Design Documentation of Excavation Support and Vault Bracing

301 Mission Street
San Francisco, CA
5 December 2018

SGH Project 147041.10

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

PREPARED FOR:

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1. INTRODUCTION

1.1 Background

The concrete tower at 301 Mission will undergo a voluntary upgrade and foundation improvement. New piles will be added adjacent to the existing mat slab. The tower mat slab will be extended to incorporate the new piles along Fremont St. and Mission St. A braced shoring system will be used excavate and construct the mat extension.

1.2 Objective

Our design objective in this report is to develop a Support of Excavation (SOE) system to allow retrofit work on the tower foundation to be completed. Our shoring system will provide support for an excavation to the bottom of the tower mat foundation.

1.3 Scope of Work

Our scope of work includes the following tasks:

- Develop shoring system design to support foundation improvement construction
- Develop specifications for the shoring system
- Develop a support system for PG&E vaults during construction

1.4 Project Description

The tower at 301 Mission is located on the corner of Fremont Street and Mission Street in San Francisco. The shoring wall will consist of soldier piles installed in drilled holes, with lagging and jet grout columns in between the soldier piles. The excavation will be 10 ft wide; 27 ft deep and will be braced by a single level of waler and strut system. To maximize the excavation access, the waler will be installed directly above the soldier piles and the struts will be raised above the waler to also support excavator platforms. The shoring system will be constructed along the street and sidewalk for approximately 175 ft along Fremont Street, and 125 ft along Mission Street.

2. DESIGN REFERENCES

We used the following codes and documents for our design:

- CBC 2016
- AASHTO LRFD Bridge Design Specifications 2016
- ACI 318-14
- ASCE 7-10
- AISC 360-10
- "Jet Grouting – Technology, Design, and Control". Croce, Flora, Modoni. 2014.
- "Single Piles and Pile Groups under Lateral Loading, 2nd Edition". Impe, Reese. 2011.

3. ANALYSIS AND DESIGN OF SHORING SYSTEM

We conducted parallel analyses with two software packages to verify the results and better predict the deflection response of the shoring system. The proposed design groundwater level for this location is El. -7 ft SFCD. However, recent explorations have shown the groundwater much deeper. Our analysis will account for both conditions:

- Design groundwater table present at El. -7 ft SFCD (GWH), and
- Expected groundwater table present at El. -28 ft SFCD (GWL)

Note that all elevations shown are for reference and may vary in the field. Contractor shall verify all elevations in the field prior to installation of the SOE system.

3.1 SAP2000 Finite Element Analysis

We modeled the complete shoring system around the tower foundation along Fremont Street and Mission Street. We modeled the soldier piles, walers, struts, and other bracing as frame elements. We modeled P-Y springs which provided the lateral resistance for the soldier piles. Figure 3-1 below shows the model along with the location of the soil springs.

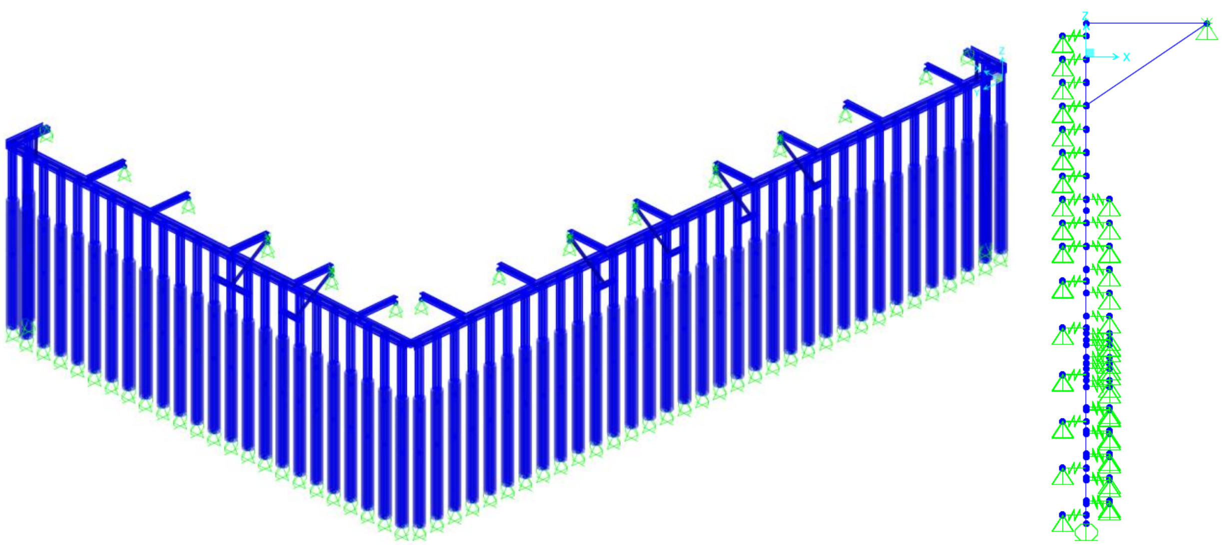
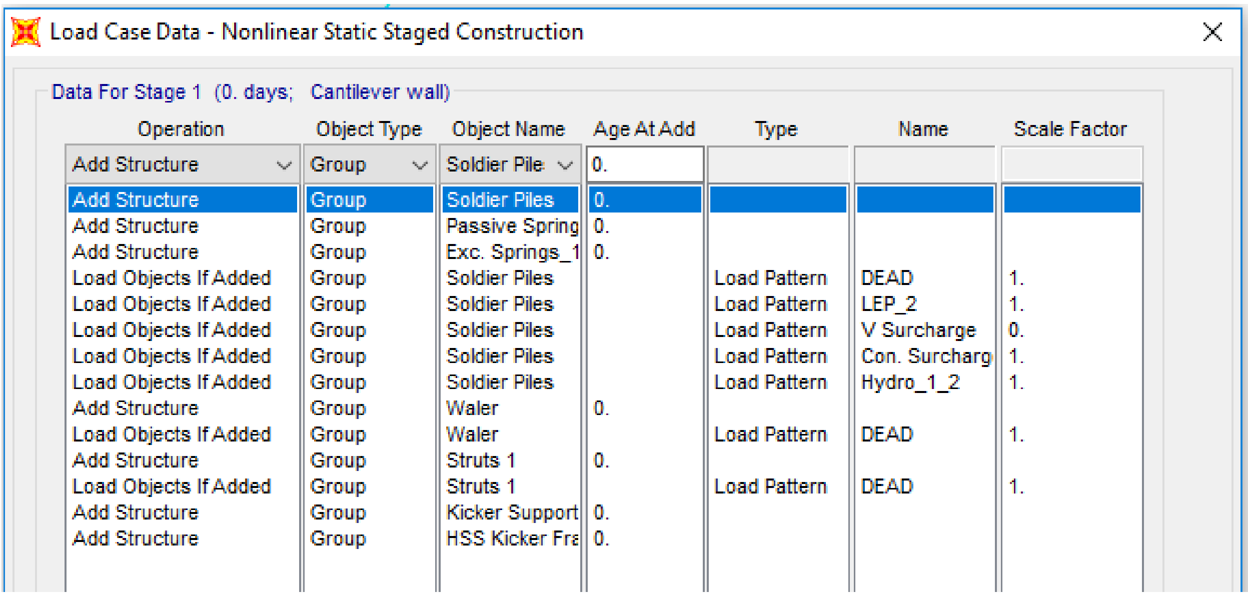


Figure 3-1: SAP Model (Left – Frame Elements, Right – Elevation View with Soil Springs)

We used nonlinear staged analysis to evaluate the successive stages of excavation. We developed the following 2 stages:

1. Install soldier piles, waler, and type 1 struts for initial excavation of 14 ft
2. Install type 2 struts and additional kicker bracing and excavate remaining depth to jet grout plug

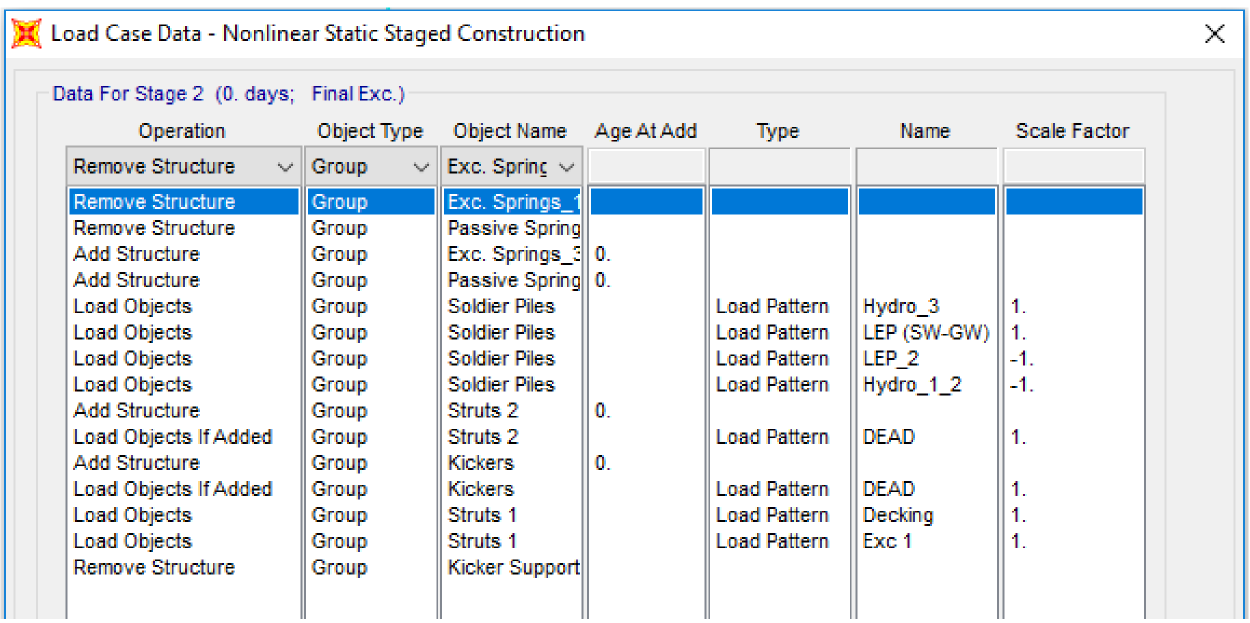
Figure 3-2 below shows the detailed SAP2000 staged construction steps.



Load Case Data - Nonlinear Static Staged Construction

Data For Stage 1 (0. days; Cantilever wall)

Operation	Object Type	Object Name	Age At Add	Type	Name	Scale Factor
Add Structure	Group	Soldier Pile	0.			
Add Structure	Group	Soldier Piles	0.			
Add Structure	Group	Passive Spring	0.			
Add Structure	Group	Exc. Springs_1	0.			
Load Objects If Added	Group	Soldier Piles		Load Pattern	DEAD	1.
Load Objects If Added	Group	Soldier Piles		Load Pattern	LEP_2	1.
Load Objects If Added	Group	Soldier Piles		Load Pattern	V Surcharge	0.
Load Objects If Added	Group	Soldier Piles		Load Pattern	Con. Surcharg	1.
Load Objects If Added	Group	Soldier Piles		Load Pattern	Hydro_1_2	1.
Add Structure	Group	Waler	0.			
Load Objects If Added	Group	Waler		Load Pattern	DEAD	1.
Add Structure	Group	Struts 1	0.			
Load Objects If Added	Group	Struts 1		Load Pattern	DEAD	1.
Add Structure	Group	Kicker Support	0.			
Add Structure	Group	HSS Kicker Fra	0.			



Load Case Data - Nonlinear Static Staged Construction

Data For Stage 2 (0. days; Final Exc.)

Operation	Object Type	Object Name	Age At Add	Type	Name	Scale Factor
Remove Structure	Group	Exc. Sprinç				
Remove Structure	Group	Exc. Springs_1				
Remove Structure	Group	Passive Spring				
Add Structure	Group	Exc. Springs_3	0.			
Add Structure	Group	Passive Spring	0.			
Load Objects	Group	Soldier Piles		Load Pattern	Hydro_3	1.
Load Objects	Group	Soldier Piles		Load Pattern	LEP (SW-GW)	1.
Load Objects	Group	Soldier Piles		Load Pattern	LEP_2	-1.
Load Objects	Group	Soldier Piles		Load Pattern	Hydro_1_2	-1.
Add Structure	Group	Struts 2	0.			
Load Objects If Added	Group	Struts 2		Load Pattern	DEAD	1.
Add Structure	Group	Kickers	0.			
Load Objects If Added	Group	Kickers		Load Pattern	DEAD	1.
Load Objects	Group	Struts 1		Load Pattern	Decking	1.
Load Objects	Group	Struts 1		Load Pattern	Exc 1	1.
Remove Structure	Group	Kicker Support				

Figure 3-2: Staged Construction Analysis Parameters for High Groundwater
(Top) – Stage 1: Initial Excavation
(Bottom) – Stage 3: Final Excavation

3.1.1 Soil P-Y Springs

We developed P-Y springs in LPILE v2016 to evaluate the lateral resistance of the soldier piles in our SAP2000 model. We developed soil parameters from geotechnical data available in the original geotechnical design report and collaborated with John Egan of Slate Geotechnical and Shannon & Wilson. We modeled a 32 in. diameter concrete soldier piles and calculated a 1.875 spacing to diameter ratio. We used recommendations for side-by-side piles in the reference, "Single Piles and Pile Groups under Lateral Loading", to calculate a 0.79 p-multiplier for the soldier piles.

Table 3-1 and Table 3-2 show the LPILE soil parameters for the GWH and GWL scenarios respectively. Figure 3-3 through Figure 3-6 plot the LPILE p-y springs for the backfill side and the initial excavation for both the GWH and GWL scenarios. Figure 3-7 shows the p-y springs for the final excavation, note that we replaced the top 5 ft of p-y springs below the final excavation with the jet grout spring shown in Figure 3-8.

Table 3-1: LPILE soil parameters for GWH scenario

Soil Type	LPile soil model	Top Elevation (SFCD ft)	Bottom Elevation (SFCD ft)	Effective Unit Weight γ_{eff} (pcf)	Friction angle ϕ (deg)	k (pci)	undrained cohesion (psf)	Strain factor E50
Fill	Sand (Reese)	2.8	-26	51	30	30	-	-
Marine Deposits	Soft Clay	-26	-40	43	-	-	880	0.02
Silty Sand	Sand (Reese)	-40	-44	64	35	60	-	-
Clayey Sand	Stiff Clay w/o free water	-44	-54	43	-	-	1595	0.007
Lower Silty Sands	Sand (Reese)	-54	-83	66	34	75	-	-
Old Bay Clay	Stiff Clay w/o free water	-83	-93	51	-	-	3960	0.005

Table 3-2: LPILE soil parameters for GWL scenario

Soil Type	LPile soil model	Top Elevation (SFCD ft)	Bottom Elevation (SFCD ft)	Effective Unit Weight γ_{eff} (pcf)	Friction angle ϕ (deg)	k (pci)	undrained cohesion (psf)	Strain factor E50
Fill	Sand (Reese)	2.8	-26	115	30	37.5	-	-
Marine Deposits	Soft Clay	-26	-40	43	-	-	880	0.02
Silty Sand	Sand (Reese)	-40	-44	64	35	60	-	-
Clayey Sand	Stiff Clay w/o free water	-44	-54	43	-	-	1595	0.007
Lower Silty Sands	Sand (Reese)	-54	-83	66	34	75	-	-
Old Bay Clay	Stiff Clay w/o free water	-83	-93	51	-	-	3960	0.005

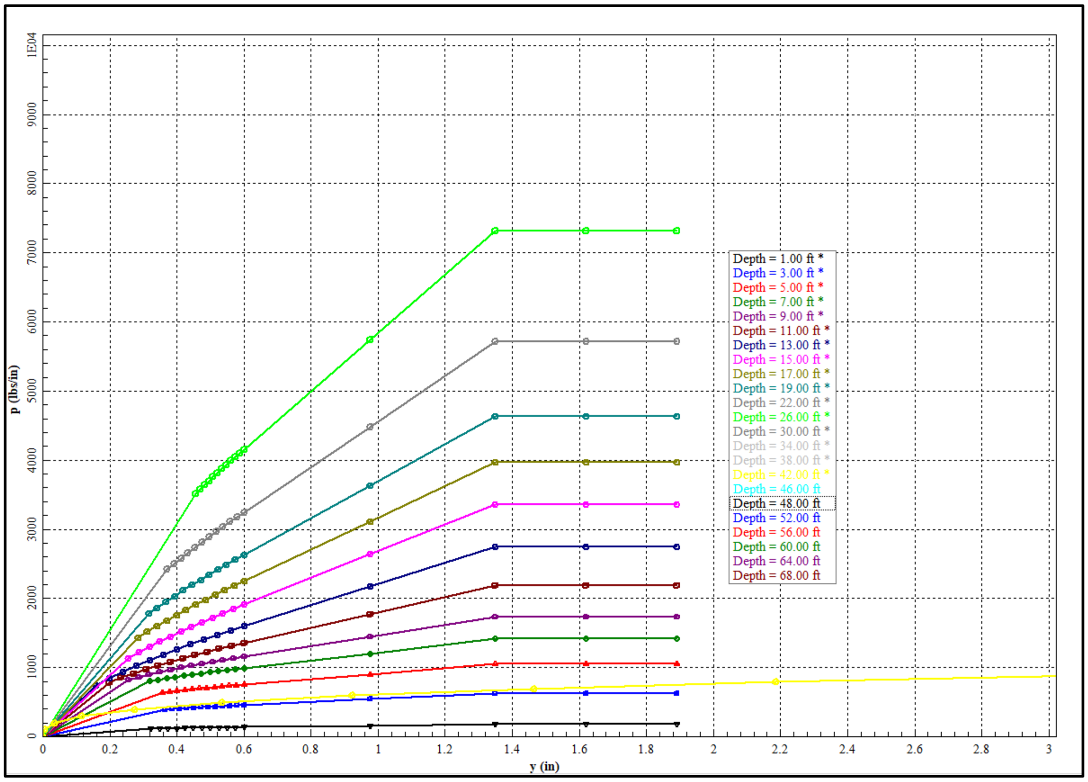


Figure 3-3: LPILE p-y springs backfill side (GWH)

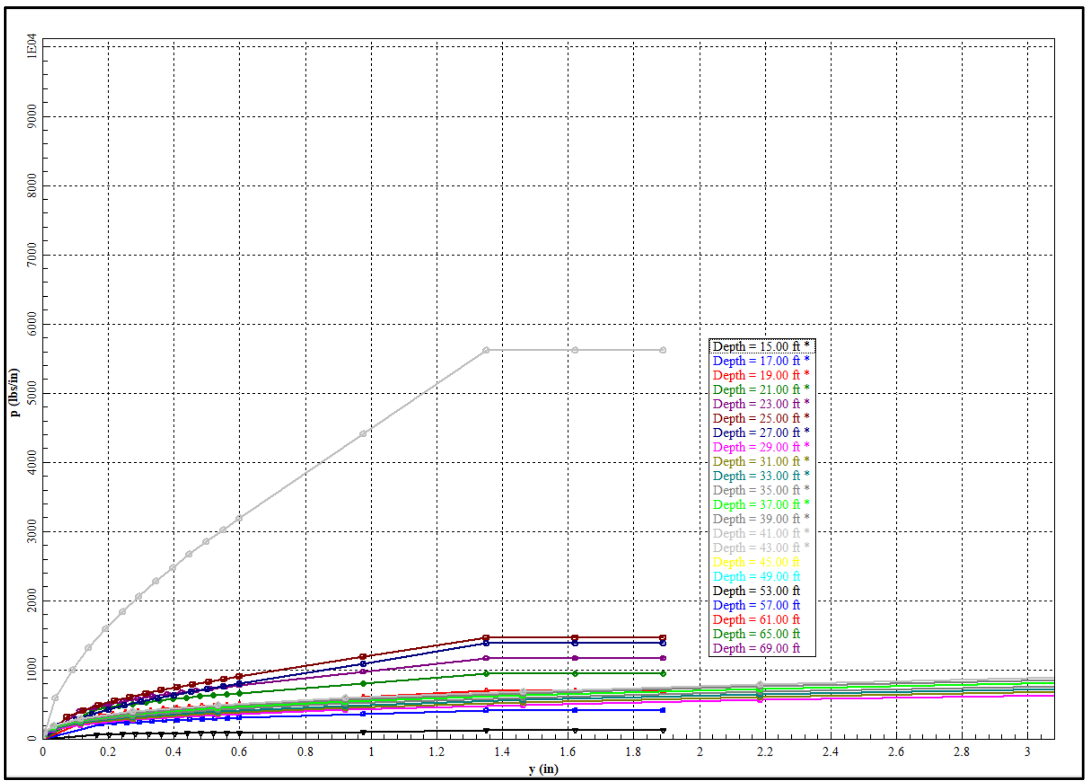


Figure 3-4: LPILE p-y springs initial excavation (GWH)

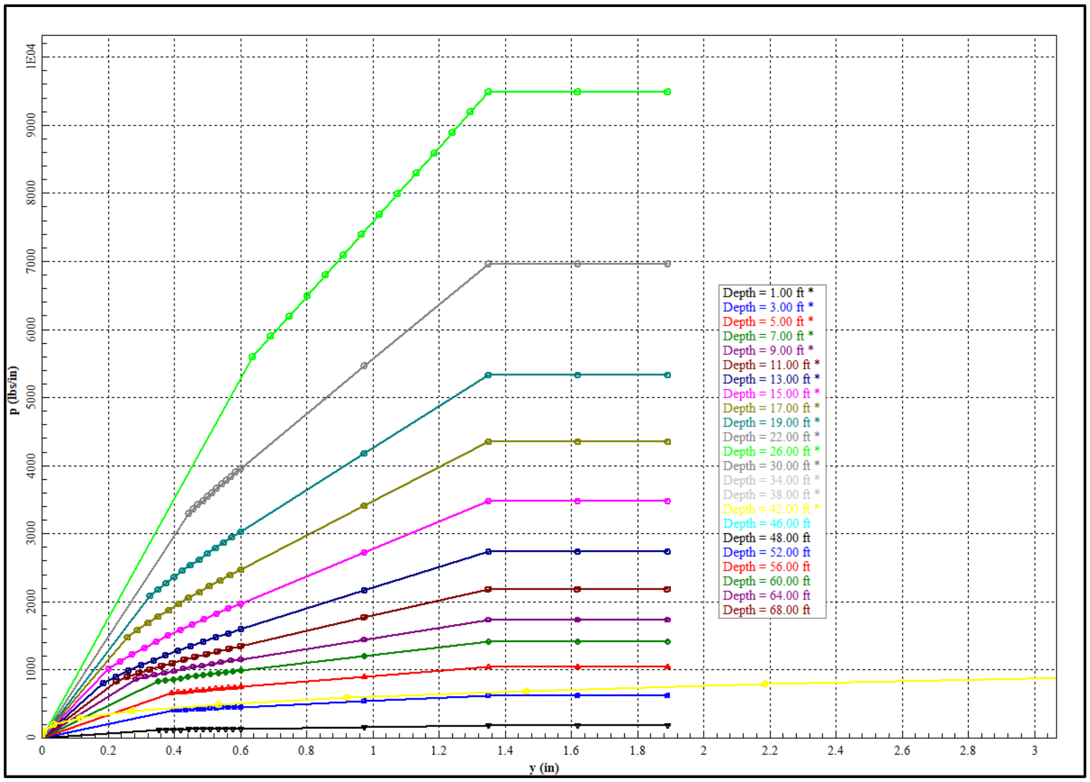


Figure 3-5: LPILE p-y springs backfill side (GWL)

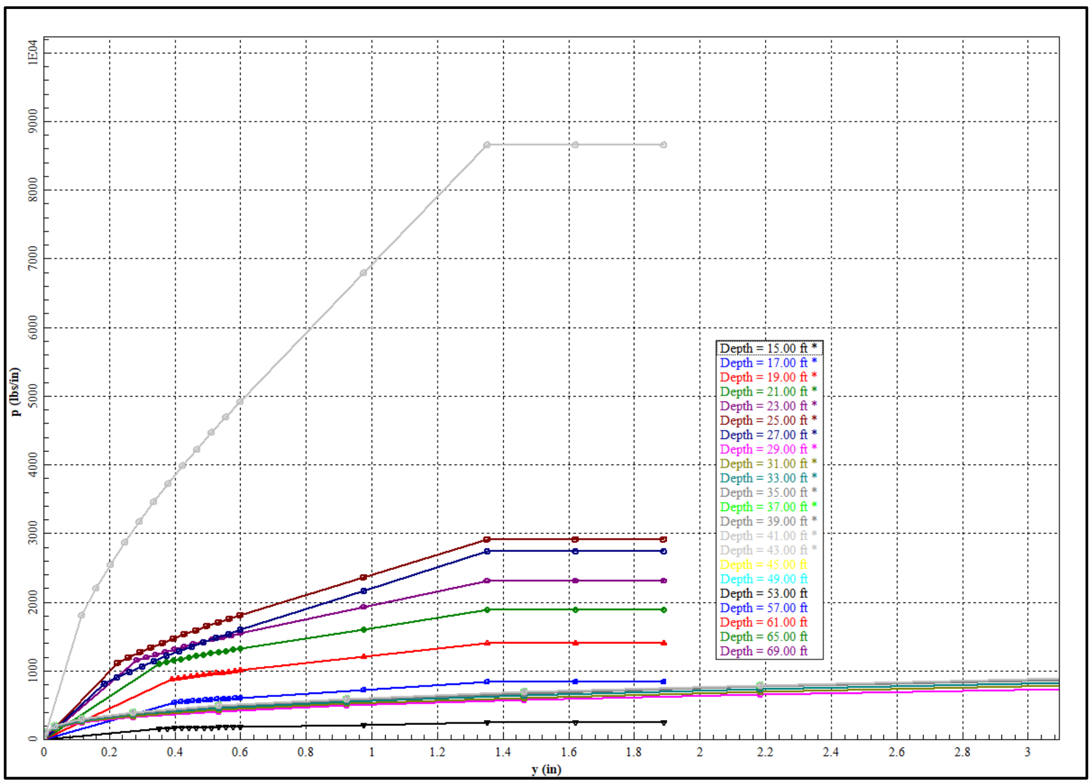


Figure 3-6: LPILE p-y springs initial excavation (GWL)

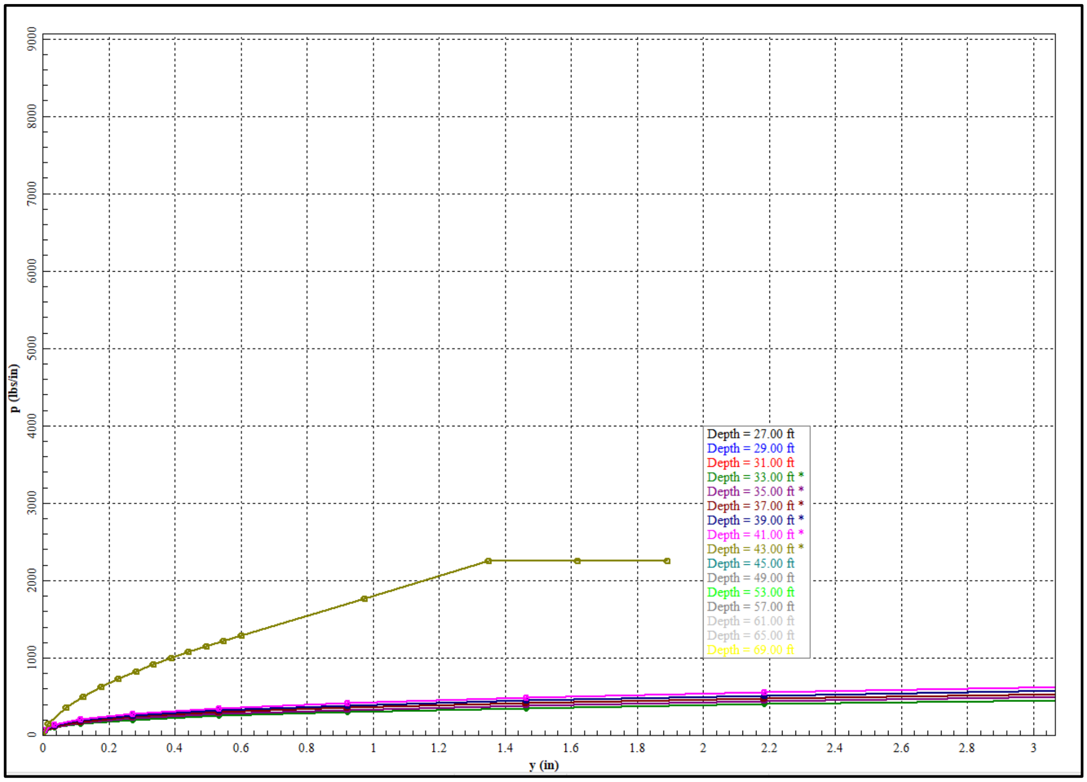


Figure 3-7: LPILE p-y springs final excavation (GWH & GWL)

3.1.2 Jet Grout Plug

The jet grout plug serves 2 purposes: an impermeable barrier at the base of the excavation to provide dry working conditions, and a compression plug at the base of the excavation to limit wall deflections. We relied on the information and experience presented in the jet grouting reference listed in Section 2 to model the plug parameters.

The plug will be installed in the marine deposits layer, which consists of very soft to medium stiff clays interbedded with very loose to medium dense sands and clayey sands. Generally, jet grouting produces better performance characteristics when installed in cohesionless soils with sufficient void space to allow cementing to better penetrate through the soil. Due to the variability of the layer, we conservatively assumed the layer will behave as a clay material which will result in a lower jet grout strength as compared to a cohesionless material.

As described in the reference, “Jet Grouting – Technology, Design, and Control”, it is common to assume quasilinear behavior before failure. We assumed the linear stiffness of the grout to be the tangent stiffness at 50% of the failure stress, consistent with studies by Fang et al. presented in the reference. We assumed an effective width of grout which resists the soldier pile wall movement equal to 3 ft, roughly corresponding to the width of the concrete soldier pile where the load is concentrated. The design strength of the jet grout will be 400 psi and the final stiffness we used for the SAP model (input as a p-y spring) is 288 kips/inch of deflection. We calculated the failure deflection of the plug to be 0.42 inches.

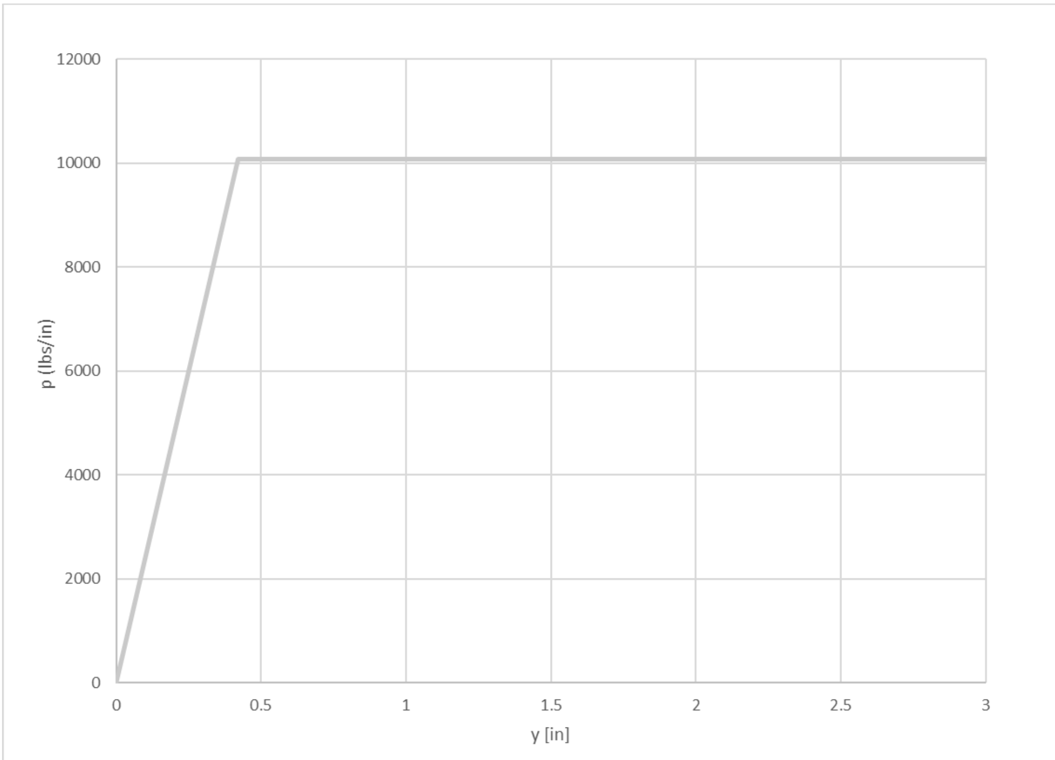


Figure 3-8: Jet grout p-y spring final excavation (GWH & GWL)

3.1.3 Loading

Shannon & Wilson (S&W) provided the apparent earth pressure for the final excavation stage for both GWL and GWH cases. The trapezoidal distribution corresponds to the braced shoring system with the strut and waler system bracing the top of the shoring wall. Figure 3-9 plots the lateral loading for the final excavation stage. Based upon our review of the planned construction loads near the shoring system, we decided to add a lateral surcharge of 250 psf due to heavy construction equipment adjacent to the excavation. Alternatively, we also checked the shoring system for a lateral surcharge due to vehicle traffic of 100 psf. Figure 3-10 shows the lateral loading we applied for the initial excavation stage. We linearly scaled the apparent earth pressure provided by S&W for the initial excavation height of 14 ft, while keeping the other loading components constant.

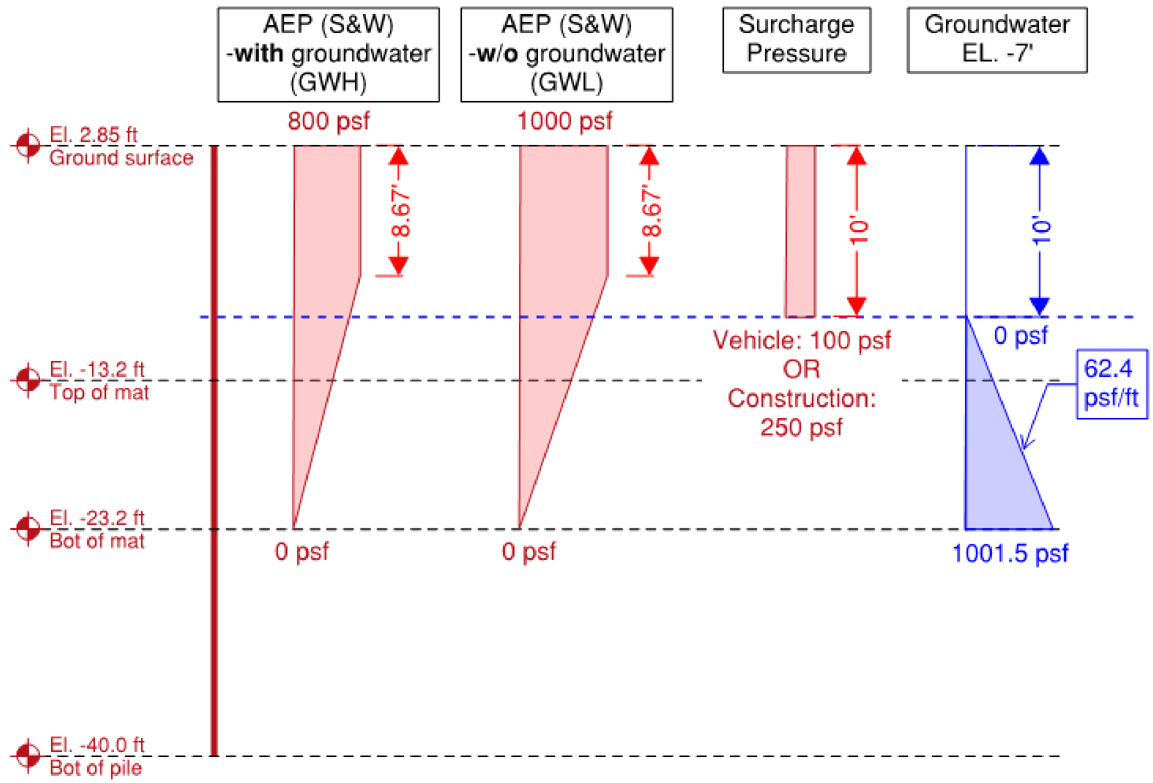


Figure 3-9: Lateral Loading for Final Excavation

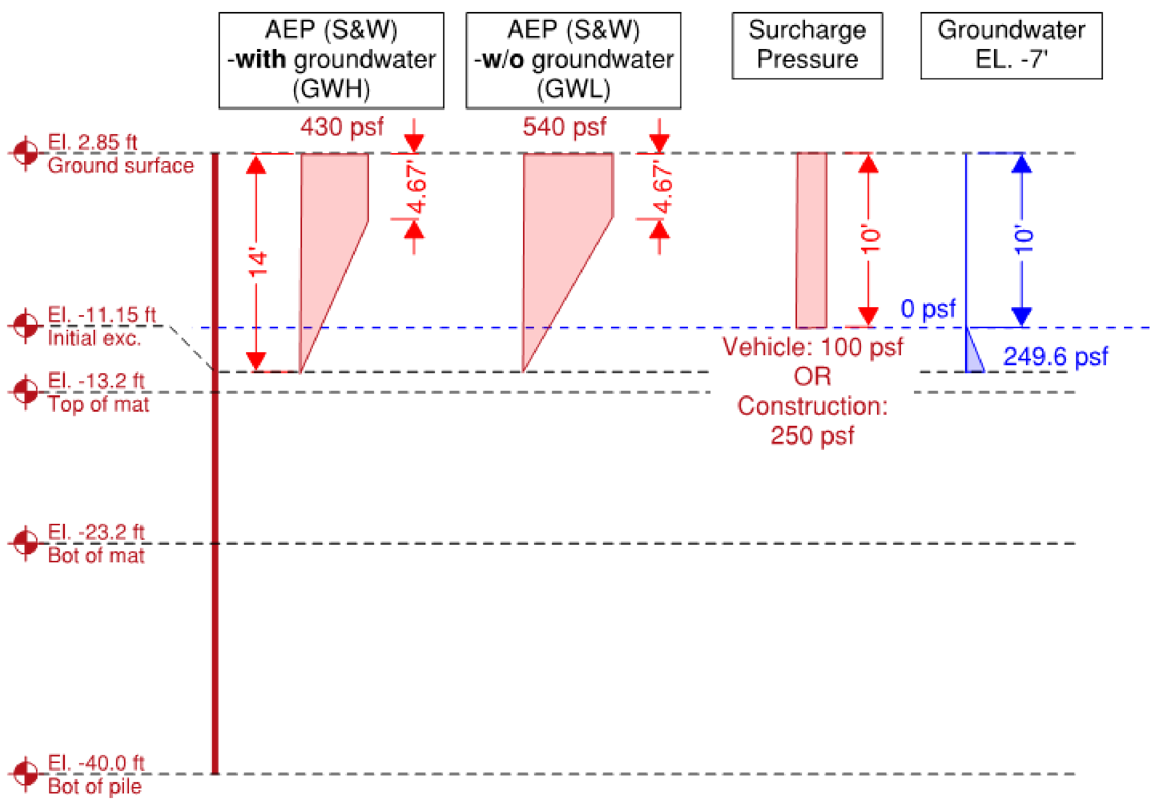


Figure 3-10: Lateral Loading for Initial Excavation

In addition to the lateral pressures, the struts will support construction equipment during excavation. We evaluated a CAT 325F excavator placed on a platform above the struts along the entire length of the excavation. We modeled worst case loading scenarios with an included impact factor of 1.33.

3.1.4 SAP2000 Analysis Results

We tabulated results and tracked deflections, member demands, and soil forces for both the design high and low groundwater conditions.

Jet Grout Plug

We observed a maximum force in the top jet grout soil spring of 36.8 kips. The springs represent an equivalent soil height of 1 ft, which corresponds to a compression demand of 85 psi on the jet grout.

Soldier Piles (W18x130)

Because the struts are spaced at 40 ft during the initial excavation, the behavior of the soldier pile wall varies with location relative to the initial struts. Piles furthest from the initial strut locations will experience the highest deflections during the initial stage. Piles close to the initial struts will attract a large portion of the apparent earth pressure due to the stiffness of the strut at the top of the pile. These piles will have the controlling shear and bending structural demands while not showing significant deflections until the final excavation (stage 2).

Figure 3-11 and Figure 3-12 plot the maximum soldier pile deflection and structural demands. As the stages progress, the location of maximum deflection moves from the top of the pile down to around mid-height of the excavation in the final step.

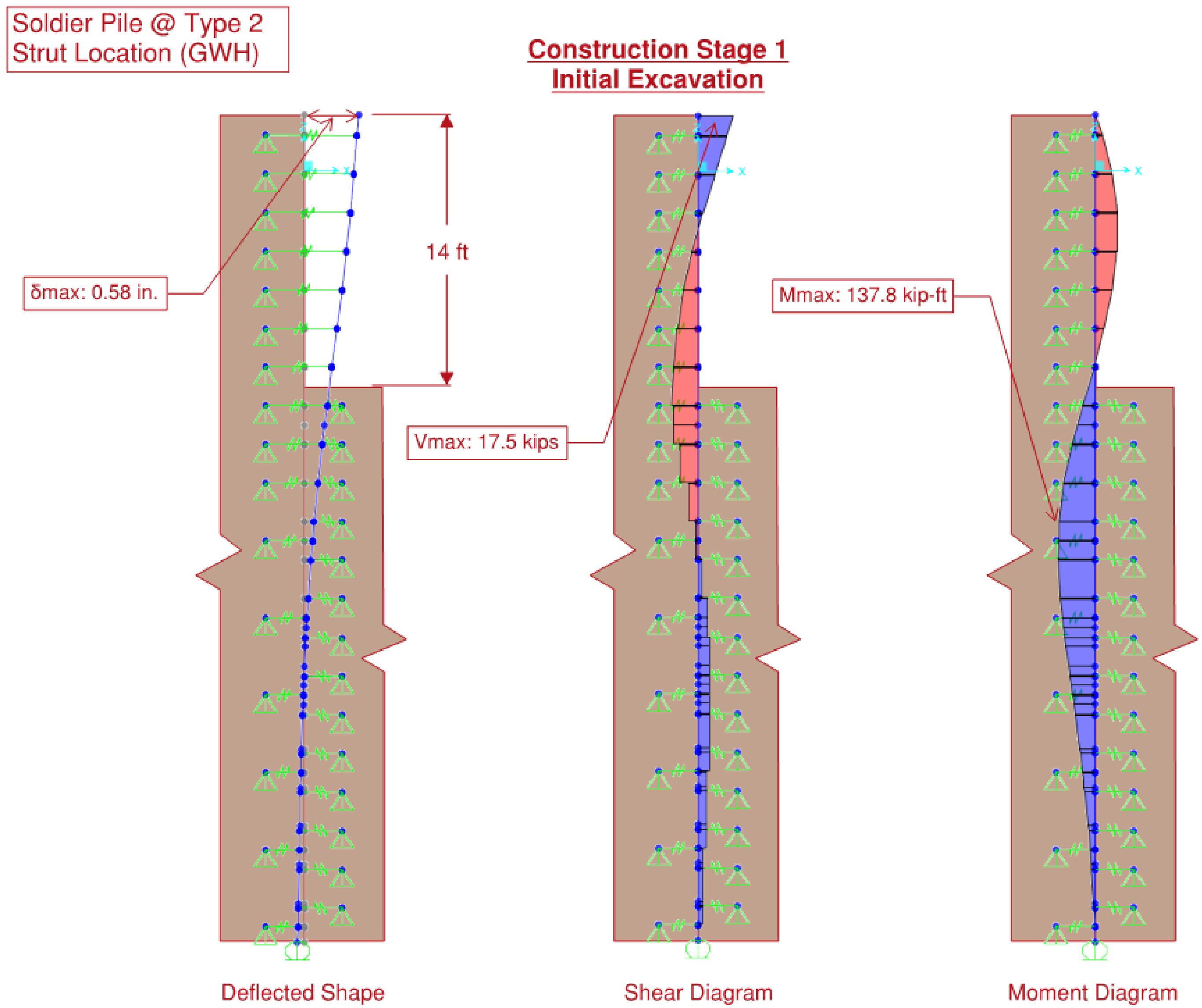


Figure 3-11: SAP Output for Soldier Pile Demands in Stage 1 (Initial Excavation)

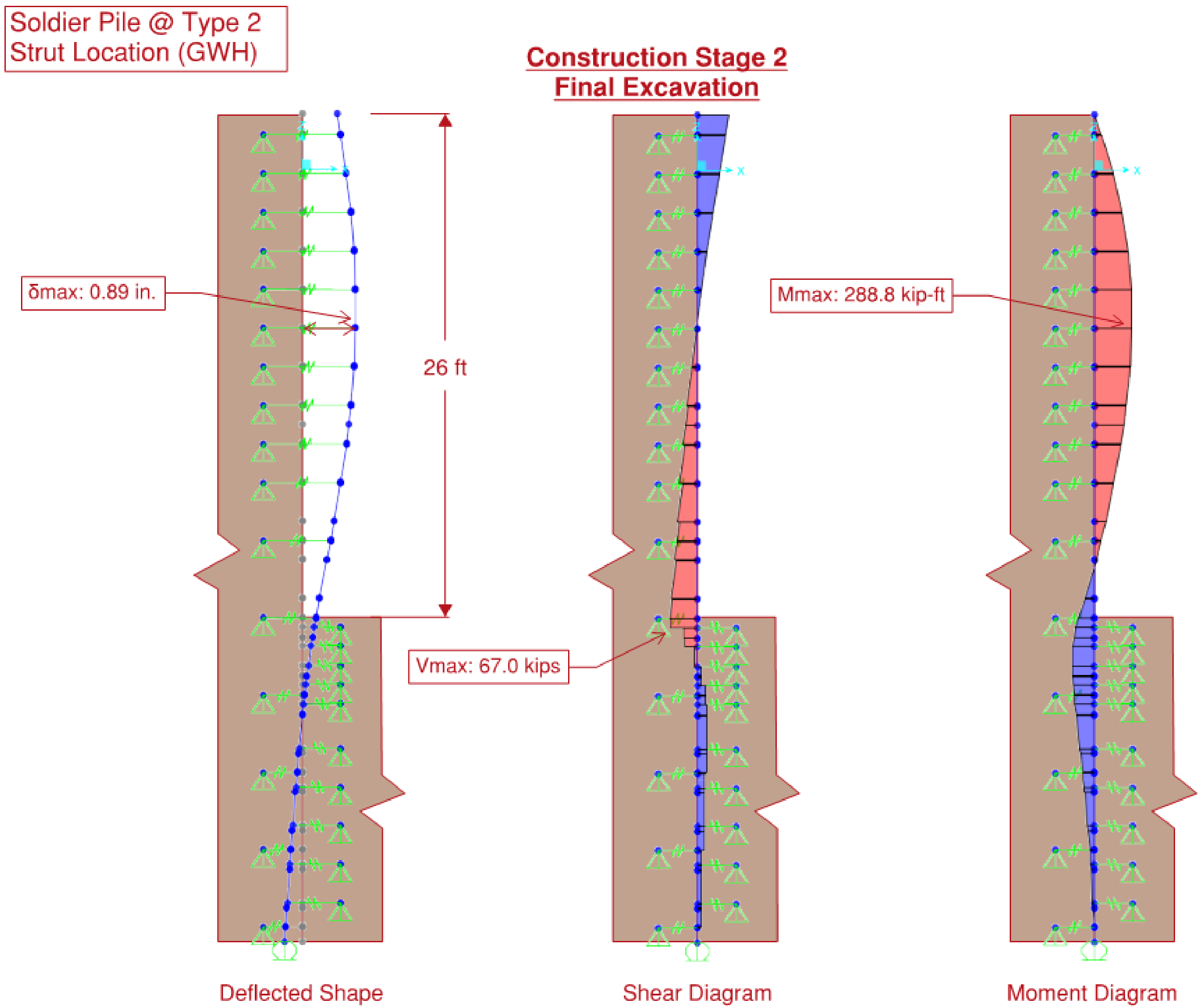


Figure 3-12: SAP Output for Soldier Pile Demands in Stage 2 (Final Excavation)

Waler (W27x146) and Struts (W12x136)

We modeled the waler as a continuous member along the length of the excavation. We sized the waler to limit deflections during the initial excavation stage and to ensure construction tolerances on soldier pile placement can be accommodated in the connection design. Figure 3-13 below shows the bending moment distribution in the waler for both construction stages. The waler experiences the highest bending moments at initial strut locations where axial deflections are small and the system is stiff.

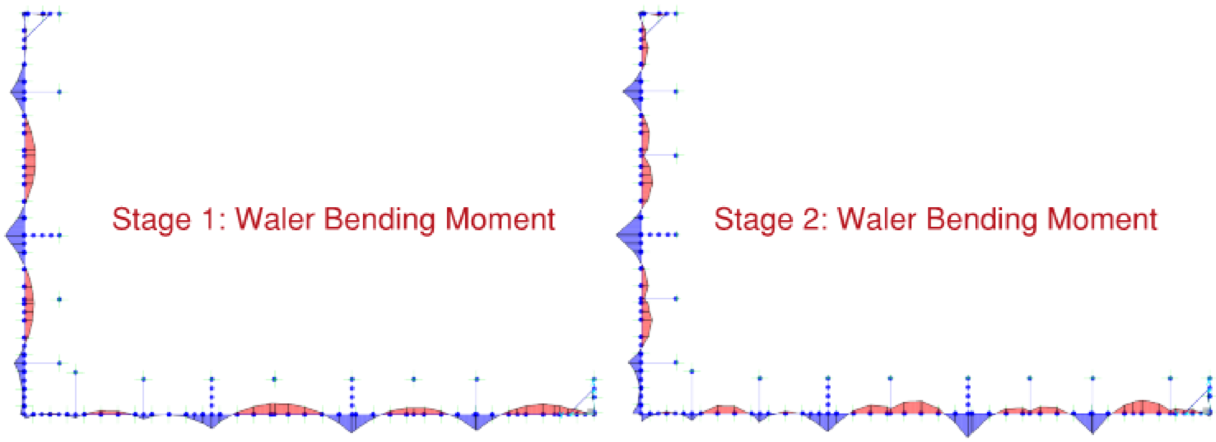


Figure 3-13: Bending Moments in Waler for GWH Case

The initial Type 1 struts carry two times the load the Type 2 struts. In addition to the axial demand, the struts will also support the excavator platform during construction. The struts are designed for combined axial and bending demands which include eccentricities in the connection design.

Design Check of Structural Members

We determined ASD level capacities for the shoring members using AISC to check demands obtained through our SAP analysis. Table 3-3 and Table 3-4 below show maximum demands for the shoring system elements for the high groundwater and low groundwater analysis cases, respectively. Table 3-5 shows the maximum deflections along the height of the soldier pile wall for each stage, and for high and low groundwater conditions. Our design calculations are presented in Appendix A.

Table 3-3: Member DCR Table for Design High Groundwater

Groundwater Condition: El. -7 ft SFCD (GWH)				
Shoring Element	Demand Type	Max ASD Demand	ASD Capacity	Controlling DCR
Soldier Pile (W18x130)	Bending (kip-ft)	342	724	0.47
	Shear (kips)	67	259	0.26
Waler (W27x146)	Bending M3 (kip-ft)	451	1160	0.78
	Bending M2 (kip-ft)	96	243	
	Shear (kips)	177	332	0.53
Strut (W12x136)	Axial (kips)	323	1030	0.88
	Bending (kips)	342	534	
Corner Brace (Pipe 8XS)	Axial (kips)	50	224	0.22
Kicker (Pipe 8XS)	Axial (kips)	172	220	0.78
HSS Kicker Support (HSS16x12x5/8)	Bending M3 (kip-ft)	90	412	0.74
	Bending M2 (kip-ft)	176	337	

Table 3-4: Member DCR Table for Design Low Groundwater

Groundwater Condition: El. -28 ft SFCD (GWL)				
Shoring Element	Demand Type	Max ASD Demand	ASD Capacity	Controlling DCR
Soldier Pile (W18x130)	Bending (kip-ft)	304	724	0.42
	Shear (kips)	62	259	0.24
Waler (W27x146)	Bending M3 (kip-ft)	460	1160	0.78
	Bending M2 (kip-ft)	93	243	
	Shear (kips)	178	332	0.54
Strut (W12x136)	Axial (kips)	335	1030	0.92
	Bending (kips)	355	534	
Corner Brace (Pipe 8XS)	Axial (kips)	51	224	0.23
Kicker (Pipe 8XS)	Axial (kips)	171	220	0.78
HSS Kicker Support (HSS16x12x5/8)	Bending M3 (kip-ft)	90	412	0.74
	Bending M2 (kip-ft)	176	337	

Table 3-5: Maximum Deflections of Shoring Wall

Groundwater Condition	Maximum Construction Stage Displacements (in.)	
	Stage 1	Stage 2
El. -7 ft SFCD	0.59	0.90
El. -28 ft SFCD	0.62	0.87

3.2 PLAXIS Soil-Structure Interaction Model

We analyzed the excavation and shoring using PLAXIS 2D Version 2017.01. The goal of our soil-structure interaction analysis is to evaluate the shoring system and confirm results of the SAP2000 analysis.

3.2.1 Methodology

We analyzed the shoring system with a PLAXIS 2D model. Figure 3-14 shows the finite element model geometry, elements and boundary conditions. Figure 3-15 shows the subsurface profile and support element elevations relative to the excavation.

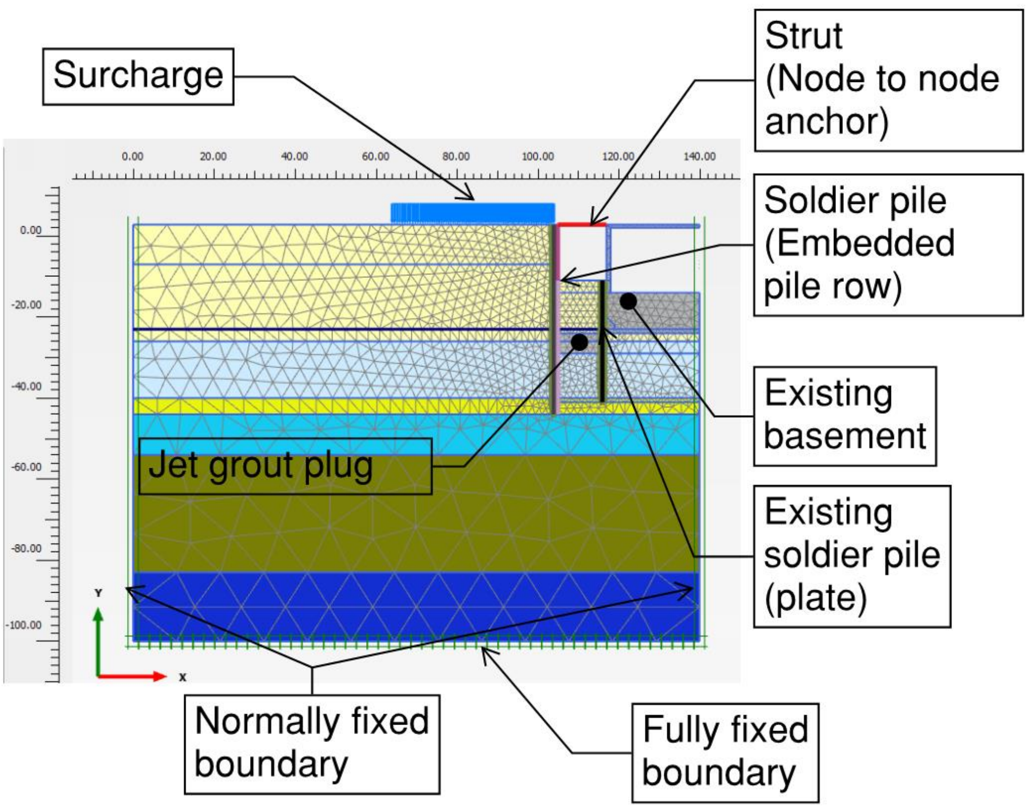


Figure 3-14: PLAXIS Model elements and boundary conditions

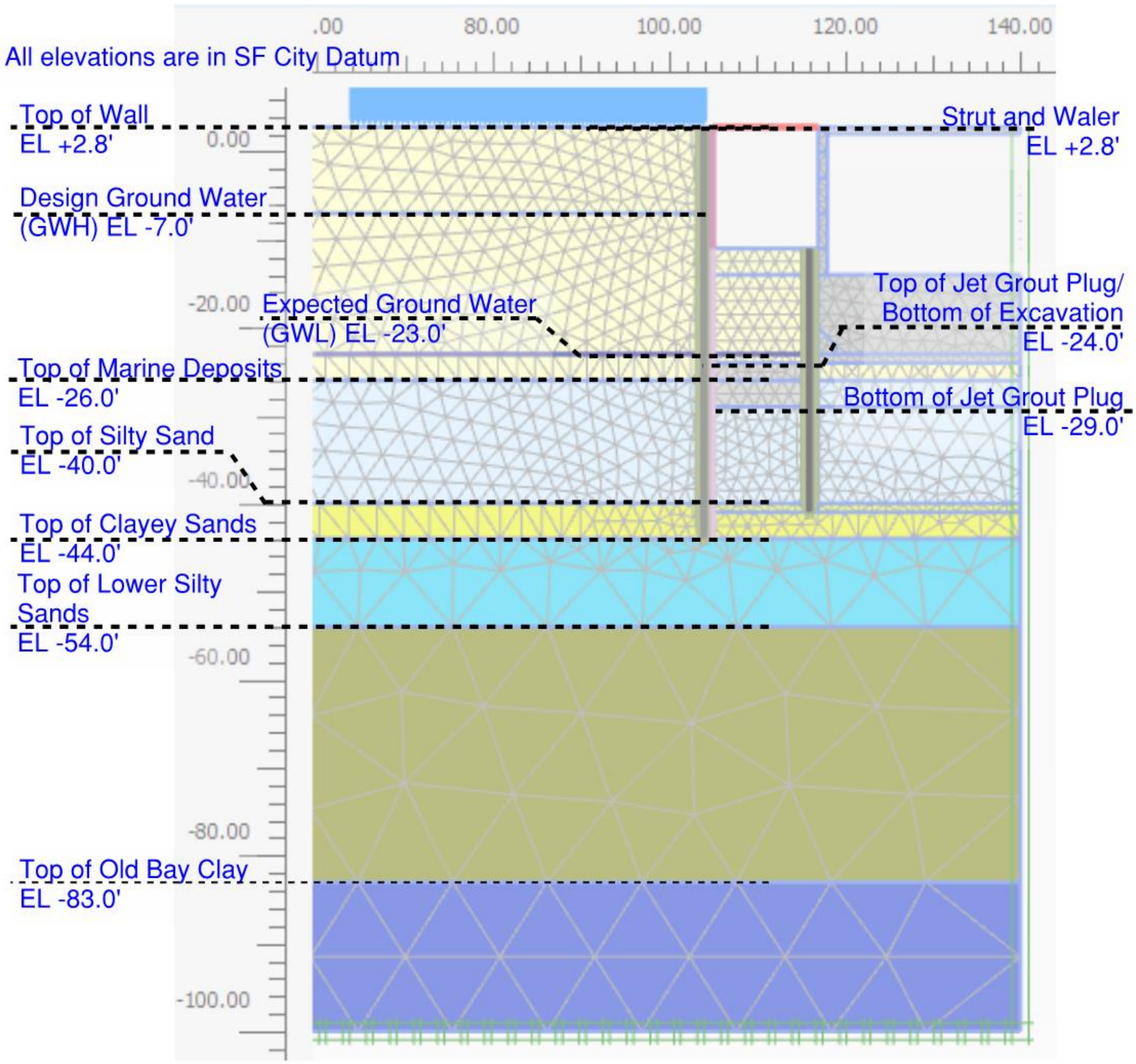


Figure 3-15: PLAXIS model subsurface profile and support element elevations

3.2.2 Soil parameters

We referenced the subsurface profile based on the Idealized Subsurface Profiles from the original geotechnical report. We developed soil parameters based on available soil parameters in collaboration with John Egan. Soils were modeled using the Hardening model with small-strain stiffnesses (HS_{small}). Tables 4-1 through 4-3 present the soil elevations and input parameters.

Table 3-6: PLAXIS generalized subsurface profile

Soil Type	Top Elevation (SFCD ft)	Depth from top of wall (ft)
Fill	2.8	0
Marine Deposits	-26	28.8
Silty Sand	-40	42.8
Clayey Sand	-44	46.8
Silty Sands	-54	56.8
Old Bay Clay	-83	85.8

Table 3-7: PLAXIS applied HS_{small} parameters for cohesive soils

Undrained Soil parameters for cohesive materials				
Identification	**units**	**Marine Deposits**	**Clayey Sand**	**Old Bay Clay**
Drainage Type		Undrained B	Undrained B	Undrained B
γ_{unsat}	lbf/ft³	107	107	115
γ_{sat}	lbf/ft³	107	107	115
E_{50}^{ref}	lbf/ft²	276.0E3	382.0E3	863.0E3
E_{oed}^{ref}	lbf/ft²	138.00E3	191.00E3	432.0E3
E_{ur}^{ref}	lbf/ft²	828.0E3	1.147E6	2.59E6
power (m)	-	1	1	1
S_u^{ref}	lbf/ft²	900	1425	2680
ϕ (phi)	deg	0	0	0
ψ	deg	0	0	2
S_u^{inc}	psf/ft	0	0	0
Z^{ref}	ft	-33	-49	0
$\gamma_{0.7}$	-	0.500E-3	0.500E-3	0.500E-3
G_o^{ref}	lbf/ft²	600.0E3	831.0E3	1.877E6
ν_{ur}	-	0.15	0.15	0.15
p_{ref}	lbf/ft²	2100	2958	6837
K_0^{nc}	-	0.5933	0.5933	0.5933
R_f	-	0.9	0.9	0.9
R_{inter}	-	0.64	0.64	0.64
K_0	-	0.7	0.67	0.7
OCR	-	1.3	1.3	1.700

Table 3-8: PLAXIS applied HS_{small} parameters for cohesionless soils

Soil parameters for **cohesionless** materials				
Identification	**units**	**Fill**	**Silty Sand**	**Lower Silty Sand**
Drainage Type		Drained	Drained	Drained
γ_{unsat}	lb/ft^3	115.0	128.0	130.0
γ_{sat}	lb/ft^3	115.0	128.0	130.0
E_{50}^{ref}	lb/ft^2	278.0E3	1.169E+6	2.151E+6
E_{oed}^{ref}	lb/ft^2	270.0E3	1.000E+6	2.000E+6
E_{ur}^{ref}	lb/ft^2	833.0E3	3.506E+6	6.455E+6
power (m)	-	0.5000	0.5000	0.5000
c_{ref}	lb/ft^2	1.000	1.000	1.000
ϕ (phi)	deg	32.00	36.00	37.00
ψ	deg	0.000	6.000	7.000
$\gamma_{0.7}$	-	1.500E-4	1.500E-4	1.500E-4
G_o^{ref}	lb/ft^2	463.0E3	1.950E+6	2.290E+7
ν_{ur}	-	0.2	0.2	0.2
p_{ref}	lb/ft^2	547.0	2508	3945.0
K_0^{nc}	-	0.4701	0.4100	0.4408
R_f	-	0.9000	0.9000	0.9000
R_{inter}	-	0.6300	0.6100	0.6100
K_0	-	0.4921	0.4260	0.4599
OCR	-	1.100	1.100	1.100

Table 3-9: PLAXIS applied parameters for jet grout plug

Identification	**units**	**Grout plug**
Material Model	Mohr-Coulomb Drained	
γ_{unsat}	lb/ft^3	140
γ_{sat}	lb/ft^3	140
E	lb/ft^2	54.30E6
v'		0.2
G	lb/ft^2	22.62E6
E_{oed}	lb/ft^2	60.33E6
c_{ref}	lb/ft^2	36.00E3
ϕ (phi)	deg	40
ψ_{unsat}	lb/ft^2	32.81E3
Ko		0.3572

3.2.3 Structural parameters

We modeled the soldier piles at 5 ft on center with plate elements and derived structural parameters for a bare W18x130 above EL. -11 SFCD and a W18x130 with 4 ksi concrete below EL -11 SFCD. We modeled the top strut as a node to node anchor and accounted for the spacing of the strut out-of-plane as either 40 ft on center for the initial excavation stage or 20 ft on center for the final excavation stage. Our modeled effective strut stiffness included the waler stiffness in series.

Table 3-10: PLAXIS embedded pile row elements (soldier pile) structural parameters

Parameter	Units	Soldier Pile	
Section	-	W18x130	W18x130 with 4 ksi concrete
spacing	ft	5	5
E	lb/ft^2	4,176.0E+6	4,176.0E+6
A	ft^2	0.2660	0.9228
I	ft^4	0.1186	0.4085
ν	-	0.2	0.2

Table 3-11: PLAXIS Fixed node anchor (Strut) structural parameters

Parameter	Units	Strut Initial Excavation	Strut Final Excavation
Strut Section	-	W12x120	W12x120
Waler Section	-	W27x146	W27x146
L_{strut}	ft	10	10
Spacing	ft	**40**	**20**
Strut EA1	lbf	1,021E+6	1,021E+6
k_{strut}	lbf/ft	102.1E+6	102.1E+6
k_{waler}	lbf/ft	4.104E+6	32.83E+6
k_{eff}	lbf/ft	3.945E+6	24.84E+6
EA_{eff}	lbf	**39.4E+6**	**248.4E+6**

3.2.4 Construction stages

Our PLAXIS 2D analysis evaluated various construction stages with varying groundwater conditions and surcharges. Table 3-12 summarizes our staged-construction analysis steps. Figure 3-16 through Figure 3-18 illustrate the construction stages for the GWH case. Table 3-13 shows the different groundwater and loading scenarios that we used to evaluate demands and deformations.

Table 3-12: PLAXIS Construction stages

<table>
<tr><th rowspan="2">Stage number</th><th rowspan="2">Stage Description</th><th colspan="2">Groundwater Elevation</th><th rowspan="2">Strut spacing (ft)</th><th rowspan="2">Vertical Surcharge</th><th rowspan="2">Excavation Depth (ft.)</th><th rowspan="2">Excavation Elevation (ft. SFCD)</th></tr>
<tr><th>GWH Case (ft. SFCD)</th><th>GWL Case (ft. SFCD)</th></tr>
<tr><td>0</td><td>Initial Phase</td><td rowspan="2">-7</td><td rowspan="9">-23</td><td>-</td><td>-</td><td rowspan="3">-</td><td rowspan="3">2.8</td></tr>
<tr><td>1</td><td>Install soldier pile, struts
Add surcharge</td><td rowspan="5">40</td><td rowspan="8">Traffic
250 psf
OR
Construction
600 psf</td></tr>
<tr><td>2</td><td>Dewater</td><td rowspan="5">-14</td></tr>
<tr><td>3</td><td>Initial Excavation to remove (E) structure and tiebacks</td><td rowspan="5">13.8</td><td rowspan="5">-11</td></tr>
<tr><td>4</td><td>Backfill</td></tr>
<tr><td>5</td><td>Install jet grout plug</td></tr>
<tr><td>6</td><td>Install and preload additional struts</td><td rowspan="3">20</td></tr>
<tr><td>7</td><td>Dewater to B.O.E</td><td rowspan="2">-28</td></tr>
<tr><td>8</td><td>Excavate to B.O.E</td><td>26.8</td><td>-24</td></tr>
</table>

Table 3-13: Groundwater and loading scenarios

<table>
<tr><th>Scenario</th><th>Groundwater case</th><th>Surcharge</th><th>Shoring system</th></tr>
<tr><td>A</td><td>GWH</td><td>Construction Surcharge</td><td rowspan="4">Initial: Top strut at 40 ft spacing,
Final: Top struts at 20 ft spacing</td></tr>
<tr><td>B</td><td>GWH</td><td>Traffic surcharge</td></tr>
<tr><td>C</td><td>GWL</td><td>Construction Surcharge</td></tr>
<tr><td>D</td><td>GWL</td><td>Traffic surcharge</td></tr>
</table>

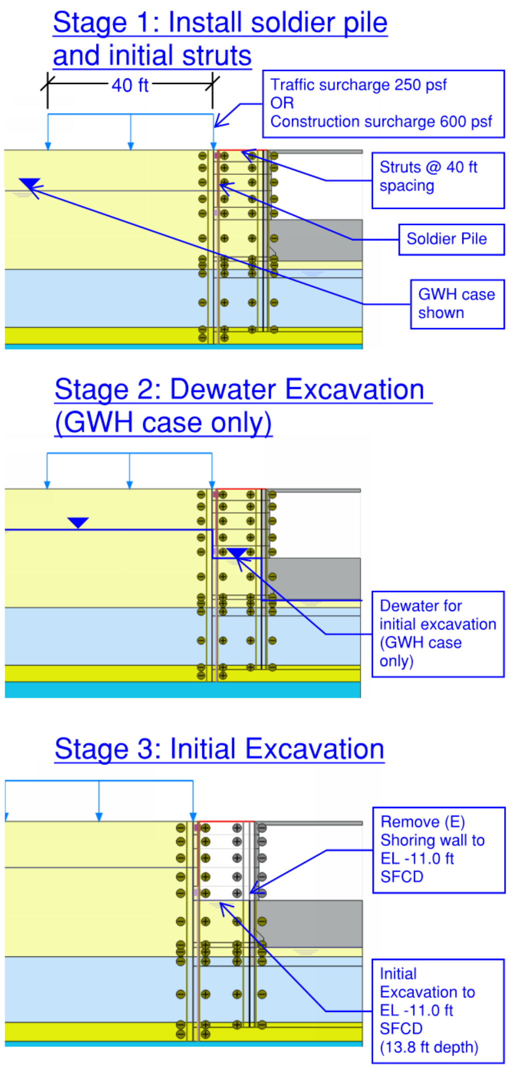


Figure 3-16: PLAXIS construction stages 1-3

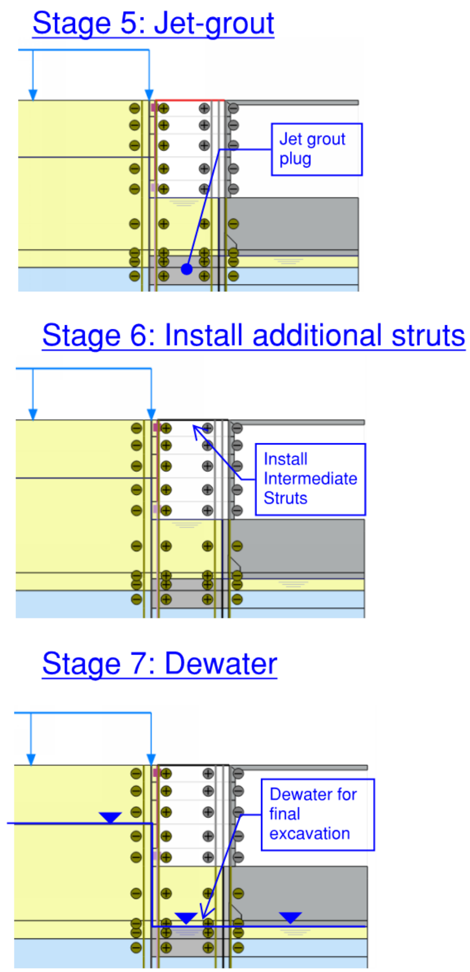


Figure 3-17: PLAXIS construction stages 5-7

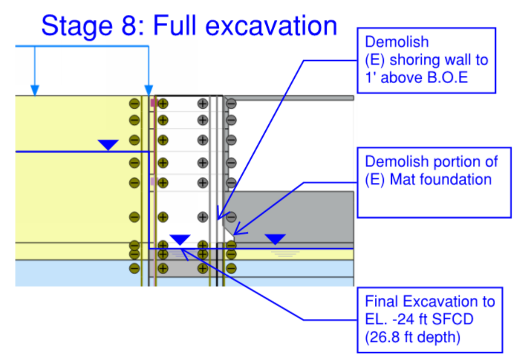


Figure 3-18: PLAXIS final construction stage

3.2.5 PLAXIS soil-structure interaction results

Table 3-14 briefly describes and presents the deflection and **Table 3-15** summarizes the strut axial demands at critical stages for each scenario. **Figure 3-19 through Figure 3-21** show the Plaxis output for Scenario A.

Table 3-14: Description of scenarios and maximum horizontal wall movement

Scenario		**A GWH-Construction**	B GWH-Traffic	**C GWL-Construction**	D GWL-Traffic
Stage		**Maximum horizontal wall movement (+ towards excavation in inches)**			
3	Initial Excavation	**0.80 in.**	0.39 in.	0.48 in.	0.26 in.
8	Final Excavation	**0.90 in.**	0.56 in.	0.59 in.	0.42 in.

Table 3-15: Strut axial demands

Scenario		**A GWH-Surcharge**	B GWH-Traffic	C GWL-Surcharge	D GWL-Traffic
Stage		**Maximum Anchor force (- compression in kip)**			
3	Initial Excavation	-238 kip	-120 kip	-137 kip	-97 kip
8	Final Excavation	-260 kip	-161 kip	-187 kip	-134 kip

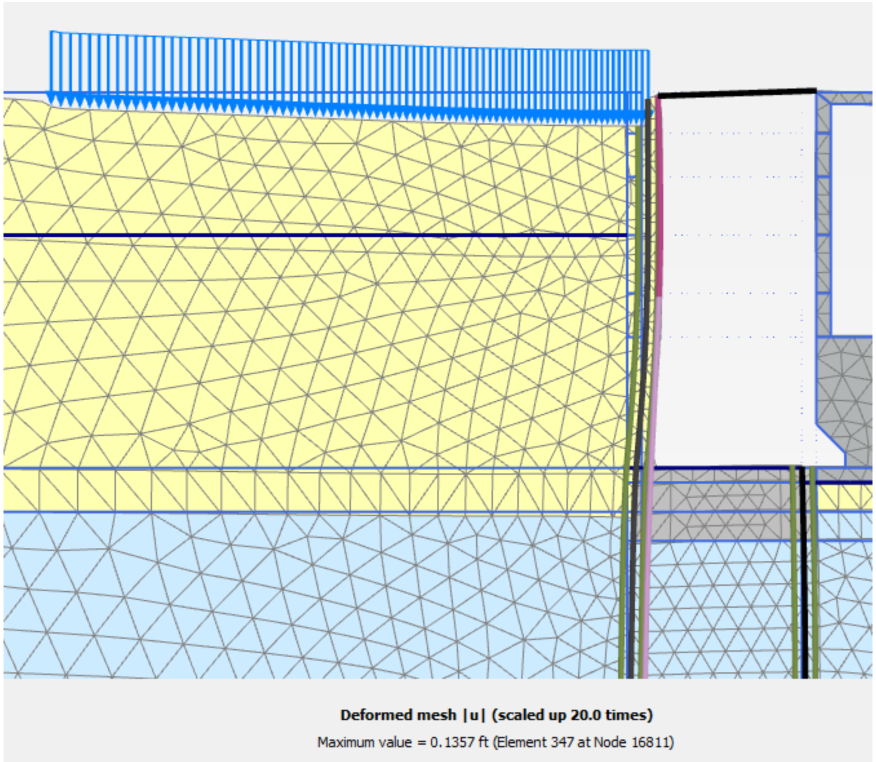


Figure 3-19: Deformed shape (Scenario A Stage 8 Final Excavation, Scale x20)

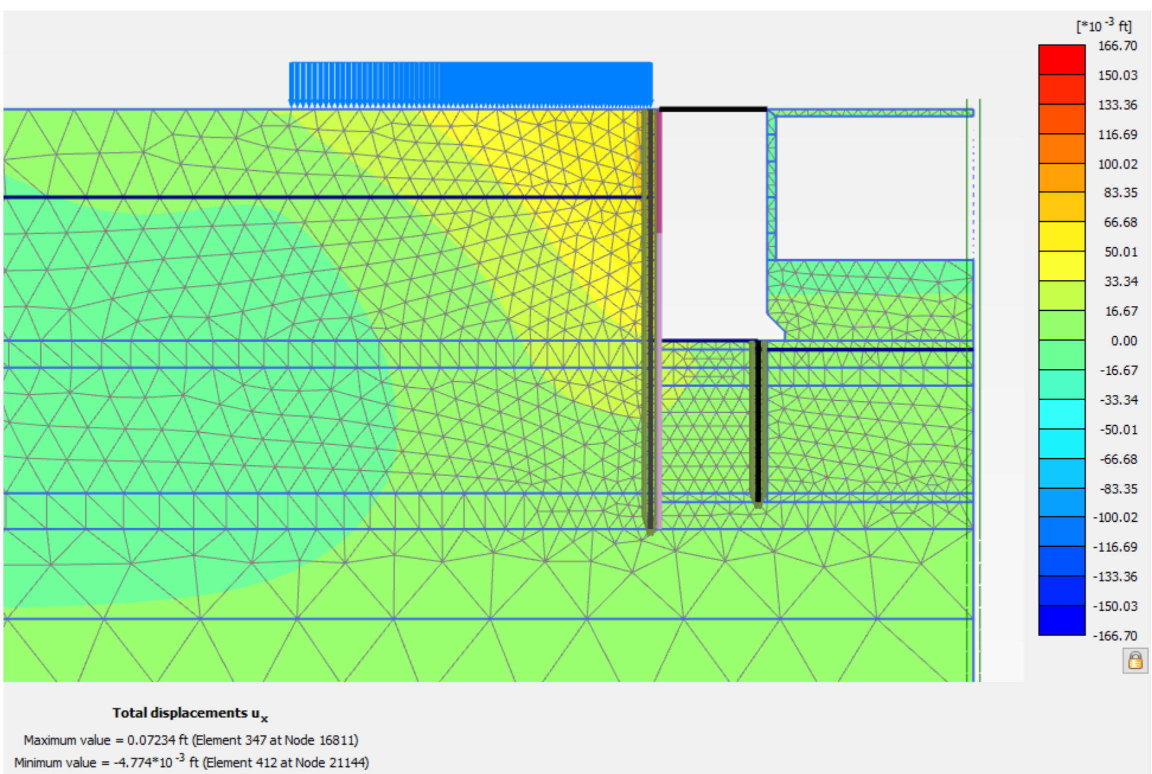


Figure 3-20: Horizontal displacement (Scenario A Stage 8 Final Excavation)

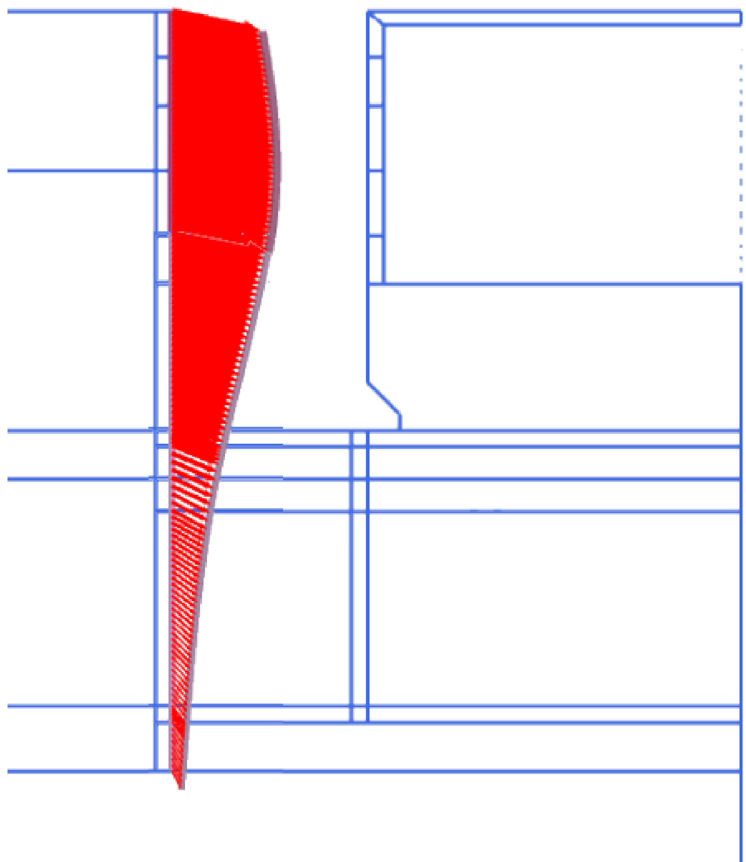


Figure 3-21: Solider pile deformed shape (Case A Step 8 Final Excavation)

3.3 Considerations for Existing Structure

The support of excavation will actively brace against the existing tower structure at the ground level through the HSS struts. We positioned the struts to align with the Level 1 floor slab of the tower to ensure load transfer directly through bearing. The floor is a 12" normal weight concrete slab with a design compressive strength of 5 ksi. We are providing a bearing plate capable of engaging a 24 in. by 12 in. area of the existing structure. We calculated the LRFD capacity of the concrete in bearing using ACI and applied a 1.6 load factor to our strut load per ASCE 7. Table 3-16 below shows the design DCR.

Table 3-16: Existing Tower Concrete Bearing DCR Table

Bearing on Existing Concrete			
Shoring Member	LRFD Demand	LRFD Capacity	Controlling DCR
Strut Bearing Plate	536	795	0.67

3.4 Design of Excavator Platform

The Contractor plans to support an excavator above the shoring wall struts during construction. The platform is designed to span between the shoring struts. In the first stage of excavation to remove the existing tiebacks, the struts will be spaced at a maximum of 41 feet. In the second stage of excavation, the spacing of the struts is reduced to 22 feet. Therefore, the platform design is governed by the first excavation stage. We designed the platform capable to support the proposed CAT 325F excavator and up to 250 psf live load for the governing span.

We created a SAP2000 beam model to evaluate the demands on the platform. We used section designer to model the actual section and applied moving loads which represented the full load of the excavator. We also included a 1.33 impact factor and applied a 1.2 dead load factor and 1.6 live load factor consistent with ASCE 7-10. Figure 3-22 below shows our analysis model.

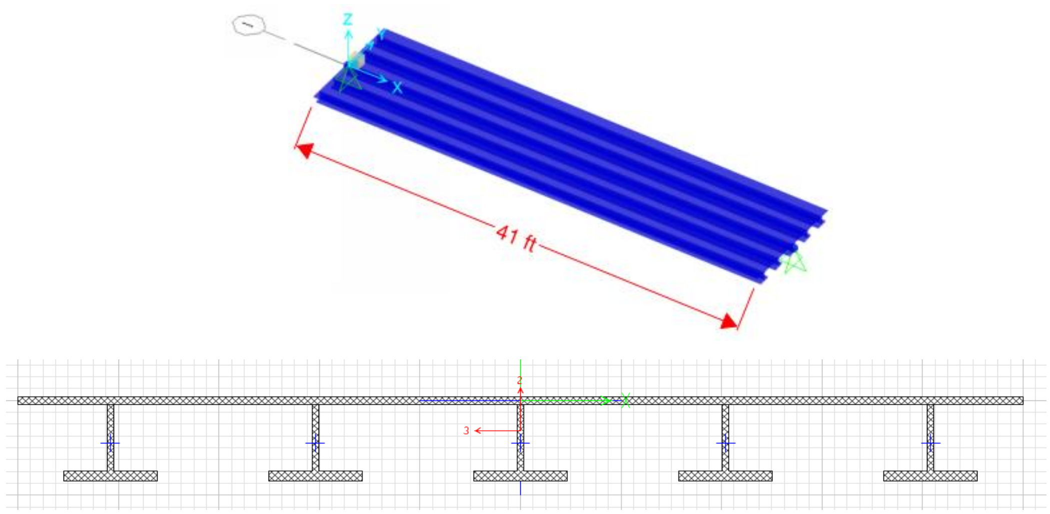


Figure 3-22: SAP2000 Analysis Model

The design section utilizing a 1 in. thick plate meets demands for a 41 ft span and provides deflection performance of L/250 for the dead load and excavator. Table 3-17 below presents the excavator platform DCR summary table for bending and shear demands. Our design calculations are presented in Appendix A.

Table 3-17: Excavator Platform DCR Table

Excavator Platform			
Demand	LRFD Demand	LRFD Capacity	Controlling DCR
Bending (kip-ft)	2175	3227	0.67
Shear (kips)	217	960	0.23

4. ANALYSIS AND DESIGN OF PG&E VAULT BRACING

During construction, two (2) existing PG&E vaults along Fremont Street will remain in place and be supported on the installed shoring system.

4.1 Vault Details

We received vault details and estimated weights from the Civil Engineer. The total weights listed include the vault self-weight and equipment located in the vault. The #5 and #7 vaults have approximate weights of 5.6 kips and 20.5 kips, respectively. Figure 4-1 below shows schematic drawings we received of the PG&E vaults. The boxes are equipped with “pull irons” which were used to lift the vaults into positions during their initial installation.

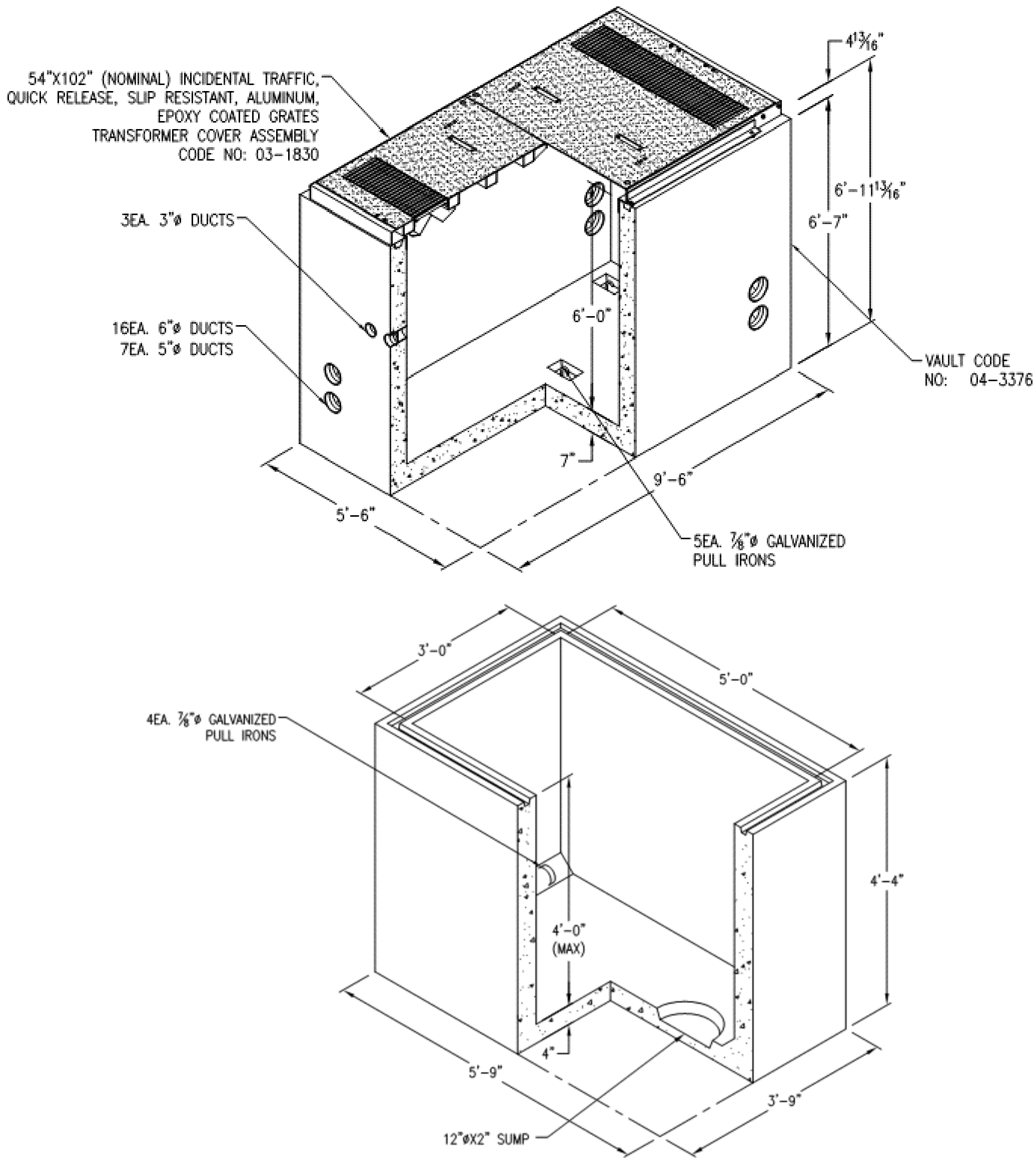


Figure 4-1: PG&E Vault Drawings (#7 top, #5 bottom)

4.2 Design of Supports

We designed the vaults to be supported from the bottom with steel channel members. The channels will be hung by angles positioned at the vault corners. The angles will be stabilized temporarily by epoxy anchors before the channels are installed beneath the vaults. We designed the supports at the bottom at locations near the pull irons that were used to lift and install the vaults initially. This allows a similar loading of the vault in it's intended direction and ensures the vault will not be under excessive stress due to its self-weight.

The drawings show the construction sequence and support framing of the vaults. The angles will then attach to W24 deck beams which are supported on the shoring wall and the existing building. The HSS braces on the W24 beams restrain lateral movement of the vaults during construction. The channel supports are also connected to the existing soldier piles to provide lateral restraint during jet grouting. Our design calculations are presented in Appendix A. Figure 4-2 below shows an overview of the support framing in relation to the shoring system and existing structure.

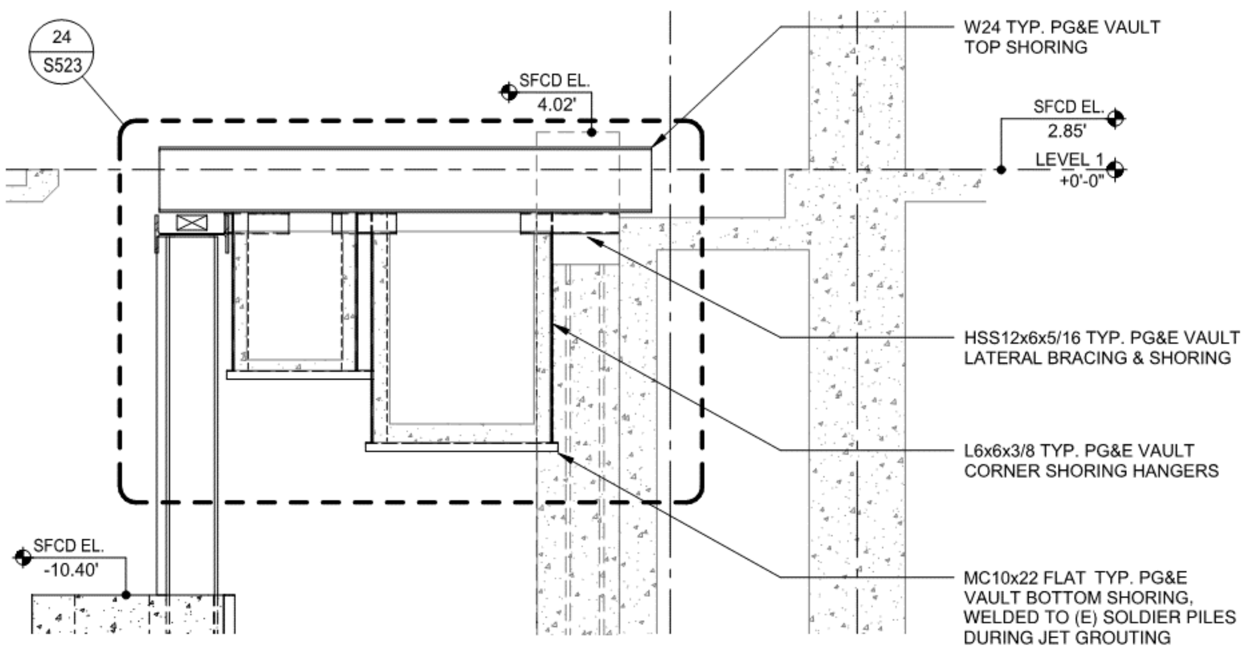


Figure 4-2: Elevation View of PG&E Vaults and Support Framing

5. SUMMARY

We analyzed and designed the support of excavation system for the excavation for the voluntary retrofit. Our shoring plan includes the support of the existing PG&E vault in place during excavation.

We modeled the shoring system in SAP2000 with the loads and support conditions for each stage. Our SAP2000 analysis model included non-linear p-y springs to evaluate the deflection of the shoring system for each construction stage. We considered the different ground water tables based upon field measurements and construction/traffic surcharge cases and applied them to our SAP2000 analysis model. Section 3.1.4 summarizes our SAP2000 output and demand to capacity checks. Appendix A shows the detailed SOE structural calculations. Our SAP2000 analysis shows a maximum deflection 0.90 in. and strut axial compression of 335 kips (ASD).

We also modeled an independent PLAXIS 2D analysis model to verify the deformations from our SAP2000 analysis model. Our PLAXIS model also included a detailed construction sequence that considered the different ground water and surcharge cases. Section 3.2.5 summarizes our PLAXIS output for the SOE. Our PLAXIS analysis results are generally consistent with the SAP2000 analysis results.

Our design calculations of the structural support to the PG&G vaults concluded that the PG&E vaults should be secured against all undesirable movements during all of the planned construction stages.

Note that all elevations shown are for reference and may vary in the field. Contractor shall verify all elevations in the field prior to installation of the SOE system.

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APPENDIX A:
SHORING CALCULATIONS

Structural Design Check Sample Calculations

Simpson Gumpertz & Heger 500 12th St, Suite 270 Oakland, CA 94607	Project 301 Mission Street				Job Ref. 147041.10	
	Section Support of Excavation				Sheet no./rev. 1	
	Calc. by MAN	Date 11/21/2018	Chk'd by JLC	Date 11/21/2018	App'd by SXY	Date 11/21/2018

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.50**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

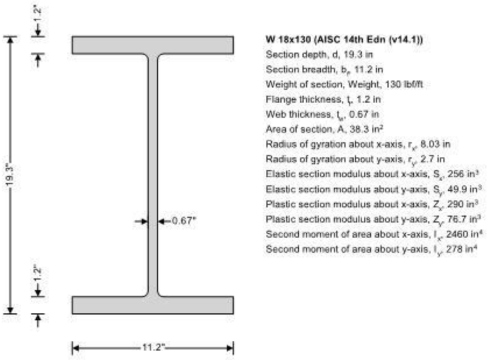
Section type W 18x130 (AISC 14th Edn (v14.1))

ASTM steel designation A992

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **342** kips_ft

Required shear strength - Major axis $V_{r,x}$ = **67** kips

Required compressive strength P_r = **100** kips

Section s1 results summary	**Unit**	**Capacity**	**Maximum**	**Utilization**	**Result**
Shear resistance (x-x)	kips	258.6	67	0.259	PASS
Bending resistance (x-x)	kips_ft	723.6	342	0.473	PASS
Compression resistance	kips	978.9	100	0.102	PASS
Combined forces				0.524	PASS

Restraint spacing

Major axis lateral restraint L_x = **15** ft

Minor axis lateral restraint L_y = **0** ft

Torsional restraint L_z = **15** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f)$ = **4.67**

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **24.03**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Classification of flanges in uniform compression - Table B4.1a (case 1)

Width to thickness ratio $b_f / (2 \times t_f)$ = **4.67**

Limiting ratio for non-compact section $\lambda_{rfc} = 0.56 \times \sqrt{[E / F_y]}$ = **13.49** Nonslender

Classification of web in uniform compression - Table B4.1a (case 5)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **24.03**

Limiting ratio for non-compact section $\lambda_{rwc} = 1.49 \times \sqrt{[E / F_y]}$ = **35.88** Nonslender

Section is nonslender in compression

Design of members for compression - Chapter E

Required compressive strength P_r = **100** kips

Slenderness limitations and effective length - Section E2

Unbraced length $L_{b,x}$ = **15** ft

Effective length factor K_x = **1.00**

Column slenderness $\lambda_x = K_x \times L_{b,x} / r_x$ = **22.416**

Major axis column slenderness ratio does not exceed recommended limit of 200

Flexural buckling of members without slender elements - Section E3

Elastic critical buckling stress - eq E3-4 $F_{e,x} = \pi^2 \times E / (K_x \times L_{b,x} / r_x)^2$ = **569.6** ksi

Flexural buckling stress - eq E3-2 $F_{cr,x} = [0.658^{F_y / F_{e,x}}] \times F_y$ = **48.2** ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,x} = F_{cr,x} \times A$ = **1845.9** kips

Torsional and torsional-flexural buckling of members without slender elements - Section E4

Unbraced length $L_{b,z}$ = **15** ft

Effective length factor K_z = **1.00**

Flexural-torsional elastic buckling stress - eq E4-4 $F_e = [\pi^2 \times E \times C_w / (K_z \times L_{b,z})^2 + G \times J] / (I_x + I_y)$ = **132.3** ksi

Flexural-torsional buckling stress - eq E3-2 $F_{cr} = [0.658^{F_y / F_e}] \times F_y$ = **42.7** ksi

Nominal compressive strength for torsional and flexural-torsional buckling - eq E4-1

$P_{n,ftb} = F_{cr} \times A$ = **1634.8** kips

Allowable compressive strength - E1

Nominal compressive strength $P_n = \min(P_{n,fb,x}, P_{n,ftb})$ = **1634.8** kips

Allowable compressive strength $P_c = P_n / \Omega_c$ = **978.9** kips

P_r / P_c = **0.102**

PASS - Nominal compressive strength exceeds required compressive strength

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **67** kips

Web area $A_w = d \times t_w$ = **12.931** in^2

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **387.9** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **258.6** kips

$V_{r,x} / V_{c,x}$ = **0.259**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **342** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **1208.3** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **1208.3** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **723.6** kips_ft

$M_{r,x} / M_{c,x}$ = **0.473**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b $P_r / (2 \times P_c) + M_{r,x} / M_{c,x}$ = **0.524**

PASS - Combined flexure and axial force is within acceptable limits

Simpson Gumpertz & Heger 500 12th St, Suite 270 Oakland, CA 94607	Project 301 Mission Street				Job Ref. 147041.10	
	Section Support of Excavation - Waler design				Sheet no./rev. 1	
	Calc. by MAN	Date 11/21/2018	Chk'd by JLC	Date 11/21/2018	App'd by SXY	Date 11/21/2018

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.50**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

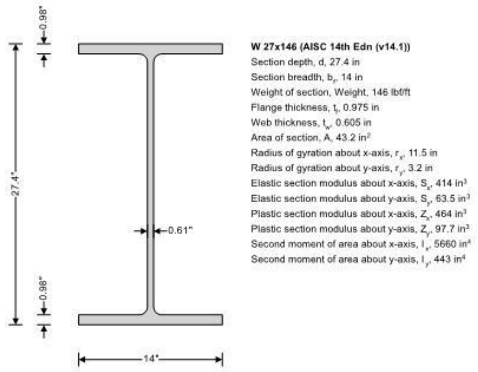
Section type W 27x146 (AISC 14th Edn (v14.1))

ASTM steel designation A992

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **451** kips_ft

Required flexural strength - Minor axis $M_{r,y}$ = **96** kips_ft

Required shear strength - Major axis $V_{r,x}$ = **177** kips

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Shear resistance (x-x)	kips	331.5	177	0.534	PASS
Bending resistance (x-x)	kips_ft	1157.7	451	0.390	PASS
Bending resistance (y-y)	kips_ft	243.8	96	0.394	PASS
Combined forces				0.783	PASS

Restraint spacing

Major axis lateral restraint L_x = **5** ft

Minor axis lateral restraint L_y = **5** ft

Torsional restraint L_z = **5** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f)$ = **7.18**

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **39.47**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **177** kips

Web area $A_w = d \times t_w$ = **16.577** in^2

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **497.3** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **331.5** kips

$V_{r,x} / V_{c,x}$ = **0.534**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **451** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **1933.3** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{y\_s1}$ = **5** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **11.303** ft

Distance between flange centroids h_o = **26.4** in

c = **1**

r_{ts} = **3.76** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2))}$ = **33.343** ft

L_b <= L_p - Lateral-torsional buckling does not apply

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **1933.3** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **1157.7** kips_ft

$M_{r,x} / M_{c,x}$ = **0.390**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,y}$ = **96** kips_ft

Yielding - Section F6.1

Nominal flexural strength for yielding - eq F6-1 $M_{n,yld,y} = M_{p,y} = \min(F_y \times Z_y, 1.6 \times F_y \times S_y)$ = **407.1** kips_ft

Compression flange local buckling - Section F6.2

$\lambda = b_f / (2 \times t_f)$ = **7.179**

Nominal flexural strength for compression flange local buckling - eq F6-2

$M_{n,flb,y} = M_{p,y}$ = **407.1** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,y} = \min(M_{n,yld,y}, M_{n,flb,y})$ = **407.1** kips_ft

Allowable flexural strength $M_{c,y} = M_{n,y} / \Omega_b$ = **243.8** kips_ft

$M_{r,y} / M_{c,y}$ = **0.394**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b $M_{r,x} / M_{c,x} + M_{r,y} / M_{c,y}$ = **0.783**

PASS - Combined flexure and axial force is within acceptable limits

Simpson Gumpertz & Heger 500 12th St, Suite 270 Oakland, CA 94607	Project 301 Mission Street				Job Ref. 147041.10	
	Section Support of Excavation - Strut design				Sheet no./rev. 1	
	Calc. by MAN	Date 11/21/2018	Chk'd by JLC	Date 11/21/2018	App'd by SXY	Date 11/21/2018

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.50**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

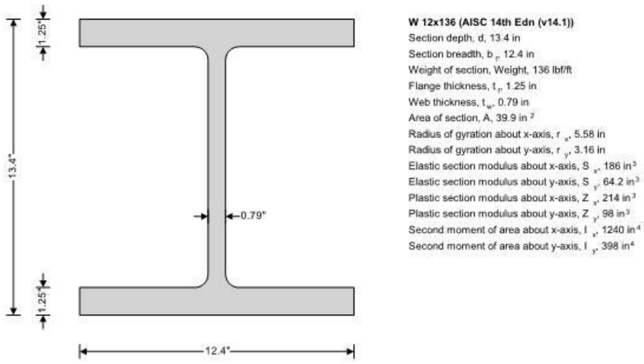
Section type W 12x136 (AISC 14th Edn (v14.1))

ASTM steel designation A992

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **342** kips_ft

Required compressive strength P_r = **323** kips

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending resistance (x-x)	kips_ft	530.6	342	0.645	PASS
Compression resistance	kips	1026.3	323	0.315	PASS
Combined forces				0.888	PASS

Restraint spacing

Major axis lateral restraint L_x = **12** ft

Minor axis lateral restraint L_y = **12** ft

Torsional restraint L_z = **12** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f)$ = **4.96**

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **12.28**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Classification of flanges in uniform compression - Table B4.1a (case 1)

Width to thickness ratio $b_f / (2 \times t_f)$ = **4.96**

Limiting ratio for non-compact section $\lambda_{rfc} = 0.56 \times \sqrt{[E / F_y]}$ = **13.49** Nonslender

Classification of web in uniform compression - Table B4.1a (case 5)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **12.28**

Limiting ratio for non-compact section $\lambda_{rwc} = 1.49 \times \sqrt{[E / F_y]}$ = **35.88** Nonslender

Section is nonslender in compression

Design of members for compression - Chapter E

Required compressive strength P_r = **323** kips

Slenderness limitations and effective length - Section E2

Unbraced length $L_{b,x}$ = **12** ft

Effective length factor K_x = **1.00**

Column slenderness $\lambda_x = K_x \times L_{b,x} / r_x$ = **25.806**

Major axis column slenderness ratio does not exceed recommended limit of 200

Slenderness limitations and effective length - Section E2

Unbraced length $L_{b,y}$ = **12** ft

Effective length factor K_y = **1.00**

Column slenderness $\lambda_y = K_y \times L_{b,y} / r_y$ = **45.570**

Minor axis column slenderness ratio does not exceed recommended limit of 200

Flexural buckling of members without slender elements - Section E3

Elastic critical buckling stress - eq E3-4 $F_{e,x} = \pi^2 \times E / (K_x \times L_{b,x} / r_x)^2$ = **429.8** ksi

Flexural buckling stress - eq E3-2 $F_{cr,x} = [0.658^{F_y / F_{e,x}}] \times F_y$ = **47.6** ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,x} = F_{cr,x} \times A$ = **1900.2** kips

Elastic critical buckling stress - eq E3-4 $F_{e,y} = \pi^2 \times E / (K_y \times L_{b,y} / r_y)^2$ = **137.8** ksi

Flexural buckling stress - eq E3-2 $F_{cr,y} = [0.658^{F_y / F_{e,y}}] \times F_y$ = **43** ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,y} = F_{cr,y} \times A$ = **1714** kips

Torsional and torsional-flexural buckling of members without slender elements - Section E4

Unbraced length $L_{b,z}$ = **12** ft

Effective length factor K_z = **1.00**

Flexural-torsional elastic buckling stress - eq E4-4 $F_e = [\pi^2 \times E \times C_w / (K_z \times L_{b,z})^2 + G \times J] / (I_x + I_y)$ = **249.8** ksi

Flexural-torsional buckling stress - eq E3-2 $F_{cr} = [0.658^{F_y / F_e}] \times F_y$ = **46** ksi

Nominal compressive strength for torsional and flexural-torsional buckling - eq E4-1

$P_{n,ftb} = F_{cr} \times A$ = **1834.7** kips

Allowable compressive strength - E1

Nominal compressive strength $P_n = \min(P_{n,fb,x}, P_{n,fb,y}, P_{n,ftb})$ = **1714** kips

Allowable compressive strength $P_c = P_n / \Omega_c$ = **1026.3** kips

P_r / P_c = **0.315**

PASS - Nominal compressive strength exceeds required compressive strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **342** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **891.7** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{y\_s1}$ = **12** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **11.162** ft

Distance between flange centroids h_o = **12.2** in

c = **1**

r_{ts} = **3.61** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)})}$ = **63.166** ft

LTB modification factor C_b = **1.000**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **886** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **886** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **530.6** kips_ft

$M_{r,x} / M_{c,x}$ = **0.645**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1a $P_r / P_c + 8 / 9 \times (M_{r,x} / M_{c,x})$ = **0.888**

PASS - Combined flexure and axial force is within acceptable limits

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.67**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

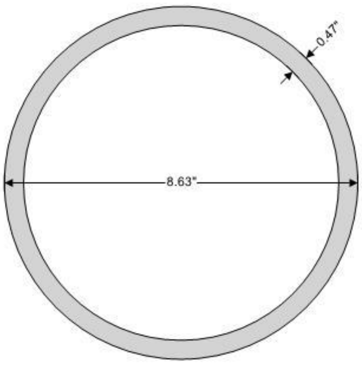
Section type Pipe XS x8 (AISC 14th Edn (v14.1))

ASTM steel designation A53 Gr.B

Steel yield stress F_y = **35** ksi

Steel tensile stress F_u = **60** ksi

Modulus of elasticity E = **29000** ksi



Pipe XS x8 (AISC 14th Edn (v14.1))
Diameter, D_o, 8.63 in
Weight of section, Weight, 43.4 lbf/ft
Section thickness, t, 0.465 in
Area of section, A, 11.9 in^2
Radius of gyration about x-axis, r_x, 2.89 in
Radius of gyration about y-axis, r_y, 2.89 in
Elastic section modulus about x-axis, S_x, 23.1 in^3
Elastic section modulus about y-axis, S_y, 23.1 in^3
Plastic section modulus about x-axis, Z_x, 31 in^3
Plastic section modulus about y-axis, Z_y, 31 in^3
Second moment of area about x-axis, I_x, 100 in^4
Second moment of area about y-axis, I_y, 100 in^4

Analysis results

Required compressive strength P_r = **50** kips

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Compression resistance	kips	224.1	50	0.223	PASS

Restraint spacing

Major axis lateral restraint L_x = **11** ft

Minor axis lateral restraint L_y = **11** ft

Torsional restraint L_z = **11** ft

Classification of sections for local buckling - Section B4

Classification of section in flexure - Table B4.1b (case 20)

Width to thickness ratio D_o / t = **18.56**

Limiting ratio for compact section $\lambda_{pff} = 0.07 \times E / F_y$ = **58.00**

Limiting ratio for non-compact section $\lambda_{rff} = 0.31 \times E / F_y$ = **256.86** Compact

Section is compact in flexure

Classification of section in uniform compression - Table B4.1a (case 9)

Width to thickness ratio D_o / t = **18.56**

Limiting ratio for non-compact section $\lambda_{rfc} = 0.11 \times E / F_y$ = **91.14** Nonslender

Section is nonslender in compression

Design of members for compression - Chapter E

Required compressive strength P_r = **50** kips

Slenderness limitations and effective length - Section E2

Unbraced length $L_{b,x}$ = **11** ft

Effective length factor K_x = **1.00**

Column slenderness $\lambda_x = K_x \times L_{b,x} / r_x$ = **45.675**

Major axis column slenderness ratio does not exceed recommended limit of 200

Slenderness limitations and effective length - Section E2

Unbraced length $L_{b,y}$ = **11** ft

Effective length factor K_y = **1.00**

Column slenderness $\lambda_y = K_y \times L_{b,y} / r_y$ = **45.675**

Minor axis column slenderness ratio does not exceed recommended limit of 200

Flexural buckling of members without slender elements - Section E3

Elastic critical buckling stress - eq E3-4 $F_{e,x} = \pi^2 \times E / (K_x \times L_{b,x} / r_x)^2$ = **137.2** ksi

Flexural buckling stress - eq E3-2 $F_{cr,x} = [0.658^{F_y / F_{e,x}}] \times F_y$ = **31.5** ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,x} = F_{cr,x} \times A$ = **374.3** kips

Elastic critical buckling stress - eq E3-4 $F_{e,y} = \pi^2 \times E / (K_y \times L_{b,y} / r_y)^2$ = **137.2** ksi

Flexural buckling stress - eq E3-2 $F_{cr,y} = [0.658^{F_y / F_{e,y}}] \times F_y$ = **31.5** ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,y} = F_{cr,y} \times A$ = **374.3** kips

Allowable compressive strength - E1

Nominal compressive strength $P_n = \min(P_{n,fb,x}, P_{n,fb,y})$ = **374.3** kips

Allowable compressive strength $P_c = P_n / \Omega_c$ = **224.1** kips

P_r / P_c = **0.223**

PASS - Nominal compressive strength exceeds required compressive strength

Simpson Gumpertz & Heger 500 12th St, Suite 270 Oakland, CA 94607	Project 301 Mission Street				Job Ref. 147041.10	
	Section Support of Excavation - Kicker brace				Sheet no./rev. 1	
	Calc. by MAN	Date 11/21/2018	Chk'd by JLC	Date 11/21/2018	App'd by SXY	Date 11/21/2018

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.67**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

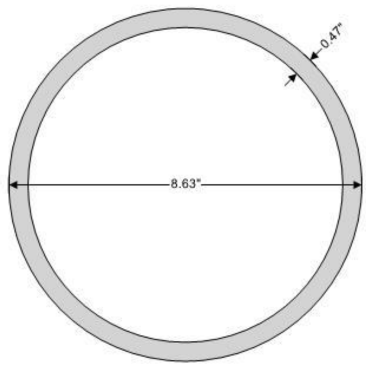
Section type Pipe XS x8 (AISC 14th Edn (v14.1))

ASTM steel designation A53 Gr.B

Steel yield stress F_y = **35** ksi

Steel tensile stress F_u = **60** ksi

Modulus of elasticity E = **29000** ksi



Pipe XS x8 (AISC 14th Edn (v14.1))
Diameter, D_o, 8.63 in
Weight of section, Weight, 43.4 lbf/ft
Section thickness, t, 0.465 in
Area of section, A, 11.9 in^2
Radius of gyration about x-axis, r_x, 2.89 in
Radius of gyration about y-axis, r_y, 2.89 in
Elastic section modulus about x-axis, S_x, 23.1 in^3
Elastic section modulus about y-axis, S_y, 23.1 in^3
Plastic section modulus about x-axis, Z_x, 31 in^3
Plastic section modulus about y-axis, Z_y, 31 in^3
Second moment of area about x-axis, I_x, 100 in^4
Second moment of area about y-axis, I_y, 100 in^4

Analysis results

Required compressive strength P_r = **172** kips

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Compression resistance	kips	219.6	172	0.783	PASS

Restraint spacing

Major axis lateral restraint L_x = **12** ft

Minor axis lateral restraint L_y = **12** ft

Torsional restraint L_z = **12** ft

Classification of sections for local buckling - Section B4

Classification of section in flexure - Table B4.1b (case 20)

Width to thickness ratio — $D_o / t = \mathbf{18.56}$

Limiting ratio for compact section — $\lambda_{pff} = 0.07 \times E / F_y = \mathbf{58.00}$

Limiting ratio for non-compact section — $\lambda_{rff} = 0.31 \times E / F_y = \mathbf{256.86}$ — Compact

Section is compact in flexure

Classification of section in uniform compression - Table B4.1a (case 9)

Width to thickness ratio — $D_o / t = \mathbf{18.56}$

Limiting ratio for non-compact section — $\lambda_{rfc} = 0.11 \times E / F_y = \mathbf{91.14}$ — Nonslender

Section is nonslender in compression

Design of members for compression - Chapter E

Required compressive strength — $P_r = \mathbf{172}$ kips

Slenderness limitations and effective length - Section E2

Unbraced length — $L_{b,x} = \mathbf{12}$ ft

Effective length factor — $K_x = \mathbf{1.00}$

Column slenderness — $\lambda_x = K_x \times L_{b,x} / r_x = \mathbf{49.827}$

Major axis column slenderness ratio does not exceed recommended limit of 200

Slenderness limitations and effective length - Section E2

Unbraced length — $L_{b,y} = \mathbf{12}$ ft

Effective length factor — $K_y = \mathbf{1.00}$

Column slenderness — $\lambda_y = K_y \times L_{b,y} / r_y = \mathbf{49.827}$

Minor axis column slenderness ratio does not exceed recommended limit of 200

Flexural buckling of members without slender elements - Section E3

Elastic critical buckling stress - eq E3-4 — $F_{e,x} = \pi^2 \times E / (K_x \times L_{b,x} / r_x)^2 = \mathbf{115.3}$ ksi

Flexural buckling stress - eq E3-2 — $F_{cr,x} = [0.658^{F_y / F_{e,x}}] \times F_y = \mathbf{30.8}$ ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,x} = F_{cr,x} \times A = \mathbf{366.8}$ kips

Elastic critical buckling stress - eq E3-4 — $F_{e,y} = \pi^2 \times E / (K_y \times L_{b,y} / r_y)^2 = \mathbf{115.3}$ ksi

Flexural buckling stress - eq E3-2 — $F_{cr,y} = [0.658^{F_y / F_{e,y}}] \times F_y = \mathbf{30.8}$ ksi

Nominal compressive strength for flexural buckling - eq E3-1

$P_{n,fb,y} = F_{cr,y} \times A = \mathbf{366.8}$ kips

Allowable compressive strength - E1

Nominal compressive strength — $P_n = \min(P_{n,fb,x}, P_{n,fb,y}) = \mathbf{366.8}$ kips

Allowable compressive strength — $P_c = P_n / \Omega_c = \mathbf{219.6}$ kips

$P_r / P_c = \mathbf{0.783}$

PASS - Nominal compressive strength exceeds required compressive strength

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.3.01

Safety factors

Shear Ω_v = **1.67**

Flexure Ω_b = **1.67**

Tensile yielding $\Omega_{t,y}$ = **1.67**

Tensile rupture $\Omega_{t,r}$ = **2.00**

Compression Ω_c = **1.67**

Design section 1

Section details

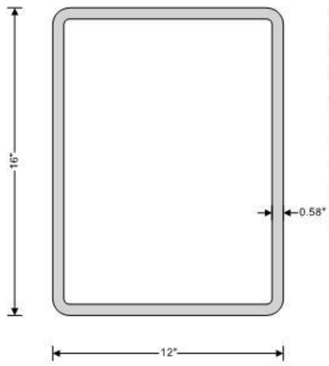
Section type HSS 16x12x5/8 (AISC 14th Edn (v14.1))

ASTM steel designation A500 Gr.C

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **62** ksi

Modulus of elasticity E = **29000** ksi



HSS 16x12x5/8 (AISC 14th Edn (v14.1))
Section depth, d, 16 in
Section breadth, b_f, 12 in
Weight of section, Weight, 110.4 lbf/ft
Section thickness, t, 0.581 in
Area of section, A, 30.3 in^2
Radius of gyration about x-axis, r_x, 6 in
Radius of gyration about y-axis, r_y, 4.8 in
Elastic section modulus about x-axis, S_x, 136 in^3
Elastic section modulus about y-axis, S_y, 117 in^3
Plastic section modulus about x-axis, Z_x, 165 in^3
Plastic section modulus about y-axis, Z_y, 135 in^3
Second moment of area about x-axis, I_x, 1090 in^4
Second moment of area about y-axis, I_y, 700 in^4

Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **90** kips_ft

Required flexural strength - Minor axis $M_{r,y}$ = **176** kips_ft

Required shear strength - Major axis $V_{r,x}$ = **98** kips

Required shear strength - Minor axis $V_{r,y}$ = **139** kips

Section s1 results summary	**Unit**	**Capacity**	**Maximum**	**Utilization**	**Result**
Shear resistance (x-x)	kips	297.6	98	0.329	PASS
Shear resistance (y-y)	kips	214.1	139	0.649	PASS
Bending resistance (x-x)	kips_ft	411.7	90	0.219	PASS
Bending resistance (y-y)	kips_ft	336.8	176	0.523	PASS
Combined forces				0.741	PASS

Restraint spacing

Major axis lateral restraint L_x = **12** ft

Minor axis lateral restraint L_y = **12** ft

Torsional restraint L_z = **12** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 17)

Width to thickness ratio max(d - 3 × t, b_f - 3 × t) / t = **24.54**

Limiting ratio for compact section λ_{pff} = 1.12 × √[E / F_y] = **26.97**

Limiting ratio for non-compact section λ_{rff} = 1.40 × √[E / F_y] = **33.72** Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio max(d - 3 × t, b_f - 3 × t) / t = **24.54**

Limiting ratio for compact section λ_{pwf} = 2.42 × √[E / F_y] = **58.28**

Limiting ratio for non-compact section λ_{rwf} = 5.70 × √[E / F_y] = **137.27** Compact

Section is compact in flexure

Classification of flanges in uniform compression - Table B4.1a (case 6)

Width to thickness ratio max(d - 3 × t, b_f - 3 × t) / t = **24.54**

Limiting ratio for non-compact section λ_{rfc} = 1.40 × √[E / F_y] = **33.72** Nonslender

Classification of web in uniform compression - Table B4.1a (case 6)

Width to thickness ratio max(d - 3 × t, b_f - 3 × t) / t = **24.54**

Limiting ratio for non-compact section λ_{rwc} = 1.49 × √[E / F_y] = **35.88** Nonslender

Section is nonslender in compression

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **98** kips

Web area A_w = 2 × (d - 3 × t) × t = **16.567** in^2

Web plate buckling coefficient k_v = **5**

(d - 3 × t) / t <= 1.10 × √(k_v × E / F_y)

Web shear coefficient - eq G2-3 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x}$ = 0.6 × F_y × A_w × C_v = **497** kips

Safety factor Ω_v = 1.67

Allowable shear strength $V_{c,x}$ = $V_{n,x}$ / Ω_v = **297.6** kips

$V_{r,x}$ / $V_{c,x}$ = **0.329**

PASS - Allowable shear strength exceeds required shear strength

Required shear strength $V_{r,y}$ = **139** kips

Web area A_w = 2 × (b_f - 3 × t) × t = **11.919** in^2

Web plate buckling coefficient k_v = **5**

(b_f - 3 × t) / t <= 1.10 × √(k_v × E / F_y)

Web shear coefficient - eq G2-3 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,y}$ = 0.6 × F_y × A_w × C_v = **357.6** kips

Safety factor Ω_v = 1.67

Allowable shear strength $V_{c,y}$ = $V_{n,y}$ / Ω_v = **214.1** kips

$V_{r,y}$ / $V_{c,y}$ = **0.649**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **90** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **687.5** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **687.5** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **411.7** kips_ft

$M_{r,x} / M_{c,x}$ = **0.219**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,y}$ = **176** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 $M_{n,yld,y} = M_{p,y} = F_y \times Z_y$ = **562.5** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,y} = M_{n,yld,y}$ = **562.5** kips_ft

Allowable flexural strength $M_{c,y} = M_{n,y} / \Omega_b$ = **336.8** kips_ft

$M_{r,y} / M_{c,y}$ = **0.523**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b $M_{r,x} / M_{c,x} + M_{r,y} / M_{c,y}$ = **0.741**

PASS - Combined flexure and axial force is within acceptable limits

PG&E Vault Calculations

SIMPSON GUMPERTZ & HEGER Engineering of Structures and Building Enclosures	CLIENT: Millennium SUBJECT: PG&E Vault Calculations	PROJECT NO: 147041.10 DATE: 12/3/2018 BY: JLC CHECKED BY: MAN

Title: Precast Vault Calculations

References: *AISC 360*

Description: *Check vault anchor and channel supports.*

Concrete strength $f_c := 4.5ksi$

Concrete unit weight $\gamma := 150pcf$

Rebar yield strength $f_y := 60ksi$

STD-3660 Utility Vault Parameters (#5 Box) - Check bottom slab span for cracking

Slab thickness $t_{slab} := 4in$

Slab moment of inertia $I_{slab} := \dfrac{12in \cdot t_{slab}^{\;3}}{12} = 64 \cdot in^4$

Wall thickness $t_{wall} := 4.5in$

Length of slab span $L_{slab} := (3ft + 9in) - 2 \cdot t_{wall} = 3\ ft$

Estimated weight $W_5 := 5560lbf$

Cracking moment of slab $M_{cr} := \dfrac{\left(7.5 \cdot \sqrt{f_c \cdot psi}\right) \cdot I_{slab}}{0.5 \cdot t_{slab}} = 16.1 \cdot kip \cdot in$

Distibuted line load self-weight $w_{slab} := (12in) \cdot t_{slab} \cdot \gamma = 50 \cdot plf$

Cable weight in box $W_{cable} := 114lbf$

Conservative max moment based on simple span $M_u := 1.4 \cdot \left[\dfrac{\left(w_{slab}\right) \cdot L_{slab}^{\;2}}{8} + \dfrac{W_{cable} \cdot L_{slab}}{4}\right] = 2.381 \cdot kip \cdot in$

$\boxed{DCR\left(0.65 \cdot M_{cr}, M_u\right) = (0.228\quad \text{"OK"})}$

4686 Incidental Enclosure Vault Parameters (#7 Box) - Check bottom slab span for cracking

Slab thickness $t_{slab.7} := 7in$

Slab moment of inertia $I_{slab.7} := \frac{12in \cdot t_{slab.7}^3}{12} = 343 \cdot in^4$

Wall thickness $t_{wall.7} := 6in$

Length of slab span $L_{slab.7} := (5ft + 6in) - 2 \cdot t_{wall.7} = 4.5\,ft$

Estimated weight $W_7 := 19330lbf$

Cracking moment of slab $M_{cr.7} := \frac{\left(7.5 \cdot \sqrt{f_c \cdot psi}\right) \cdot I_{slab.7}}{0.5 \cdot t_{slab.7}} = 49.31 \cdot kip \cdot in$

Distibuted line load self-weight $w_{slab.7} := (12in) \cdot t_{slab.7} \cdot \gamma = 87.5 \cdot plf$

Cable weight in box $W_{equip} := 1146lbf$

Conservative max moment based on simple span $M_{u.7} := 1.4 \cdot \left[\frac{\left(w_{slab.7}\right) \cdot L_{slab.7}^2}{8} + \frac{W_{equip} \cdot L_{slab.7}}{4}\right] = 25.38 \cdot kip \cdot in$

$DCR\left(0.65 \cdot M_{cr.7}, M_{u.7}\right) = (0.792\ \ \text{"OK"})$

Post-Installed Anchor - Check capacity

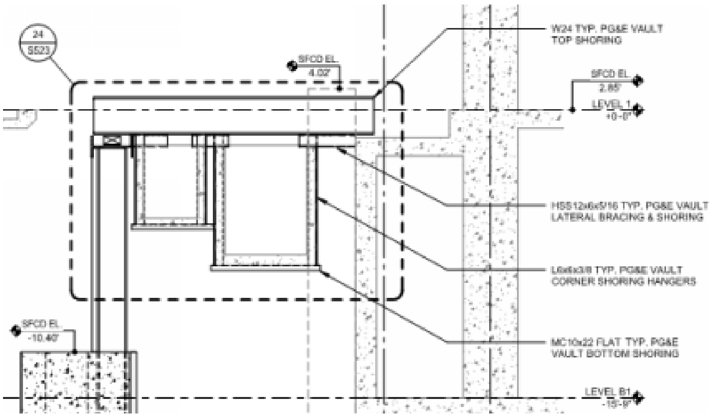
Anchor capacity $\phi V_n := 4kip$

Number required for #5 box $N_5 := \frac{1.4 \cdot W_5}{\phi V_n} = 1.946$

Number required for #7 box $N_7 := \frac{1.4 \cdot W_7}{\phi V_n} = 6.766$

Provide (8) anchors per box for stability prior to channel installation

Size Channel Support Beams



Length of box #7 $L_{7box} := 5ft + 6in$

Distributed load of 1/2 of #7 box $w_{dist} := \dfrac{\left[0.5 \cdot \left(W_7 + W_{equip}\right)\right]}{L_{7box}} = 1.861 \cdot klf$

Yield strength of channel $f_{y.36} := 36ksi$

Dead load factor for LRFD $\gamma_D := 1.4$

Factored moment demand $M_{u.channel} := \gamma_D \cdot \left(\dfrac{w_{dist} \cdot L_{7box}^2}{8}\right) = 118.2 \cdot kip \cdot in$

Bending strength reduction factor $\phi_b := 0.9$

Required plastic section modulus $Z_{req} := \dfrac{M_{u.channel}}{\phi_b \cdot f_{y.36}} = 3.65 \cdot in^3$ Check MC10x22

Plastic seciton modulus $Z_{MC} := 5.29in^3$

Elastic section modulus $S_{MC} := 2.75in^3$

Factored bending demand with self-wt $M_{u.MC} := \gamma_D \cdot \left[\dfrac{\left(w_{dist} + 22plf\right) \cdot L_{7box}^2}{8}\right] = 119.646 \cdot kip \cdot in$

Nominal bending capacity $M_n := \min\left(f_{y.36} \cdot Z_{MC}, 1.6 \cdot f_{y.36} \cdot S_{MC}\right) = 158.4 \cdot kip \cdot in$

Factored bending capacity $\boxed{\phi M_n := \phi_b \cdot M_n = 142.56 \cdot kip \cdot in}$

$\boxed{DCR\left(\phi M_n, M_{u.MC}\right) = (0.84 \ \ \text{"OK"})}$

Excavator Platform Design Calculations

SIMPSON GUMPERTZ & HEGER Engineering of Structures and Building Enclosures	CLIENT: Millennium SUBJECT: Excavator Platform	PROJECT NO: 147041.10 DATE: 12/4/2018 BY: JLC CHECKED BY: MAN

Title: Excavator Platform Calculations

References: *(AISC) Steel Construction Manual - 2017*

Description: *Calculation for excavator platform member size and weld design*

325F L Hydraulic Excavator Specifications

Dimensions

All dimensions are approximate.

Boom Options	**Reach Boom 5.7 m (18'8")**	
Stick Options	**R2.9 m* (9'6")**	
1 Shipping Height	3180 mm	10'5"
2 Shipping Length	8920 mm	29'3"
3 Tail Swing Radius	1720 mm	5'8"
4 Length to Center of Rollers	3650 mm	12'0"
5 Track Length	4460 mm	14'8"
6 Ground Clearance	450 mm	1'6"
7 Track Gauge	2380 mm	7'10"
8 Transport Width		
600 mm (24") Shoes	2980 mm	9'9"
790 mm (31") Shoes	3170 mm	10'5"
9 Handrail Height	3180 mm	10'5"
10 Counterweight Clearance	960 mm	3'2"

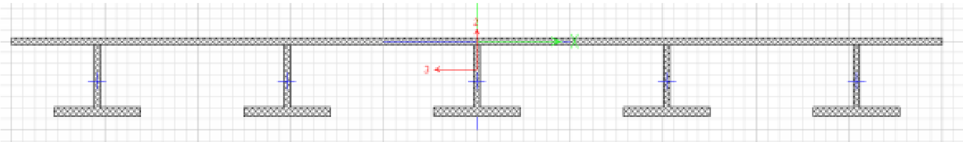
*With 1.19 m^3 (1.56 yd^3) Bucket

Material Properties

Steel yield strength $f_y := 50\text{ksi}$ A992 for members, A572 for plate

Weld material yield strength $f_{exx} := 70\text{ksi}$

Steel modulus $E_s := 29000\text{ksi}$



Design Forces

Operating weight of CAT 325F (includes bucket capacity) $W_{cat} := 63\text{kip}$

Factored moment demand from SAP $M_u := 2175\text{kip}\cdot\text{ft}$

Factored shear demand from SAP $V_u := 217\text{kip}$

Section Properties - 1" plate with (5) WT9x71.5 stiffeners

Section moment of inertia $I_3 := 4434.12\text{in}^4$

Plastic section modulus $Z_3 := 860.6\text{in}^3$

Bending strength reduction factor $\phi_b := 0.9$

Factored moment capacity $\boxed{\phi M_n := 0.9\cdot\left(Z_3\cdot f_y\right) = 3227.2\cdot\text{kip}\cdot\text{ft}}$

$\boxed{\text{DCR}\left(\phi M_n, M_u\right) = \left(0.67\ \ \text{"OK"}\right)}$

Area of webs $A_w := 5\cdot(9.75\text{in}\cdot 0.73\text{in}) = 35.6\cdot\text{in}^2$

Shear strength reduction factor $\phi_v := 0.9$

Factored shear strength $\boxed{\phi V_n := \phi_v\cdot\left(0.6\cdot A_w\cdot f_y\right) = 960.9\cdot\text{kip}}$

$\boxed{\text{DCR}\left(\phi V_n, V_u\right) = \left(0.23\ \ \text{"OK"}\right)}$

Properties

Base Material A992Fy50
Xcg 0.
Ycg -3.7994

Axis Angle 90 >>

A	224.6201
J	88.1627
I33	4434.1191
I22	270170
I23	0.
AS2	39.6764
AS3	135.4727
S33(+face)	1031.3863
S33(-face)	687.9279
S22(+face)	4502.8348
S22(-face)	4502.8348
Z33	860.5701
Z22	6715.9172
r33	4.443
r22	34.6812
d33pna	3.3633
d22pna	0.

OK

Design Weld to WT Stiffeners

Thickness of plate	$t_{plate} := 1in$
Width of plate	$w_{plate} := 10ft$
Area of plate	$A_{plate} := t_{plate} \cdot w_{plate} = 120 \cdot in^2$
Distance from plate centroid to section centroid	$y_{bar} := 3.7994in$
First moment of area of plate	$Q_{plate} := A_{plate} \cdot y_{bar} = 455.9 \cdot in^3$
Shear flow across plate	$q_{plate} := \frac{V_u \cdot Q_{plate}}{I_3} = 22.3 \cdot \frac{kip}{in}$
Distribute shear flow amongst 5 stiffeners	$q_{weld} := \frac{q_{plate}}{5} = 4.5 \cdot \frac{kip}{in}$
Use 7/16 inch fillet stitch welds	$\phi R_n := 9.74 \frac{kip}{in}$
Find required weld length per foot	$\boxed{L_{weld.req} := \frac{q_{weld} \cdot 1ft}{\phi R_n} = 5.5 \cdot in}$

Provide 7/16 inch fillet welds at 6 on 12 staggered

Check Transverse Capacity of Plate

CAT 325F Track Width	$L_{track} := 24in$
Spacing between stiffeners	$L_{stiff} := 27in$
Factored moment transverse on plate	$M_{u.plate} := 1.6 \cdot \left[\left(\frac{0.5 \cdot W_{cat}}{L_{stiff}}\right) \cdot \frac{L_{stiff}^2}{8}\right] = 170.1 \cdot kip \cdot in$
Width of hard-point roller support	$w_{roller} := 2ft$
Plastic section modulus of plate	$Z_{trans} := \frac{w_{roller} \cdot t_{plate}^2}{4} = 6 \cdot in^3$
Factored moment capacity	$\boxed{\phi M_{n.plate} := \phi_b \cdot (Z_{trans} \cdot f_y) = 270 \cdot kip \cdot in}$

$\boxed{DCR(\phi M_{n.plate}, M_{u.plate}) = (0.6 \ \ \text{"OK"})}$