GEODESIGN INC
AN NV5 COMPANY

Memorandum

To:	Aaron Van Dyke	From:	Jordan L. Melby, P.E. Scott McDevitt, P.E., G.E. Brett A. Shipton, P.E., G.E.
Company:	Lincoln Property Company	Date:	February 11, 2020
Address:	1211 SW 5th Avenue, Suite 700 Portland, OR 97204		

cc:	Bonnie Chiu, TVA Architects, Inc. (via email only) Pearse O'Moore, TVA Architects, Inc. (via email only) Chris Ferrera, DCI Engineers (via email only)

GDI Project:	LPC-1-02
RE:	Responses to City of Portland Review Comments – 19-246252-FND-01-CO Fremont Place 1650 NW Naito Parkway Portland, Oregon

INTRODUCTION

GeoDesign, Inc. is providing geotechnical engineering services for the proposed Fremont Place project in Portland, Oregon. We provided a geotechnical engineering report and addenda for site development.[1,2,3] We also prepared a report providing design recommendations for ground improvement to Pacific Foundation.[4] This memorandum provides our responses to City of Portland Site Development review comments (project number 19-246252-FND-01-CO) regarding these documents. City of Portland comments with corresponding reference numbers are presented below in italics, followed by our responses.

CITY COMMENTS AND GEODESIGN RESPONSES

REFERENCE #145

Please provide calculations for the shoring wall (CDSM with embedded soldier piles) proposed along the east side of the basement excavation. The calculation must demonstrate lateral stability for sliding, overturning, and strength calculations for shear and bending moments induced in the proposed CDSM/Soldier-pile system.

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[1] GeoDesign, Inc., 2017. *Report of Geotechnical Engineering Services; Fremont Place; 1650 NW Naito Parkway; Portland, Oregon,* dated March 22, 2017. GeoDesign Project: LPC-1-02

[2] GeoDesign, Inc., 2019. *Memorandum; Addendum 1; Driven Piling and CFA Pile Recommendations; Fremont Place; 1650 NW Naito Parkway; Portland, Oregon,* dated March 18, 2019. GeoDesign Project: LPC-1-02

[3] GeoDesign, Inc., 2019. *Memorandum; Addendum 2; Additional Geotechnical Recommendations; Fremont Place; 1650 NW Naito Parkway; Portland, Oregon,* dated September 30, 2019. GeoDesign Project: LPC-1-02

[4] GeoDesign, Inc., 2019. *Report of Geotechnical Engineering Services; Fremont Apartments; 1650 NW Naito Parkway; Portland, Oregon,* dated November 12, 2019. GeoDesign Project: PacFound-20-01

GEODESIGN RESPONSE

We used the PLAXIS finite element modeling program to evaluate the deformations and stresses of the cement deep soil mix (CDSM) columns used for shoring. The model was staged from installation of the columns through completion of the basement excavation. The tangential columns were modeled as a continuous 4-foot-wide zone of soil with parameters weighted for the area replacement of the deep soil mixing. The columns on both sides of the tangential row of columns were modeled as elasto-plastic piles 8 feet on-center with an estimated maximum bending strength based on a limited yield strength equal to 10 percent of the design strength of 150 pounds per square inch (psi). The PLAXIS model evaluated external stability for both sliding and overturning of the CDSM columns. The PLAXIS model also evaluated internal failure mechanisms such as shear and bending moments of the CDSM columns. Our analysis indicates maximum horizontal deflection of the interior face of the CDSM column wall will be less than 1 inch, which is acceptable in our opinion. A report presenting the material strengths, lateral deformations, and bending stresses from the PLAXIS analysis is presented in the Attachment.

REFERENCE #167

OSSC 107.2, ASCE 7-10 21.1.3: (GeoDesign) Please provide the Spectral Amplification Ratios period by period that were used to generate the site-specific response spectra.

GEODESIGN RESPONSE

The spectral amplifications ratios summarized in Table 1 were calculated at selected periods based on the results of our site response analysis.

Table 1. Spectral Amplification Ratios

Period	Spectral Amplification Ratio
0	0.52
0.1	0.29
0.2	0.45
0.3	0.66
0.5	0.87
1	1.45
2	1.85
3	1.38
4	1.29
5	1.15

REFERENCE #168

OSSC 107.2, ASCE 7-10 11.4.7: (GeoDesign) Please describe the pore pressure generation (effective stress) model used to generate the site response analysis.

GEODESIGN RESPONSE

Site response and pore pressure generation were determined using the DeepSoil V6.1 computer application. The parameters summarized in Table 2 were used according to the Dobry and Matasovic model.

Table 2. DeepSoil Parameters for Pore Water Pressure

Material Type	Depth (feet)	Pore Water Pressure Model				γ_{tvp}	v
		f	p	F	s		
Fill	0 – 10	1.0	0	1.0	0	0	1.0
Owl Island Sand	10 – 20	2.0	1.005	3.0	1.8	0.025	1.0
Pacific Northwest Silt	20 – 50	2.0	1.0	0.32	1.3	0.025	1.0
Santa Monica Beach Sand	50 – 100+	1.0	1.0	0.73	1.0	0.02	0.38

REFERENCE #169

OSSC 1810.3: (GeoDesign) – Please clarify if unit skin friction in the lower gravel is neglected for uplift pile capacity (seismic and flood load conditions). The recommendations are unclear in this regard.

GEODESIGN RESPONSE

Since it is uncertain how deep the piles will penetrate the gravel unit, uplift capacity estimates have not included skin friction from the gravel.

REFERENCE #170

OSSC 107.2, 1613.1: (GeoDesign): Please work with DCI to check the potential for pile buckling during liquefaction. Key to this analysis is the thickness and location of liquefied soils zones and soil stiffness during cyclic shaking. If "fixity" of the pile tip is required to prevent buckling, please provide the minimum embedment required into the gravel layer (or lowest non-liquefied layer) and associated L-Pile analyses estimating the depth to zero moment.

GEODESIGN RESPONSE

GeoDesign performed LPile analyses on the proposed piles. The response to Reference #171 (below) presents the LPile parameters used in our analysis, which includes residual strength values for liquefied sand. The results of the analyses indicate that the liquefied soil will provide enough lateral support to prevent buckling during the design seismic event. The LPile application will return an error message if the pile being analyzed yields and excessive deflection is calculated. In our opinion, this is a more suitable analysis method than the Euler method (or similar), which treats liquefied soil the same as air, providing no confinement at all. Additionally, in our opinion it is unlikely that the entire sand layer from 47 to 120 feet below ground surface will completely liquefy during/after the seismic event. It is more likely that portions of the sand layer will liquefy while some portions will retain full strength, which makes our LPile results conservative.

REFERENCE #171

OSSC 107.2, 1810.2.4: (GeoDesign) Please provide the L-Pile output showing all input parameters for each soil layer and section properties of the piles.

GEODESIGN RESPONSE

We used the LPile computer application to evaluate lateral capacity for 18- and 24-inch-diameter steel pipe piles with wall thicknesses of 1/2 to 5/8 inch. The soil parameters summarized in Table 3 were used in our analysis.

Table 3. LPile Soil Parameters

LPile Soil Type	Depth[1] (feet)	Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (degrees)	Strain Factor	k (pcf)	Residual Strength (psf)
Soft Clay	17 – 47	48	300	N/A	0.02	N/A	N/A
Liquefied Sand	47 – 120	53	N/A	N/A	0.01	N/A	300
Sand[2]	120+	78	0	45	N/A	800	N/A

1. Measured from ground surface. First soil layer begins at basement elevation.
2. Gravel layer

NA: not applicable
pcf: pounds per cubic foot
psf: pounds per square foot

REFERENCE #162

OSSC 1810.2: (DCI/GeoDesign) Please check if pile embedment into the lower gravel is required to prevent pile buckling during liquefaction. If pile buckling is a concern, which Site Development assumes could be depending on the strength, thickness, and location of liquified soil zones, the pile may require additional embedment into the lower gravel to maintain a "fixed" condition understanding that buckling is influenced by pinned/fixed assumptions.

GEODESIGN RESPONSE

18-inch-diameter piles will be embedded at least 5 feet into the native gravel in order to avoid buckling during the seismic event. A 5-foot embedment is not necessary for 24-inch-diameter piles per our response to Reference #170.

REFERENCE #152

OSSC 107.2, 1803.1, 1810.1: Please contact the geotechnical engineer and clarify the minimum embedment depth of the piles below the top of the gravel layer. Show minimum embedment on Details 4/S3.02 and 10/S3.02.

GEODESIGN RESPONSE

18-inch-diameter piles should be embedded at least 5 feet into the dense gravel. 24-inch-diameter piles should be embedded at least 2 feet into the dense gravel.

REFERENCE #182
Please provide GeoDesign's review of the soil improvement plan for conformance to their recommendations.

GEODESIGN RESPONSE
GeoDesign prepared the soil improvement plan in accordance with our recommendations.

REFERENCE #183
Please provide specifications for ground improvement in the plan set or specification manual.

GEODESIGN RESPONSE
TVA Architects, Inc. will include the specifications for ground improvement in the specification manual.

REFERENCE #184
Site Development observes that the type of liquefaction mitigation changed between the March 22, 2017 soils report and November 22, 2019 letter (from a roughly 30-foot wide zone of stone columns to a 12-foot zone of cement deep soil mix columns). What is the reason for this change? Was the impact of the easterly adjoining greenway a consideration?

GEODESIGN RESPONSE
Due to project constraints, we understood the project team did not want to impact the east adjoining greenway. Accordingly, we designed a ground improvement system that would provide the required lateral support outside of the greenway.

REFERENCE #185
Site Development has significant concerns regarding CDSM columns and their ability to mitigate the impact of lateral spreading at the site. Key to our concerns is the potential for bending and overturning (tilting) of the slender columns subject lateral soil deformation. These failure modes are not addressed, and it appears that the CDSM columns may fail in bending (tension) or adverse rotation. Please evaluate the potential for bending and tilting of the CDSM columns. GeoDesign should consider a deformation-based analysis (FLAC, etc.) to evaluate deformation behavior. Clarifying the soil displacement profile from the bottom of the lowest liquefaction layer to the surface will help Site Development's understanding of the mitigation.

GEODESIGN RESPONSE
We used the PLAXIS finite element modeling program to evaluate the deformations and stresses of the CDSM columns for post-liquefied lateral spreading. The model was staged from installation of the columns through excavation for construction and the application of liquefied strength and stiffness parameters. The tangential columns were modeled as a continuous 4-foot-wide zone of soil with parameters weighted for the area replacement of the deep soil mixing. The columns on both sides of the tangential row of columns were modeled as elasto-plastic piles 8 feet on-center with an estimated maximum bending strength based on a limited yield strength equal to 10 percent of the design strength of 150 psi. Both the PLAXIS and SLOPE/W analyses indicate lateral spreading

movement in the slope outside of the CDSM ground improvement area from the upper liquefiable zone. To allow for easier convergence of the PLAXIS model, a portion of soil was removed from the upper slope when the liquefied soil parameters were applied to represent the shallower lateral spreading. Based on the results of our analyses, the CDSM columns will not fail in bending or adverse rotation from liquefaction-induced lateral spreading. A report presenting the material strengths, lateral deformations, and bending stresses from the PLAXIS analysis is presented in the Attachment.

We also used the PLAXIS finite element modeling program to evaluate deformations of the CDSM-improved soil from the maximum seismic induced shear loading from the building foundation. A uniform load of 5,000 psf was applied over a height of 4 feet at the edge of the CDSM-improved ground to estimate maximum ground deformations from the shear loading before estimating liquefaction-induced strengths and lateral spreading. The piles supporting the vertical building loads were conservatively neglected from the PLAXIS model. Our analysis indicates the horizontal loading will result in acceptable horizontal deformations of less than 0.1 inch. A report presenting the material strengths, lateral deformations, and bending stresses from the PLAXIS analysis is presented in the Attachment.

REFERENCE #186

The CDSM ground improvement model does not accommodate horizontal ground deformation beyond the leading row of improvement. The wedge of soil in front of the wall will likely displace away from the improvement zone during cyclic shaking potentially resulting in unsupported CDSM columns above the bottom of the liquefied zone (when viewed as a simple force-diagram). Please evaluate the deformation of the soil mass in front of the wall and its impact on the proposed ground improvement.

GEODESIGN RESPONSE

Please see responses to References #185 and #190.

REFERENCE #187

In the soil model, please extend the slope eastward to the bottom of the adjoining Willamette River. It's unclear what the bathymetric elevations are beyond east side of the slope model.

GEODESIGN RESPONSE

The riverward slope within our stability model has been extended to the east by over 250 feet from our original model. The slope modeling was performed using available bathymetry data.[5]

REFERENCE #188

Is lateral spreading and soil deformation limited to soils above Elev -13 and within Liquefaction Layer 1 of the soil model? Is lateral deformation anticipated in Liquefaction Layer 2?

[5] David Evans and Associates, Inc., 2018. *Willamette River, Oregon; River Mile 1.9 to 11.8; Hydrographic Survey Report*, dated July 2018.

GEODESIGN RESPONSE
Based on our analysis (see Attachment), Liquefiable Layer #2 has a factor of safety greater than 1.23 against lateral deformation when subjected to a horizontal ground motion equal to two-thirds of the maximum considered earthquake geometric mean peak ground acceleration adjusted for site affects (PGA_M), which is 0.25 g, and a base shear of 5,050 psf distributed over a 4-foot-thick mat slab after soils have liquefied.

REFERENCE #189
Do the soil properties for the non-liquefied soil between Elev 0 and -13 accommodate some loss of strength due to strain softening?

GEODESIGN RESPONSE
We conservatively assumed a friction angle of 28 degrees with a cohesion of 100 psf for the non-liquefiable soil layers in our analyses. Based on our data collected from CPT-4, the clay soil layer in the east portion of the site is correlated to have an approximate undrained shear strength of 960 psf between elevations of 0 and -13 feet.[6] If we reduce the undrained shear strength to 300 psf in our stability model, we calculate similar factors of safety to those calculated assuming a friction angle of 28 degrees with a cohesion of 100 psf (see Attachment).

Lab testing of the soil collected from these elevations indicates the soil at plasticity indices between 16 and 18. Accordingly, it is our opinion that the soil at these elevations are not susceptible to strain softening, and the strength parameters used in our models are conservative.

REFERENCE #190
Please provide a check of flow liquefaction in the soil wedge in front of the improvement zone. It's unclear if static forces will induce flow failure of the wedge of soil in front of the improved zone following liquefaction.

GEODESIGN RESPONSE
Based on our analysis (see Attachment), a portion of the soil wedge east of the improvement zone is susceptible to flow failure. Accordingly, we have conservatively updated our stability model to account for complete loss of the zone of soil that is susceptible to flow failure. Our updated stability model shows that the ground improvement resists anticipated lateral forces due to a horizontal ground motion of 0.25 g (two-thirds of the PGA_M) and a base shear of 5,050 psf distributed over a 4-foot-thick mat slab during liquefied soil conditions.

REFERENCE #191
Please clarify how the block failure surface was developed for the slope stability model.

[6] Robertson, P.K., 2012. *Interpretation of In-Situ Tests – Some Insights.*

GEODESIGN RESPONSE

The block model was performed by evaluating sliding block failures at each node within the green meshes shown on the attached stability plots. The modelled block failure with the lowest calculated factor of safety is highlighted and is what was used in our analysis.

REFERENCE #192

Please clarify how the inertial loads of the lateral spreading mass behind the wall were developed and incorporated into the slope stability model. Since the soil wedge in front of the wall may displace/separate from the improved zone (with increasing deformation from the bottom to top of the liquefied layer), it's unclear if the failure surface is appropriately modeled.

GEODESIGN RESPONSE

Please see response to Reference #190.

REFERENCE #193

Please clarify the yield acceleration used in the lateral spread analysis.

GEODESIGN RESPONSE

A yield acceleration of 0.35 g was determined by our original stability model and was used in the lateral spreading analysis conducted with SLAMMER.

REFERENCE #194

There appears to be a distributed load normal to the ground surface along that portion of the slope below the water surface. What does this distributed load represent? Is this an artifact of the stability model?

GEODESIGN RESPONSE

The distributed load described above represents hydrostatic pressure from the Willamette River.

REFERENCE #198

The inertial loading of the basement and pile caps is not considered in the CDSM mitigation model. Please work with DCI to establish where base shear of the building is resolved (which is unclear at this time – see other comments). If the shear resistance of the CDSM columns is required to help resist base shear of the building, please revise the ground modification accordingly.

GEODESIGN RESPONSE

The base shear of the structure is resolved within the 4- to 5-foot-thick mat slab at the base of the basement. We have updated our stability models to account for the horizontal loading resulting from base shear at the mat slab location.

♦ ♦ ♦

We appreciate the opportunity to be of continued service to you on this project. Please call if you have questions concerning the information provided.

JLM:SPM:BAS:sn

Attachment

One copy submitted (via email only)

Document ID: LPC-1-02-021120-geom-rev.docx

REGISTERED PROFESSIONAL ENGINEER
63094
OREGON
JULY 11, 2000
SCOTT P. McDEVITT
EXPIRES: 12/31/20

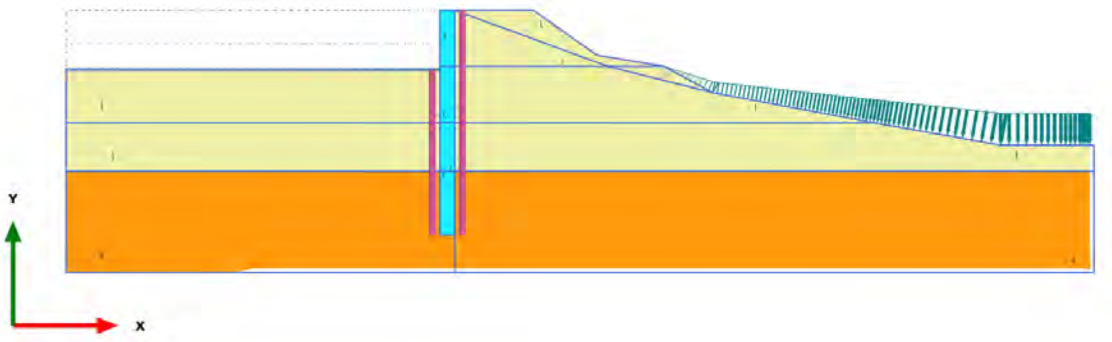
ATTACHMENT

Reference #185, 186, 190, 198

PLAXIS Report

Filename	Pacfound-20-01-012820c.p2dx
Directory	C:\Users\baw\Documents\Plaxis \
Title	Freemont Apartments
Model	Plane strain
Elements	15-Noded
PLAXIS Version	Version 20.0.0.119

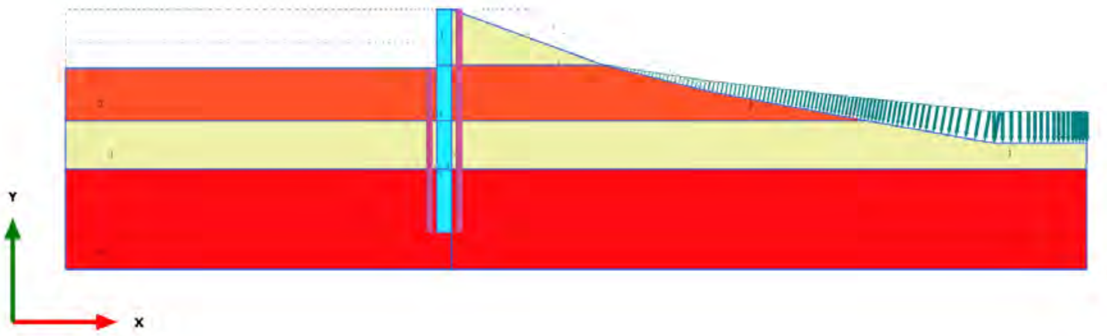
1.1.1.1 Calculation results, Excavate 2 [Phase_3] (3/53), Materials plot



Materials plot

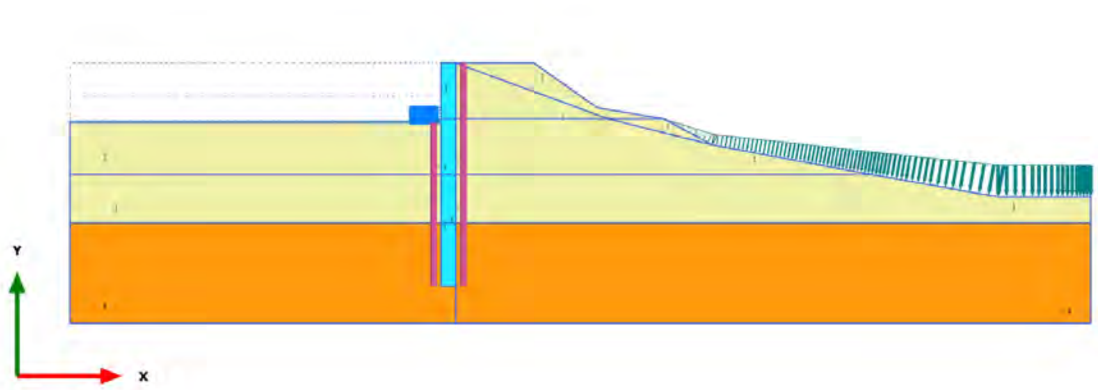
Reference #185, 186, 190, 198

1.1.1.2 Calculation results, Liquefied Parameters [Phase_4] (4/95), Materials plot



Materials plot

1.1.1.3 Calculation results, Apply horizontal load [Phase_5] (5/200), Materials plot



Materials plot

1.1.2.1.1 Materials - Soil and interfaces - Mohr-Coulomb

Identification		Non-Liquefiable	Liquefiable 1	Liquefiable 2	Non-liquefied Sand	Tangential CDSM Column Row
Identification number		1	2	3	4	5
Drainage type		Drained	Undrained (C)	Undrained (C)	Drained	Non-porous
Colour						
Comments						
γ_{unsat}	lbf/ft^3	110.0	110.0	110.0	110.0	125.0
γ_{sat}	lbf/ft^3	115.0	110.0	110.0	110.0	125.0
Dilatancy cut-off		No	No	No	No	No
e_{init}		0.5000	0.5000	0.5000	0.5000	0.5000
e_{min}		0.000	0.000	0.000	0.000	0.000
e_{max}		999.0	999.0	999.0	999.0	999.0
Rayleigh α		0.000	0.000	0.000	0.000	0.000
Rayleigh β		0.000	0.000	0.000	0.000	0.000
E	lbf/ft^2	400.0E3	50.00E3	70.00E3	800.0E3	6.500E6
ν (nu)		0.3500	0.4950	0.4950	0.3500	0.3000
G	lbf/ft^2	148.1E3	16.72E3	23.41E3	296.3E3	2.500E6
E_{oed}	lbf/ft^2	642.0E3	1.689E6	2.365E6	1.284E6	8.750E6
c_{ref}	lbf/ft^2	50.00	180.0	1120	0.000	6900
φ (phi)	°	30.00	0.000	0.000	32.00	0.000

Reference #185, 186, 190, 198

Freemont Apartments

Identification		Non-Liquefiable	Liquefiable 1	Liquefiable 2	Non-liquefied Sand	Tangential CDSM Column Row
ψ (psi)	°	0.000	0.000	0.000	0.000	0.000
V_s	ft/s	208.2	69.95	82.76	294.4	802.3
V_p	ft/s	433.4	703.0	831.8	612.9	1501
Set to default values		Yes	No	No	Yes	Yes
E_{inc}	lbf/ft²/ft	0.000	0.000	0.000	0.000	0.000
y_{ref}	ft	0.000	15.00	0.000	0.000	0.000
c_{inc}	lbf/ft²/ft	0.000	10.00	0.000	0.000	0.000
y_{ref}	ft	0.000	15.00	0.000	0.000	0.000
Tension cut-off		Yes	No	No	Yes	Yes
Tensile strength	lbf/ft²	0.000	208.9E6	208.9E6	0.000	0.000
Undrained behaviour		Standard	Standard	Standard	Standard	Standard
Skempton-B		0.9699	0.000	0.000	0.9699	0.9783
ν_u		0.4950	0.4950	0.4950	0.4950	0.4950
$K_{w,ref}$ / n	lbf/ft²	14.32E6	0.000	0.000	28.64E6	243.7E6
$C_{v,ref}$	ft²/s	0.000	0.000	0.000	0.000	0.000
Stiffness		Standard	Standard	Standard	Standard	Standard
Strength		Rigid	Rigid	Rigid	Rigid	Rigid
R_{inter}		1.000	1.000	1.000	1.000	1.000
Consider gap closure		Yes	Yes	Yes	Yes	Yes
δ_{inter}		0.000	0.000	0.000	0.000	0.000
Cross permeability		Impermeable	Impermeable	Impermeable	Impermeable	Impermeable
Drainage conductivity, dk	ft³/s/ft	0.000	0.000	0.000	0.000	0.000

Reference #185, 186, 190, 198

Freemont Apartments

Identification		Non-Liquefiable	Liquefiable 1	Liquefiable 2	Non-liquefied Sand	Tangential CDSM Column Row
K_0 determination		Automatic	Automatic	Automatic	Automatic	Automatic
$K_{0,x} = K_{0,z}$		Yes	Yes	Yes	Yes	Yes
$K_{0,x}$		0.5000	0.5000	0.5000	0.4701	0.5000
$K_{0,z}$		0.5000	0.5000	0.5000	0.4701	0.5000
Data set		Standard	Standard	Standard	Standard	Standard
Type		Coarse	Coarse	Coarse	Coarse	Coarse
< 2 µm	%	10.00	10.00	10.00	10.00	10.00
2 µm - 50 µm	%	13.00	13.00	13.00	13.00	13.00
50 µm - 2 mm	%	77.00	77.00	77.00	77.00	77.00
Use defaults		None	None	None	None	None
k_x	ft/s	0.000	0.000	0.000	0.000	0.000
k_y	ft/s	0.000	0.000	0.000	0.000	0.000
$-\psi_{unsat}$	ft	32.81E3	32.81E3	32.81E3	32.81E3	32.81E3
e_{init}		0.5000	0.5000	0.5000	0.5000	0.5000
S_s	1/ft	0.000	0.000	0.000	0.000	0.000
c_k		1000E12	1000E12	1000E12	1000E12	1000E12

1.1.2.2 Materials - Embedded beam row -

Identification		CDSM Columns 8' oc
Identification number		1
Comments		
Colour		
Material type		Elastoplastic
E	lbf/ft^2	6.500E6
γ	lbf/ft^3	120.0
Beam type		User-defined
A	ft^2	12.57
I_2	ft^4	12.57
I_3	ft^4	12.57
Rayleigh α		0.000
Rayleigh β		0.000
Axial skin resistance		Layer dependent
T_{max}	lbf/ft	68.52E12
F_{max}	lbf	3700
Identification number		1

Reference #185, 186, 190, 198

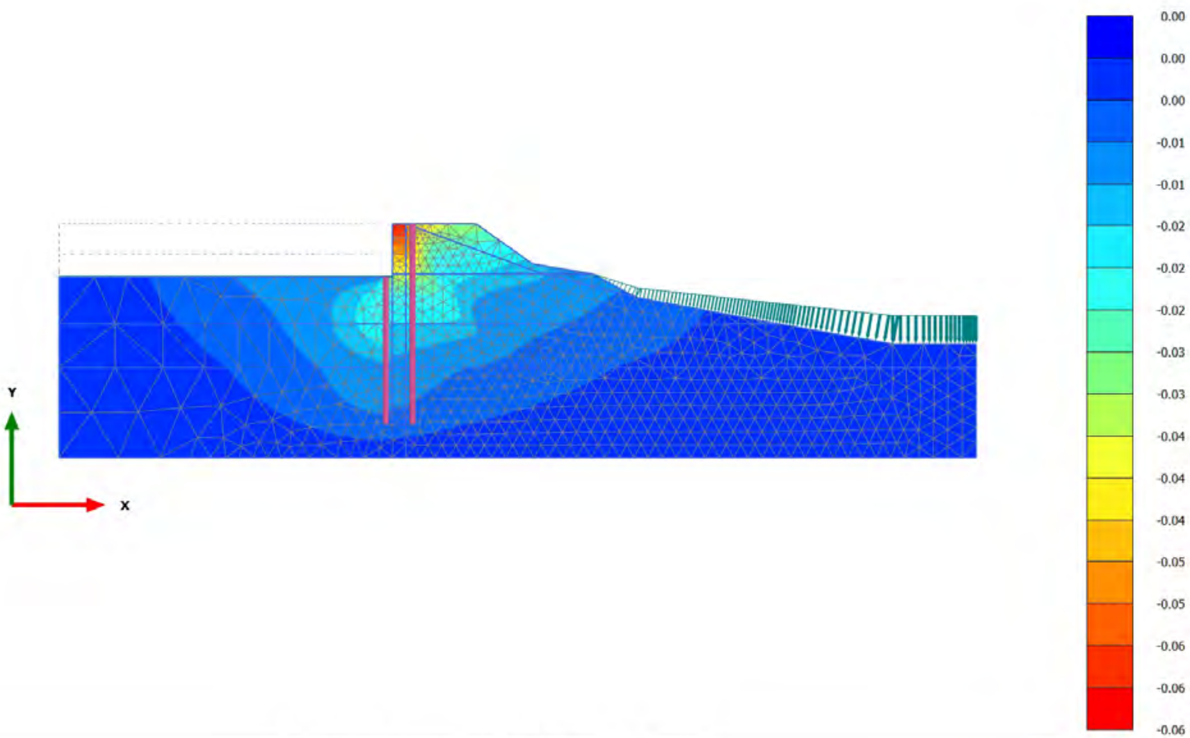
Freemont Apartments

Identification		CDSM Columns 8' oc
Comments		
Colour		
Material type		Elastoplastic
E	lbf/ft²	6.500E6
γ	lbf/ft³	120.0
Beam type		User-defined
A	ft²	12.57
I	ft⁴	12.57
$L_{spacing}$	ft	8.000
M_p	lbf ft	1700
N_p	lbf	33.9E3
Rayleigh α		0.000
Rayleigh β		0.000
Axial skin resistance		Layer dependent
T_{max}	lbf/ft	68.52E12
Lateral resistance		Linear
$T_{lat, start, max}$	lbf/ft	630.0
$T_{lat, end, max}$	lbf/ft	370.0
F_{max}	lbf	3700

Reference #185, 186, 190, 198

Identification	CDSM Columns 8' oc
Default values	Yes
Axial stiffness factor	1.334
Lateral stiffness factor	1.334
Base stiffness factor	13.34
Identification number	1

4.1.1 Calculation results, Excavate 2 [Phase_3] (3/53), Total displacements u_x



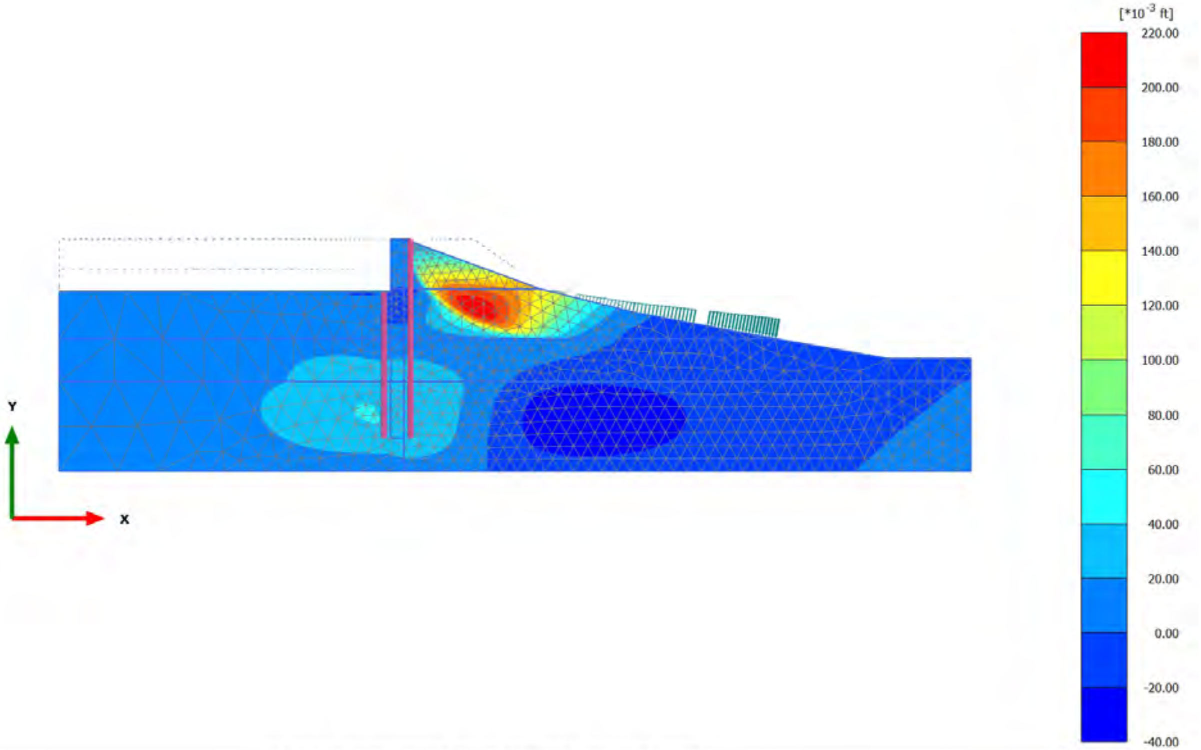
Total displacements u_x (scaled up 100 times)

Maximum value = 0.000 ft (Element 304 at Node 13337)

Minimum value = -0.06162 ft (Element 83 at Node 10862)

Reference #185, 186, 190, 198

4.1.2 Calculation results, Liquefied Parameters [Phase_4] (4/95), Total displacements u_x



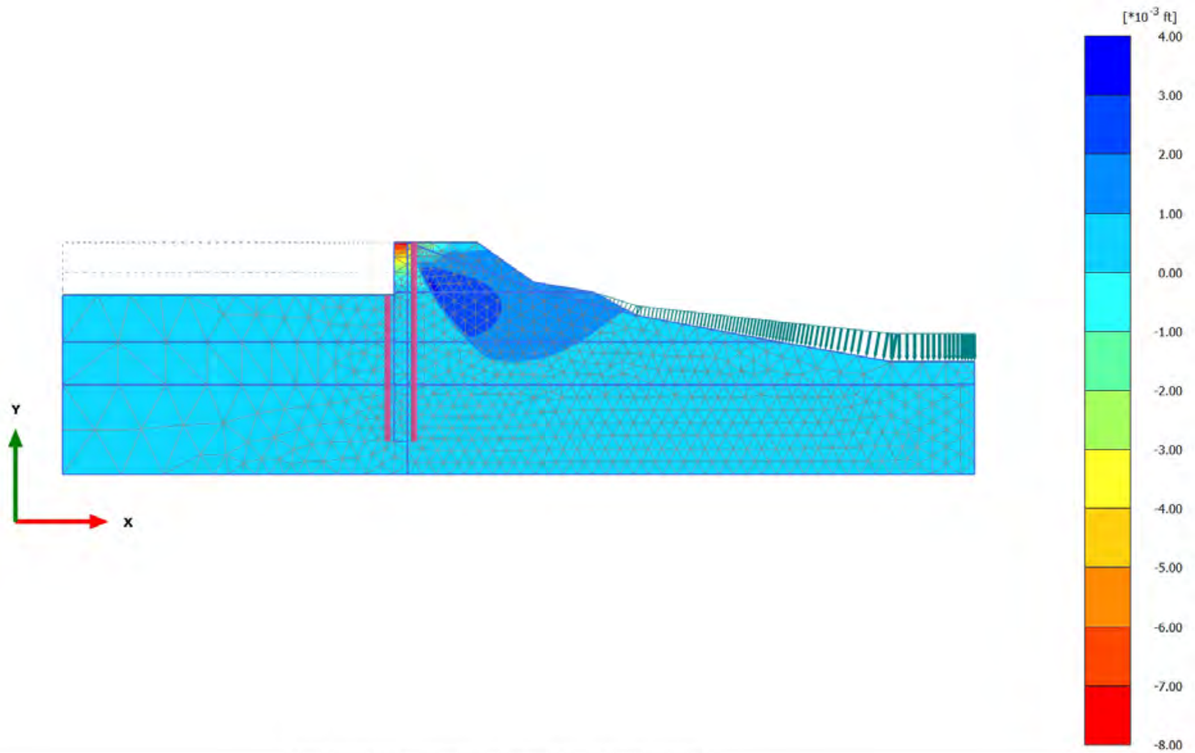
Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.2180 ft (Element 478 at Node 6962)

Minimum value = -0.03764 ft (Element 1233 at Node 4876)

Reference #185, 186, 190, 198

4.1.3 Calculation results, Apply horizontal load [Phase_5] (5/200), Total displacements u_x

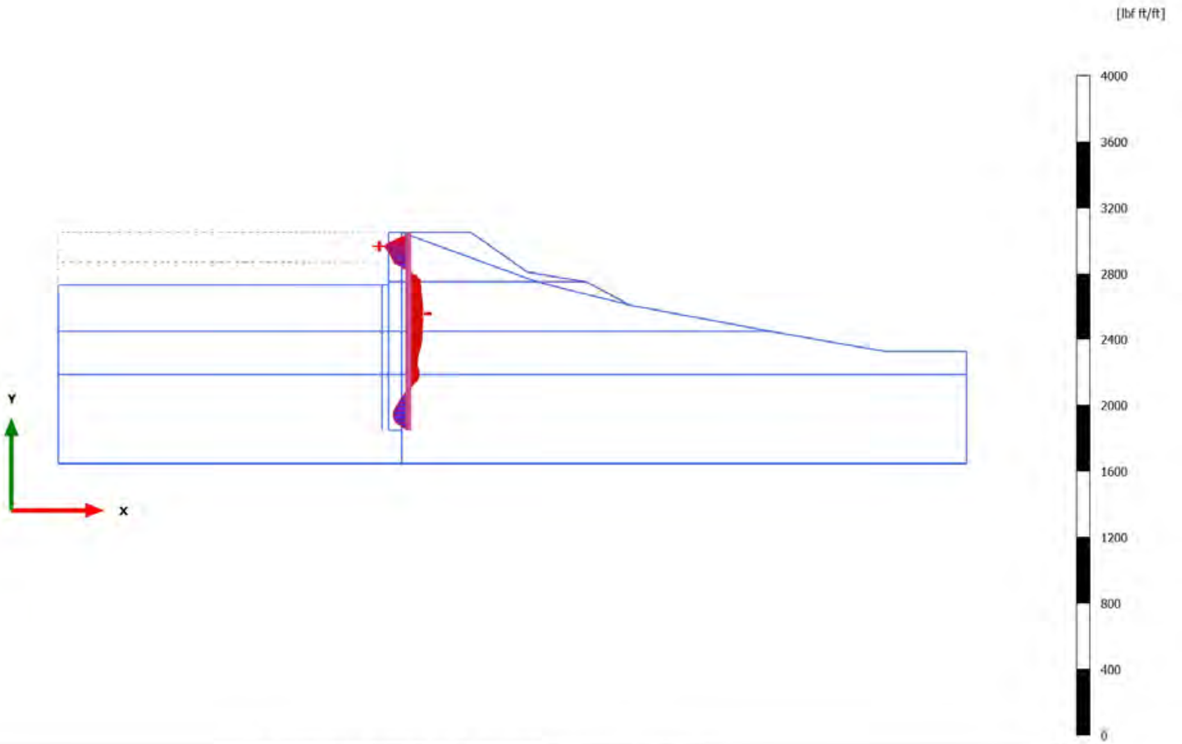


Total displacements u_x (scaled up 500 times)

Maximum value = $3.055*10^{-3}$ ft (Element 148 at Node 8945)

Minimum value = $-7.691*10^{-3}$ ft (Element 83 at Node 10862)

4.2.1 Calculation results, Embedded beam row, Excavate 2 [Phase_3] (3/53), Bending moments M



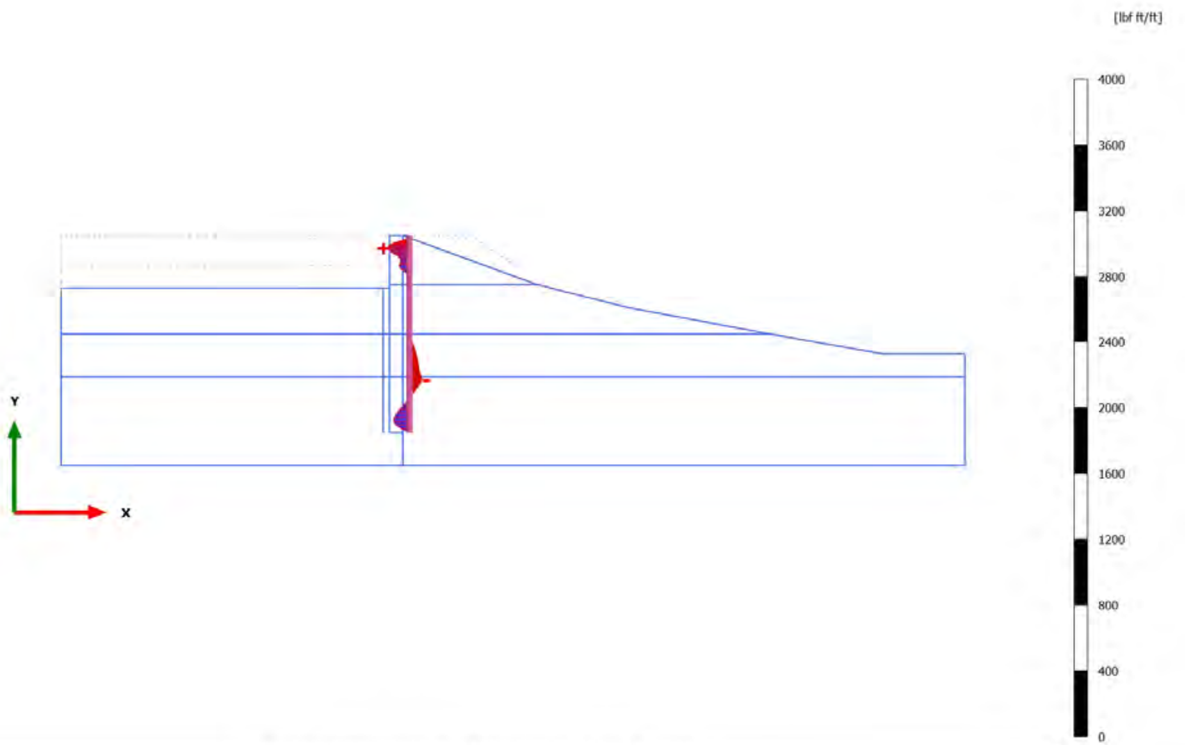
Bending moments M (scaled up 0.0500 times)

Maximum value = 146.3 lbf ft/ft (Element 20 at Node 13382)

Minimum value = -86.89 lbf ft/ft (Element 12 at Node 13414)

Reference #185, 186, 190, 198

4.2.2 Calculation results, Embedded beam row, Liquefied Parameters [Phase_4] (4/95), Bending moments M



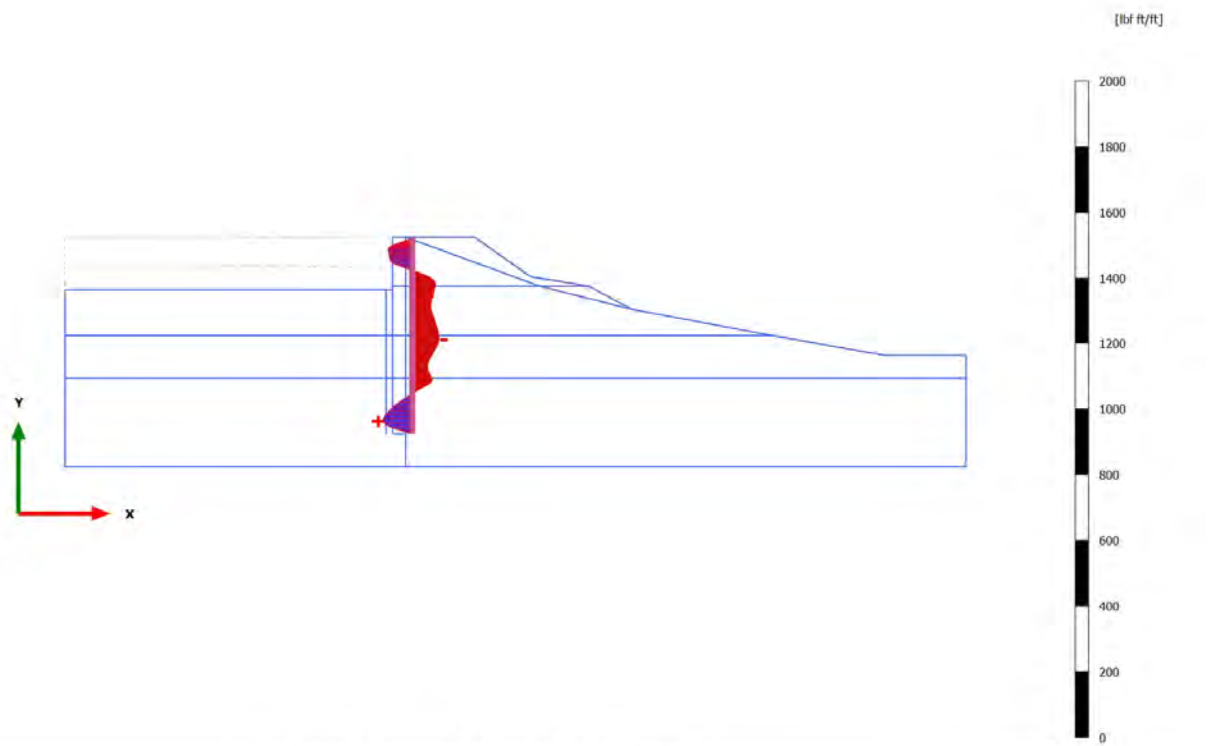
Bending moments M (scaled up 0.0500 times)

Maximum value = 130.8 lbf ft/ft (Element 21 at Node 13381)

Minimum value = -74.96 lbf ft/ft (Element 6 at Node 13439)

Reference #185, 186, 190, 198

4.2.3 Calculation results, Embedded beam row, Apply horizontal load [Phase_5] (5/200), Bending moments M



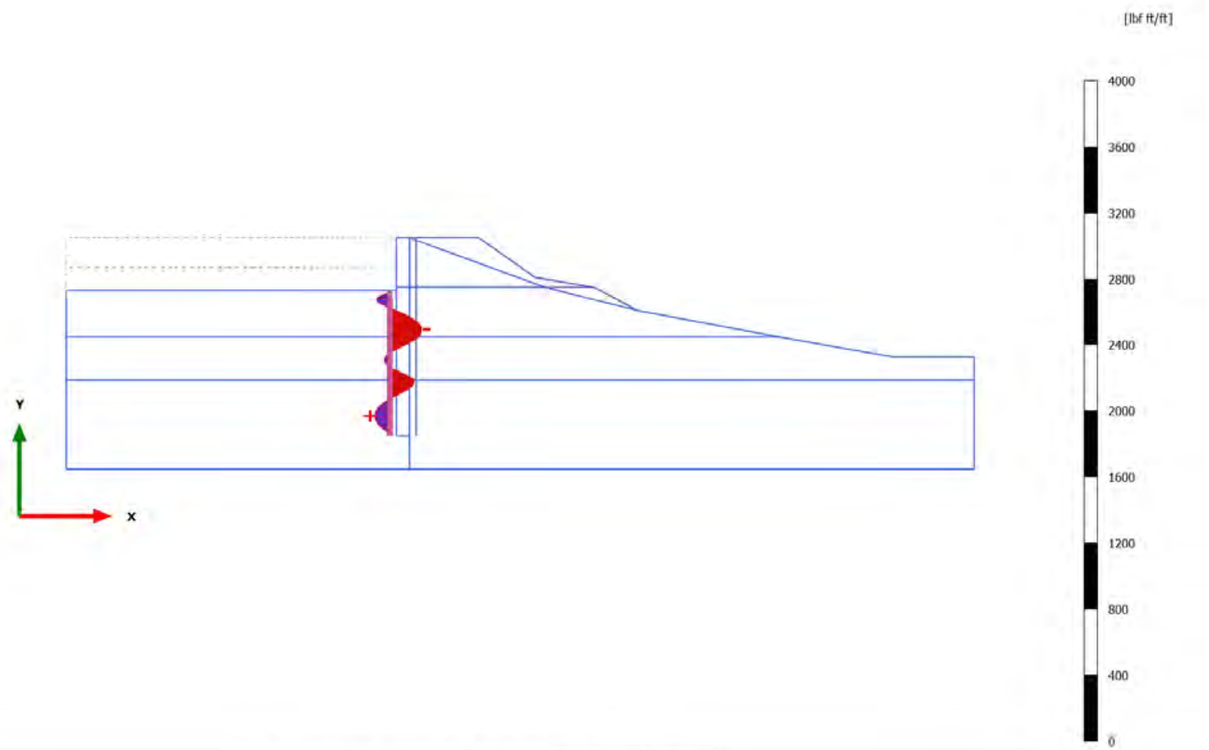
Bending moments M (scaled up 0.100 times)

Maximum value = 88.43 lbf ft/ft (Element 2 at Node 13456)

Minimum value = -81.66 lbf ft/ft (Element 10 at Node 13423)

Reference #185, 186, 190, 198

4.3.1 Calculation results, Embedded beam row, Excavate 2 [Phase_3] (3/53), Bending moments M

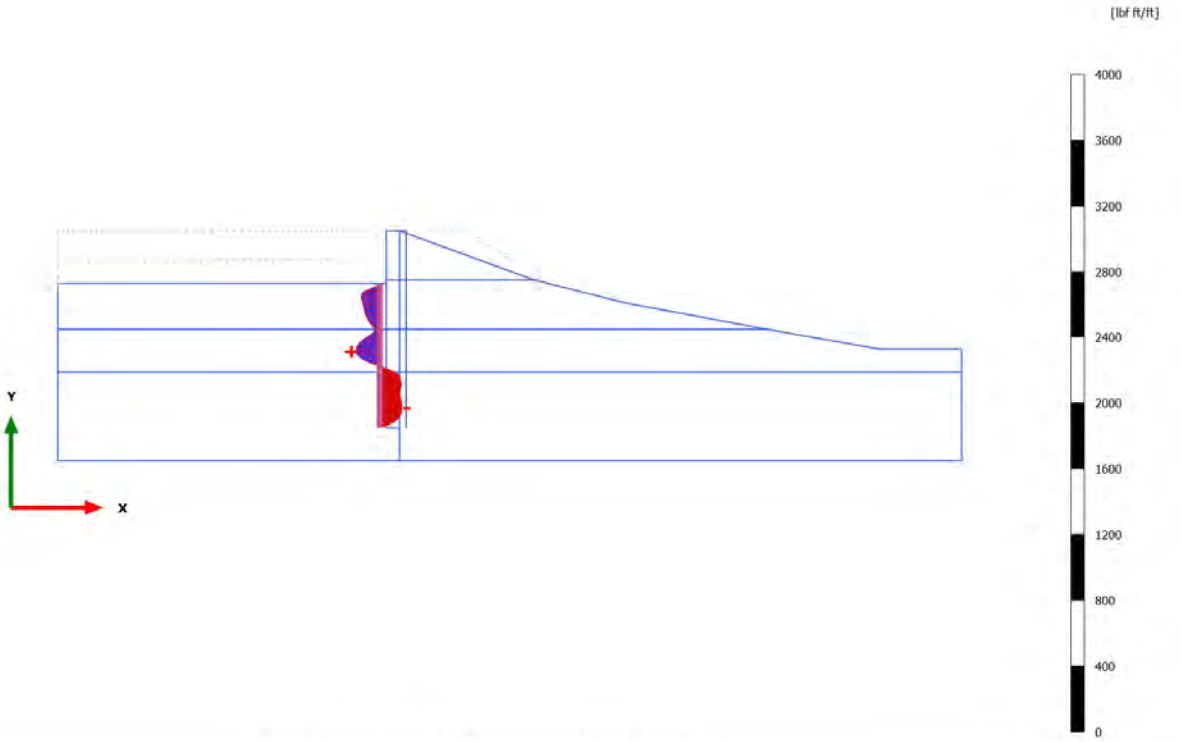


Bending moments M (scaled up 0.0500 times)

Maximum value = 90.72 lbf ft/ft (Element 27 at Node 13514)

Minimum value = -193.4 lbf ft/ft (Element 36 at Node 13479)

4.3.2 Calculation results, Embedded beam row, Liquefied Parameters [Phase_4] (4/95), Bending moments M



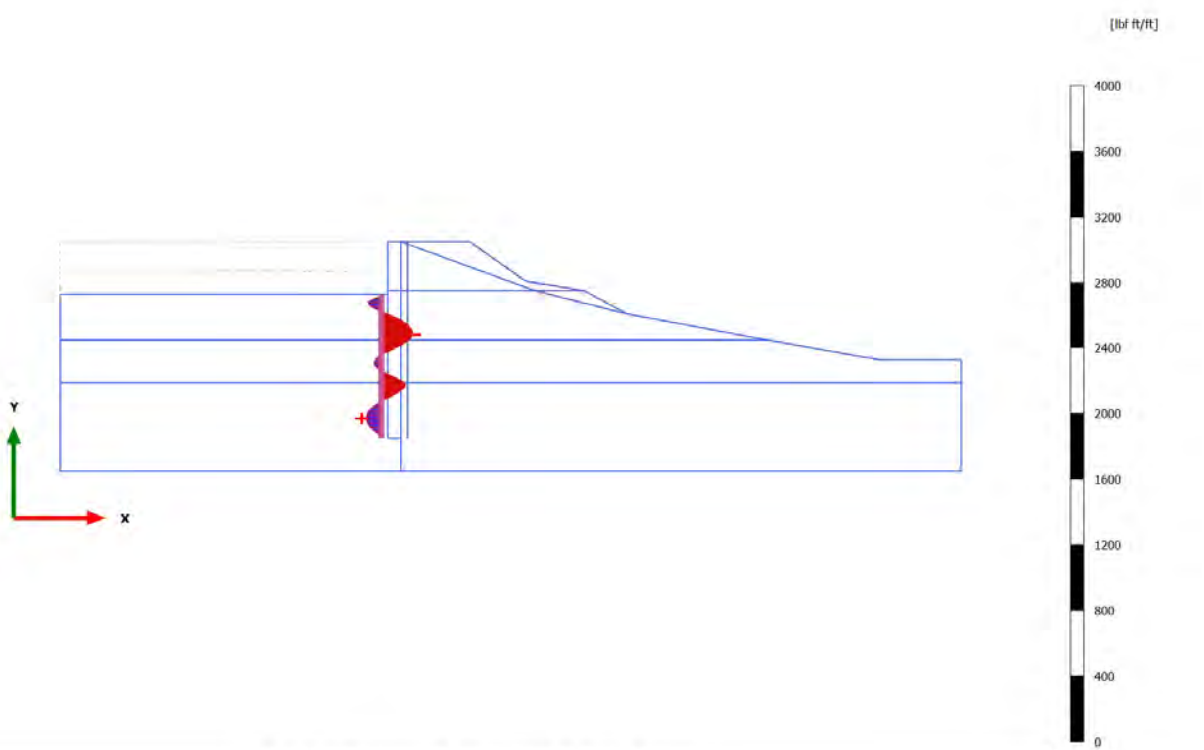
Bending moments M (scaled up 0.0500 times)

Maximum value = 146.7 lbf ft/ft (Element 32 at Node 13491)

Minimum value = -133.6 lbf ft/ft (Element 27 at Node 13514)

Reference #185, 186, 190, 198

4.3.3 Calculation results, Embedded beam row, Apply horizontal load [Phase_5] (5/200), Bending moments M

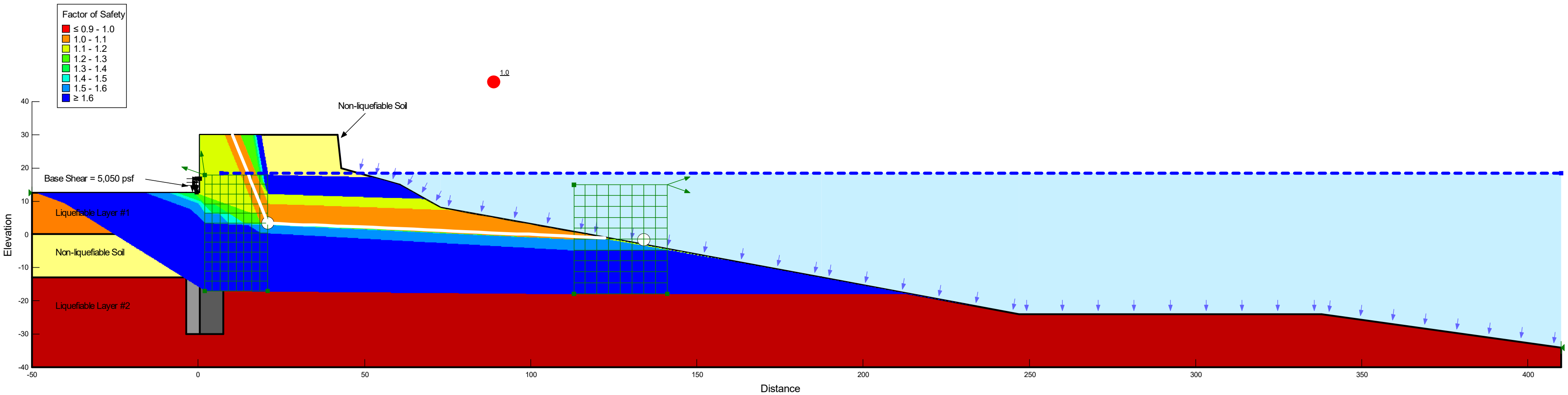


Bending moments M (scaled up 0.0500 times)

Maximum value = 91.48 lbf ft/ft (Element 27 at Node 13514)

Minimum value = -192.2 lbf ft/ft (Element 35 at Node 13480)

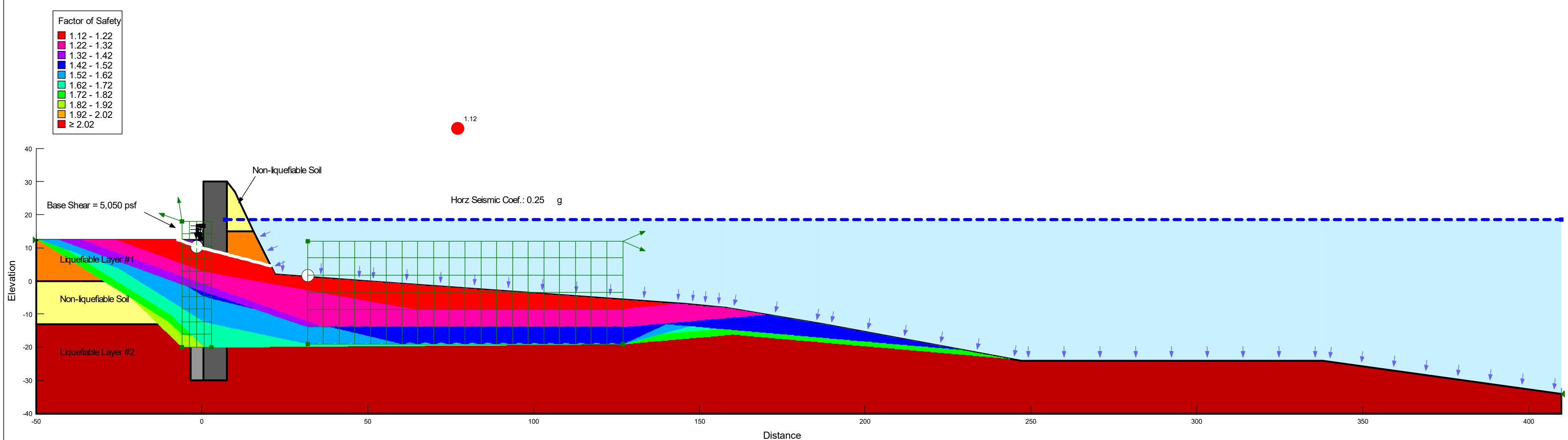
Reference #185, 186, 187, 190, 198



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)
	Additional CDSM Column Row (8' o.c.)	110			3,955
	Liquefiable 1	110			179
	Liquefiable 2	110			1,116
	Non-Liquefiable Soil	110	100	28	
	Tangential CDSM Column Row (3' o.c.) and 2nd Row (8' o.c.)	110			6,867

Sliding Block Failure with Ground Improvement
Flow Failure Model
Tangential 2nd Row (8 feet o.c.)
Fremont Apartments
Portland, OR

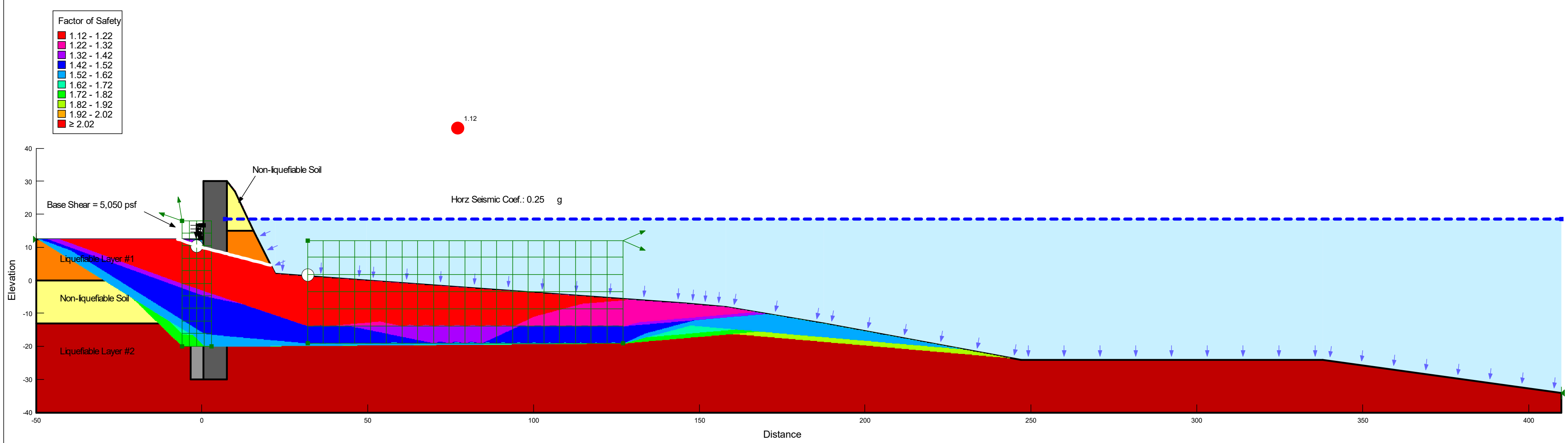
Reference #185, 186, 187, 190, 198



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)
	Additional CDSM Column Row (8' o.c.)	110			3,955
	Liquefiable 1	110			179
	Liquefiable 2	110			1,116
	Non-Liquefiable Soil	110	100	28	
	Tangential CDSM Column Row (3' o.c.) and 2nd Row (8' o.c.)	110			6,867

Sliding Block Failure with Ground Improvement
Model with Soil Loss
Tangential 2nd Row (8 feet o.c.)
Fremont Apartments
Portland, OR

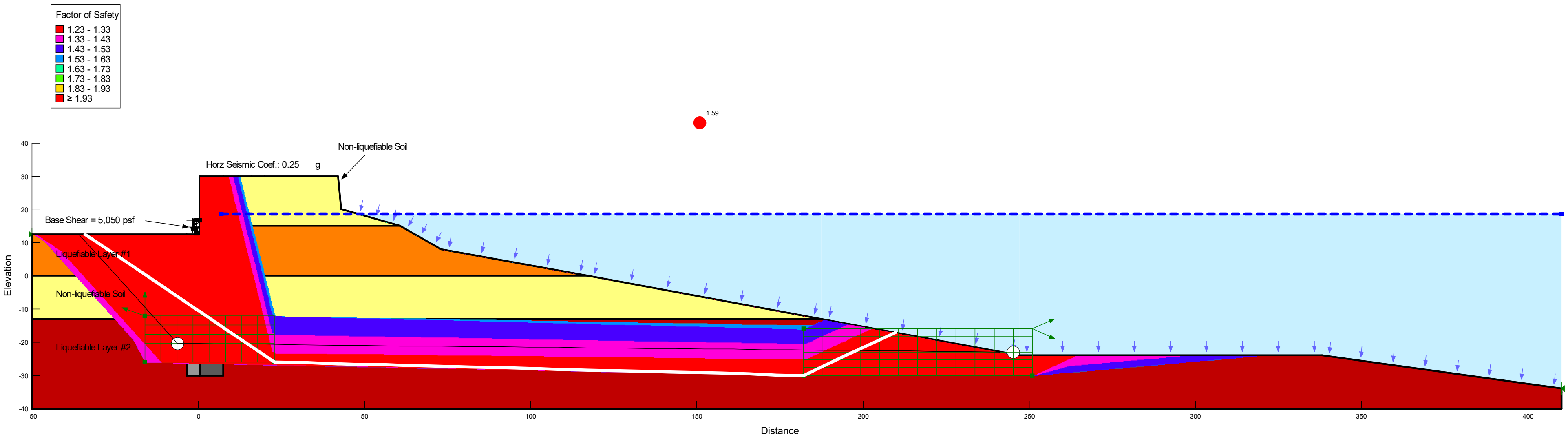
Reference #189



Color	Name	Unit Weight (pcf)	Cohesion (psf)
	Additional CDSM Column Row (8' o.c.)	110	3,955
	Liquefiable 1	110	179
	Liquefiable 2	110	1,116
	Non-Liquefiable Soil	110	300
	Tangential CDSM Column Row (3' o.c.) and 2nd Row (8' o.c.)	110	6,867

Sliding Block Failure with Ground Improvement
Model with Soil Loss - Undrained Strength for Non-Liq
Tangential 2nd Row (8 feet o.c.)
Fremont Apartments
Portland, OR

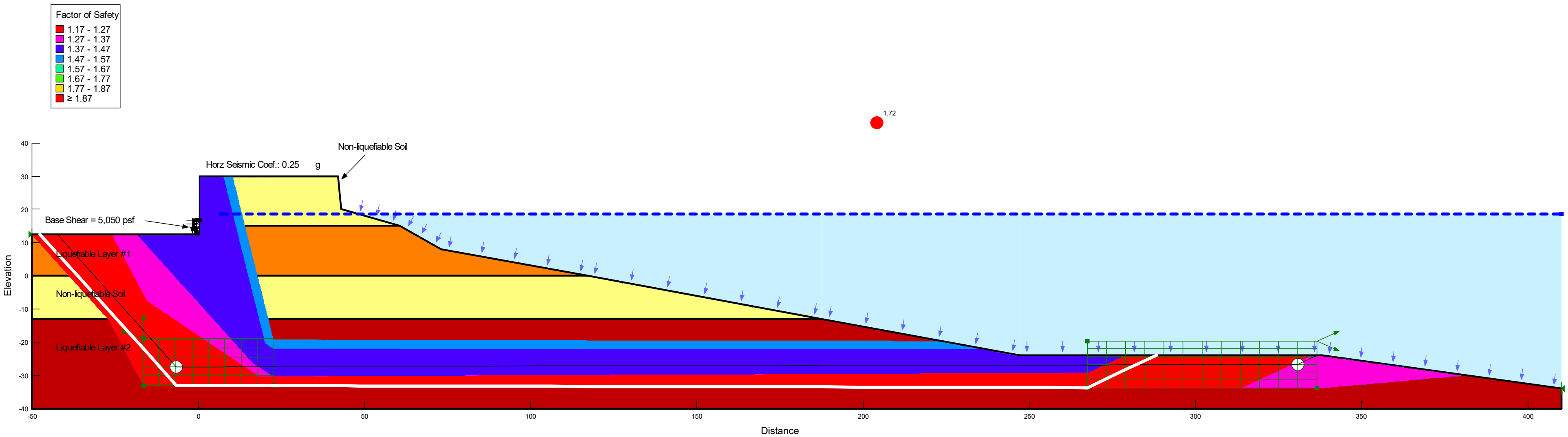
Reference #187, 188, 198



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)
	Additional CDSM Column Row (8' o.c.)	110			3,955
	Liquefiable 1	110			179
	Liquefiable 2	110			1,116
	Non-Liquefiable Soil	110	100	28	
	Tangential CDSM Column Row (3' o.c.) and 2nd Row (8' o.c.)	110			6,867

Sliding Block Failure with Ground Improvement
Evaluation of Liquefied Layer #2
Tangential 2nd Row (8 feet o.c.)
Fremont Apartments
Portland, OR

Reference # 187, 188, 198



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)
	Additional CDSM Column Row (8' o.c.)	110			3,955
	Liquefiable 1	110			179
	Liquefiable 2	110			1,116
	Non-Liquefiable Soil	110	100	28	
	Tangential CDSM Column Row (3' o.c.) and 2nd Row (8' o.c.)	110			6,867

Sliding Block Failure with Ground Improvement
Evaluation of Liquefied Layer #2
Tangential 2nd Row (8 feet o.c.)
Fremont Apartments
Portland, OR