832	200352	-17964610
25000	186206	-12955919	137593	-11232623
30000	168528	-12035432	121426	-10228854
35000	159070	-11585913	113682	-9797637
40000	152477	-11288242	110642	-9632287
45000	150662	-11199908	108145	-9500084
50000	148162	-11085614	106110	-9394700

Table 7.2-9
Fresno Viaduct - Stiffness Matrix for Vertical Load, Long Term Concrete Stiffness

Vertical Load (kips)	K_{t-z} (kips/in)
0	4257
500	4257
1000	4239
1500	4214
2000	4186
2500	4151
3000	4110
3500	3340
4000	2118
4500	1480
5000	1115
5500	877

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7.3 Fresno Grade Separation

The predominantly granular nature of the ground conditions at the Fresno Grade Separation location favor continuous wall types without gaps to mitigate possible ground loss. Given the proximity of the HST alignment to adjacent structures, the Union Pacific Railroad, and potential shallow groundwater conditions due to Dry Creek Canal and groundwater detention basins, "stiff" excavation support systems capable of minimizing water inflows are anticipated.

The most suitable wall components are therefore expected to be either diaphragm slurry wall, secant pile, or a deep soil mixing/cast-in-place (temporary/permanent) wall combination. A soldier-pile-and-lagging/cast-in-place wall combination is also a possibility at dry locations or where adequate dewatering is employed. For 30% Design, a secant pile wall with a diameter of 3 feet and center-to-center spacing of 2.5 feet was determined to be appropriate as a temporary excavation support system (ESS) during construction.

Loads on the temporary shoring wall were developed in accordance with Section 6.7 of TM 2.9.10. For this purpose, excavation support modeling was performed using the computer program Oasys FREW, which models the excavation construction sequence, incorporating the following:

- Effects of surcharge loading
- Construction dewatering
- Temporary and permanent struts
- Excavation support wall stiffness
- Different soil layers and properties

The ESS consists of a temporary secant pile wall and the base slab. More specifically, the stiffness of the secant pile wall was estimated assuming that all the piles are composed of reinforced concrete and the area of steel is 1% of the total area of the pile cross section. The combined Young's modulus of the composite cross section of each pile was then calculated based on the above assumption, and finally, the stiffness of the secant pile wall was estimated based on a center-to-center spacing of 2.5 feet for the piles. The base slab stiffness was estimated assuming a slab thickness of 4 feet. The secant pile wall and base slab stiffness calculations used in the software were evaluated using concrete Young's modulus for long-term conditions.

Temporary strut stiffness was estimated assuming 24-inch steel pipe struts spaced 15 feet apart horizontally. The size of the pipe struts was estimated through an iterative process, to verify that the struts can sustain the anticipated applied axial loads without buckling. The permanent struts, used throughout the Fresno Grade Separation except for the jacked box location, were modeled as concrete beams, having a 4 feet by 4 feet cross section and 15 feet spacing along the excavation. The individual permanent strut stiffness was estimated based on this configuration using concrete Young's modulus for long-term conditions.

Groundwater conditions inside and outside of the excavation support system were also modeled. Groundwater table outside the excavation was modeled at a constant depth of 10 feet below existing ground surface. While the 10-foot deep groundwater assumption has been removed from the program, further analyses of seepage and seasonal rainfall have dictated that the ground immediately around the grade separation structure could become saturated. During modeling of the excavation, groundwater was maintained at 2 feet below the bottom of the excavation for each excavation stage. Because of the granular nature of soils at the Grade Separation location, all the analyses were performed for drained conditions.

A surcharge was included in the models to simulate embankment and construction equipment. The surcharge was 30 feet wide at 15 feet from the edge of the excavation.

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7.3.1 Lateral Earth Pressures

According to the SC Solutions (2011) seismic modeling, the study area is in a seismic zone with PGAs 0.08g and 0.28g, respectively. Therefore, per Section 6.10.13 of TM 2.9.10, seismic earth pressures are not considered for either rigid or yielding walls for the permanent wall design. Table 7.3-1 shows the lateral earth pressure coefficients that were used for the permanent wall design of the Fresno Grade Separation structure.

Table 7.3-1
Earth Pressures

Type of Wall	Lateral Earth Pressure Coefficients
Unrestrained Permanent Trench Wall	Active condition, $k_a = \tan^2(45- \phi'/2)$
Restrained Permanent Trench Wall	At-rest condition $k_o = 1\text{-}\sin \phi'$

7.3.2 Results of Excavation Sequence Modeling

A total of four sections were modeled in FREW for the Fresno Grade Separation structure based on the two design soil profiles presented in Section 5.5. Table 7.3-2 presents the stations where each section is applicable. The ground conditions change at Station 10924+00; thus, Sections 1 and 2 located north of this station have the same soil parameters. Sections 3 and 4, extending south of Station 10924+00, refer to the same ground conditions. However, Section 4 was specifically developed for the case of the jacked box. Tables 7.3-3 and 7.3-4 summarize the soil parameters used for each model.

Table 7.3-2
Section Stationing

Section	From Station	To Station
1	10885+00	10913+00
2	10913+00	10924+00
3	10924+00	10975+00
4	10935+95	10939+40

Table 7.3-3
Grade Separation FREW Input for Sections 1 and 2

Layer	Thickness (ft)	γ (pcf)	φ′ (deg)	E (ksf)
Silty Sand (Layer A)	40	125	35	800
Silty Sand (Layer B)	>60	125	40	1540

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Table 7.3-4
Grade Separation FREW Input for Sections 3 and 4

Layer	Thickness (ft)	γ (pcf)	φ' (deg)	E (ksf)
Silty Sand (Layer A)	10	125	37	940
Silty Sand (Layer B)	10	125	40	1540
Silty Sand (Layer C)	>80	125	39	1420

Tables 7.3-5, 7.3-6, and 7.3-7 summarize the construction sequence modeled in FREW for all sections.

Table 7.3-5
Section 1 Construction Sequence as Modeled in FREW

Stages	Section 1 (EL) **Sta. 10885+00 – 10913+00**
1. Install secant pile wall	220
2. Excavate Dewater	281 279
3. Install Strut 1 and preload 50%	283
4. Excavate Dewater	265 263
5. Install Strut 2 and preload 50%	267
6. Excavate to Bottom of excavation Dewater	248 246
7. Install base slab	251
8. Partially construct U-wall	From 253.437 to 265
9. Remove Strut 2	267
10. Complete Permanent Wall	From 265 to 278
11. Install Permanent Strut 3	280
12. Remove Strut 1	283

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Table 7.3-6
Section 2 and 3 Construction Sequence as Modeled in FREW

Stages	Section 2 (EL) Sta. 10913+00 – 10924+00	Section 3 (EL) Sta. 10924+00 – 10975+00
1. Install secant pile wall	210	210
2. Excavate	284	284
3. Install Strut 1 and preload 50%	286	286
4. Excavate Dewater	268 266	268 266
5. Install Strut 2 and preload 50%	270	270
6. Excavate Dewater	253 251	253 251
7. Install Strut 3 and preload 50%	255	255
8. Excavate to Bottom of excavation Dewater	239.5 237.5	238 236
9. Install base slab	242.6	241
10. Partially construct permanent wall	From 245.133 to 253	From 243.508 to 253
11. Remove Strut 3	255	255
12. Partially construct permanent wall	From 253 to 268	From 253 to 268
13. Remove Strut 2	270	270
14. Complete permanent wall	From 268 to 281	From 268 to 281
15. Install Permanent Strut	283	283
16. Remove Strut 1	286	286

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Table 7.3-7
Section 4 Construction Sequence as Modeled in FREW

Stages	Section 4 (EL) Sta. 10935+95 – 10939+40
1. Install secant pile wall	299 to 220
2. Excavate both sides of box	289 (ground level)
3. Excavate	282
4. Install Strut 1 and preload 50%	284
5. Excavate Dewater	267 265
6. Install Strut 2 and preload 50%	269
7. Excavate Dewater	256 254
8. Install Strut 3 and preload 50%	258
9. Excavate to Bottom of excavation Dewater	244 242
10. Install base slab	246.5
11. Remove Strut 3	258
12. Remove Strut 2	269
13. Install Strut 4 and preload 10%	298
14. Remove Strut 1	284

The results of the analyses are summarized in Table 7.3-8 and show that deflections and bending moments are within the structural limits of the system. The allowable bending moments for the ESS are presented in Table 7.3-8.

Table 7.3-8
Excavation Sequencing Modeling Results

Section	Embedment Depth of ESS Wall Below Bottom of Excavation (ft)	Maximum Deflection of ESS Wall (in)	Service Bending Moment of ESS Wall (kip-ft/ft)
1	28	0.6	125
2	30	1.2	235
3	28	1.3	250
4	24	1.5	235

7.3.3 Base Stability Evaluation

The base stability of the temporary excavation is evaluated to avoid instability due to seepage from construction dewatering. The excavation is considered safe against base instability for an FOS greater

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than 2. The FOS is defined as the ratio of the critical hydraulic gradient over the hydraulic gradient of flow exiting the base:

$$FOS = \frac{i_c}{i_e} = \frac{\frac{\gamma_T}{\gamma_w} - 1}{\frac{\Delta h}{\Delta L}} > 2$$

Where

i_c = critical hydraulic gradient
i_e = exit hydraulic gradient
γ_T = total unit weight of soil
γ_w = total unit weight of water
Δh = difference in hydraulic head over distance ΔL

The numerator of the above equation is approximately 1, since the total unit weight of soil γ_T is about twice the unit weight of water γ_w. From the above equation for a factor of safety of 2, it can be shown that the minimum wall embedment to ensure base stability is half the difference in groundwater level of the two sides of the wall:

$$h_e \geq \frac{H_w}{2}$$

Where

h_e = wall embedment below base of excavation
H_w = groundwater level difference between inside and outside of the U-Walls

Table 7.3-9 presents the wall embedment as well as the groundwater level differences for the four sections. It can be observed that for all four cases the wall embedment is adequate and base instability is avoided.

Table 7.3-9
Base Stability Evaluation for the Four Sections

Section	Embedment depth, h_e (ft)	$H_w/2$
1	28	17
2	30	21
3	28	22
4	24	18.5

7.3.4 Tie-Downs/Tension Piles

Over certain sections of the Fresno Grade Separation, there is a possibility that groundwater may be encountered at shallower depths of around 10 feet below existing ground surface. Should this occur, tie-down or tension piles will be required to counteract the water buoyancy forces. The design soil profile for the area of high groundwater is as provided in Table 5.5-2. Design of tie-downs is based on the concept of Section 10.9 of AASHTO – Micropiles.

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For this design, Type A micropile in very dense sand was used, which yields an ultimate bond stress of 3 kips per square foot. Applying a resistance factor of 0.7 per California Amendments to AASHTO yields a factored bond stress of 2.1 kips per square foot.

To resist a hydrostatic pressure equivalent to 30 feet of water, it is recommended that two rows of tie-downs, one below each track, spaced at 5 feet center-to-center with a minimum embedment length of 35 feet be provided.

7.4 Jensen Trench

The design of the Jensen Trench U-trough structure should assume the same geotechnical parameters and lateral earth pressure coefficients as are used for the Fresno Grade Separation.

8.0 Limitations and Further Information

The 30% Design effort for Package 1A and 1B is based on limited information included in historical geotechnical reports. The 30% Design effort for the Fresno Viaduct (Package 1C) is based on 30% Design GI data and conclusions. For design based on historical data, some extrapolations were necessary to provide parameters deeper than historical data provided. These assumptions have been checked against the results of the 30% Design GI values and found to be reasonable or conservative for 30% Design. The results of this memorandum should be considered preliminary and refined by the Contractor during final design once site-specific information gathered by the Contractor is available.

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