# CHAPTER 20.3

# SEISMIC DESIGN OF CUT-AND-COVER TUNNELS

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## 20.3.1 INTRODUCTION

This chapter presents an example of seismic design of cut-and-cover Tunnels (CCT). In the example detailed annotations for every design step are provided, following the requirements in STP 20.32, BDM 20.32, Caltrans Seismic Design Criteria (SDC), Version 2.0 (Caltrans 2019), AASHTO LRFD Bridge Design Specifications with CA Amendment (AASHTO 2017a), and AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 1st Edition (AASHTO 2017b).

A tunnel is generally considered an underground structure fully or partially surrounded by geomaterials, resembling other underground structures such as culverts. However, to clarify design responsibility and practices for tunnel seismic design in Caltrans, the definition of a tunnel is narrowed to "an enclosed roadway with vehicle access limited to portals," as stated in STP 20.32. This definition distinguishes tunnels from culverts. The tunnels, with public access, need to be designed for public safety, which is different from culverts. Culverts are for wildlife or water passage, which have less demanding design requirements.

Seismic analysis and design of underground structures differ from above-ground structures (bridges). For above-ground structures, seismic inertia induced by seismic ground acceleration is the primary effect that deform the structures. Earthquakes cause seismic excitation of the elevated mass (superstructure), causing deformations in the columns, movements in the joints, and damage in the engineered Plastic Hinges (PH). In other words, bridge structures are engineered to withstand deformations due to inertia exerted by ground acceleration. For underground structures (tunnels), seismic deformation of surrounding geomaterials, instead of seismic inertia, is the major effect on the structures. In the tunnel seismic design, seismic deformation demands are applied in the form of geomaterial deformation induced by an earthquake. Therefore, the design needs to include modeling of the geomaterials around the structure to capture Soil Structure Interaction (SSI).

The design procedures for CCT presented in this BDP follows Caltrans seismic design philosophy for bridge structures that is "Strong Beam – Weak Column" proportioning principle. It is suggested that a CCT situated close to the ground level (shallow tunnel) will respond closer to a single-span bridge with diaphragm abutments than a conventional tunnel. Regardless of whether it is classified as a tunnel or bridge, the vertical structural members, such as side walls or columns, must be designed as seismically critical members intended to be ductile during seismic events.

## 20.3.2 DESIGN PROCEDURES

In general, the seismic performance of a tunnel is evaluated using three primary engineering demand parameters (EDPs): racking, axial, and curvature deformations.

The magnitudes and effects of axial and curvature deformations on a tunnel are significantly reduced if the tunnel is relatively short. Section 20.32.4.2.1 of BDM 20.32 defines short tunnels as the ratio of tunnel length (L) to average height ($H_{avg}$) less than 8. The primary seismic effect on short tunnels is racking deformation, while curvature deformation has a secondary effect. The curvature deformation can be ignored for the seismic design of short tunnels. The tunnel presented in the example is classified as a short tunnel as its length/height ratio is less than 8.

The racking deformation is the lateral seismic induced displacement at the top of a tunnel relative to the bottom of a tunnel. It is analogous to the displacement at the top of a column in the seismic design of a bridge structure. The tunnel used in this example is classified as a short tunnel because it has a ratio of tunnel length (L) to average height ($H_{avg}$) less than 8 per Section 20.32.4.2.1 of BDM 20.32. For short tunnels, the primary design consideration is racking deformation.

In this example, racking deformation and the resulting strains of reinforcing steels and concrete are estimated using nonlinear time history analysis (NTHA) in which the effect of the structure's inertia is properly captured. The strain demands from NTHA are evaluated against the threshold in Section 20.32.5.3 of BDM 20.32.

Following the NTHA, each structural element is designed and detailed to withstand the flexural and shear demands associated with the plastic moment of SCM. The shear associated with curvature along the tunnel axis is also evaluated under Section 20.32.4.2.1 of BDM 20.32. The overall step-by-step procedure is outlined in Table 1. The details and annotation of each step are presented in the next section.

Table 1. Step-by-Step Procedure of Seismic Design of Tunnel Structure

| Step | Description |
|---|---|
| 1 | Prepare ground displacement time histories (DTHs) to be applied along the perimeter of the tunnel liner. |
| 2 | Develop soil-spring models by performing geotechnical pushover analysis |
| 3 | Perform transverse pushover analysis to estimate displacement capacity, $\Delta_C$` |
| 4 | Perform NTHA for strain and displacement demands under FEE and SEE |
| 5 | Check ductility demand and capacity |
| 6 | Check P-Δ effects in the transverse direction (SDC Eq. 4.4.4-1) |
| 7 | Check wall flexural capacity in the transverse direction (SDC Section 5.3.6) |
| 8 | Check wall shear capacity in transverse and longitudinal directions (SDC Section 5.3.7) |

| | |
|---|---|
| 9 | Design joint shear reinforcement (SDC Section 7.4.2) |
| 10 | Check plastic hinge region and splicing option (SDC Section 5.3.2) |
| 11 | Check roof slab flexural and shear capacity against overstrength demands (SDC Section 4.4.2) |
| 12 | Check base slab flexural and shear capacity against overstrength demands (SDC Section 4.4.2) |

## 20.3.3 DESIGN EXAMPLE

### 20.3.3.1 DESIGN SCENARIO

This example tunnel is a 300-ft long reinforced concrete CCT. The tunnel is 40 ft wide and 30 ft high (clear span of 35 ft and clear height of 24 ft). The thickness of the walls and slabs are 2'-6" and 3'-0", as shown in Figure 1. It is situated 10 ft below finished grade (FG) within a 100-ft thick medium dense sand layer underlain by bedrock. Note that the 10 ft of embedment is measured from the FG to the soffit of the roof slab.

The construction of CCT requires excavation and backfill with engineering fill (structure backfill) within the proximity of the proposed tunnel. Excavation types may vary from open cut with a back slope to vertical cut with temporary shoring, depending on the constraint of a project. In this design scenario, open cut is assumed with a 1.5H:1V back slope. The idealized subsurface profile is shown in Figure 2. The structural and geotechnical material properties used in this design scenario are listed as follows.

*Structural Material Properties:*

| | |
|---|---|
| Concrete: | $f'_c$ = 4 ksi, $f'_{ce}$ = 5 ksi |
| | $E_c$ = 4,266 ksi, |
| Reinforcing steel (A706 Grade 60): | $f_y$ = 60 ksi, $f_{ye}$ = 68 ksi, $f_{ue}$ = 95 ksi |
| | $E_s$ = 29,000 ksi |

*Geotechnical Material Properties:*

| | |
|---|---|
| Structure Backfill | |
| Unit weight: | $\gamma$ = 120 pcf |
| Friction Angle: | $\phi$ = 29° |
| Elastic Modulus: | E = 794 ksf |
| Medium Dense Sand | |
| Unit weight: | $\gamma$ = 125 pcf |
| Friction Angle: | $\phi$ = 30° |

Elastic Modulus: E = 2,089 ksf

The tunnel shall be designed for the seismic design hazard levels per STP 20.32.4.1 which are

Functional Evaluation Earthquake (FEE): 300-year return period
Safety Evaluation Earthquake (SEE): 1500-year return period

The seismic design uses the FEE and SEE level design ARS curves at the bedrock level for the input motion (DTH) preparation. The ARS curves at bedrock are shown along with those at ground surface in Figure 3. They were developed using the following seismic parameters.

*Seismic Parameters*

Seismic Moment Magnitude ($M_w$) for SEE: 7.41
Peak Ground Acceleration (PGA): 1.07g
Shear wave velocity for the upper 30 m ($V_{s30}$): 685 ft/s (210 m/s)
Shear wave velocity for bedrock: 3,200 ft/s (1,000 m/s)

It is assumed in this scenario that the sections of walls and slabs were designed for other limit states in accordance with AASHTO 2017b. The reinforcement layout is depicted in Figure 1, with details regarding the size and spacing listed below.

Cross Tie Size and Spacing ($S_{CT}$ & $S_{CL}$): No. 5 @ 6”, No. 5 @ 8”
Transverse (Main) Reinforcement Size and Spacing ($S_T$): No. 9 @ 8”
Confinement Tie Size and Spacing ($S_L$): No. 5 @ 6”

More details for the reinforcements can be found in the Appendix.

For seismic analysis of tunnel structures, an NTHA is required using an FE model to estimate demands. Two different FE modeling techniques are available per Section 20.32.4.2.1.2 of BDM 20.32, which are de-coupled modeling and coupled modeling. Coupled modeling integrates geotechnical and structural components into one model. The soil domain vertically extends to the top of competent materials (bedrock), where input motions are applied and propagated upward in the model. In contrast, de-coupled modeling uses structural models supported by a series of soil springs. Displacement time histories are applied at the spring fixities as input ground deformation. De-coupled modeling requires GD to prepare sets of nonlinear soil springs and displacement time histories at each spring location. This example uses the de-coupled modeling, and the following sections 20.3.2 and 20.3.3 discuss the development of the displacement time histories and the soil springs.

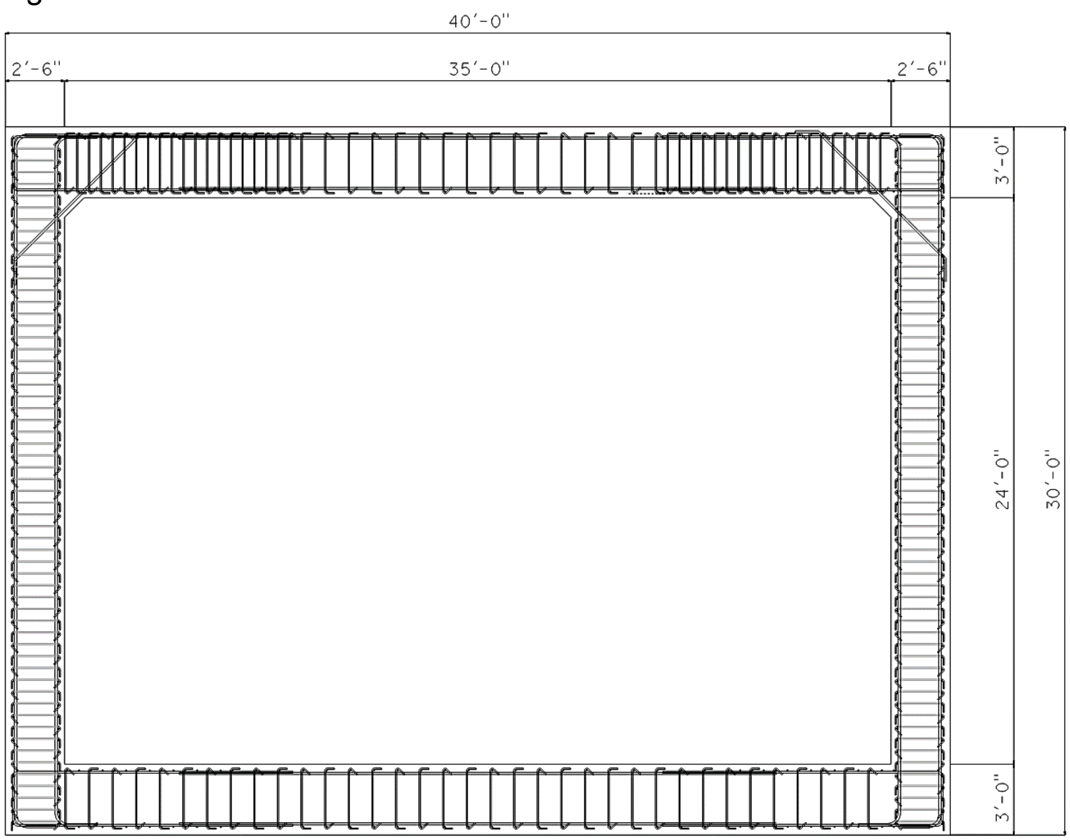



Figure 1 General Plan of Design Example

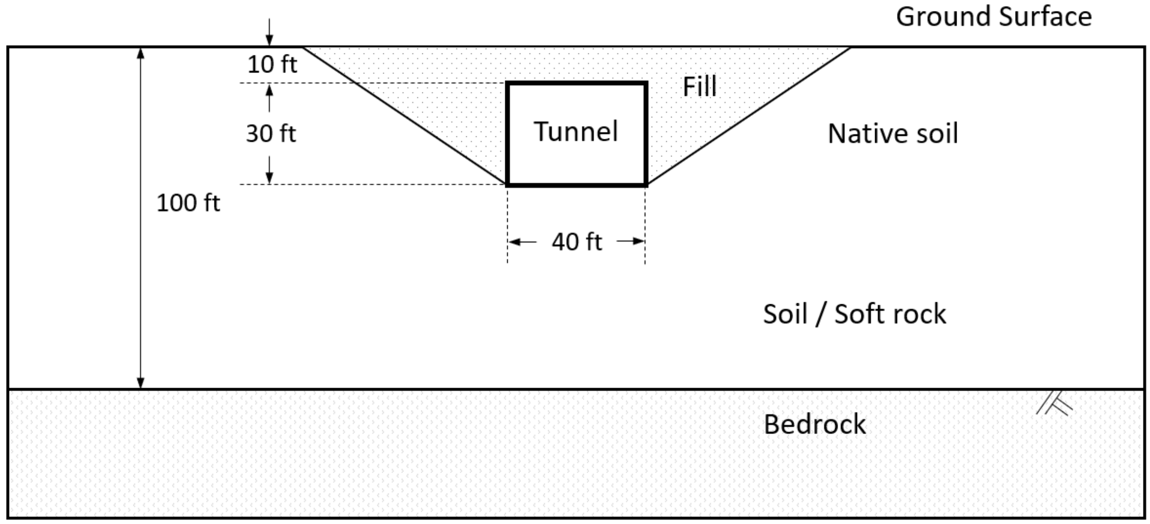



Figure 2 Generalized Subsurface Profile at Project Site

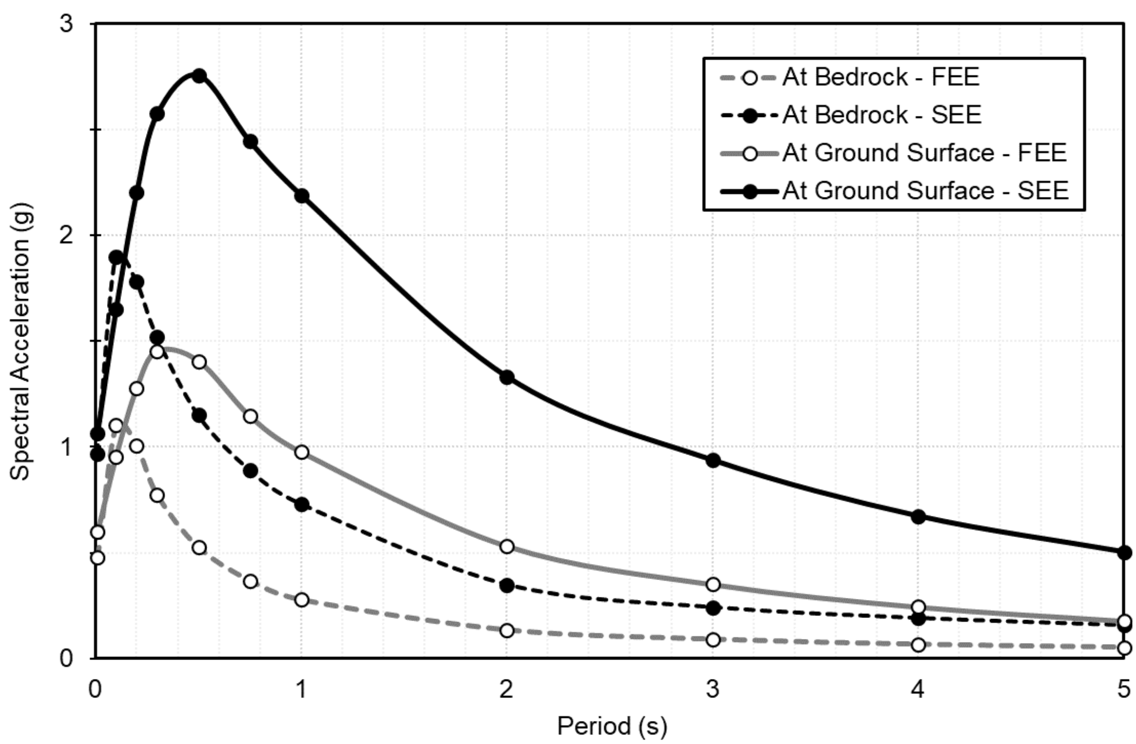



Figure 3 FEE and SEE ARS Curves at Ground Surface Level and Bedrock Level

### 20.3.3.2 STEP 1 – DEVELOP INPUT GROUND MOTIONS

The analysis requires seven sets of DTHs along the perimeter of the tunnel lining as input motions prepared by the Geotechnical Designer (GD). As outlined in Section 20.32.4.1 of BDM 20.32, the procedure of the input motion preparation is mainly divided into two parts:

1. Prepare DTHs at the bedrock level
2. Perform two-dimensional (2D) site response analysis

The following subsections discuss each part to be performed by GD.

#### 20.3.3.2.1 Prepare Seed Motions

The first step for this input motion preparation is to obtain the ATHs at the top of the bedrock (commonly referred to as “Seed Motions”). In this example, the seed motions were generated and spectrally matched to the design (FEE and SEE) ARS curves at the top of the bedrock using the program “GMGen”. The spectrally matched DTHs and their ARS curves are shown in Figure 4 and Figure 5, respectively.

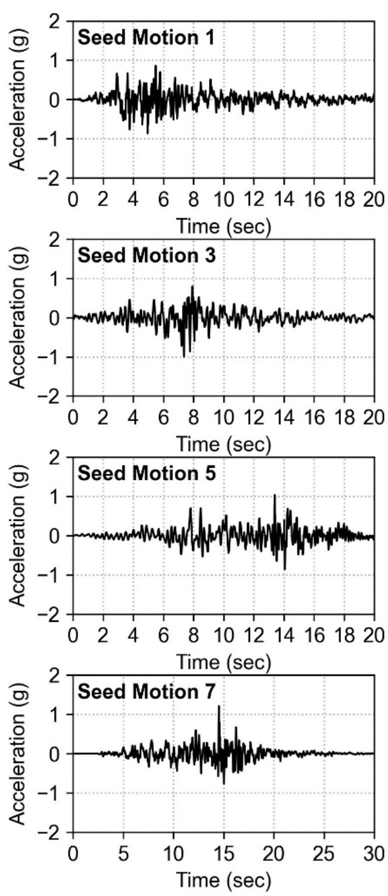



Figure 4 Spectrally matched ATHs at the top of bedrock

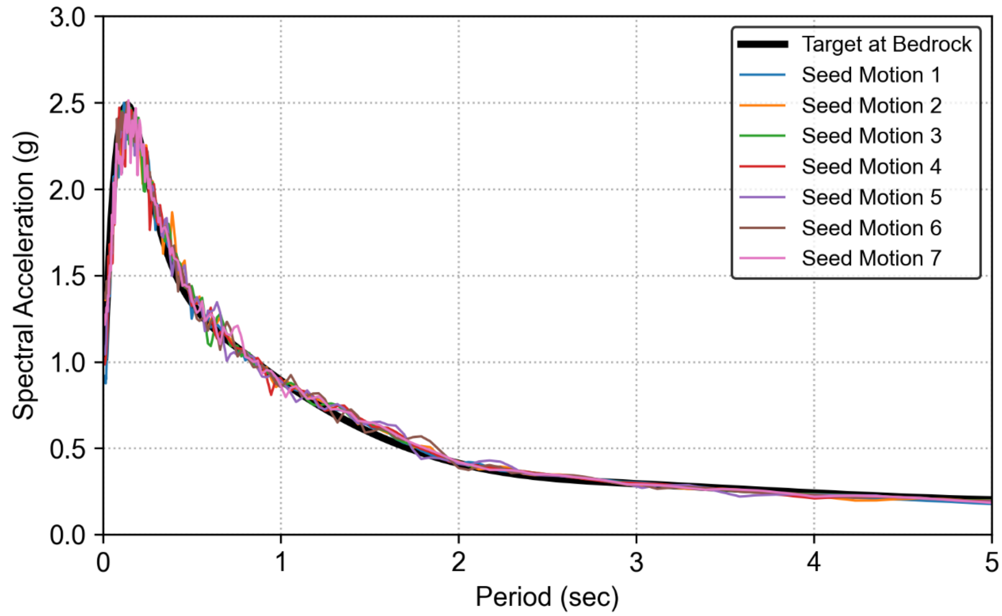



Figure 5 ARS curves of the seed motions

#### 20.3.3.2.2 Perform Two-Dimensional Site-Response Analysis

A 2D site response analysis is performed by applying the seed motions shown in Figure 4 at the top of the bedrock. In this example, the geotechnical FE analysis software, “PLAXIS" was used. Figure 6 depicts the PLAXIS model, including the tunnel cavity and structure backfill placed around the tunnel perimeter.

The stiffness of tunnel structure elements in the model is set to be near zero so that the seismic-induced displacement within the tunnel height is not reduced by the presence of structure elements, which is a conservative approach. To prevent the collapse of the tunnel with near-zero stiffness of the tunnel structure elements, gravity is set to zero. Seismic motions then propagate upward through the soil layer. Note that the zero-gravity setting does not affect seismic wave propagation.

Figure 7 shows total displacements measured at seven points along the left-hand side tunnel wall when subjected to the seed motion 1. The peak displacement at the top of the tunnel relative to the bottom is estimated to be less than 6 in. The seismic-induced maximum shear strains can be calculated by dividing this peak relative displacement by the tunnel height, resulting in a 1.4% shear strain. This maximum shear strain was checked against the result (1.7%) from the closed-form solution conducted as an independent check analysis. The details of the independent check analysis are discussed in Section 20.3.3.13.

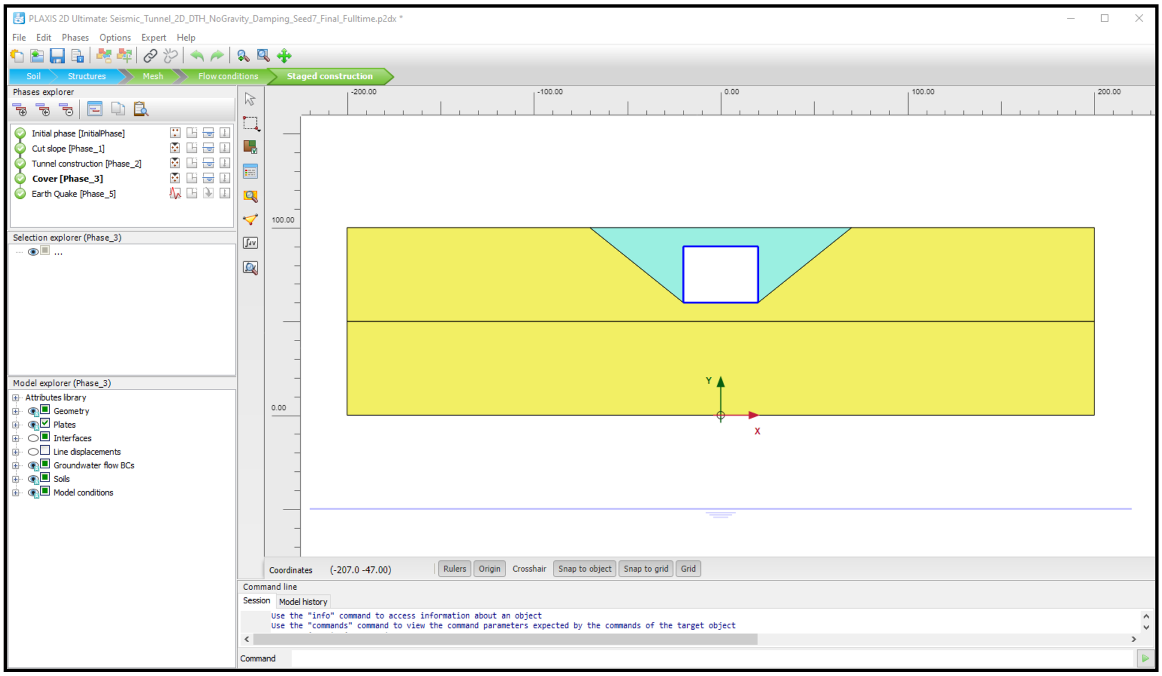
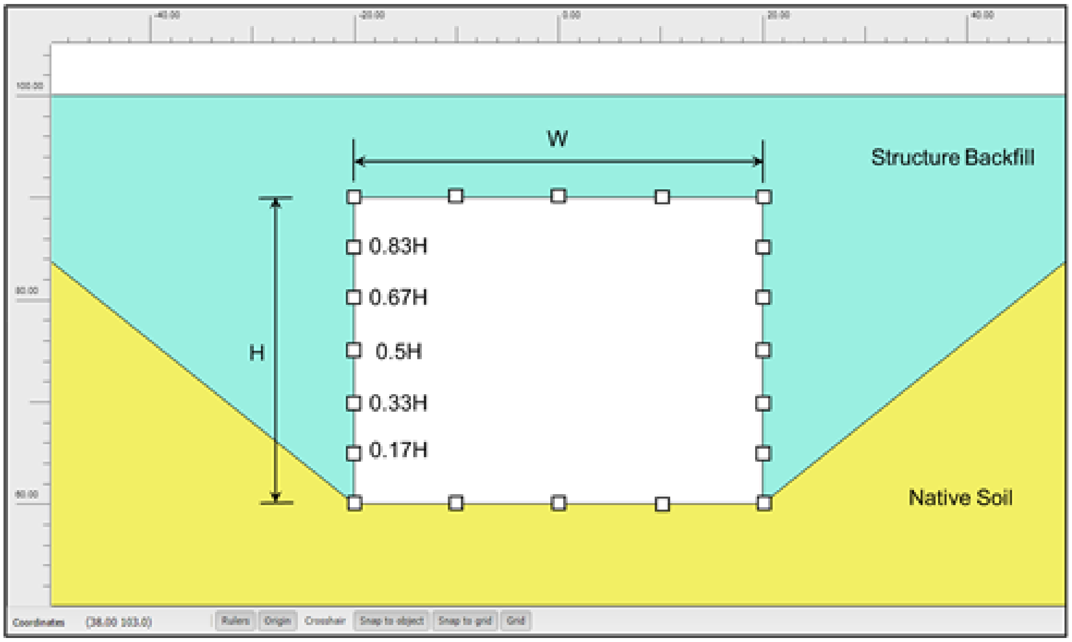
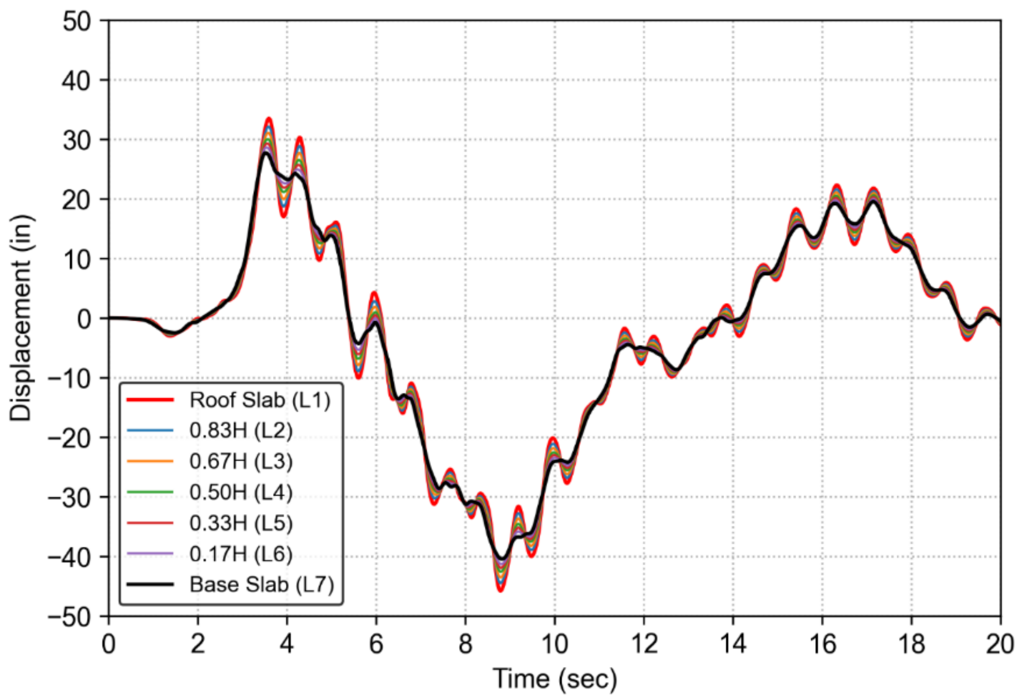



Figure 6 Snapshot of PLAXIS Model




(a)




(b)

Figure 7 (a) Monitoring nodes for total ground DTHs along the tunnel wall in the PLAXIS model (b) Total ground displacement measured at the nodes (H: Wall height, 30 ft)

### 20.3.3.3 STEP 2- DEVELOP SOIL SPRINGS

As mentioned in the earlier section, de-coupled modeling requires a series of soil springs around the tunnel lining, in addition to sets of displacement time histories. Soil springs in the NTHA model are discretized and oriented perpendicular to the surface of the tunnel lining so that they engage normal to the tunnel lining element. Figure 8 shows a schematic of discretized soil springs attached to the structural model for the NTHA, each with a bilinear force-displacement curve.

To develop such soil springs, the GD performs two pushover analyses using the same PLAXIS model created in Section 20.3.3.2 (STEP 1). As illustrated in Figure 9a, one involves bidirectional lateral pushing of the tunnel lining for lateral soil springs attached to the wall. If the model is reflective symmetric along the center line, pushover analysis on one side of tunnel wall elements is sufficient.

The other analysis is to push the tunnel lining upward and downward for vertical springs attached to the top and floor slabs (See Figures 9b and 9c). In both cases, the stiffness of tunnel elements in the PLAXIS model is artificially increased to ten times higher than the actual lining stiffness to minimize the influence of structural deflection on the soil force-deflection curve (For reference, parametric studies were performed to determine the required increase in stiffness).

In the pushover process, forces or displacements are incrementally applied perpendicular to tunnel elements. For this particular example, the method involves using incremental forces. From the pushover analyses, stress-displacement responses are first obtained at the soil-structure interface of the tunnel. Then, the stresses are multiplied by the tributary width to determine the spring force.

Figure 10 depicts the force-displacement response obtained from the tunnel lining nodes. The initial stiffness is summarized in Table 2 and Table 3.

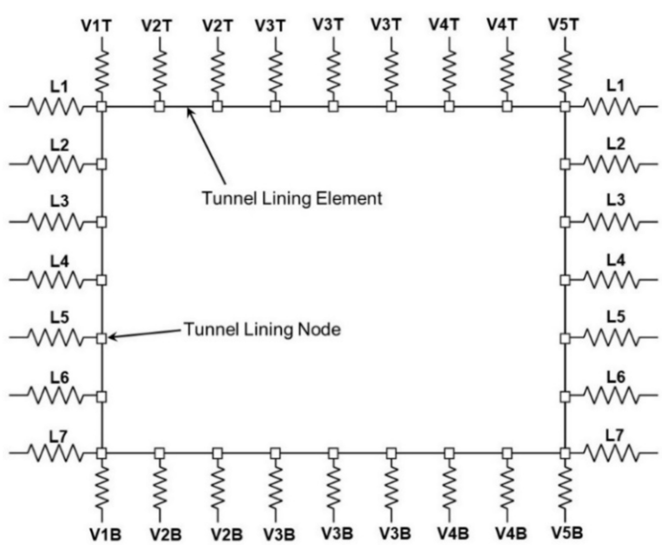



Figure 8. Schematic of soil springs along the perimeter of the tunnel

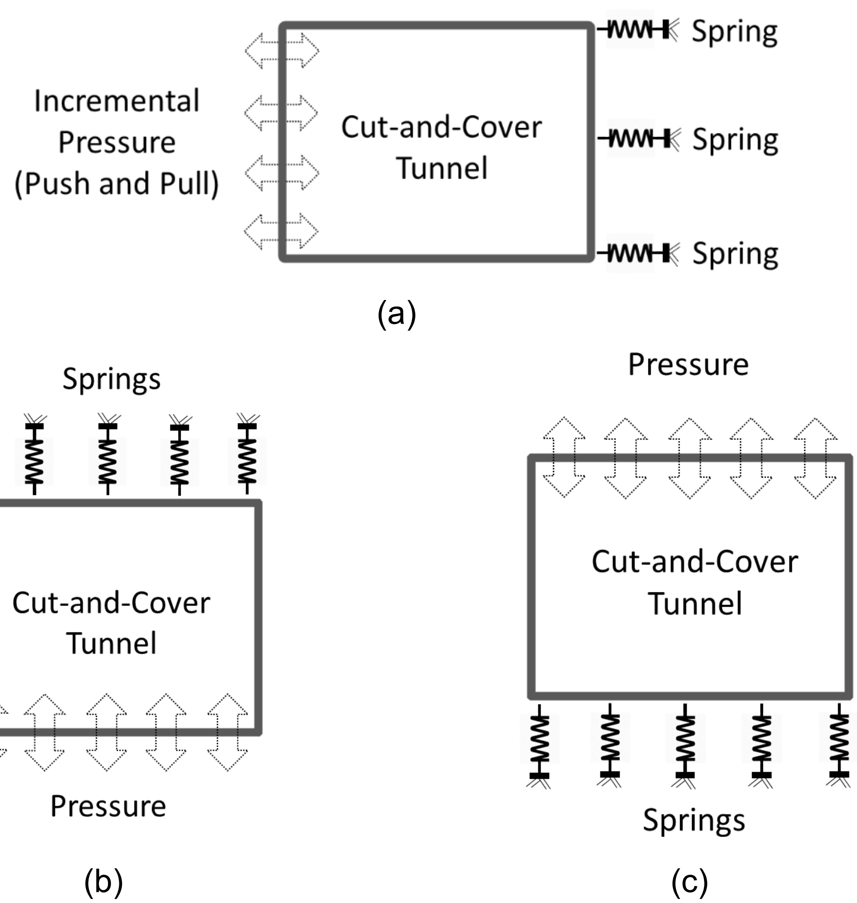



Figure 9. Schematic of geotechnical pushover analysis to develop soil springs normal to the tunnel lining: (a) lateral pushover, and (b) and (c) vertical pushover for roof slab and base slab, respectively

Table 2. Elastic stiffness of soil springs along tunnel walls

| Direction | Soil Spring Stiffness (kip/in) | | | | | | |
|---|---|---|---|---|---|---|---|
| | L1 | L2 | L3 | L4 | L5 | L6 | L7 |
| Transverse (normal) | 5.47 | 17.71 | 24.48 | 31.25 | 57.29 | 83.33 | 54.69 |

Table 3. Elastic stiffness of soil springs along roof and base slabs

| Location | Soil Spring Stiffness (kip/in) | | | | |
|---|---|---|---|---|---|
| | V1 | V2 | V3 | V4 | V5 |
| Roof Slab (T) | 0.74 | 1.47 | 2.35 | 1.47 | 0.74 |
| Base Slab (B) | 47.84 | 95.69 | 97.5 | 95.69 | 47.84 |

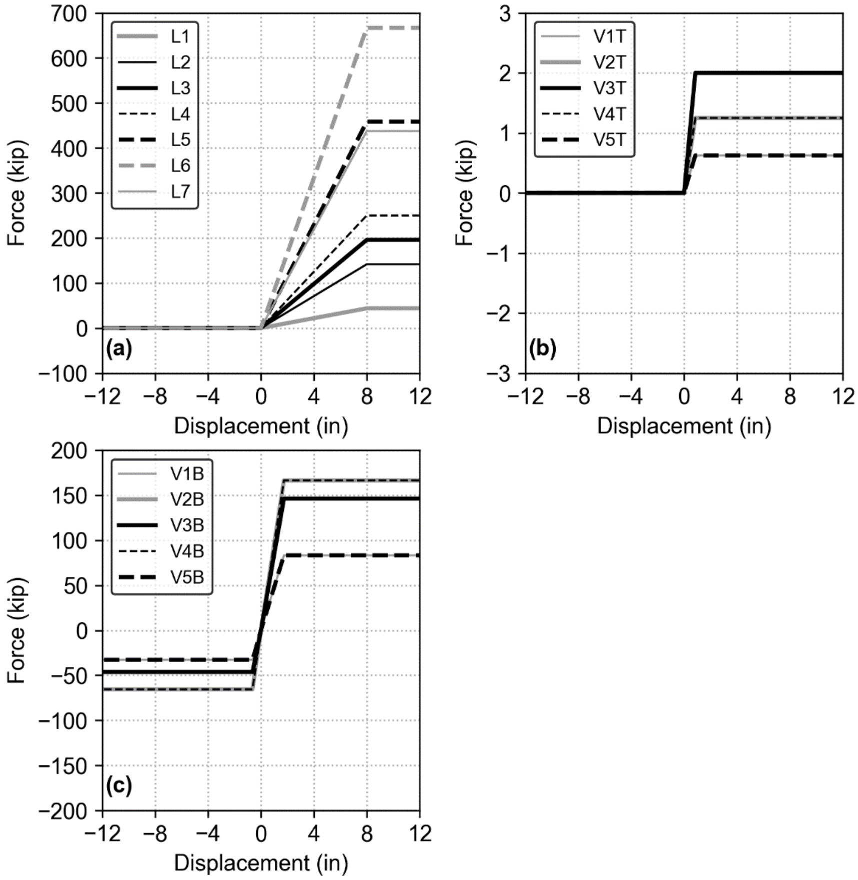



Figure 10. Force-displacement relationship of soil springs (+: movement toward tunnel lining and -: away from tunnel lining)

### 20.3.3.4 STEP 3 – PERFORM TRANSVERSE PUSHOVER ANALYSIS

A pushover analysis, also known as “inelastic static analysis”, serves three main purposes. Firstly, it identifies locations where maximum flexural demands occur so that plastic hinges are assigned to the proper locations. Secondly, it evaluates the displacement ductility capacity of the tunnel system in the transverse direction. Thirdly, it determines the yield displacement, $\Delta_Y$, and the associated shear force, $V_P$, necessary for the initial stiffness of the frame ($V_P / \Delta_Y$). This stiffness will be used in the independent check analysis, as discussed later in 20.3.13.

For pushover analysis, a three-dimensional (3D) frame model built in CSiBridge is used, as shown in Figure 11. In the frame model, the tunnel components (walls and top/bottom slabs) are modeled using beam elements in longitudinal and transverse directions. The cross sections of walls and slabs are defined using the section designer in the program, as shown in Figure 12.

Capturing nonlinearities of soil and structures is crucial in the analysis. Soil springs determined in the previous section are attached along the beam elements. For structural nonlinearity, the concentrated plastic hinges are incorporated into the wall section. The plastic hinges are assigned at locations where maximum bending moments are expected along the walls: the top and bottom of the wall and the bottom two-thirds of the wall height. These locations are determined by preliminary pushover analysis without requiring detailed plastic hinge modeling (see Appendix 20.3.3).

The plastic hinge is at mid-height of the plastic hinge length ($L_p$). The $L_p$ of the wall is determined in accordance with SDC 5.3.4 Case A for this design example as follows:

L = 12 ft (half of the total wall height, H/2)

$L_p = 0.08L + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl}$ (SDC 5.3.4-1)

$L_p = 0.08(12 \times 12) + 0.15(68)(1.27) = 24.5 \text{ in} > 0.3(68)(1.27) = 25.9 \text{ in}$

Use $L_p$ = 25.9 in

Element lengths are properly proportioned in a way that allows for the inclusion of a plastic hinge and its length within a single element. The wall elastic segments have their area based on gross section dimensions, 2.5 $ft^2$/ft, and their moment of inertia based on cracked section properties obtained from moment-curvature analysis, 0.22 $ft^4$/ft, which is defined as 0.17 times the gross moment of inertia.

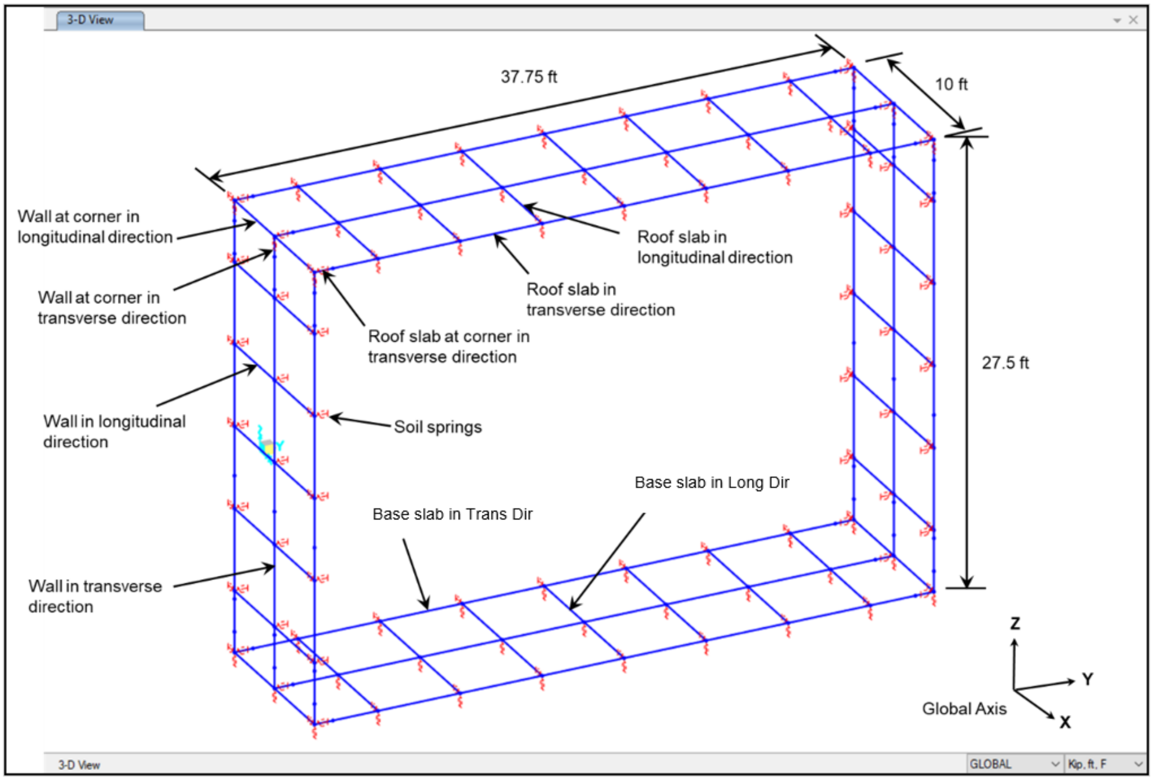



Figure 11. 3D frame model built in CSiBridge model

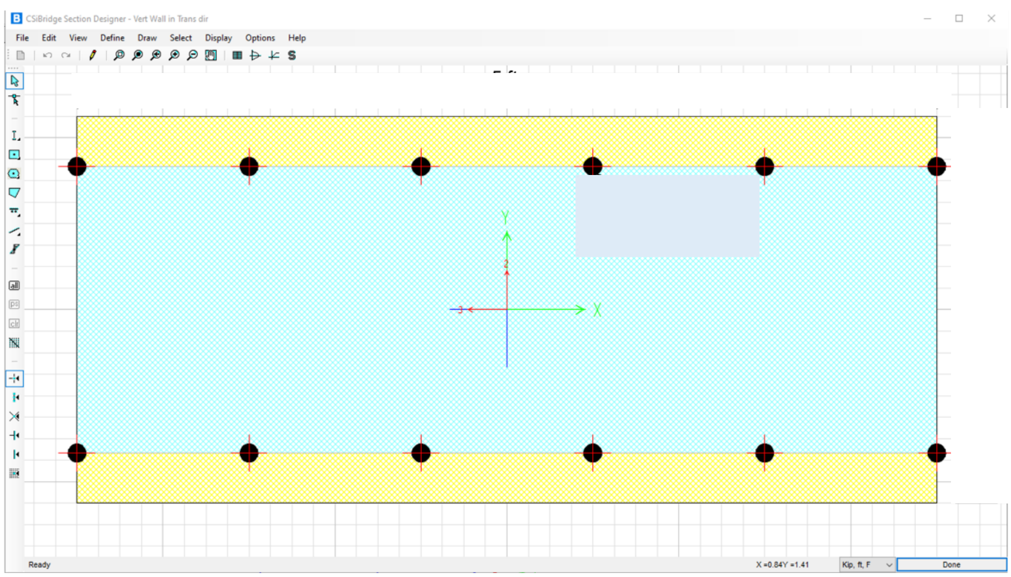



Figure 12. Cross-section of wall in transverse direction defined in CSiBridge

As the frame is pushed in the positive Y direction, the resulting overturning moment causes the redistribution of the axial forces in the walls. Consequently, an additional compression force is applied on the right wall, while the left wall experiences the tension, thereby reducing the net axial load. For this mechanism, the axial forces in the plastic hinge region of the two walls, as read from CSiBridge analysis outputs, are 43.2 kips and 8.4 kips (in compression) per unit length, as summarized in Table 4.

At the instant the first plastic hinge forms (in this case, at the corner between the roof slab and the vertical wall, as shown in Figure 13), the yield lateral displacement and corresponding lateral force values are obtained from CSiBridge outputs (see Appendix 20.3.3). Additionally, the displacement capacity is determined when the plastic hinge reaches ultimate strain first, as summarized in Table 4.

### Axial Load Limits due to Overturning Check

Per SDC 5.3.3, the axial load ratio due to the load and the overturning is checked as follows:

$$\rho_{dl} = \frac{P_{dl}}{f'_c A_g} = \frac{25}{(4)(360)} = 0.017 < 0.15$$

OK (SDC 5.3.3-1)

$$\rho_c = \frac{P_c}{f'_c A_g} = \frac{41.2}{(4)(360)} = 0.029 < 0.22$$

OK (SDC 5.3.3-2)

Table 4. Section Properties with Updated Axial Forces

| Wall | $P_{dl}$ 1) (kip/ft) | $P_c$ 1) (kip/ft) | $M_n$ (kip-ft/ft) | $I_{eff}$ ($ft^4$/ft) | $\Delta_Y$ (in) | $\Delta_C$ (in) | $V_P$ (kips/ft) |
|---|---|---|---|---|---|---|---|
| Right | -25 | -43.2 | 258 | 0.234 | 3.0 | 20 | 45.6 |
| Left | -25 | -8.4 | 224 | 0.200 | 3.0 | 20 | 45.6 |

1) -: Comp +: Ten

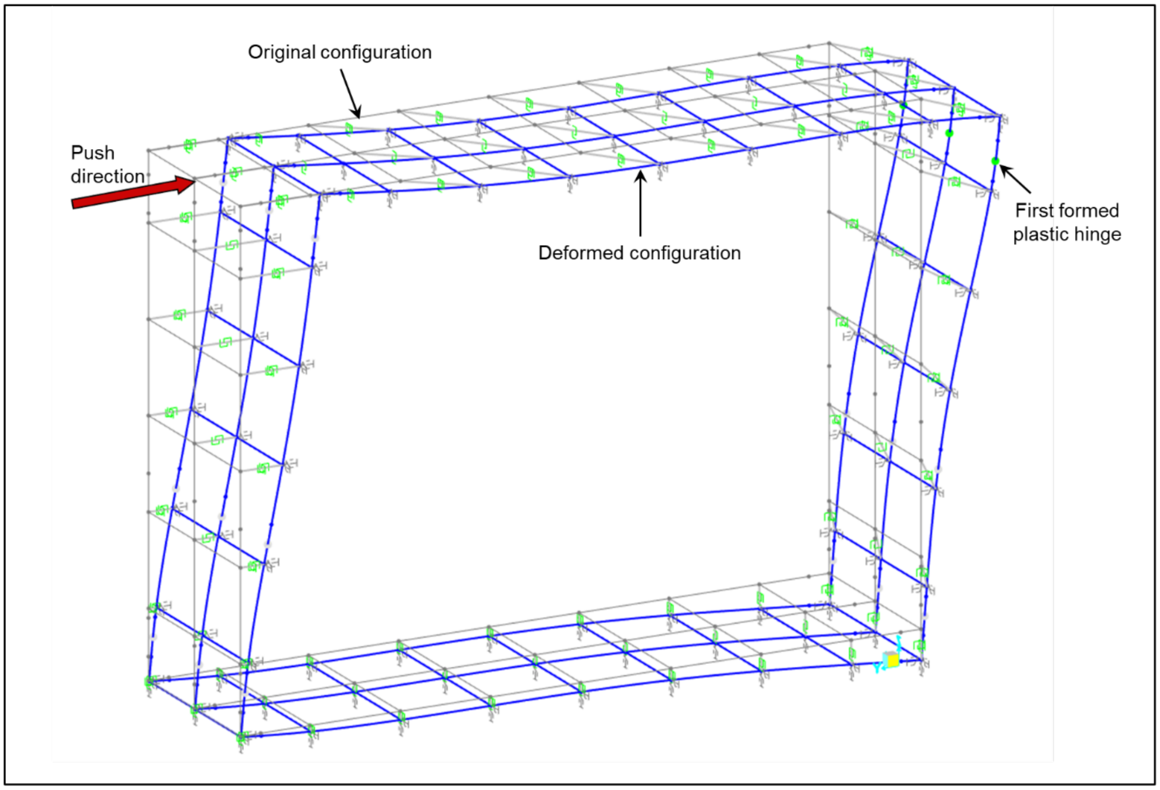



Figure 13. Schematic view of pushover analysis and resulting deformed configuration at first plastic hinge formed at the corner between the roof slab and the vertical wall denoted as green dots

### 20.3.3.5 STEP 4 – PERFORM NONLINEAR TIME HISTORY ANALYSIS

For NTHA, the same 3D frame model in CSiBridge developed for the pushover analysis (Section 20.3.3.4) in CSiBridge is used (Figure 11). Seven NTHAs, named THA1 thru THA7, are performed using the seven input motion sets selected in Section 20.3.2. The input motion sets were applied at the fixities of each soil spring into the transverse direction for each analysis. Per SDC 4.2.3, Rayleigh damping of 3% is specified at two periods where 80% modal mass participation is achieved.

**Analysis Results**

Table 5 summarizes the strain demands in the plastic hinge regions recorded from the seven NTHAs.

For the performance levels established in STP 20.32 Section 20.32.4.2, BDM 20.32.5.3 provides strain limits. The SCMs in this example are designed not to exceed the BDM 20.32 strain limits in plastic hinge regions for SEE events:

$\varepsilon_{steel} < 1/2\ \varepsilon_{su}^{R} = 0.045$

$\varepsilon_{con'c} < 2/3\ \varepsilon_{cu} = 0.0107$

Where,

Reduced ultimate tensile strain, $\varepsilon_{su}^{R}$ = 0.09 for #10 bar and
Ultimate compression strain for confined concrete, $\varepsilon_{cu}$ = 0.016

Comparing the demands with the limits, the resulting strain demands meets the requirements specified above.

Table 5. Summary of strain demands within PHs under SEE events

| **Event No.** | **Location**[1)] | $\varepsilon_{steel}$ | $\varepsilon_{con'c}$ | $\varepsilon_{steel} < 1/2\ \varepsilon_{su}^{R}$ | $\varepsilon_{con'c} < 2/3\ \varepsilon_{cu}$ |
|---|---|---|---|---|---|
| THA1 | Right Top | 0.0233 | 0.00137 | Yes | Yes |
| THA2 | Left Top | 0.0227 | 0.00107 | Yes | Yes |
| THA3 | Right Top | 0.0268 | 0.00123 | Yes | Yes |
| THA4 | Right Top | 0.0280 | 0.00154 | Yes | Yes |
| THA5 | Right Middle | 0.0246 | 0.00153 | Yes | Yes |
| THA6 | Left Top | 0.0221 | 0.00119 | Yes | Yes |
| THA7 | Right Middle | 0.0291 | 0.00133 | Yes | Yes |

1) where highest strain demand is observed during the THA.

Table 6 summarizes the racking deformations (peak displacement demands ($\Delta_D$) at the top of tunnel walls relative to the base slab) resulting from the seven THAs. The average $\Delta_D$ is 7.93 in. In the plastic hinge region near the roof slab, the maximum wall bending moment ($M_{eq}$) on average is estimated to be 291 kip-ft/ft.

Table 6. Peak racking deformation and bending moment at top of tunnel walls

| Event No. | Racking deformation (in) | |
|---|---|---|
| | Left Wall | Right Wall |
| THA1 | 7.81 | 7.81 |
| THA2 | 7.25 | 7.25 |
| THA3 | 7.85 | 7.86 |
| THA4 | 9.64 | 9.64 |
| THA5 | 7.82 | 7.82 |

| | | |
|---|---|---|
| THA6 | 7.08 | 7.08 |
| THA7 | 8.08 | 8.08 |
| Average | 7.93 | 7.93 |

### 20.3.3.6 STEP 5 – DUCTILITY DEMAND AND CAPACITY CHECK

Displacement ductility demand is checked:

$$\mu_D = \frac{\Delta_D}{\Delta_y} = \frac{7.93}{3.0} = 2.64 < 3$$

OK (SDC 4.4.1-1)

The global displacement demand is checked against the global capacity.

$\Delta_D = 7.93$ in $< \Delta_c = 20$ in OK (SDC 3.5.1)

### 20.3.3.7 STEP 6 – CHECK P-Δ EFFECTS

The wall section is checked to meet the P-Δ effect requirement.

$P_{dl} = 24$ kip/ft, $M_p = 246$ kip-ft/ft.

The maximum seismic displacement $\Delta_D = 7.93$ in.

$$\frac{P_{dl}\Delta_D}{M_p} = \frac{(24)(7.93)}{(246)(12)} = 0.064 < 0.25$$

OK (SDC 4.4.4-1)

### 20.3.3.8 STEP 7 – WALL MINIMUM FLEXURAL CAPACITY CHECK

The tunnel walls shall have a minimum plastic moment capacity to resist a lateral force of 10% of the tributary weight of the roof slab and the overburden soil applied as a static load at center of gravity of the roof slab.

Weigh of overburden soil above the roof slab =0.121(10)(30) = 36.3 kips/ft

Weight of roof slab = 0.15(3)(30) = 13.5 kips/ft

Total weight = 36.3 + 13.5 = 49.8 kips/ft

10% of tributary weight to each wall = 0.1(49.8)(0.5) =2.5 kips/ft

Given the lateral load, the resulting bending moment can be checked using simple hand calculations as described below:

$M_{p,min}$ = 224 kip-ft/ft (see Table 4)

$M_{D,10\% \text{ of weight}}$ = 2.5(25.5) = 63.75 kip-ft/ft

Where 25.5 is the distance measured from the top of base floor to CG of the top slab in ft.

$M_{p,min} > M_D$

OK (SDC 5.3.6.1)

### 20.3.3.9 STEP 8 –WALL SHEAR CAPACITY CHECK

BDM 20.32 requires SCMs (walls) to meet a shear force ratio D/C of less than 1.0 in both transverse and longitudinal directions (BDM 20.32.5.3). The shear demand, D, in the BDM 20.32 refers to the overstrength shear associated with the overstrength moment of the SCM in the transverse direction. In the longitudinal direction, the shear demand is $1.2V_{EQ}$ times the mass of top slab. In this example, the shear is checked first in the transverse direction followed by the check in the longitudinal direction.

**Transverse Direction**

In the pushover (toward the right side), a larger plastic moment is observed at the right wall. The overstrength moment of the right wall is calculated as:

$M_o = 1.2M_p = (1.2)(258) = 310$ kip-ft/ft (SDC 4.4.2.1-1)

As mentioned in previous sections, the max bending moments are located at the top of the wall and at the bottom two-third of the wall height (24 ft). The distance from the point of max moment to the point of contra-flexure, L, is estimated as:

2/3 x 24 ft x 1/2 = 8 ft

The shear demand associated with the overstrength moment in the transverse direction is determined by dividing the overstrength by L:

$$V_o = \frac{M_o}{L} = \frac{310}{8} = 38.8 \text{ kips/ft}$$

<u>Concrete Shear Capacity, $V_c$</u>

For cross ties (#5 @ 6 in vertical and @ 8 in horizontal), $A_v$ = 0.465 (= 0.31 x 12/8) $in^2$, $D'_c$ = 8 in per unit length, s = 6 in.

Where $D'_c$ = confined wall cross-section dimension, measured out-to-out of ties, in the direction parallel to the axis of bending (SDC C5.3.8.2-2)

$$\rho_s = \frac{A_v}{D_c^{'}s} = \frac{0.31 \text{ x } (12 \text{ in} / 8\text{in})}{(8)(6)} = 0.0097$$

(SDC C5.3.8.2-2)

$f_{yh}$ = 60 ksi

$\rho_s f_{yh}$ = (0.0097) (60) = 0.58 ksi

If the calculated value is greater than 0.35 ksi, then 0.35 ksi is used.

Table 20.32.6.1-1 of BDM 20.32 shows that the minimum volumetric reinforcement ratio is 0.0025 for cross ties and confinement ties. The above ratio meets the requirement.

The global displacement ductility demand is used in lieu of the local demand per SDC 5.3.7.2, ($\mu_d$ in this example is 2.64), and the shear capacity factor F1 is calculated as:

$$F1 = \frac{\rho_s f_{yh}}{0.15} + 3.67 - \mu_d = \frac{0.35}{0.15} + 3.67 - 2.64 = 3.36 > 3$$

(SDC 5.3.7.2-5)

Since the calculated F1 is greater than 3, 3 is used for F1.

$$F2 = 1 + \frac{P_c}{2{,}000A_g} = 1 + \frac{8.4(1{,}000)}{2{,}000(360)} = 1.012 < 1.5$$

OK (SDC 5.3.7.2-6)

The shear capacity is reduced when the axial load is decreased. The controlling shear capacity will be found near the top of the left wall as the tunnel is deformed toward the left-hand side.

The nominal concrete shear capacity inside and outside the plastic hinge regions are equal due to the shear capacity factor F1 = 3.

$$v_c = (F1)(F2)\sqrt{f'_c} = (3.0)(1.012)\sqrt{4{,}000} = 192.2 \text{ psi} < 4\sqrt{4{,}000} = 253 \text{ psi}$$

OK (SDC 5.3.7.2-3)

$A_e = 0.8A_g = (0.8)(360) = 288 \text{ in}^2/\text{ft}$ (SDC 5.3.7.2-2)

$V_c = v_c A_e = (192.2 \text{ psi})(288 \text{ in}^2/\text{ft}) = 55.4 \text{ kips/ft}$ (SDC 5.3.7.2-1)

Reinforcement Shear Capacity, $V_s$, is

$$V_s = \frac{A_v f_{yh} D'}{s} = \frac{0.31(12\text{ in}/8\text{ in})(60)(8)}{6} = 37.2 \text{ kips/ft}$$

(SDC 5.3.7.3-1)

The maximum shear reinforcement requirement is checked as:

$$V_s < 8\sqrt{f'_c}A_e = (8)\left(\sqrt{4000}\right)(288) = 145.7 \text{ kips/ft}$$

OK (SDC 5.3.7.4-1)

The minimum shear reinforcement requirement is checked as:

$$A_v = 0.465\,\text{in}^2/\text{ft} > 0.025\left(\frac{D's}{f_{yh}}\right) = 0.025\frac{(8)(6)}{60} = 0.02\ \text{in}^2/\text{ft}$$

(SDC 5.3.7.5-1)

The shear capacity is:

$$\phi V_n = (1.0)(V_c + V_s) = (1.0)(55.4 + 37.2) = 92.6 \text{ kips/ft} > V_o = 38.8 \text{ kips/ft}$$

OK

**Longitudinal Direction**

Per BDM 20.32.4.2.1, the shear demand is $1.2V_{EQ}$ times the mass applied to the wall, where $V_{EQ}$ represents the PGA at the mid-height of the tunnel, estimated to be 1.0 g. The larger axial load, $P_c$, determined in Section 20.3.3.5 is 43.2 kips/ft.

$$V_o = 1.2 \times 1.0 \text{ g} \times 43.2 \text{ kips/ft/g} = 51.84 \text{ kips/ft}$$

The shear capacity in the longitudinal direction is calculated using the SDC 5.3.7 equations shown above using the total sectional area of the confinement tie ($A_v$ = 0.31 in$^2$) spaced at $D'_C$ of 24.125 in (= 30 – 4 – 0.625 x 2 – 0.625) horizontally and 6 in vertically (s = 6 in).

$$\rho_s = \frac{A_v}{D'_c s} = \frac{0.31 \times 2 \text{ legs}}{(24.125)(6)} = 0.0043$$

(SDC C5.3.8.2-2)

Table 20.32.6.1-1 of BDM 20.32 shows the minimum volumetric reinforcement ratio is 0.0025 for cross ties and confinement ties. The above ratio meets the requirement.

$$\rho_s f_{yh} = (0.0043)(60) = 0.258 \text{ ksi}$$

$$F1=\frac{\rho_s f_{yh}}{0.15}+3.67-\mu_d=\frac{0.258}{0.15}+3.67-2.64=2.75<3$$

Since the calculated F1 is less than 3, 2.75 is used for F1.

$$F2=1+\frac{P_c}{2{,}000A_g}=1+\frac{43.2(1000)}{2{,}000(360)}=1.06<1.5$$

$$v_c=(F1)(F2)\sqrt{f'_c}=(2.75)(1.06)\sqrt{4{,}000}=184.4\text{ psi}<4\sqrt{4{,}000}=253\text{ psi}$$

OK (SDC 5.3.7.2-3)

$A_e = 0.8A_g = (0.8)(360) = 288\text{ in}^2/\text{ft}$ (SDC 5.3.7.2-2)

$V_c = v_cA_e = (184.4\text{ psi})(288\text{ in}^2/\text{ft}) = 53.4\text{ kips/ft}$ (SDC 5.3.7.2-1)

Reinforcement Shear Capacity, $V_s$ is

$$V_s=\frac{A_v f_{yh} D'}{s}=\frac{(0.31\times 2)(60)(24.125)}{6}=149.6\text{ kips/ft}$$

(SDC 5.3.7.3-1)

The maximum shear reinforcement requirement is checked as:

$V_s < 8\sqrt{f'_c}A_e = (8)\left(\sqrt{4000}\right)(288) = 145.7\text{ kips/ft}$

Use 145.7 kips/ft (SDC 5.3.7.4-1)

The minimum shear reinforcement requirement is checked as:

$$A_v = 0.31\text{ in}^2/\text{ft} > 0.025\left(\frac{D's}{f_{yh}}\right) = 0.025\left(\frac{24.125\text{ x }6}{60}\right) = 0.06\text{ in}^2/\text{ft}$$

(SDC 5.3.7.5-1)

The shear capacity is:

$\phi V_n = (1.0)(V_c + V_s) = (1.0)(53.4 + 145.7) = 199\text{ kips/ft} > V_o = 51.84\text{ kips/ft}$

OK

### 20.3.3.10 STEP 9 – DESIGN JOINT SHEAR REINFORCEMENT

Moment resisting connections between the wall and the roof slab shall be designed to resist the wall overstrength demands, $M_o$ and $V_o$, while remaining essentially elastic in accordance with SDC 7.4. The connection at the top of the wall to the roof in this example is considered “Case 1” knee joint per SDC 7.4.4.2-2. To satisfy the joint proportioning, the principal stresses shall be limited as follows:

**Closing Failure Mode**

The closing failure mode is expected at the wall under compression (the right wall in the pushover analysis discussed in Section 20.3.3.4). Firstly, principal stresses, $p_t$ and $p_c$, are calculated. Note that $B_{cap}$ in SDC is considered the unit width of the top slab (1 ft) in this example.

Vertical shear stress, $v_{jv}$

$$T_c = M_o/h = 1.2M_p/h = (1.2)(258 \text{ kips-ft/ft})/(2.2 \text{ ft}) = 141 \text{ kips/ft}$$

where, h = the distance between CG of compressive force and tension force on the wall section

This $T_c$ is checked using the tension force taken directly from the Section Designer of CSiBridge (122 kips/ft).

$$A_{jv} = l_{ac,provided}(B_{cap}) = (48)(12) = 576 \text{ in}^2 \qquad \text{(SDC 7.4.2-9)}$$

Where $l_{ac,provided}$ = 36 in top slab depth – 3 in cover + 12 $d_b$ after 90-degree hook

$$v_{jv} = T_c / A_{jv} = (141)/(576) = 0.245 \text{ ksi/ft} \qquad \text{(SDC 7.4.2-7)}$$

Normal stress (vertical), $f_v$

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c+D_s)B_{cap}} = \frac{43.2}{(30+36)(12)} = 0.055 \text{ ksi/ft}$$

(SDC 7.4.2-6)

Horizontal normal stress, $f_h$, resulting from the lateral earth pressure.

$$P_b = 371 \text{ kips} / 5 \text{ ft} = 74 \text{ kips/ft (from CSiBridge pushover)}$$

$$f_h = \frac{P_b}{B_{cap}D_s} = \frac{74}{(12)(36)} = 0.171 \text{ ksi/ft}$$

(SDC 7.4.2-5)

The principal stresses acting on a roof-to-wall joint are calculated as:

$$p_t = \frac{(f_h+f_v)}{2} - \sqrt{\left(\frac{(f_h-f_v)}{2}\right)^2 + v_{jv}^2} = \frac{(0.171 + 0.055)}{2} - \sqrt{\left(\frac{0.171 - 0.055}{2}\right)^2 + 0.245^2}$$

= -0.139 ksi/ft (- for joint in tension) (SDC 7.4.2-3)

$$p_c = \frac{(f_h+f_v)}{2} + \sqrt{\left(\frac{(f_h-f_v)}{2}\right)^2 + v_{jv}^2} = \frac{(0.171 + 0.055)}{2} + \sqrt{\left(\frac{0.171 - 0.055}{2}\right)^2 + 0.245^2}$$

= 0.365 ksi/ft (+ for joint in compression) (SDC 7.4.2-4)

Joint size adequacy can be checked using the computed principal compression and tension stresses as follows.

Principal compression,

$p_c$= 0.365 ksi/ft < 0.25f'$_c$ = 0.25 (4) = 1.0 ksi OK

Principal tension,

$p_t$ = 0.139 ksi/ft < 12√f'$_c$ = 12(√4000)/1000 = 0.849 ksi OK

Minimum joint shear reinforcement can be checked by following the SDC 7.4.5.1 requirement. Since $p_t$ is less than 3.5√f'c (= 0.22 ksi = (3.5)(√4000)/1000), only the minimum joint shear reinforcement shall be provided (SDC 7.4.5). The required minimum ratio is

$$\rho_{s,min} = \frac{3.5\sqrt{f'c}}{f_{yh}} = \frac{(3.5)(\sqrt{4000})}{60{,}000} = 0.0037$$

(SDC 7.4.5.1-1)

SDC C7.4.5.1 states that minimum joint shear reinforcement may be provided in the form of wall transverse steel continued to the bent cap. In this example, this requirement can be met by extending the cross ties and confinement ties in the wall to the joint section.

**Opening Failure Mode**

The closing failure mode is expected at the wall under tension (the left wall in the pushover analysis discussed in Section 20.3.3.4). The same procedure for the closing failure mode can be repeated using:

Pc = -8.4 kip/ft and $M_p$ = 224 kip-ft/ft

Vertical shear stress, $v_{jv}$

$T_c = M_o/h = 1.2M_p/h$ = (1.2)(224)/(2.2) = 122.2 kips/ft (SDC Section 7.4.2)

$A_{jv} = l_{ac,provided}(B_{cap})$ = (48) (12) = 576 in$^2$ (SDC 7.4.2-9)

Where $l_{ac,provided}$ = 36 in top slab depth – 3 in cover + 12 $d_b$ after 90-degree hook

$v_{jv} = T_c / A_{jv}$ = (122.2)/(576) = 0.212 ksi/ft (SDC 7.4.2-7)

Normal stress (vertical), $f_v$

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c+D_s)B_{cap}} = \frac{8.4}{(30+36)(12)} = 0.0106 \text{ ksi/ft}$$

(SDC 7.4.2-6)

Horizontal normal stress, $f_h$ resulting from the lateral earth pressure

$$f_h = \frac{P_b}{B_{cap}D_s} = \frac{74}{(12)(36)} = 0.171 \text{ ksi/ft}$$

(SDC 7.4.2-5)

The principal stresses acting on a roof-to-wall joint are calculated as:

$$p_t = \frac{(f_h+f_v)}{2} - \sqrt{\left(\frac{(f_h-f_v)}{2}\right)^2 + v_{jv}^2} = \frac{(0.171 + 0.011)}{2} - \sqrt{\left(\frac{0.171 - 0.011}{2}\right)^2 + 0.212^2}$$

= -0.136 ksi (- for joint in tension) (SDC 7.4.2-3)

$$p_c = \frac{(f_h+f_v)}{2} + \sqrt{\left(\frac{(f_h-f_v)}{2}\right)^2 + v_{jv}^2} = \frac{(0.171 + 0.011)}{2} + \sqrt{\left(\frac{0.171 - 0.011}{2}\right)^2 + 0.212^2}$$

= 0.318 ksi (+ for joint in compression) (SDC 7.4.2-4)

Joint size adequacy can be checked using the computed principal compression and tension stresses as follows.

Principal compression,

$p_c$= 0.318 ksi/ft < 0.25f'$_c$ = 0.25 (4) = 1.0 ksi OK

Principal tension,

$p_t$ = 0.136 ksi/ft < 12√f'$_c$ = 12(√4000)/1000 = 0.849 ksi OK

Minimum joint shear reinforcement can be checked by following the SDC 7.4.5.1 requirement. Since $p_t$ is less than 3.5√f'c (0.22 ksi = (3.5)(√4000)/1000), only the minimum joint shear reinforcement shall be provided (SDC 7.4.5). The required minimum ratio is:

$$\rho_{s,min}=\frac{3.5\sqrt{f'c}}{f_{yh}}=\frac{(3.5)(\sqrt{4000})}{60,000}=0.0037$$

(SDC 7.4.5.1-1)

SDC C7.4.5.1 states that minimum joint shear reinforcement may be provided in the form of wall transverse steel continued to the bent cap. In this example, this requirement can be met by extending the cross ties and confinement ties in the wall to the joint section at the top of the left wall.

### 20.3.3.11 STEP 10 – PLASTIC HINGE REGION AND SPLICING OPTION CHECK

**Plastic Hinge Region**

Lateral confinement (#5 @ 6 in) shall be positioned within the extent of the plastic hinge region as per SDC 5.3.2, which is the larger of:

1.5 x width of wall (2.5 ft) = 3.75 ft

0.25 x L (8 ft) = 2 ft

The region of the moment greater than $0.75M_p$ = 7.5 ft

Where L represents the distance from the point of max moment to the contra flexure point.

#5 @ 6 shall be placed within 7.5 ft around the midpoint of the plastic hinge location. However, due to the constructability concerns, the wall is designed and detailed with #5 cross ties and confinement ties spaced 5 inches vertically throughout the entire height of the walls.

BDM 20.32 stipulates that the confinement in SCMs (walls in this example) should extend into the roof slab within the same plastic hinge length defined for the SCMs (see Section 20.32.6.1). Therefore, the same confinements in the walls are placed within 10 ft of the roof slab, measured from the interior face of the wall.

**Splicing Requirement**

According to SDC 8.2.2.1, splicing of the transverse (main) reinforcement is not needed within the walls due to its height being less than 60 ft, which allows for the use of a single length of commercially available reinforcing bar.

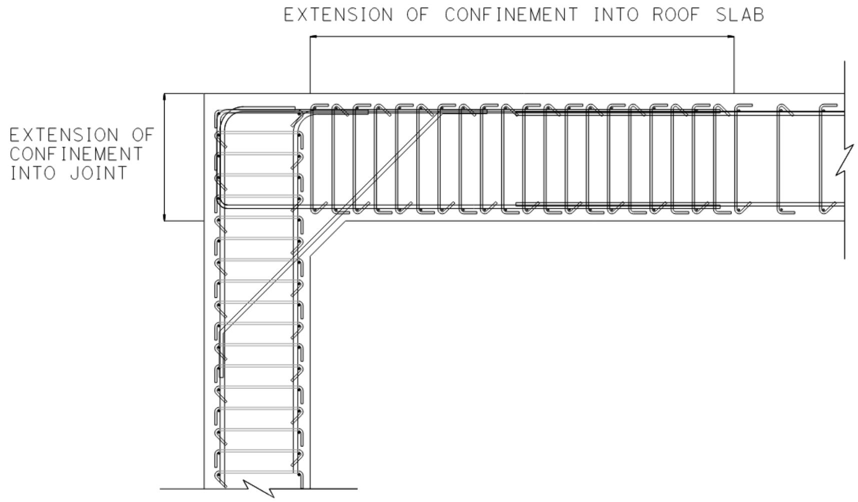



Figure 14. Reinforcement details at the corner of the roof and wall

### 20.3.3.12 Step 11 – Check Roof Slab Flexural and Shear Capacity

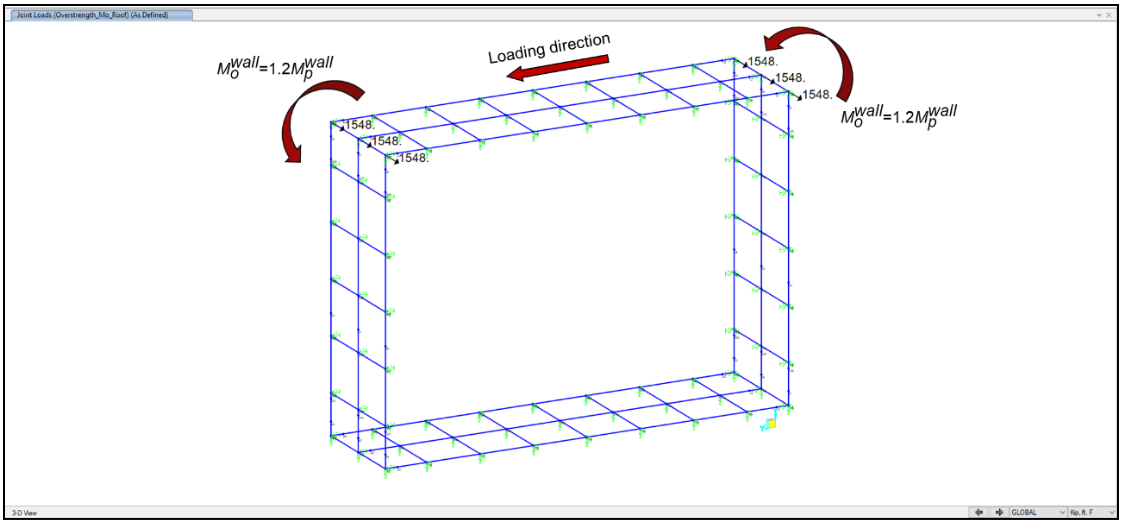
As a capacity protected member (CPM), the roof slab shall be designed to resist the overstrength demands resulting from the side walls. The overstrength moment of the side wall shall be taken as:

$M_o = 1.2\ M_p = (1.2)(258 \text{ kip-ft/ft}) = 309.6 \text{ kip-ft/ft}.$

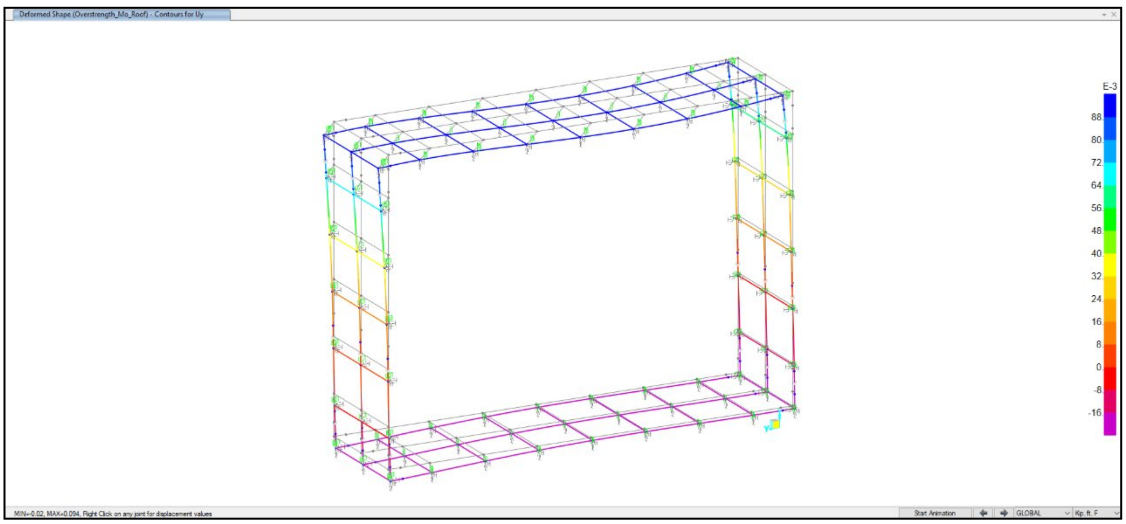
This overstrength moment is applied to the roof slab of the CSiBridge model, as illustrated in Figure 15. The shear and bending moment demands associated with $M_o$ are denoted as EQ thereafter. Their diagrams along the roof slab are shown in Figure 16 and the values are summarized in Table 7. Additionally, the demands due to DC, EH, and EV are depicted and tabulated. Note that the demands presented are per unit length in the longitudinal direction. The required reinforcement in the roof slab is evaluated against the factored shear demand and bending moment. STP 20.32.4 mandates Extreme Event T-1 in the AASHTO LRFD Road Tunnel Design and Construction Guide Specification, where the load factors for DC, EH, EV, and EQ are 1.25, 1.35, 1.35, and 1.0, respectively.

**Vertical Acceleration Analysis Requirement**

SDC 7.2.2 discusses the requirement for the effect of vertical ground motion. In this example, the PGA is greater than 0.6g, which is the minimum PGA for the consideration of the vertical ground motion in seismic design. The effect is accounted for by increasing the demand due to DC by 25%. Since the flexural check in this section uses the 25% increased DC and the 35% increased EV, this requirement is implicitly fulfilled.




(a) Schematic view of applying the overstrength moment




(b) Deformed configuration (color contour indicates the transverse deformation)

Figure 15. CSiBridge model applying the overstrength moment of the side walls to the roof slab

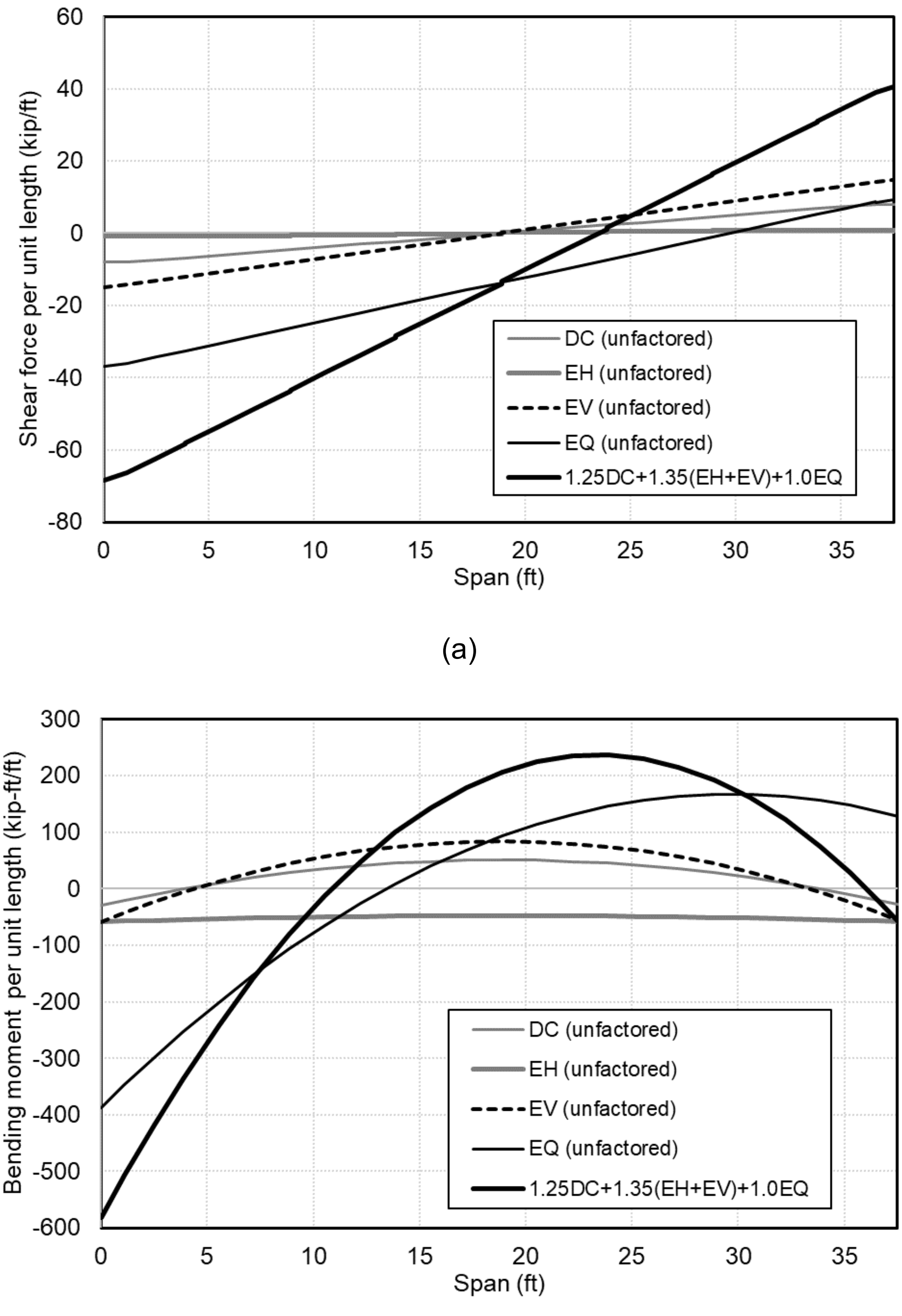



Figure 16. Shear force (a) and bending moment (b) along the roof slab applying the overstrength moment of side walls (Note: span length is measured from the interior wall face)

Table 7. Forces along roof slab resulting from CSiBridge

| Location | Span Length (ft) | Load Combination: 1.25DC + 1.35 (EH + EV) + 1.0EQ | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Axial Force (kip/ft)[1] | | | | | Bending Moment (kip-ft/ft) | | | | | Shear Force (kip/ft) | | | | |
| | | DC | EH | EV | EQ | Combined | DC | EH | EV | EQ | Combined | DC | EH | EV | EQ | Combined |
| 0 | 0 | -1.7 | -15.4 | -3.8 | -19.3 | -47.4 | -20.7 | -56.9 | -42.0 | -347.3 | -506.6 | -8.0 | -0.8 | -14.2 | -36.0 | -66.3 |
| 0.1 | 3.5 | -1.7 | -15.4 | -3.8 | -19.3 | -47.4 | 4.9 | -54.0 | 3.2 | -228.3 | -290.9 | -6.4 | -0.7 | -11.4 | -31.5 | -55.9 |
| 0.2 | 7 | -1.7 | -15.4 | -3.8 | -19.3 | -47.4 | 24.9 | -51.6 | 38.5 | -124.5 | -111.0 | -4.8 | -0.7 | -8.5 | -27.1 | -45.6 |
| 0.3 | 10.5 | -1.7 | -15.4 | -3.8 | -19.3 | -47.5 | 39.2 | -49.6 | 63.8 | -36.3 | 31.9 | -3.2 | -0.5 | -5.7 | -22.6 | -35.0 |
| 0.4 | 14 | -1.7 | -15.4 | -3.8 | -19.4 | -47.5 | 47.8 | -48.3 | 79.0 | 36.3 | 137.6 | -1.6 | -0.2 | -2.8 | -18.1 | -24.2 |
| 0.5 | 17.5 | -1.7 | -15.4 | -3.8 | -19.4 | -47.6 | 50.7 | -47.6 | 84.2 | 93.1 | 205.9 | 0.0 | 0.2 | 0.0 | -13.6 | -13.4 |
| 0.6 | 21 | -1.7 | -15.4 | -3.8 | -19.4 | -47.6 | 47.8 | -48.3 | 79.0 | 133.6 | 234.8 | 1.6 | 0.2 | 2.8 | -9.2 | -3.1 |
| 0.7 | 24.5 | -1.7 | -15.4 | -3.8 | -19.4 | -47.5 | 39.2 | -49.6 | 63.8 | 158.2 | 226.3 | 3.2 | 0.5 | 5.7 | -4.7 | 7.7 |
| 0.8 | 28 | -1.7 | -15.4 | -3.8 | -19.4 | -47.5 | 24.9 | -51.6 | 38.5 | 166.9 | 180.3 | 4.8 | 0.7 | 8.5 | -0.3 | 18.3 |
| 0.9 | 31.5 | -1.7 | -15.4 | -3.8 | -19.4 | -47.5 | 4.9 | -54.0 | 3.2 | 159.6 | 97.1 | 6.4 | 0.7 | 11.4 | 4.2 | 28.5 |
| 1 | 35 | -1.7 | -15.4 | -3.8 | -19.4 | -47.4 | -20.7 | -56.9 | -42.0 | 136.9 | -22.4 | 8.0 | 0.8 | 14.2 | 8.6 | 39.0 |

[1] -: Compression, +: Tension

#### 20.3.3.12.1 Check at Wall Face

Required roof slab flexural capacity can be checked by following the steps below.

Determine the factored ultimate moment at the Extreme Limit State, $M_u$ (see Table 7):

$$M_u = 1.25M_{DC} + 1.35M_{EH} + 1.35M_{EV} + 1.0M_{EQ}$$

$$= 1.25(-20.7) + 1.35(-56.9 - 42.0) + 1.0(-347.3) = -507 \text{ kip-ft /ft}$$

Compute the nominal moment resistance of the section:

For the transverse reinforcement (#9 @ 8 inch – bundle bars) along the unit length of the tunnel,

$$A_s = (1.00)(12/8)(3) = 4.5 \text{ in}^2\text{/ft}$$

Assuming no compression steel (i.e., $A'_s$ is zero).

b = unit length of wall cross section = 12 in

$f'_c = 4$ ksi

Per AASHTO 5.6.2.2,

$$\alpha_1 = 0.85$$

$$\beta_1 = 0.85$$

$$c = \frac{A_s f_s}{\alpha_1 f'_c \beta_1 b} = \frac{(4.5)(60)}{(0.85)(4)(0.85)(12)} = 7.79 \text{ in}$$

(AASHTO 5.6.3.1.1-4)

Using Equation AASHTO 5.6.3.2.2-1,

$$a = \beta_1 c = (0.85)(7.79) = 6.62 \text{ in}$$

$$M_n = A_s f_s \left(d_p - \frac{a}{2}\right) = (4.5)(60)\left(32.625 - \frac{6.62}{2}\right) = 7915 \text{ kip-in/ft} = 660 \text{ kip-ft/ft}$$

Per SDC 3.6, the resistance factor for flexure, $\phi = 1.0$.

$\phi M_n = (1.0)(660) = 660$ kip-ft/ft > $M_u = 507$ kip-ft/ft OK

Check Roof Slab Shear Capacity

For cross ties (#5 @ 6 in transverse and @ 8 in longitudinal direction),

$A_v$ = (0.31)(12/8) = 0.465 $in^2$/ft

$d_v$ = effective shear depth = 32.625 - 5.6/2 = 29.825 in

The shear stress on the concrete is calculated in accordance with AASHTO 5.7.2.8-1:

$$v_u = \frac{|V_u|}{\phi b d_v} = \frac{|-66.3|}{(1.0)(12)(29.825)} = 0.185 \text{ ksi}$$

$$\frac{v_u}{f_c'} = \frac{0.185}{4} = 0.04625$$

Determine the factored axial force:

$N_u = 1.25P_{DC} + 1.35P_{EH} + 1.35P_{EV} + 1.0P_{EQ}$

= (1.25)(-1.7) + (1.35)(-15.4) + (1.35)(-3.8) + (1.0)(-19.3) = -47.4 kip/ft

Using AASHTO B5.2-3 with the factored axial force, begin with $\theta$ = 45° (i.e., cot$\theta$ =1), calculate $\varepsilon_x$:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u|\cot\theta\right)}{2E_sA_s} = \frac{\left(\frac{507 \times 12}{29.825} + (0.5)(-47.4) + (0.5)(66.3)(1)\right)}{(2)(29000)(4.5)} = 0.817 \times 10^{-3}$$

From AASHTO Table B5.2-1 using the calculated values of $v_u/f'_c$ = 0.04625 and $\varepsilon_x$ = 0.817 x $10^{-3}$,

$\theta$ = 34.4° and $\beta$ = 2.34

The nominal shear resistance, $V_n$ is the lesser of the following per AASHTO 5.7.3.3:

$V_n = V_c + V_s$

$V_n = 0.25f'_cb_vd_v$

In which:

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_vd_v$$

where:

$V_c$ = nominal shear resistance provided by the tensile stresses in the concrete (kip)

$\beta$ = factor relating effect of longitudinal strain on the shear capacity of concrete as indicated by the ability of diagonally cracked concrete to transmit tension

$\lambda$ = concrete density modification factor

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

where:

$A_v$ = area of transverse reinforcement within distance s (in$^2$)

$\alpha$ = angle of inclination of transverse reinforcement to the longitudinal axis (degree)

s = spacing of reinforcing bars (in)

The nominal shear resistance provided by the concrete is calculated as:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\,b_v d_v = (0.0316)(2.34)(1)\left(\sqrt{4}\right)(12)(29.825) = 52.9 \text{ kips}$$

Using $\alpha$ = 90°, the nominal shear resistance provided by cross ties is calculated as:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.465)(60)(29.825)(\cot 36.4)}{6} = 188.1 \text{ kips}$$

$$V_n = V_c + V_s = 52.9 + 188.1 = 241 \text{ kips}$$

$$V_n = 241 \text{ kips} < 0.25f'_c b_v d_v = (0.25)(4)(12)(29.825) = 357.9 \text{ kips.}$$

The factored nominal shear capacity is:

$$\phi V_n = (1.0)(241) = 241 \text{ kips} > V_u = 66 \text{ kips} \qquad \text{OK}$$

The flexural and shear capacity calculations at other locations (regions) along the roof slab are summarized in Table 8 and Table 9, respectively.

It is important to note that the required reinforcements to withstand the flexural demands in each section must have a development length extending beyond the region.

**Splicing requirement**

Splicing is not permitted in the critical zone where the demand is greater than 75% of maximum factored bending moment (380.25 kip-ft/ft = 0.75 x 507). This region is within 4 ft measured from the interior wall face on both sides. Outside these regions, service splice shall be used per SDC 8.2.3.1.

| Location | Span | Factored Forces | | | Flexural Capacity (AASHTO LRFD BDS 5.6.3.2) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Axial | Bending Moment | Shear Force | Rebar | Spacing[1)] | As | c | a | $\phi M_n$ | Check | $\varepsilon_t$ | Check |
| | ft | kip/ft | kip-ft/ft | kip/ft | | in | $in^2$ | in | in | kip-ft/ft | | | |
| 0 | 0 | -47.4 | -506.6 | -66.3 | #9 (3 bundle) | 8 (T & B) | 4.5 | 7.8 | 6.6 | 660 | OK | 0.0133 | OK |
| 0.1 | 3.5 | -47.4 | -290.9 | -55.9 | #9 (3 bundle) | 8 (T & B) | 4.5 | 7.8 | 6.6 | 660 | OK | 0.0133 | OK |
| 0.2 | 7 | -47.4 | -111.0 | -45.6 | #9 | 8 (T & B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.3 | 10.5 | -47.4 | 31.9 | -35.0 | #9 | 8 (B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.4 | 14 | -47.4 | 137.6 | -24.2 | #9 | 8 (B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.5 | 17.5 | -47.4 | 205.9 | -13.4 | #9 | 8 (B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.6 | 21 | -47.4 | 234.8 | -3.1 | #9 | 8 (B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.7 | 24.5 | -47.4 | 226.3 | 7.7 | #9 | 8 (B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.8 | 28 | -47.4 | 180.3 | 18.3 | #9 | 8 (T & B) | 2.25 | 3.9 | 3.3 | 348 | OK | 0.0296 | OK |
| 0.9 | 31.5 | -47.4 | 97.1 | 28.5 | #9 (3 bundle) | 8 (T & B) | 4.5 | 7.8 | 6.6 | 660 | OK | 0.0133 | OK |
| 1 | 35 | -47.4 | -22.4 | 39.0 | #9 (3 bundle) | 8 (T & B) | 4.5 | 7.8 | 6.6 | 660 | OK | 0.0133 | OK |

[1)] T: Top Mat, B: Bottom Mat

Table 8. Flexural check along the roof-slab subjected to overstrength moment

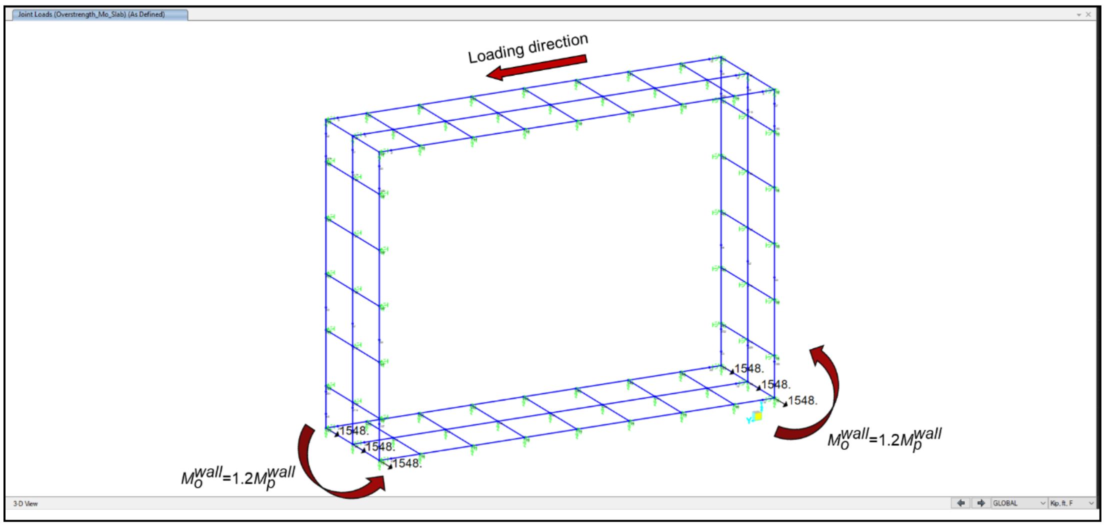
Table 9. Shear check along the roof-slab subjected to overstrength moment

| Location | Span | Shear Resistance (AASHTO LRFD BDS 5.7.3.3) | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cross tie | Spacing | No. bar per ft | $A_v$ | $v_u$ | $v_u/f'_c$ | $\varepsilon_x$ (x1000) | θ | b | $\varepsilon_x$ (x1000) | θ | b | $V_c$ | $V_s$ | $\phi V_n$ | Vu | Check |
| | ft | | in | | in² | ksi | | (1st trial) | deg | | (2nd trial) | deg | | kip | kip | kip | kip | |
| 0 | 0.0 | #5 | 6 | 1.5 | 0.465 | 0.185 | 0.046 | 0.817 | 34.4 | 2.34 | 0.876 | 35.1 | 2.3 | 52.0 | 197.3 | 249.4 | -66.3 | OK |
| 0.1 | 3.5 | #5 | 6 | 1.5 | 0.465 | 0.156 | 0.039 | 0.465 | 31.4 | 2.53 | 0.533 | 30.9 | 2.56 | 57.9 | 231.7 | 289.6 | -55.9 | OK |
| 0.2 | 7.0 | #5 | 12 | 1.5 | 0.465 | 0.127 | 0.032 | 0.335 | 27.9 | 2.82 | 0.491 | 30.4 | 2.6 | 58.8 | 118.2 | 177.0 | -45.6 | OK |
| 0.3 | 10.5 | #5 | 12 | 1.5 | 0.465 | 0.098 | 0.024 | 0.051 | 22.8 | 3.54 | 0.236 | 26.3 | 2.97 | 67.2 | 140.3 | 207.5 | -35 | OK |
| 0.4 | 14.0 | #5 | 12 | 1.5 | 0.465 | 0.068 | 0.017 | 0.335 | 27.9 | 2.82 | 0.418 | 29.2 | 2.7 | 61.1 | 124.1 | 185.1 | -24.2 | OK |
| 0.5 | 17.5 | #5 | 12 | 1.5 | 0.465 | 0.037 | 0.009 | 0.505 | 30.6 | 2.59 | 0.540 | 31 | 2.56 | 57.9 | 115.4 | 173.3 | -13.4 | OK |
| 0.6 | 21.0 | #5 | 12 | 1.5 | 0.465 | 0.009 | 0.002 | 0.554 | 31.2 | 2.54 | 0.562 | 31.3 | 2.54 | 57.5 | 114.0 | 171.5 | -3.1 | OK |
| 0.7 | 24.5 | #5 | 12 | 1.5 | 0.465 | 0.022 | 0.005 | 0.546 | 31.1 | 2.55 | 0.565 | 31.3 | 2.54 | 57.5 | 114.0 | 171.5 | 7.7 | OK |
| 0.8 | 28.0 | #5 | 12 | 1.5 | 0.465 | 0.051 | 0.013 | 0.444 | 29.6 | 2.67 | 0.498 | 30.5 | 2.59 | 58.6 | 117.7 | 176.3 | 18.3 | OK |
| 0.9 | 31.5 | #5 | 6 | 1.5 | 0.465 | 0.080 | 0.020 | 0.113 | 24.1 | 3.29 | 0.181 | 25.4 | 3.01 | 68.1 | 292.1 | 360.2 | 28.5 | OK |
| 1 | 35.0 | #5 | 6 | 1.5 | 0.465 | 0.109 | 0.027 | 0.018 | 22.2 | 3.68 | 0.127 | 24.3 | 3.23 | 73.1 | 307.2 | 380.2 | 39 | OK |

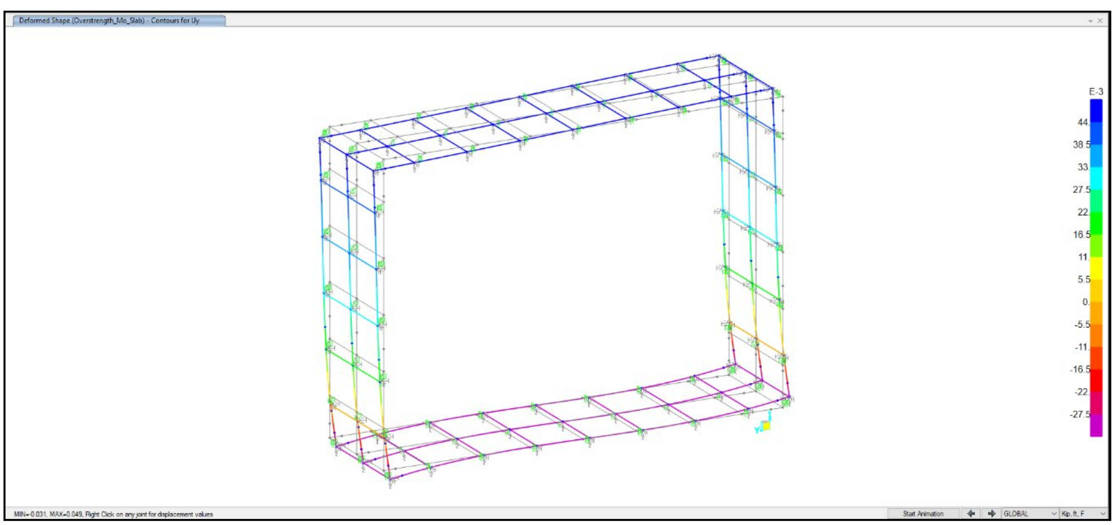
### 20.3.3.13 STEP 12 – CHECK BASE SLAB FLEXURAL AND SHEAR CAPACITY

Similar to STEP 11, the base slab shall be designed to resist the overstrength demands resulting from the side walls. The overstrength moment of 309.6 kip-ft/ft is applied to the base slab, as illustrated in Figure 17.

The resulting shear and bending moment demands per unit length are shown in Figure 18, and the values per unit length are summarized in Table 10. Figure 18b illustrates that the factored shear diagram exhibits a saw tooth pattern, which is a result of the lumped soil springs assigned along the bottom slab.

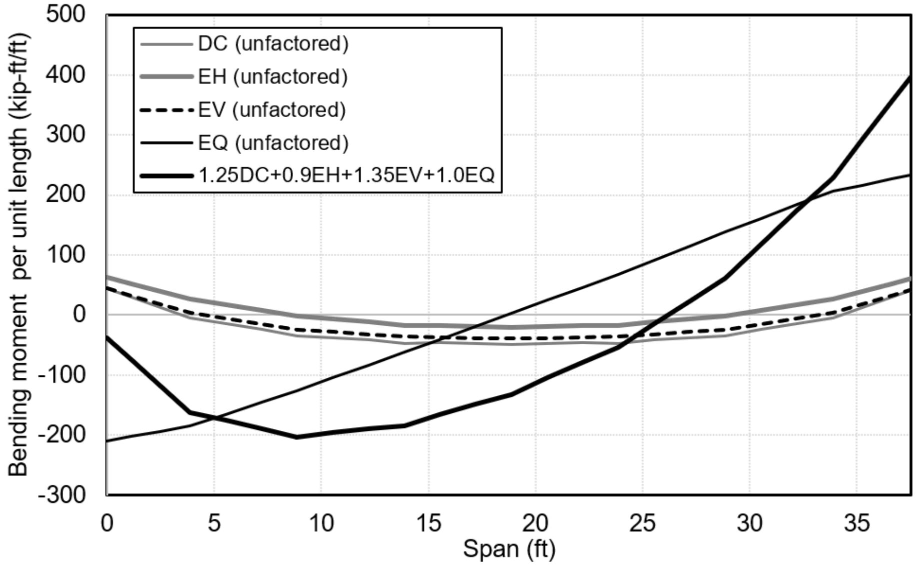



(a) Schematic view of applying the overstrength moment

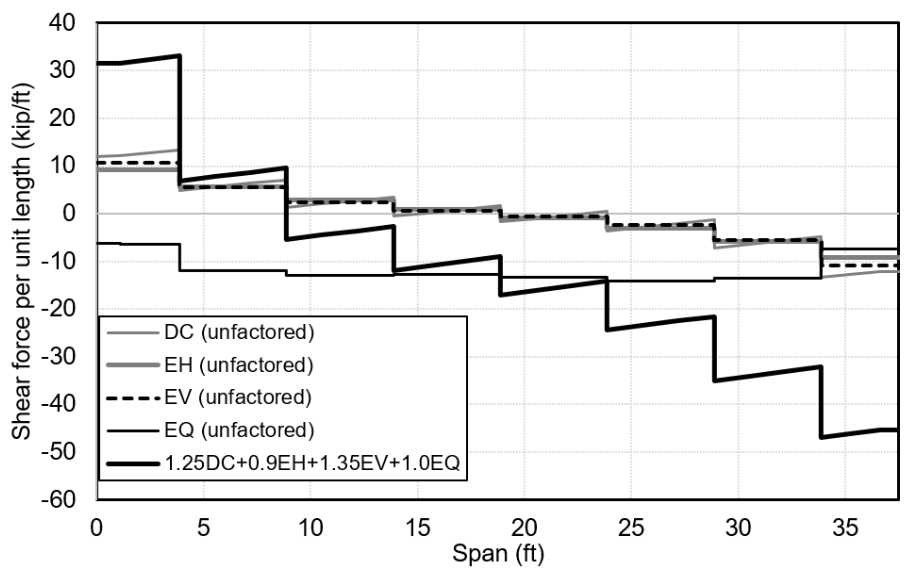



(b) Deformed configuration (color contour indicates the transverse deformation)

Figure 17. CSiBridge model applying the overstrength moment of the side walls to the base slab




(a)




(b)

Figure 18. Shear force (a) and bending moment (b) along the base slab, applying the overstrength moment from the side walls

Table 10. Forces along base slab resulting from CSiBridge

| Location | Span Length (ft) | Load Combination: 1.25DC + 1.35 (EH + EV) + 1.0EQ | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Axial Force (kip/ft) [1] | | | | | Bending Moment (kip-ft/ft) | | | | | Shear Force (kip/ft) | | | | |
| | | DC | EH | EV | EQ | Combined | DC | EH | EV | EQ | Combined | DC | EH | EV | EQ | Combined |
| 0 | 0 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | 30.1 | 52.0 | 32.9 | -202.8 | -50.6 | 12.1 | 9.1 | 10.8 | -6.3 | 35.7 |
| 0.1 | 3.5 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | -9.1 | 22.3 | -1.1 | -176.0 | -158.8 | 5.2 | 5.7 | 5.5 | -11.8 | 9.9 |
| 0.2 | 7.0 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | -30.4 | 1.9 | -20.7 | -134.0 | -197.4 | 6.8 | 5.7 | 5.5 | -11.8 | 11.9 |
| 0.3 | 10.5 | -4.1 | -10.1 | -6.1 | -7.5 | -34.6 | -40.6 | -10.5 | -31.1 | -88.9 | -195.8 | 2.6 | 3.0 | 2.3 | -12.9 | -2.4 |
| 0.4 | 14.0 | -4.1 | -10.1 | -6.1 | -7.5 | -34.5 | -46.7 | -18.1 | -36.9 | -43.2 | -175.8 | 0.1 | 0.9 | 0.6 | -12.8 | -10.6 |
| 0.5 | 17.5 | -4.1 | -10.1 | -6.1 | -7.5 | -34.5 | -49.8 | -21.3 | -39.1 | 2.2 | -141.5 | -1.7 | -0.9 | -0.6 | -13.2 | -17.4 |
| 0.6 | 21.0 | -4.1 | -10.1 | -6.1 | -7.5 | -34.5 | -46.7 | -18.1 | -36.9 | 49.2 | -83.3 | -0.1 | -0.9 | -0.6 | -13.2 | -15.4 |
| 0.7 | 24.5 | -4.1 | -10.1 | -6.1 | -7.5 | -34.6 | -40.6 | -10.5 | -31.1 | 98.0 | -8.9 | -2.6 | -3.0 | -2.3 | -14.1 | -24.5 |
| 0.8 | 28.0 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | -30.4 | 1.9 | -20.7 | 147.6 | 84.2 | -6.8 | -5.7 | -5.5 | -13.5 | -37.2 |
| 0.9 | 31.5 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | -9.1 | 22.3 | -1.1 | 195.5 | 212.8 | -5.2 | -5.7 | -5.5 | -13.5 | -35.2 |
| 1 | 35.0 | -4.0 | -10.1 | -6.1 | -7.6 | -34.6 | 30.1 | 52.0 | 32.9 | 226.7 | 378.9 | -12.1 | -9.1 | -10.8 | -7.4 | -49.4 |

1) -: Compression, +: Tension

### 20.3.3.13.1 Check at Wall Face (Right-Hand Side Wall)

Check Base Slab Flexural Capacity

Determine the factored ultimate moment at the Extreme Limit State, $M_u$ (see Table 10).

$$M_u = 1.25M_{DC} + 1.35M_{EH} + 1.35M_{EV} + 1.0M_{EQ}$$

$$= 1.25(30.1) + 1.35(52 + 32.9) + 1.0(226.7) = 378.9 \text{ kip-ft /ft}$$

Compute the nominal moment resistance of the section:

For the transverse reinforcement (#9 @ 8, bundled) along the unit length of the tunnel,

$$A_s = (1.0)(12/8) \times 2 = 3.0 \text{ in}^2$$

Assuming no compression steel (i.e., $A'_s$ is zero).

$$B = \text{unit length of wall cross-section} = 12 \text{ in}$$

$$f'_c = 4 \text{ ksi}$$

Per AASHTO 5.6.2.2,

$$\alpha_1 = 0.85$$

$$\beta_1 = 0.85$$

$$c = \frac{A_s f_s}{\alpha_1 f'_c \beta_1 b} = \frac{(3.0)(60)}{(0.85)(4)(0.85)(12)} = 5.19 \text{ in}$$

(AASHTO 5.6.3.1.1-4)

Using Equation AASHTO 5.6.3.2.2-1,

$$a = \beta_1 c = (0.85)(5.19) = 4.41 \text{ in}$$

$$M_n = A_s f_s \left(d_p - \frac{a}{2}\right) = (3.0)(60)\left(32.625 - \frac{4.41}{2}\right) = 5{,}475 \text{ kip-in/ft} = 456 \text{ kip-ft/ft}$$

Per SDC 3.6, the resistance factor for flexure, $\phi = 1.0$.

$$\phi M_n = (1.0)(456) = 456 \text{ kip-ft/ft} > M_u = 378.9 \text{ kip-ft/ft} \quad \text{OK}$$

Check Base Slab Shear Capacity

For cross ties (#4 @ 12 in transverse and @ 16 in longitudinal direction),

$A_v$ = (0.20)(12/16) = 0.15 $in^2$/ft

$d_v$ = effective shear depth = 32.625 – 4.41/2= 30.42 in

The shear stress on the concrete is calculated in accordance with AASHTO 5.7.2.8-1:

$$v_u = \frac{|V_u|}{\phi b d_v} = \frac{|49.4|}{(1.0)(12)(30.42)} = 0.135 \text{ ksi}$$

$$\frac{v_u}{f'_c} = \frac{0.135}{4} = 0.034$$

Determine the factored axial force:

$$N_u = 1.25P_{DC} + 1.35P_{EH} + 1.35P_{EV} + 1.0P_{EQ}$$

$$= (1.25)(4.0) + (1.35)(-10.1) + (1.35)(-6.1) + (1.0)(-7.6) = -34.6 \text{ kip/ft}$$

Using AASHTO B5.2-3 with the factored axial force, begin with θ = 45° (i.e., cot θ =1), calculate $\varepsilon_x$:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u|\cot\theta\right)}{2E_sA_s} = \frac{\left(\frac{378.9\times 12}{30.42} + (0.5)(-34.6) + (0.5)(49.4)(1)\right)}{(2)(29000)(3)} = 0.902\times 10^{-3}$$

From AASHTO Table B5.2-1 using the calculated values of $v_u/f'_c$ = 0.034 ksi and $\varepsilon_x$ = 0.902x $10^{-3}$,

θ = 35.3° and β = 2.29

The nominal shear resistance provided by the concrete is calculated as:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}b_vd_v = (0.0316)(2.23)(1)\left(\sqrt{4}\right)(12)(30.42) = 52.8 \text{ kips}$$

Using α = 90°, the nominal shear resistance provided by cross ties is calculated as:

$$V_s = \frac{A_vf_yd_v\cot\theta}{s} = \frac{(0.15)(60)(30.42)(\cot 35.3)}{16} = 24.2 \text{ kips}$$

$$V_n = V_c + V_s = 52.8 + 24.2 = 77.0 \text{ kips}$$

$$V_n < 0.25f'_cb_vd_v = (0.25)(4)(12)(30.42) = 365 \text{ kips}$$

The factored nominal shear capacity is:

$\phi V_n$ = (1.0)(77.0) = 77.0 kips > $V_u$ = 49.4 kips OK

The flexural and shear capacity calculations at other locations along the base slab are summarized in Table 11 and Table 12, respectively.

Table 11. Flexural check along the base-slab subjected to overstrength moment

| Location | Span | Factored Forces | | | Flexural Capacity (AASHTO LRFD BDS 5.6.3.2) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Axial | Bending Moment | Shear Force | Rebar | Spacing[1)] | As | c | a | $\phi M_n$ | Check | $\varepsilon_t$ | Check |
| | ft | kip/ft | kip-ft/ft | kip/ft | | in | in$^2$ | in | in | kip-ft/ft | | | |
| 0 | 0 | -34.6 | -50.6 | 35.7 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |
| 0.1 | 3.5 | -34.6 | -158.8 | 9.9 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |
| 0.2 | 7.0 | -34.6 | -197.4 | 11.9 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |
| 0.3 | 10.5 | -34.6 | -195.8 | -2.4 | #9 | 8 (T & B) | 1.5 | 2.6 | 2.2 | 236.4 | OK | 0.0383 | OK |
| 0.4 | 14.0 | -34.5 | -175.8 | -10.6 | #9 | 8 (T) | 1.5 | 2.6 | 2.2 | 236.4 | OK | 0.0383 | OK |
| 0.5 | 17.5 | -34.5 | -141.5 | -17.4 | #9 | 8 (T) | 1.5 | 2.6 | 2.2 | 236.4 | OK | 0.0383 | OK |
| 0.6 | 21.0 | -34.5 | -83.3 | -15.4 | #9 | 8 (T) | 1.5 | 2.6 | 2.2 | 236.4 | OK | 0.0383 | OK |
| 0.7 | 24.5 | -34.6 | -8.9 | -24.5 | #9 | 8 (T & B) | 1.5 | 2.6 | 2.2 | 236.4 | OK | 0.0383 | OK |
| 0.8 | 28.0 | -34.6 | 84.2 | -37.2 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |
| 0.9 | 31.5 | -34.6 | 212.8 | -35.2 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |
| 1 | 35.0 | -34.6 | 378.9 | -49.4 | #9 (bundle) | 8 (T & B) | 3 | 5.2 | 4.4 | 456.3 | OK | 0.0179 | OK |

[1)] T: Top Mat, B: Bottom Mat

Table 12. Shear check along the base-slab subjected to overstrength moment

| Location | Span | Shear Resistance (AASHTO LRFD BDS 5.7.3.3) | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cross tie | Spacing | No. bar per ft | $A_v$ | $v_u$ | $v_u/f'_c$ | $\varepsilon_x$ (x1000) | θ | b | $\varepsilon_x$ (x1000) | θ | b | $V_c$ | $V_s$ | $\phi V_n$ | Vu | Check |
| | ft | | in | | in$^2$ | ksi | | (1st trial) | deg | | (2nd trial) | deg | | kip | kip | kip | kip | |
| 0 | 0 | #4 | 16 | 0.75 | 0.15 | 0.098 | 0.024 | 0.118 | 24.2 | 3.27 | 0.244 | 26.5 | 2.95 | 68.1 | 34.3 | 102.4 | 35.7 | OK |
| 0.1 | 3.5 | #4 | 16 | 0.75 | 0.15 | 0.027 | 0.007 | 0.289 | 27.2 | 2.89 | 0.316 | 27.6 | 2.85 | 65.8 | 32.7 | 98.5 | 9.9 | OK |
| 0.2 | 7.0 | #4 | 16 | 0.75 | 0.15 | 0.033 | 0.008 | 0.382 | 28.7 | 2.76 | 0.411 | 29.1 | 2.71 | 62.5 | 30.7 | 93.3 | 11.9 | OK |
| 0.3 | 10.5 | #4 | 16 | 0.75 | 0.15 | 0.007 | 0.002 | 0.703 | 33.1 | 2.42 | 0.710 | 33.2 | 2.41 | 55.6 | 26.1 | 81.7 | -2.4 | OK |
| 0.4 | 14.0 | #4 | 16 | 0.75 | 0.15 | 0.029 | 0.007 | 0.660 | 32.5 | 2.46 | 0.694 | 33 | 2.43 | 56.1 | 26.3 | 82.4 | -10.6 | OK |
| 0.5 | 17.5 | #4 | 16 | 0.75 | 0.15 | 0.048 | 0.012 | 0.543 | 31.1 | 2.55 | 0.609 | 31.9 | 2.5 | 57.7 | 27.5 | 85.2 | -17.4 | OK |
| 0.6 | 21.0 | #4 | 16 | 0.75 | 0.15 | 0.042 | 0.011 | 0.268 | 26.9 | 2.9 | 0.354 | 28.2 | 2.79 | 64.4 | 31.9 | 96.3 | -15.4 | OK |
| 0.7 | 24.5 | #4 | 16 | 0.75 | 0.15 | 0.067 | 0.017 | -0.018 | 21.3 | 3.88 | 0.203 | 25.7 | 3.05 | 70.4 | 35.6 | 105.9 | -24.5 | OK |
| 0.8 | 28.0 | #4 | 16 | 0.75 | 0.15 | 0.102 | 0.025 | 0.198 | 25.6 | 3.06 | 0.315 | 27.6 | 2.85 | 65.8 | 32.7 | 98.5 | -37.2 | OK |
| 0.9 | 31.5 | #4 | 16 | 0.75 | 0.15 | 0.096 | 0.024 | 0.484 | 30.3 | 2.61 | 0.556 | 31.2 | 2.54 | 58.6 | 28.3 | 86.9 | -35.2 | OK |
| 1 | 35.0 | #4 | 16 | 0.75 | 0.15 | 0.135 | 0.034 | 0.902 | 35.3 | 2.29 | 0.960 | 35.3 | 2.29 | 52.8 | 24.2 | 77.0 | -49.4 | OK |

### 20.3.3.14 Independent Check

BDM 20.32 allows designers to use the simplified methods (closed-form solutions) in accordance with Section 13.5.1.3 of FHWA (2009) if the design meets the conditions specified in Section 20.32.4.2.2.1 of BDM 20.32. Since this design example fulfills the requirements, the closed-form solution is performed as an analysis method for the independent and preliminary check.

Section 13.5.1.3 of FHWA (2009) introduces two methods. The difference between the methods is whether the lateral stiffness of the structure is considered or not:

1) Free-Field Racking Deformation Method

The free-filed racking deformation method is performed assuming that the structure's response $\Delta_s$ under the seismic event is equal to the soil deformation ($\Delta_{free\text{-}field} = \Delta_s$). Thus, the structure's stiffness is ignored. Its application is limited to design cases where seismic intensity is low or the ground is very stiff, which is different from the conditions of this design example

2) Tunnel-Ground Interaction Analysis.

The tunnel-ground interaction analysis includes the effect of structure stiffness into the analysis by using "flexibility ratio" that is a ratio of structure to soil stiffness. The detailed calculation following Tunnel-Ground Interaction Analysis is presented later in this section.

In this example, the free-field racking deformation is shown for illustrative purpose even though it might be directly relevant to this particular design example. Table 13 summarizes the geotechnical properties used in the dependent check.

Table 13. Soil Properties in Estimating Free-Field Deformation ($\Delta_{free\text{-}field}$)

| Parameter | Value |
|---|---|
| Magnitude[1] | 7.41 |
| Peak Ground Acceleration[1] (PGA) | 1.07 g |
| Average Shear Wave Velocity[2] ($V_{s30}$) | 210 m/sec (689 ft/sec) |
| Soil Type | Medium Dense Sand |
| Total Unit Weight, $\gamma_{soi}$ | 120 pcf |
| Corrected Standard Penetration Test (SPT) Value, $N_{60}$ | 20 |
| Age Scaling Factor[3] (ASF) | 1 |

Notes:
1. 1500-year return period
2. Vs30 is estimated using Wair et al. (2012)
3. Factor is for Quaternary

#### 20.3.3.14.1 Free-Field Racking Deformation Method

The soil profile is subdivided into layers of uniform thickness. The calculation steps for each discretized soil layer are as follows.

*Step 1: Calculate average shear wave velocity ($V_s$)*

In accordance with the Caltrans Geotechnical Manual, Design Acceleration Response Spectrum Module, the shear wave velocity is estimated using the following empirical correlation with Standard Penetration Test (SPT) blow count corrected for hammer efficiency ($N_{60}$) and effective overburden stress ($\sigma'_{vo}$) at the discretized soil layer.

This example considers a soil layer between a depth (D) of 10 ft and 15 ft.

$$\sigma'_{vo} = \gamma_{soil}D = (120\text{ pcf})(12.5\text{ ft}) = 1500\text{ psf}$$

Note that the overburden stress is taken at midpoint of the layer (12.5 ft).

$$V_s = 30(\text{ASF})(N_{60})^{0.23}(\sigma'_{vo})^{0.23} = 30(1)(20)^{0.23}\left(1500\text{ psf}\times 0.048\frac{\text{kPa}}{\text{psf}}\right)^{0.23} = 159.8\text{ m/sec}$$

*Step 2: Calculate small-strain shear modulus ($G_{max}$)*

$$G_{max} = (V_s)^2\rho_{soil} = (159.8\text{ m/sec}\times 3.28\text{ ft/m})^2\times\frac{120\text{ pcf}}{32.2\,ft/sec^2}\times 0.001\frac{\text{ksf}}{\text{psf}} = 1{,}023\text{ ksf}$$

where, $\rho_{soil}$ is mass density of soil.

*Step 3: Calculate the maximum earthquake-induced shear stress ($\tau_{max}$)*

$$\tau_{max} = \left(\frac{\text{PGA}}{g}\right)(\gamma_{soil}D)(R_d) = (1.07)(120\text{ pcf})(12.5\text{ ft})(0.971) = 1{,}558\text{ psf}$$

where $R_d$: depth-dependent stress reduction factor.

$R_d$ = 1.0 – 0.00233z for z < 30 ft for z < 30 ft (Section 13.5.1.1, FHWA 2009)

= 1.0 – (0.00233)(12.5) = 0.971

*Step 4: Calculate free-field shear deformation ($\Delta_{free\text{-}field}$) over tunnel height*

Through trial/error iteration, find the $G/G_{max}$ with the calculated effective shear strain match the value shown in the selected shear modulus reduction curve (EPRI 1993). The procedure of the iteration is as follows:

1) Select a $G/G_{max}$ value to start the iteration. In this example, the $G/G_{max}$ = 0.75 is selected.

2) Calculate strain-compatible shear modulus ($G_m$)

$$G_m=(G/G_{max})G_{max}=(0.75)(1{,}023\ \text{ksf})=768\ \text{ksf}$$

3) Calculate maximum shear strain ($\gamma_{max}$)

$$\gamma_{max}=\frac{\tau_{max}}{G_m}=\frac{1{,}558\ \text{psf}}{768\ \text{ksf}}\times 0.001\frac{\text{ksf}}{\text{psf}}=0.20\%$$

Calculate effective shear strain ($\gamma_{eff}$). The effective shear strain is determined as follows:

$$\gamma_{eff} = R\gamma_{max}$$

Where R is a ratio of equivalent uniform strain divided by maximum strain. This value typically varies from 0.4 to 0.75, depending on the earthquake magnitude, *M* (Idriss and Sun, 1992). In this example, *R* is expressed as:

$$R = \frac{M - 1}{10} = \frac{7.41\text{-}1}{10} = 0.641$$

$$\gamma_{eff} = R(\gamma_{max}) = (0.641)(0.20\%) = 0.13\%$$

4) Use the calculated effective shear strain, $\gamma_{eff}$ = 0.13%, to find the $G/G_{max}$ from the selected shear modulus reduction curve (EPRI 1993). In this example, matching $G/G_{max}$ = 0.53 yields the difference in $G/G_{max}$ exceeding a tolerance of 0.01.

$$G/G_{max} - G/G_{max\ (\text{from curve})} = 0.75 – 0.53 = 0.22 > \text{tolerance; repeat}$$

5) Repeat steps 1) through 5) until the difference between the tried and the matching $G/G_{max}$ is within tolerance. In this example, $G/G_{max}$ = 0.38 with the corresponding $\gamma_{max}$ = 0.40% (Figure 19). The iteration process is illustrated in Figure 17.

6) Calculate free-field shear deformation ($\delta_{free\text{-}field}$)

$$\delta_{\text{free-field}}=(\gamma_{max})(\text{Thickness of Soil Layer})=(0.40\%)\left(5\ \text{ft}\times\frac{12\ \text{in}}{\text{ft}}\right)=0.24\ \text{in}$$

7) Calculate free-field shear deformation at other discretized soil layers. This calculation may be performed using a spreadsheet (Table 14). The cumulative free-field shear deformation profile within the tunnel height can then be developed by summing the calculated free-field shear deformations from the bottom of soil layer (40 ft) to the top of soil layer (10 ft) (Figure 20). The relative lateral soil deformation at the top of the tunnel to the bottom is estimated as follow.

$$\Delta_{free\text{-}field} = 15.6 - 11.6 = 4.0 \text{ in}$$

*Step 5: Calculate tunnel racking deformation ($\Delta_s$)*

Assume racking deformation ($\Delta_s$) is equal to the relative lateral soil deformation between at the depths of the top and bottom of the tunnel structure ($\Delta_{free\text{-}field}$).

$$\Delta_s = \Delta_{free\text{-}field} = 4.0 \text{ in}$$

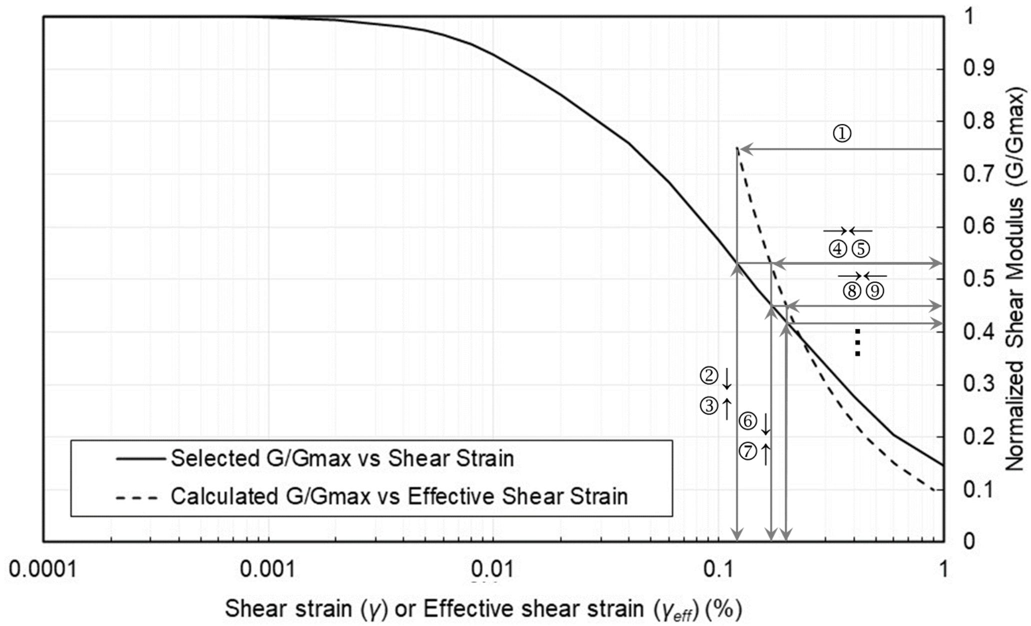



Figure 19. Selected $G/G_{max}$ vs Shear Strain Curve and Estimated $G/G_{max}$ vs Effective Shear Strain Curve to Determine $G/G_{max}$

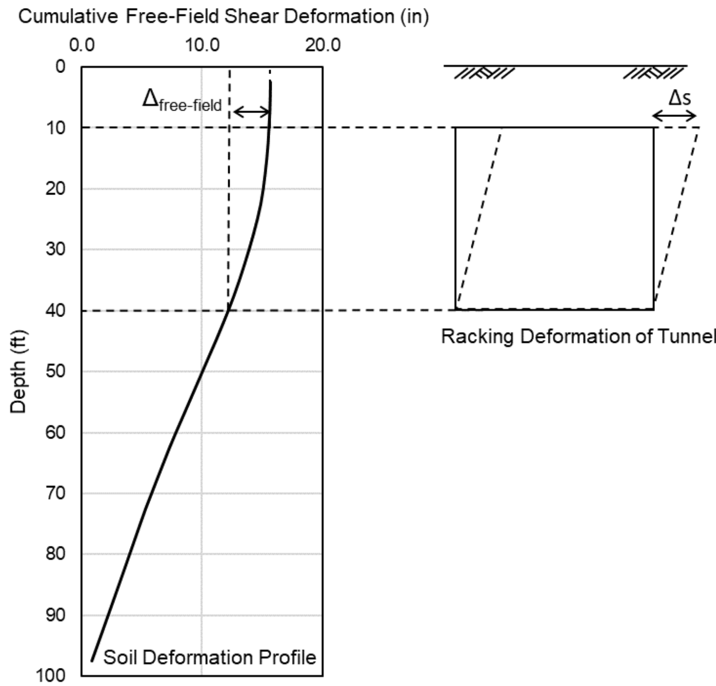



Figure 20. Cumulative Free-Field Shear Deformation Profile

Table 14. Calculation Spreadsheet for the Analytical Solution of Tunnel Racking Deformation

| Top Depth (ft) | Bottom Depth (ft) | Mid Depth (ft) | Thickness (ft) | $N_{60}$ | Total Unit Weight (pcf) | $\sigma'_{vo}$ (psf) | Vs (ft/s) | $G_{max}$ (ksf) | $G/G_{max}$ | Strain Comp. G (ksf) | Peak Accel. (g) | $R_d$ | $\tau_{max}$ (psf) | Shear Strain (%) | Effective Shear Strain (%) | Free-Field Shear Deform. (in) | Cumulative Free-Field Shear Deform. (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 5 | 2.5 | 5 | 20 | 120 | 300 | 362 | 488.2 | 0.69 | 336.9 | 1.07 | 0.994 | 319 | 0.095 | 0.061 | 0.06 | 15.6 |
| 5 | 10 | 7.5 | 5 | 20 | 120 | 900 | 466 | 809.3 | 0.49 | 396.6 | 1.07 | 0.983 | 946 | 0.239 | 0.153 | 0.14 | 15.6 |
| 10 | 15 | 12.5 | 5 | 20 | 120 | 1500 | 524 | 1023.7 | 0.38 | 389.0 | 1.07 | 0.971 | 1558 | 0.401 | 0.257 | 0.24 | 15.4 |
| 15 | 20 | 17.5 | 5 | 20 | 120 | 2100 | 566 | 1195.0 | 0.30 | 358.5 | 1.07 | 0.959 | 2155 | 0.601 | 0.385 | 0.36 | 15.2 |
| 20 | 25 | 22.5 | 5 | 20 | 120 | 2700 | 600 | 1341.5 | 0.20 | 268.3 | 1.07 | 0.948 | 2738 | 1.020 | 0.654 | 0.61 | 14.8 |
| 25 | 30 | 27.5 | 5 | 20 | 120 | 3300 | 628 | 1471.2 | 0.18 | 264.8 | 1.07 | 0.936 | 3305 | 1.248 | 0.800 | 0.75 | 14.2 |
| 30 | 35 | 32.5 | 5 | 20 | 120 | 3900 | 653 | 1588.7 | 0.17 | 270.1 | 1.07 | 0.909 | 3795 | 1.405 | 0.901 | 0.84 | 13.5 |
| 35 | 40 | 37.5 | 5 | 20 | 120 | 4500 | 675 | 1696.8 | 0.15 | 254.5 | 1.07 | 0.869 | 4183 | 1.643 | 1.053 | 0.99 | 12.6 |
| 40 | 45 | 42.5 | 5 | 20 | 120 | 5100 | 695 | 1797.4 | 0.14 | 251.6 | 1.07 | 0.828 | 4519 | 1.796 | 1.151 | 1.08 | 11.6 |
| 45 | 50 | 47.5 | 5 | 20 | 120 | 5700 | 713 | 1891.7 | 0.14 | 264.8 | 1.07 | 0.787 | 4802 | 1.813 | 1.162 | 1.09 | 10.6 |
| 50 | 55 | 52.5 | 5 | 20 | 120 | 6300 | 729 | 1980.9 | 0.14 | 277.3 | 1.07 | 0.747 | 5033 | 1.815 | 1.163 | 1.09 | 9.5 |
| 55 | 60 | 57.5 | 5 | 20 | 120 | 6900 | 745 | 2065.5 | 0.14 | 289.2 | 1.07 | 0.706 | 5212 | 1.802 | 1.155 | 1.08 | 8.4 |
| 60 | 65 | 62.5 | 5 | 20 | 120 | 7500 | 759 | 2146.3 | 0.15 | 321.9 | 1.07 | 0.665 | 5339 | 1.658 | 1.063 | 0.99 | 7.3 |
| 65 | 70 | 67.5 | 5 | 20 | 120 | 8100 | 773 | 2223.6 | 0.15 | 333.5 | 1.07 | 0.625 | 5413 | 1.623 | 1.040 | 0.97 | 6.3 |
| 70 | 75 | 72.5 | 5 | 20 | 120 | 8700 | 785 | 2297.9 | 0.16 | 367.7 | 1.07 | 0.584 | 5435 | 1.478 | 0.948 | 0.89 | 5.3 |
| 75 | 80 | 77.5 | 5 | 20 | 120 | 9300 | 798 | 2369.5 | 0.16 | 379.1 | 1.07 | 0.555 | 5522 | 1.456 | 0.934 | 0.87 | 4.5 |
| 80 | 85 | 82.5 | 5 | 20 | 120 | 9900 | 809 | 2438.7 | 0.16 | 390.2 | 1.07 | 0.543 | 5749 | 1.473 | 0.944 | 0.88 | 3.6 |
| 85 | 90 | 87.5 | 5 | 20 | 120 | 10500 | 820 | 2505.6 | 0.16 | 400.9 | 1.07 | 0.531 | 5960 | 1.487 | 0.953 | 0.89 | 2.7 |
| 90 | 95 | 92.5 | 5 | 20 | 120 | 11100 | 831 | 2570.4 | 0.16 | 411.3 | 1.07 | 0.518 | 6156 | 1.497 | 0.959 | 0.90 | 1.8 |
| 95 | 100 | 97.5 | 5 | 20 | 120 | 11700 | 841 | 2633.5 | 0.16 | 421.4 | 1.07 | 0.506 | 6336 | 1.504 | 0.964 | 0.90 | 0.9 |

### 20.3.3.14.2 Tunnel-Ground Interaction Analysis

Tunnel-Ground Interaction Analysis is a simplified procedure developed by Wang (1993) that incorporates the effects of the stiffness ratio between soil and structure.

1) Calculate maximum shear strain ($\gamma_{max}$) at tunnel location. In this example, average shear wave velocity ($V_s$) over the tunnel location is used to calculate small-strain shear modulus ($G_{max}$), $G/G_{max}$, and strain-compatible shear modulus ($G_m$). Overburden stress ($\gamma D$) and depth dependent stress reduction factor ($R_d$) at tunnel invert are conservatively assumed to calculate maximum shear strain ($\gamma_{max}$). The GD provides the calculated $G_m$ and $\gamma_{max}$:

   $G_m = 263$ ksf

   $\gamma_{max} = 1.69\%$

2) Calculate free-field shear distortion ($\Delta_{free\text{-}field}$)

   $$\Delta_{\text{free-field}} = (H)(\gamma_{max}) = (30\text{ ft})(1.69\%)\left(\frac{12\text{ in}}{\text{ft}}\right) = 6.1\text{ in}$$

   Where, $H$: height of tunnel structure

3) Calculate racking stiffness ($K_s$) using the yield displacement ($\Delta_Y$) and the associated shear ($V_P$) from the pushover analysis discussed in Section 20.3.5, which are 3 in (0.25 ft) and 45.6kips/ft, respectively. The racking stiffness is as follows:

   Ks = 45.6 kips/ft / 0.25 ft = 182.4 kips/ft/ft

4) Calculate flexibility ratio ($F_r$)

   $$F_r = \left(\frac{G_m}{K_s}\right)\left(\frac{W}{H}\right) = \left(\frac{263\text{ ksf}}{182.4\text{ ksf}}\right)\left(\frac{40\text{ ft}}{30\text{ ft}}\right) = 1.92$$

   Where, $W$: width of tunnel structure

5) Calculate racking coefficient ($R_r$)

   $$R_r = \left(\frac{\Delta_s}{\Delta_{\text{free-field}}}\right)$$

   For no-slip interface condition,

$$R_r = \frac{4(1 - v_m)F_r}{3 - 4v_m + F_r} = \frac{4(1 - 0.3)(1.92)}{3 - 4(0.3) + 1.92} = 1.45$$

For full-slip interaction condition

$$R_r = \frac{4(1 - v_m)F_r}{2.5 - 3v_m + F_r} = \frac{4(1 - 0.3)(1.92)}{2.5 - 3(0.3) + 1.92} = 1.53$$

Where, $v_m$: Poisson's ratio of soil. In this example, $v_m$ = 0.3 is selected from typical ranges of drained Poisson's ratio (EPRI 1990).

6) Calculate the racking deformation of the tunnel structure ($\Delta_s$). In this example, full-slip interface condition is assumed.

$$\Delta_s = (R_r)(\Delta_{free\text{-}field}) = (1.53)(6.1) = 9.3 \text{ in}$$

Table 15 summarizes the racking deformation of the tunnel structure ($\Delta_s$) resulting from the NTHA and two simplified solutions. Compared to the free-field racking deformation method ($\Delta_s = \Delta_{free\text{-}field}$), the tunnel-ground interaction analysis approach results in a more reasonable estimate as a preliminary check in this example.

Table 15. Racking Deformation Comparison between Independent Check Analysis and NTHA Analysis

| Parameter | Independent Check Analysis | | NTHA from Section 20.3.4 |
|---|---|---|---|
| | Free-Field Racking Deformation Method | Tunnel-Ground Interaction Analysis | |
| Racking deformation of the tunnel structure $\Delta_s$ | 4.0 in | 9.3 in | 7.9 in |

# APPENDICES

# APPENDIX 20.3.1. Parametric Study for the Required Stiffness in the Soil Spring Development

As discussed in Section 20.3.3.3, a parametric study was performed to determine the stiffness of tunnel lining elements required to minimize the effects of deflection of the lining (Figure A20.3.1-1). The study demonstrates that a stiffness over ten times higher than the normal stiffness of tunnel elements (both bending and axial stiffness) is required to induce unit displacement, indicating the rigid behavior of the lining.

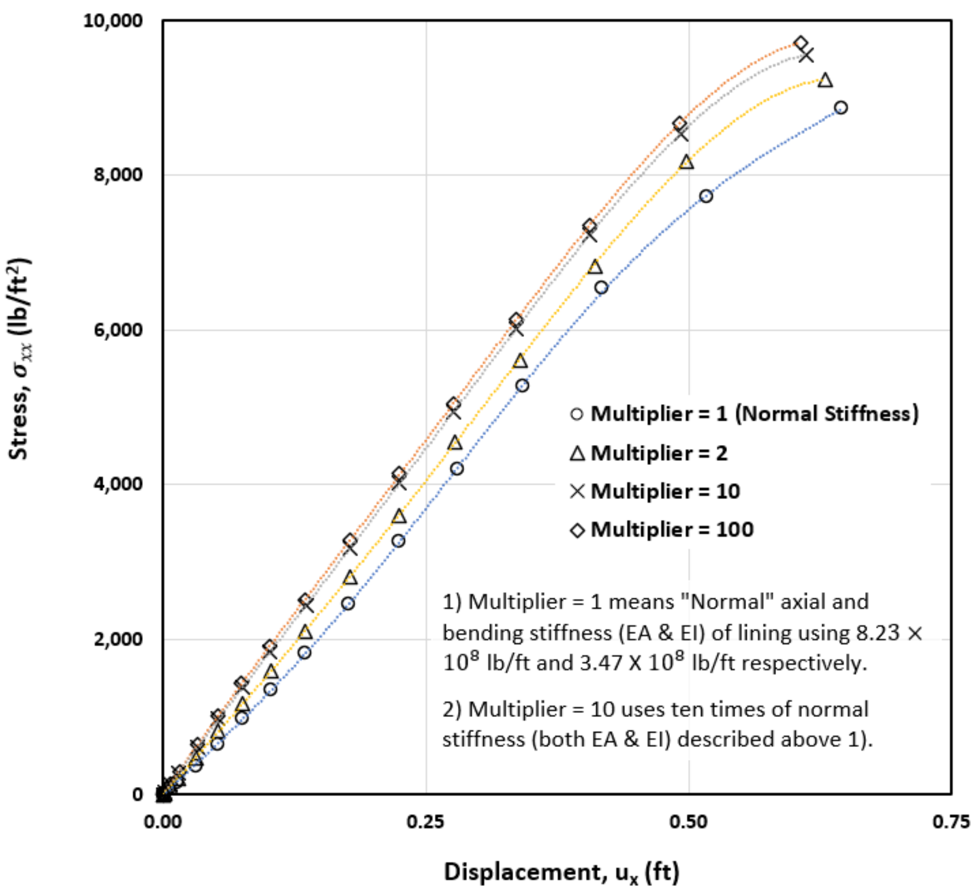


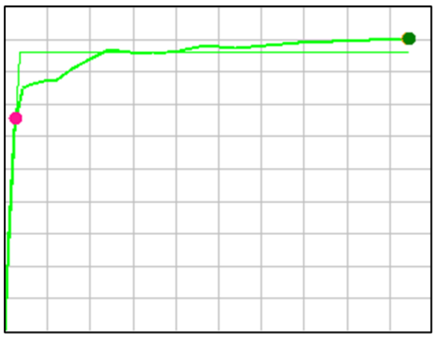

Figure A20.3.1-1. Parametric study results shown the effect of stiffness of increasing tunnel stiffness on normal stress to wall and lateral displacement

# APPENDIX 20.3.2. CSiBridge Section-Designer Output

## Right Wall

**MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F**

*Axial Load* = -206
*Moment Angle* = 0



### Results For Exact-Integration

$\phi_{y(Initial)}$ = 1.620E-03
$M_y$ = 984.482
$\phi_{y(Idealized)}$ = 2.126E-03
$M_p$ = 1292.3965
$I_{crack}$ = 1.171
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
$\phi_{steel}$ = 0.0566
$M_{steel}$ = 1351.0468

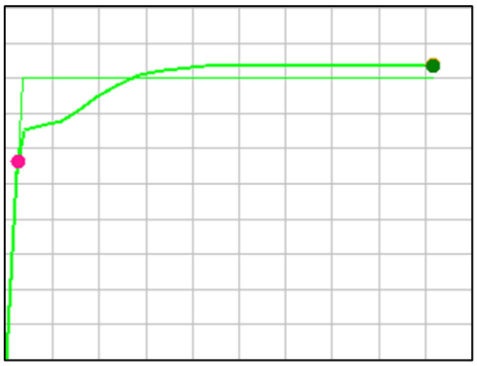
| Conc. Strain | Neutral Axis | Steel Strain | Conc. Comp. | Steel Comp. | Steel Ten. | Prestress | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|---|
| -3.25E-05 | 0 | -3.25E-05 | -191.684 | -14.3764 | 0 | 0 | -206.06 | 0 | 3.89E-16 |
| -1.97E-04 | 0.469 | 7.05E-04 | -318.531 | -43.5993 | 155.8738 | 0 | -206.256 | 5.42E-04 | 457.5121 |
| -3.48E-04 | 0.5765 | 1.91E-03 | -551.574 | -76.8461 | 421.8368 | 0 | -206.583 | 1.35E-03 | 937.9159 |
| -3.75E-04 | 0.6797 | 3.69E-03 | -641.471 | -82.7453 | 518.16 | 0 | -206.056 | 2.44E-03 | 1127.93 |
| -2.89E-04 | 0.7572 | 6.03E-03 | -660.537 | -63.755 | 518.16 | 0 | -206.132 | 3.79E-03 | 1145.399 |
| -1.68E-04 | 0.8023 | 8.86E-03 | -687.097 | -37.0924 | 518.16 | 0 | -206.029 | 5.42E-03 | 1157.295 |
| -3.50E-05 | 0.8286 | 0.0122 | -716.706 | -7.6614 | 518.16 | 0 | -206.207 | 7.31E-03 | 1165.561 |
| 3.77E-05 | 0.8373 | 0.0158 | -757.589 | 0 | 551.5289 | 0 | -206.06 | 9.48E-03 | 1211.943 |
| 3.25E-05 | 0.8361 | 0.0199 | -786.877 | 0 | 580.7292 | 0 | -206.148 | 0.0119 | 1260.731 |
| -1.11E-04 | 0.8258 | 0.0243 | -788.013 | -24.2523 | 606.083 | 0 | -206.183 | 0.0146 | 1302.693 |
| -8.40E-04 | 0.7856 | 0.0285 | -656.381 | -185.388 | 635.6752 | 0 | -206.094 | 0.0176 | 1293.079 |
| -1.47E-03 | 0.7629 | 0.0333 | -533.449 | -324.23 | 651.5839 | 0 | -206.095 | 0.0209 | 1284.62 |
| -1.95E-03 | 0.7533 | 0.0387 | -444.756 | -430.659 | 669.4796 | 0 | -205.935 | 0.0244 | 1297.538 |
| -2.33E-03 | 0.7508 | 0.0446 | -379.408 | -513.776 | 687.2941 | 0 | -205.889 | 0.0282 | 1319.042 |
| -4.07E-03 | 0.707 | 0.0496 | -383.826 | -518.16 | 695.2789 | 0 | -206.707 | 0.0322 | 1311.381 |
| -4.85E-03 | 0.7006 | 0.0561 | -393.589 | -518.16 | 705.5146 | 0 | -206.234 | 0.0366 | 1323.814 |
| -5.43E-03 | 0.7015 | 0.0632 | -400.944 | -518.16 | 713.0832 | 0 | -206.021 | 0.0412 | 1335.481 |
| -6.04E-03 | 0.7023 | 0.0707 | -406.194 | -518.16 | 718.3547 | 0 | -206 | 0.046 | 1343.651 |
| -6.68E-03 | 0.7029 | 0.0786 | -409.631 | -518.16 | 721.789 | 0 | -206.002 | 0.0512 | 1348.865 |
| -7.35E-03 | 0.7035 | 0.087 | -411.173 | -518.16 | 723.3397 | 0 | -205.993 | 0.0566 | 1351.047 |

Note: units on the table – kip, ft

## Left Wall

**MOMENT CURVATURE (*M-ϕ*) GRAPH - Kip, ft, F**

*Axial Load* = -42
*Moment Angle* = 0



### Results For Exact-Integration

$\phi_{y(Initial)}$ = 1.556E-03
$M_y$ = 847.2604
$\phi_{y(Idealized)}$ = 2.201E-03
$M_p$ = 1198.2959
$I_{crack}$ = 1.0488
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
$\phi_{steel}$ = 0.055
$M_{steel}$ = 1251.1063

| Conc. Strain | Neutral Axis | Steel Strain | Conc. Comp. | Steel Comp. | Steel Ten. | Prestress | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|---|
| -6.76E-06 | 0 | 0.0000 | -38.860 | -2.986 | 0.000 | 0 | -41.845 | 0.0000 | 0.000 |
| -1.13E-04 | 0.6182 | 0.0008 | -186.566 | -25.014 | 168.820 | 0 | -42.760 | 0.0005 | 341.206 |
| -2.49E-04 | 0.6441 | 0.0019 | -416.528 | -55.011 | 429.575 | 0 | -41.963 | 0.0013 | 807.800 |
| -2.48E-04 | 0.7285 | 0.0037 | -505.816 | -54.831 | 518.160 | 0 | -42.487 | 0.0024 | 980.570 |
| -1.35E-04 | 0.7966 | 0.0060 | -530.400 | -29.860 | 518.160 | 0 | -42.100 | 0.0037 | 994.315 |
| 1.33E-05 | 0.8359 | 0.0088 | -563.783 | 0.000 | 521.150 | 0 | -42.633 | 0.0053 | 1006.261 |
| 1.80E-04 | 0.8587 | 0.0120 | -599.915 | 0.000 | 557.974 | 0 | -41.941 | 0.0071 | 1015.570 |
| 3.06E-04 | 0.8666 | 0.0157 | -651.136 | 0.000 | 609.574 | 0 | -41.562 | 0.0092 | 1063.493 |
| 3.76E-04 | 0.8659 | 0.0197 | -696.415 | 0.000 | 655.090 | 0 | -41.325 | 0.0116 | 1117.400 |
| 3.57E-04 | 0.8585 | 0.0240 | -724.954 | 0.000 | 683.467 | 0 | -41.487 | 0.0142 | 1168.902 |
| 2.44E-04 | 0.8476 | 0.0288 | -732.029 | 0.000 | 690.585 | 0 | -41.444 | 0.0171 | 1213.137 |
| 1.00E-04 | 0.8383 | 0.0339 | -717.969 | 0.000 | 675.929 | 0 | -42.040 | 0.0203 | 1230.216 |
| -1.28E-04 | 0.8279 | 0.0393 | -685.465 | -28.017 | 671.719 | 0 | -41.764 | 0.0237 | 1245.704 |
| -4.56E-04 | 0.8167 | 0.0452 | -629.522 | -100.540 | 688.157 | 0 | -41.905 | 0.0274 | 1255.441 |
| -8.55E-04 | 0.8061 | 0.0513 | -551.430 | -188.570 | 697.979 | 0 | -42.021 | 0.0313 | 1251.459 |
| -1.26E-03 | 0.798 | 0.0580 | -474.020 | -277.193 | 708.493 | 0 | -42.720 | 0.0355 | 1252.005 |
| -1.57E-03 | 0.7942 | 0.0651 | -411.499 | -345.387 | 714.433 | 0 | -42.453 | 0.0400 | 1250.716 |
| -1.82E-03 | 0.7927 | 0.0727 | -360.062 | -401.516 | 719.790 | 0 | -41.789 | 0.0447 | 1251.887 |
| -2.02E-03 | 0.7927 | 0.0809 | -318.073 | -446.303 | 722.205 | 0 | -42.171 | 0.0497 | 1251.442 |
| -2.19E-03 | 0.7936 | 0.0895 | -283.641 | -482.668 | 723.803 | 0 | -42.505 | 0.0550 | 1251.106 |

Note: units on the table – kip, ft

## APPENDIX 20.3.3. Pushover and NTHA Modeling and Results

Capturing nonlinearities of soil and structures is crucial in the analysis. Soil springs determined in the previous section are attached along the beam elements. For structural nonlinearity, the concentrated plastic hinges are incorporated in the wall section. The plastic hinge locations can be determined through additional pushover analysis without requiring detailed plastic hinge modeling. As the lateral force incrementally increases, the bending moment is developed along the perimeter of the tunnel. As shown in Figure A20.3.3-1, the maximum bending moments are developed along the walls: the top and bottom of the wall, and the bottom two-thirds of the wall height. Based on this preliminary analysis, the plastic hinges are symmetrically assigned at those locations on both walls as illustrated in Figure A20.3.3-2.

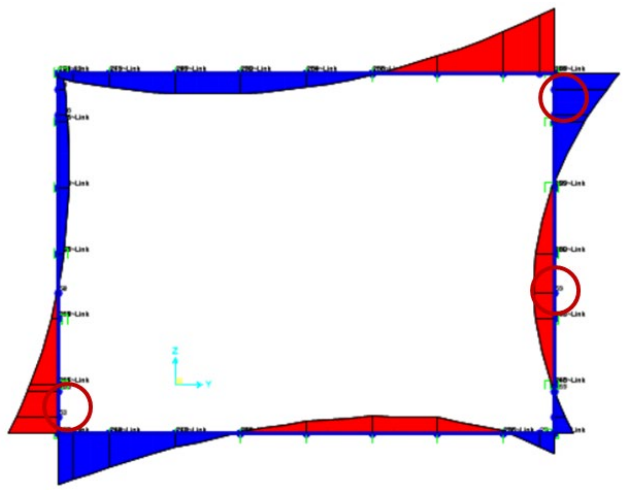


Figure A20.3.3-1. Bending moment diagram in pushover and locations of max bending moments (in red circles)

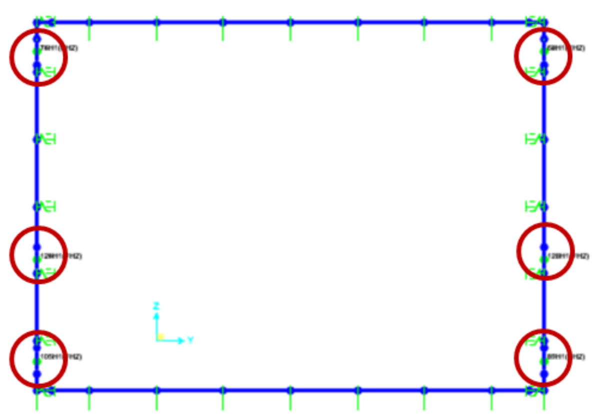
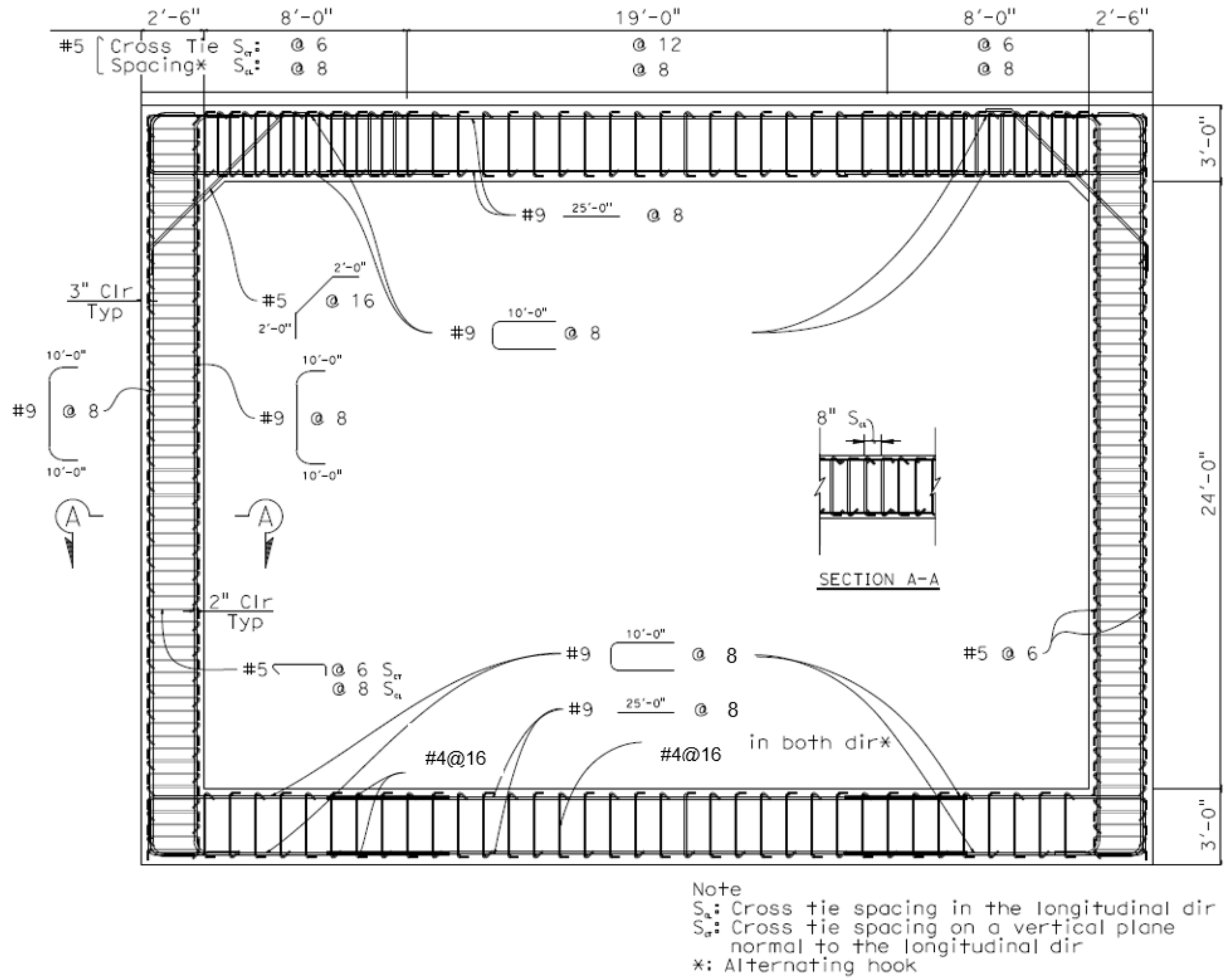
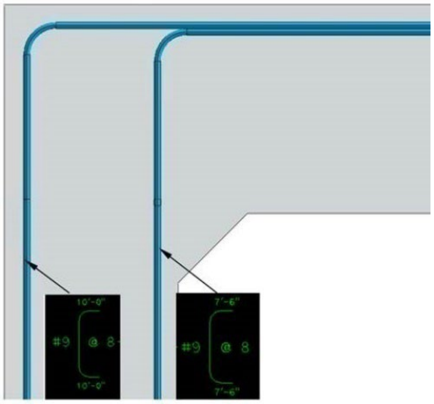


Figure A20.3.3-2. Assigned Plastic Hinge Locations (in red circles)

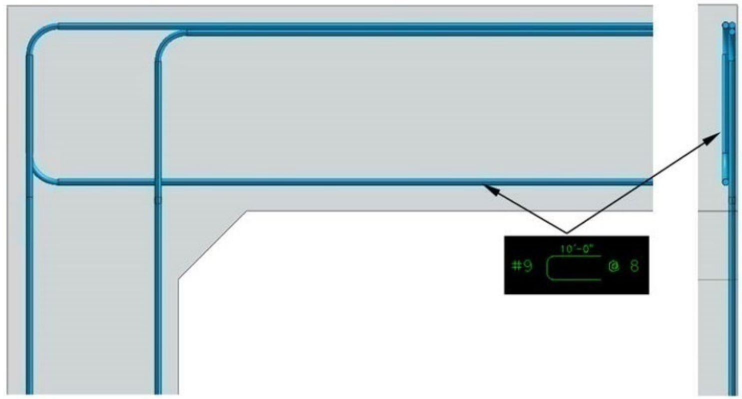
# APPENDIX 20.3.4. Details of Plans



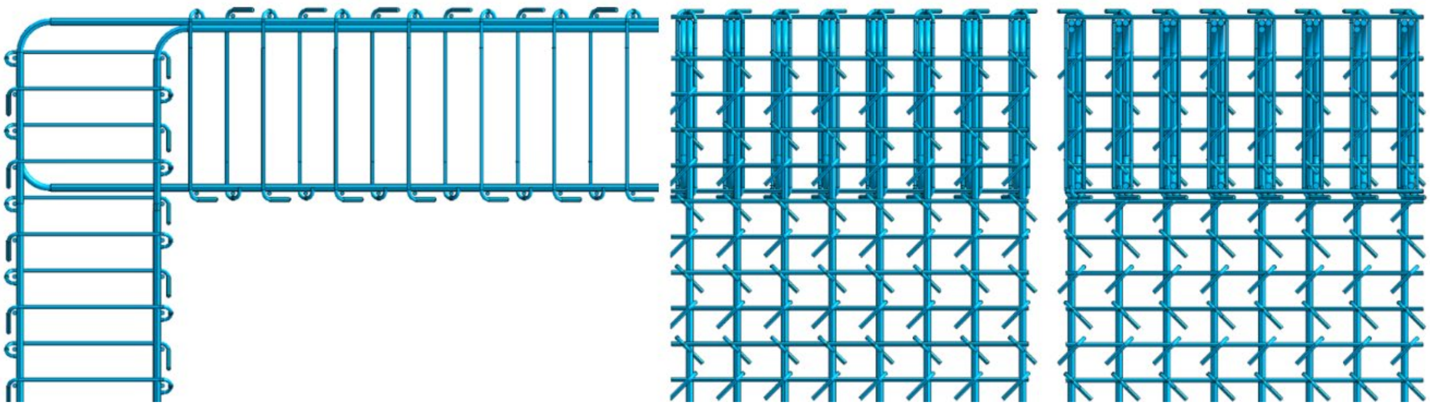

(a) Typical Section



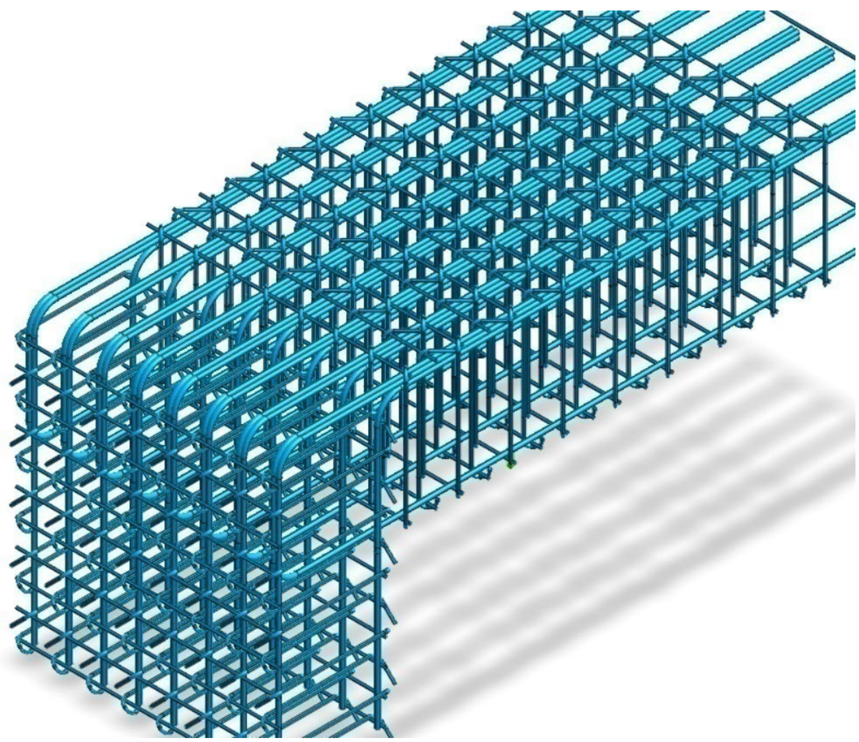

(b) Vertical Bar Layout



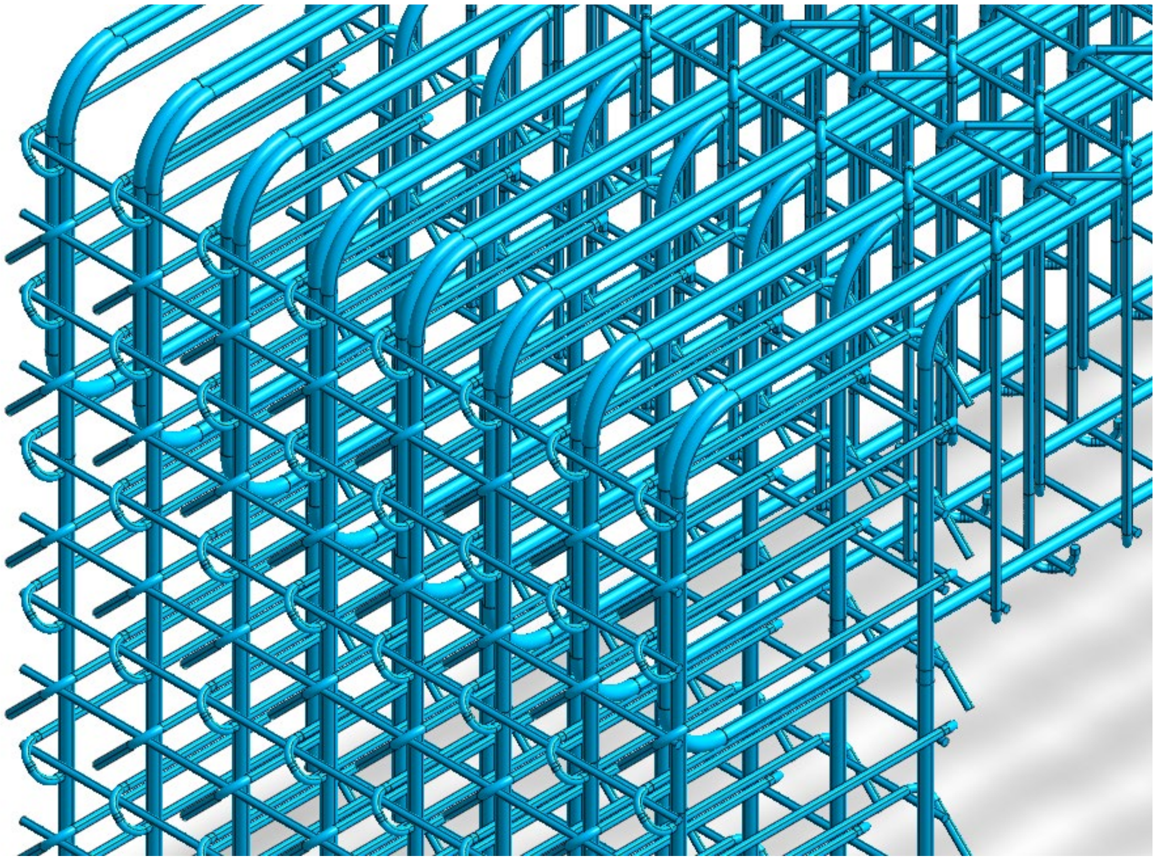

(c) "U" Bar in Top Slab in Elev and Side Views



(d) Cages in Elevation View and Side Views



(e) Isometric View 1



(f) Isometric View 2

Figure A20.3.4-1. Details of Reinforcement

# NOTATION

$A_e$ = effective shear area
$A_g$ = gross cross section area of a SCM
$A_{jh}$ = effective horizontal area of a moment resisting joint ($in^2$)
$A_{jv}$ = effective vertical area for a moment resisting joint ($in^2$)
$A_s$ = area of nonprestressed tension reinforcement ($in^2$)
$A_v$ = area of shear reinforcement perpendicular to the flexural tension reinforcement; area of transverse reinforcement within distance s ($in^2$)
a = depth of equivalent rectangular stress block (in.)
B = unit length of wall cross section
$B_{cap}$ = unit slab width (in.)
$B_{eff}$ = effective width of the superstructure for resisting longitudinal seismic moments (in.)
$b_v$ = effective web width taken as the minimum web width within the shear depth $d_v$ (in.)
c = distance from the extreme compression fiber to the neutral axis (in.)
D' = cross-sectional dimension of confined concrete core measured between the centerlines of the peripheral hoop or spiral (in.)
$D'_c$ = confined column cross-section dimension, measured out to out of ties, in the direction parallel to the axis of bending
$D_c$ = cross-sectional dimension of wall in transverse direction (width)
$D_s$ = depth of slab
$d_p$ = distance from extreme compression fiber to the centroid of the prestressing force (in.)
$d_v$ = effective shear depth
E = elastic modulus
$E_c$ = modulus of elasticity of concrete
$E_s$ = modulus of elasticity of steel
F1 = shear safety factor
F2 = shear safety factor
$F_r$ = flexibility ratio
$f'_c$ = concrete compressive strength
$f'_{ce}$ = expected compressive strength
$f_h$ = average normal stress in the horizontal direction within a moment resisting joint
$f_s$ = stress in the nonprestressed tension reinforcement at nominal flexural resistance
$f_y$ = specified minimum yield strength of reinforcement; average normal stress in the vertical direction within a moment resisting joint
$f_{ye}$ = expected yield strength of reinforcement
$f_{yh}$ = specified minimum yield strength of transverse reinforcement
$f_{ue}$ = expected tensile strength of reinforcement
G = shear modulus
$G_{max}$ = small-strain shear modulus
$G_m$ = strain-compatible shear modulus

$H$ = the distance between CG of compressive force and tension force on the wall section
$l_{ac,provided}$ = actual length of wall reinforcement embedded into the slab
$I_{crack}$ = moment of inertia of the cracked section, transformed to concrete ($in^4$)
$K_s$ = racking stiffness
$L$ = length of a SCM from the point of maximum moment to the point of contraflexure (in.)
$L_P$ = equivalent plastic hinge length
$M_{D,10\% \text{ of weight}}$ = moment due to 10% of dead load
$M_{eq}$ = seismic moment at the top of the wall
$M_n$ = nominal moment capacity based on the nominal concrete and steel strengths when the concrete strain reaches 0.003 (kip-ft)
$M_o$ = overstrength moment of a seismic critical member
$M_{p,min}$ = minimum idealized plastic moment capacity of an SCM
$M_p$ = idealized plastic moment capacity of an SCM
$M_u$ = ultimate moment capacity of an SCM
$M_w$ = seismic moment magnitude for SEE
$M_y$ = moment of a seismic critical member corresponding to the first reinforcing bar yield
$N_u$ = factored axial force, taken as positive if tensile and negative if compressive (kip)
$N_{60}$ = SPT blow count corrected for hammer efficiency
$P_c$ = axial force due to dead load and overturning
$P_{dl}$ = axial force due to dead load
$p_c$ = nominal principal compression stress in a joint
$p_t$ = nominal principal tension stress in a joint
$R$ = ratio of equivalent uniform strain divided by maximum strain
$R_d$ = depth-dependent stress reduction factor
$R_r$ = racking coefficient
$s$ = spacing of shear/transverse reinforcement
$s_{CT}$ = cross-tie spacing on a vertical plane normal to the longitudinal direction
$s_{CL}$ = cross-tie spacing in the longitudinal direction
$s_T$ = transverse reinforcement spacing in the longitudinal direction
$s_L$ = longitudinal reinforcement spacing on a vertical plane normal to longitudinal direction
$T_c$ = total tensile force in column longitudinal reinforcement associated with $M_o$
$T_{max}$ = maximum tension
$V_c$ = nominal shear capacity provided by concrete
$V_{eq}$ = seismic shear demand
$V_{jv}$ = nominal shear stress in the vertical direction within a moment resisting joint
$V_n$ = nominal shear capacity

$V_o$ = overstrength shear associated with the overstrength moment of a SCM
$V_p$ = inelastic lateral force; shear force associated with plastic moment
$V_s$ = nominal shear capacity provided by shear reinforcement
$V_{s30}$ = shear wave velocity for the upper 30 m of the soil profile
$V_u$ = shear demand
$v_m$ = Poisson's ratio of soil
W = width of the tunnel structure
$\phi_{y(Initial)}$ = yield curvature corresponding to the first reinforcing bar yield
$\phi_{steel}$ = reinforcing steel curvature
$\phi_{y(idealized)}$ = idealized yield curvature
$\phi_{concrete}$ = concrete curvature
$\phi$ = resistance factor
$\theta$ = angle of inclination of diagonal compressive stresses (deg)
$\gamma$ = soil unit weight
$\gamma_{eff}$ = effective shear strain
$\gamma_{max}$ = maximum shear strain
$\lambda$ = concrete density modification factor
$\rho$ = reinforcement ratio
$\rho_{dl}$ = axial load ratio due to dead load
$\rho_s$ = volumetric ratio of transverse reinforcement
$\rho_c$ = axial load ratio due to dead load and overturning
$\rho_{s,min}$ = minimum volumetric ratio of wall confinement reinforcement
$\rho_{soil}$ = mass density of soil
$\alpha_1$ = stress block factor specified
$\alpha$ = angle of inclination of transverse reinforcement to the longitudinal axis (deg)
$\beta_1$ = stress block factor
$\beta$ = factor relating effect of longitudinal strain on the shear capacity of concrete as indicated by the ability of diagonally cracked concrete to transmit tension.
$\sigma'_{vo}$ = effective overburden stress
$\delta_{free\text{-}field}$ = free-field shear deformation
$\tau_{max}$ = maximum earthquake-induced shear stress
$\varepsilon_{steel}$ = peak strain demand for reinforcing steel from analysis
$\varepsilon_{con'c}$ = peak strain demand for confined concrete from analysis
$\varepsilon_x$ = longitudinal strain at the middepth of the member
$\varepsilon_{su}^R$ = reduced ultimate tensile strain
$\varepsilon_{cu}$ = ultimate compression strain for confined concrete
$\Delta_D$ = peak displacement demands
$\Delta_{free\text{-}field}$ = free-field shear deformation over tunnel height
$\Delta_y$ = idealized yield displacement of a seismic critical member
$\Delta_c$ = displacement capacity
$\Delta_s$ = racking deformation of the tunnel structure

$\mu_D$ = displacement ductility demand
$\mu_d$ = local displacement ductility demand

ATH = acceleration t time history
ARS = acceleration response spectrum
CCT = Cut-And-Cover tunnels
DC = dead load of structural components and attachments
DTH = displacement time history
EH = horizontal earth pressure loads
EDP = engineering demand parameters
EV = vertical pressure from dead load of earth fill
EQ = earthquake load
FEE = functional evaluation earthquake
GD = geotechnical designer
NTHA = nonlinear time history analysis
PGA = peak ground acceleration
SCM = seismic critical members longitudinal axis of the structural member
SEE = safety evaluation earthquake
SPT = standard penetration test

For notations not included, see AASHTO (2017a), BDM 20.32, & SDC.

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