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# Deep Soil Mixing Ground Improvement Design at West Slope
## 1815 Clement Avenue
## Alameda, California

**Submitted to:**

**Alameda Marina, LLC**
**1815 Clement Avenue**
**Alameda, California 94501**

**Submitted by:**
**Keller North America**

**June 30, 2020**

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Keller North America 16141005
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Alameda Marina, LLC
1815 Clement Avenue
Alameda, California 94501

Attention:

Subject: Alameda Marina
1815 Clement Avenue
Alameda, California
Deep Soil Mixing Ground Improvement Design at West Slope

Keller North America (Keller) is pleased to present the design for ground improvement at West Slope of the proposed shoreline improvements at Alameda Marina site in Alameda, California. The purpose of the ground improvement program is to enhance the safety, stability and serviceability of the proposed improvements. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated raise-in-grade under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of Keller and our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (805) 933-1331 with any questions, comments or concerns.

Respectfully submitted,

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## 1. Executive Summary

This project site is located in 1815 Clement Avenue, Alameda, CA.

**The West Slope** extends along the waterfront from the northwest corner of the site approximately 350 feet to the southeast to the existing Building 20, with an existing grade varying from Elevation 8.9 to 9.6 feet and the mudline outboard of the slope is at approximately Elevation -7 feet. Riprap of mixed materials, including boulders and chunks of concrete, is the existing protection of the slope and the asphalt-paved parking lots and a concrete walkway behind the slope.

According to the project geotechnical report prepared by Rockridge Geotechnical, Inc, the subsurface soils at this site and in their present condition is anticipated to be unstable as a results of new fill loads. Based on the geotechnical recommendations by Rockridge Geotechnical, Inc., Keller would employ deep soil mixing technologies to support the proposed work at West Slope.

**Deep Soil Mixing (DSM)**

DSM construction involves using high torque equipment to mechanically mix grout with native soils to create a nearly homogeneous mixture of weak concrete called soilcrete. DSM is a top down construction technique. As the mixing tool is advanced into the soil, grout slurry is pumped through the hollow stem of the shaft and injected into the soil at the tip and through the tool. The auger flights and mixing blades on the tool blend the soil with grout in pug-mill fashion. When the design depth is reached, the tool is withdrawn to the surface. Left behind are stabilized soil mixed columns. Often predrilling can be using to simplify disposal of construction spoils and waste soil. Depending on project requirements DSM can be used to improve ~10% to ~90% of the soil in a given area.

At the time of this design report, Keller has not competed a lab mixing test program. Based on our experience with similar soils, a conservative cement dosage of 225 kg/m3 has been selected to achieve the target design 28-days UCS of **150 psi** as stated in Section 4 below.

Figure 1: Construction of DSM

## 2. Ground Improvement Design Basic

This design is based on Keller's understanding of the project. Our understanding of the project is based on our review of the following documents and performance requirements articulated by the project structural engineer and geotechnical engineer. Although many documents were reviewed, only those which provided information that directly affects our design are listed below.

- Alameda Marina Shoreline Improvements, Geotechnical Design Criteria Report, Alameda California, dated June 7, 2019, prepared by Rockridge Geotechnical, Inc.
- Final Geotechnical Investigation Phase II Development – Townhomes, Alameda Marina, Alameda, California, dated January 29, 2020, prepared by Rockridge Geotechnical, Inc.
- Alameda Marina - Prelim Analysis for Keller, dated April 24, 2020, provided by Rockridge Geotechnical, Inc.
- Shear Wave Velocity Plots_to Keller_20200515, dated May 15, 2020, provided by Rockridge Geotechnical, Inc.
- MSM Grading Files, dated April 27, 2020, provided by CBG, Inc.
- Alameda Marina – The Launch, Development Plan and Design review, dated September 30, 2019, prepared by BDE Architecture
- AIA A104 (Design-Build) MSM Alameda Marina DRAFT Standard Contract
- Keller General Terms and Conditions

If any of these documents are changed or altered in any way, Keller should be notified, and the design may require modifications.

### 1.1 Performance Requirements

The anticipated performance criteria are articulated by the project geotechnical engineer and summarized below. Keller was not a party to the selection of the design and performance criteria. As part of our professional due diligence, we have confirmed that these values are reasonable for a project of this type in this location.

Table 1: Performance Criteria

| | |
|---|---|
| **Design earthquake** | PGA=0.65g, Mw=7.33 |
| **Factor Safety against Global Stability in static** | >1.5 |

### 1.2 Subsurface Conditions

According to project geotechnical report prepared by Rockridge Geotechnical, Inc, we understand the subsurface conditions to generally consists of artificial fill composed of predominately granular upper fill, and clayey (potentially dredged) sands/silts overlying a transitional zone of bay deposits (weaker more compressible clays); the Temescal Formation (medium stiff to stiff clay deposits with varying amounts of sand), and the Merritt Sand deposits (dense to very dense, well-sorted and -drained, fine sands). The deepest geologic formation encountered on the site is an alluvial deposit called the San Antonio Formation which is composed predominately of clay, sand and silt with occasional lenses of gravel.

Additionally, we understand the site to be "underlain by remnants of old timber structures, foundation elements, abandoned utilities, riprap, and concrete rubble. Numerous driven timber piles that supported former piers, cranes, and bulkheads were reportedly broken off at the mud line and still remain in place. Large timber cribbing and driven timber piles were reportedly left in place prior to filling of the former graving (dry) docks in the central portion of the site. However, we do not know the exact locations and orientations of these piles. There are also buried "deadman" anchors that provide lateral restraint to existing (or former) bulkhead structures along the waterfront.

### 3. DSM Design

Keller will design and install DSM panels at Slope 2.

The DSM panel consists of 5 overlapped 6-foot-diameter DSM columns. The length of each DSM panel is 25 feet and the spacing between two panels is 20 feet. The DSM panels are design to installed from elevation +9 to the top of Clay 2 defined by Rockridge Geotechnical, Inc.

Please refer to Keller shop drawing for layout of the DSM panels at West Slope. The target average 28-days unconfined compressive strength are around 150 psi. The actual unconfined compressive strength is pending verification by laboratory results.

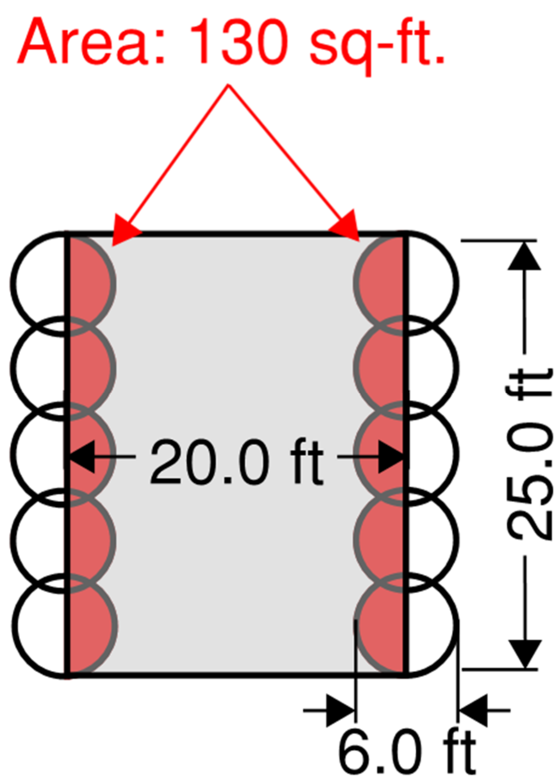



Figure 2: Illustration of a DSM Panel Design Unit

## 4. Slope Stability Analysis

Keller has performed slope stability analysis at West Slope 2 with DSM panels in place, using commercially available software Slide 2018. The geometry and soil properties, excluding the DSM zone, used in Keller's slope stability analysis developed from the SLOPE/W files provided by Rockridge Geotechnical, Inc., dated April 24, 2020.

Keller has made following adjustment to the DSM zone in the analysis:

a. Front edge of the DSM zone is 5 feet away from the outside edge of the existing concrete walkway
b. The composite shear strength of the DSM zone is computed based on Equation 1

$$\tau_{comp} = ARR \times \frac{UCS}{2} + (1 - ARR) \times \tau_{soil} \qquad \text{Equation 1}$$

where $\tau_{comp}$ is the composite shear strength of DSM zone. Please refer to Appendix B for detailed computation of composite shear strength of DSM zone used in global stability analysis.

Based on the liquefaction analysis conducted by Rockridge Geotechnical, Keller produces Figure 3 below illustrating layers susceptible to liquefaction at West Slope. The N60 correlated from CPT-16c and 17 corresponds to an estimated residual strength of 100 psf, according to Figure 4.

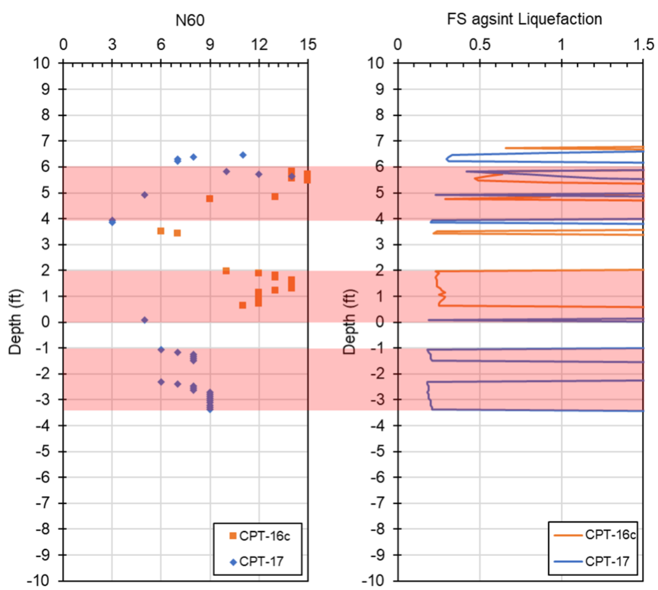



Figure 3: Liquefaction Layers at West Slope

In Keller's seismic slope stability analysis, the three layers of liquefied soil with residual strength of 100 psf have been included according to Figure 3. In the slope stability analysis, the liquefied layers only extend to 100 feet behind the edge of west slope since liquefaction was not presented in CPT-3 and 6, which is about 100 feet away from the edge of west slope. Please refer to Appendix A for detailed geometries.

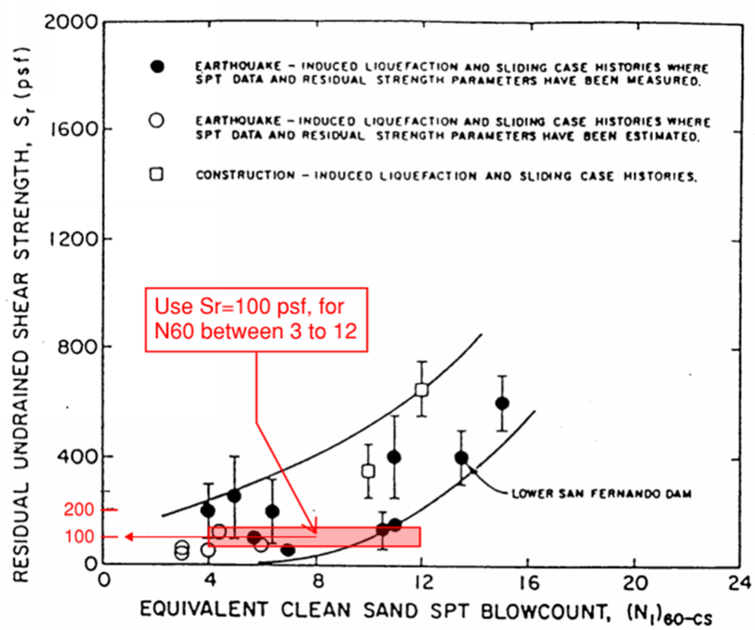



Figure 4: Relationship between Residual Strength and Corrected "Clean Sand" SPT Blowcount ($N_{160}$) from Case Histories (after Seed and Harder, 1990)

Table 2 below has summarized the slope stability results, for detailed input and output, please refer to Appendix A of this submittal file.

Table 2: Summary of Slope Stability Results

| | Janbu Corrected | Spencer | Morgenstern-Price | Average |
|---|---|---|---|---|
| Static with DSM | FS=2.474 | FS=2.436 | FS=2.442 | FS=2.45 |
| Seismic with DSM | Ky=0.233 | Ky=0.230 | Ky=0.230 | Ky=0.23 |

## 5. Static Stability Check of DSM

Please refer to Appendix B for detailed computation results.

## 6. Seismic Slope Deformation

Please refer to Appendix C for detailed computation results.

# 7. DSM Construction

## 7.1 Layout

Keller will provide an AutoCAD shop drawing for each DSM panels coordinates overlaid on the site Civil drawing. Keller understands that general contractor will be responsible and use a licensed surveyor to provide Keller with controlled points and survey benchmarks, and it is Keller's responsibility to locate every DSM panels before installation and will prepare as-built drawings after completion. DSM panels will be installed within 6 inches of the design locations as shown in the Keller shop drawing.

Keller requires the same surveyor to locate the verification coring locations.

## 7.2 Sequence of work

Once a stable working platform has been established as shown in Keller shop drawing. DSM panels will be performed sequentially based on the site superintendent and the field engineer.

## 7.3 Predrill

To minimize the mixing tool damage and maintaining soil mixing quality, Keller may pre-dill holes or excavate for better mixing quality. The holes will be filled with soilcrete up to the working elevation during the mixing stage.

## 7.4 Soil Mixing

In general, soil mixing operation parameters, such as mixing shaft speed, penetration rate, batching grout specific gravity and pumping speed will be determined based on our lab mixing result and our past experience, and will be fine-tuned at the beginning of mixing column production. The design cement content in place (cement weight/[soil volume + grout volume]) will start from approximately 225 kg/m3 with grout slurry specific gravity of 1.45. Keller engineers may adjust the cement content based on the field sample strength development.

### 7.4.1 Vertical Alignment

Vertical alignment of the mix tool stroke will be controlled by the drill rig operator. Two measurements of verticality will be monitored. These are the fore-aft and left-right vertical mast positions. Verticality will be measured by a level as measured on the mix tool prior to penetration. Intermittent measurements will be made as may be necessary during mixing operations.

### 7.4.2 Mixing shaft speed

The mixing shaft speed which is anticipated to be ranging between 20-60 RPM, and shall be adjusted to accommodate a constant rate of mixing shaft penetration based on the degree of drilling difficulty. The mixing shaft speed can be adjusted according to drilling difficulty. The mixing shaft speed can be adjusted to aid mixing of the soil column when needed or to assist penetration in hard drilling. Mixing shaft speed will be recorded.

### 7.4.3 Penetration Rate

In order to ensure adequate mixing, the penetration rate of the mixing shaft shall be maintained at about from 1.0 to 2.0 feet/minute during penetration. The bottom of the columns shall be double mixed by raising the mixing shaft 3 ft off the bottom and then reinserting them for remixing. The penetration rate and maximum depth of each stroke shall be recorded by Keller data acquisition system.

#### 7.4.4 Grout Take

The grout slurry flow per vertical foot of column will be adjusted to the requirements of the design mix. Progressive cavity pumps will be used to transfer the grout from the mix plant to the mixing rig. Flow monitoring devices will be installed in the grout line to detect any line blockage and monitor flow, total injected grout per column, and grout pressure. These parameters will be recorded.

Inevitably some variations of the grout take will occasionally occur due to field conditions. However, the overall application rate to each stroke can be monitored, calculated and controlled. Additional mixing will be used when necessary to evenly distribute the grout through the entire column. The injection of grout at each stroke will be monitored, checked by calculation and recorded. It is anticipated that a grout flow rate between 100 to 150 GPM will be used during penetration. If the mixing tool penetration below 1.5 ft/min, the grout flow rate will be reduced accordingly, according to the on-board computer display.

#### 7.4.5 Withdrawal and Remixing/Re-strocking Rate

The mixing shaft will be withdrawn at a rate of 8 to 10 foot per minute during the re-stroke operation and complete removal of the mixing shaft from the ground thus mixed.

#### 7.4.6 Obstruction/ Mixing Shaft Refusal

Keller will use a data acquisition system to monitor the mixing shaft penetration and the shaft rotation resistant in terms of the hydraulic pressure. Keller will set up the penetration criteria based on the site measurement.

### 7.5 Soil Mixing

Cement: Cement will be furnished by Keller and conform to ASTM C150 "Standard Specification for Portland Cement," Type II/V, or equivalent. The cement will be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter will not be used.

Water: Water for the slurry will be fresh, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, furnished by the owner.

### 7.6 Equipment

#### 7.6.1 Batching Equipment

The batch plant shall consist of in-line eductor (jet valve) mixers. Dry materials shall be stored in tankers and/or silos and fed to the mixers for shearing and circulation. The resulting grout slurry will be transferred to a surge tank for continuous agitation and to supply the in-situ soil mixing rig. Grout slurry quality will be assured by frequent testing prior to injection into the soil.

#### 7.6.2 Mixing Equipment

Single shaft mixing equipment that mechanically mixes the soil and cement slurry for the full dimensions of the column will be used for the Work. We anticipate using hydraulic drill rigs for the soil mixing operations. This rig is capable of up to >176,000 ft-lbs of torque at <20 rpm. Working shaft rate of rotation ranges between 7 and 55 rpm. The mixing shaft will have mixing augers and/or blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and cement slurry. The power source for driving the mixing shafts will be sufficient to maintain the required mix tool (shaft) rotation speed in revolutions per minute and penetration/ withdrawal rates from the ground surface to the maximum depth required. The design target Blade Rotation Number (BRN, defined as number of blade cut in each 1.0 meter soil) will be at least 500.

The DSM equipment will be equipped with devices to assure vertical alignment in two planes (90 degrees in plan from each other): fore-aft and left-right. The DSM equipment will be equipped with real time display of depth, rotation speed, grout flow rate; cumulative grout injected and grout pressure for each soil mix column. The cement will be mixed with water within the jet valve to create a 1.45 specific gravity mix +/- .03.

#### 7.6.3 Pumping Equipment

Grout slurry will be supplied to the drill using large size Moyno pumps. These pumps will be sized and powered so that design volumes and pressures can be maintained up to 300 ft away from the batching facility. It is anticipated that a continuous grout slurry flow of 150 gallons per minute at 100 psi to the drill rig will be necessary

#### 7.6.4 Equipment Location

The batching and pumping facility will be set up central to both in situ soil mixing areas. This will eliminate the need to move the plant once it is established.

## 8. DSM QA/QC Program

Following installation of DSM panels, verification testing will include:

- Unconfined compressive test on wet soils mixed samples
- Unconfined compressive test on cored samples
- Review of production DAQ logs

### 8.1 Wet Soils Mixed Samples

Wet Soil mix samples will be retrieved and cast into molds for one cell per rig/shift, at one random depth, typically near the end of each shift. Samples will be retrieved using an in situ wet sampler immediately after cell construction and shall consist of no fewer than 8 specimens. Soil clods greater than 10% of the mold diameter will be screened off. Appropriate curing techniques shall be implemented until testing based on ASTM D 1632.

Unconfined compression testing shall be performed by an approved laboratory working directly for the owner in pairs of specimens at 7 days. If the 7-days specimens do not reach the desired strength according the lab test curve, another pair of specimens will be tested at 14 days, 28 days, and if needed at 56 days. All specimens at 28 days and available 56-days of age will be tested and used in the statistical calculation. The Unconfined Compressive Strength (UCS) shall be determined by ASTM D1633 "Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders". Sulfur caps shall be required in the UCS tests to minimize the end effects to the test specimen. The advantage of the wet sampling is that Keller can get an early trend of the soilcrete strength development without waiting to the end of the project for coring, and can make early decisions in the field program to add additional soil mixing cells if necessary.

### 8.2 Core Samples

Additionally, Keller will core FOUR (4) DSM panels. All core locations are randomly selected, to collect core samples for unconfined compression testing. Double-tubes coring method, with utilization of vibrators to assist the core to depth can be used in lieu of conventional coring technique. Keller anticipates 4 specimens trimmed from each core hole, and a total of 16 UCS tests by ASTM D1633.

Uniformity of mixing shall be evaluated by the geotechnical engineer of record (GEOR) based on the continuous core samples recovered. The continuous core holes shall extend the entire depth of the DSM cell. An estimated recovery of 80 percent for each 5-foot-long segment of a boring and at least 90 percent when averaged over all core runs within a single boring shall be achieved. The lumps of unimproved soils shall not exceed 20 percent of the total volume of any 5-foot core segment from a boring. If the core recovery below the anticipated value due to the gravel particles in the soilcrete matrix, Keller shall be allowed to utilize downhole camera or other approved methods to verify the core hole.

Keller will calculate the average 28-day UCS value from all core samples and wet grab samples. A ceiling, not-to-exceed value of four times the average unconfined compressive strength (i.e., 600 psi) shall be used for individual specimens in calculating the average strength achieved in the field from each coring and wet sample and for the entire project. At the end of the project, to not unnecessary delay subsequent activities by waiting for 28-days test result, a correction of early strength gain will be used to approve the DSM work. However, this correlation will not relieve the contractor of the responsibility to achieve average 28-days strength of 150 psi. Based on FHWA (2013) guidelines, the following UCS aging factor correlations will be applied to this job:

- 28: 3 day, 1.72
- 28: 7 day, 1.35
- 28: 14 day, 1.15

A site-specific correlation between 3-days and 28-days strength may be used to supersede this correlation if in the opinion of the Engineer the site-specific correlation is more appropriate. No more than 10 percent of all specimens tested shall exhibit an unconfined compressive strength of less than 50 psi at 28 days.

### 8.3 Production DAQ Logs

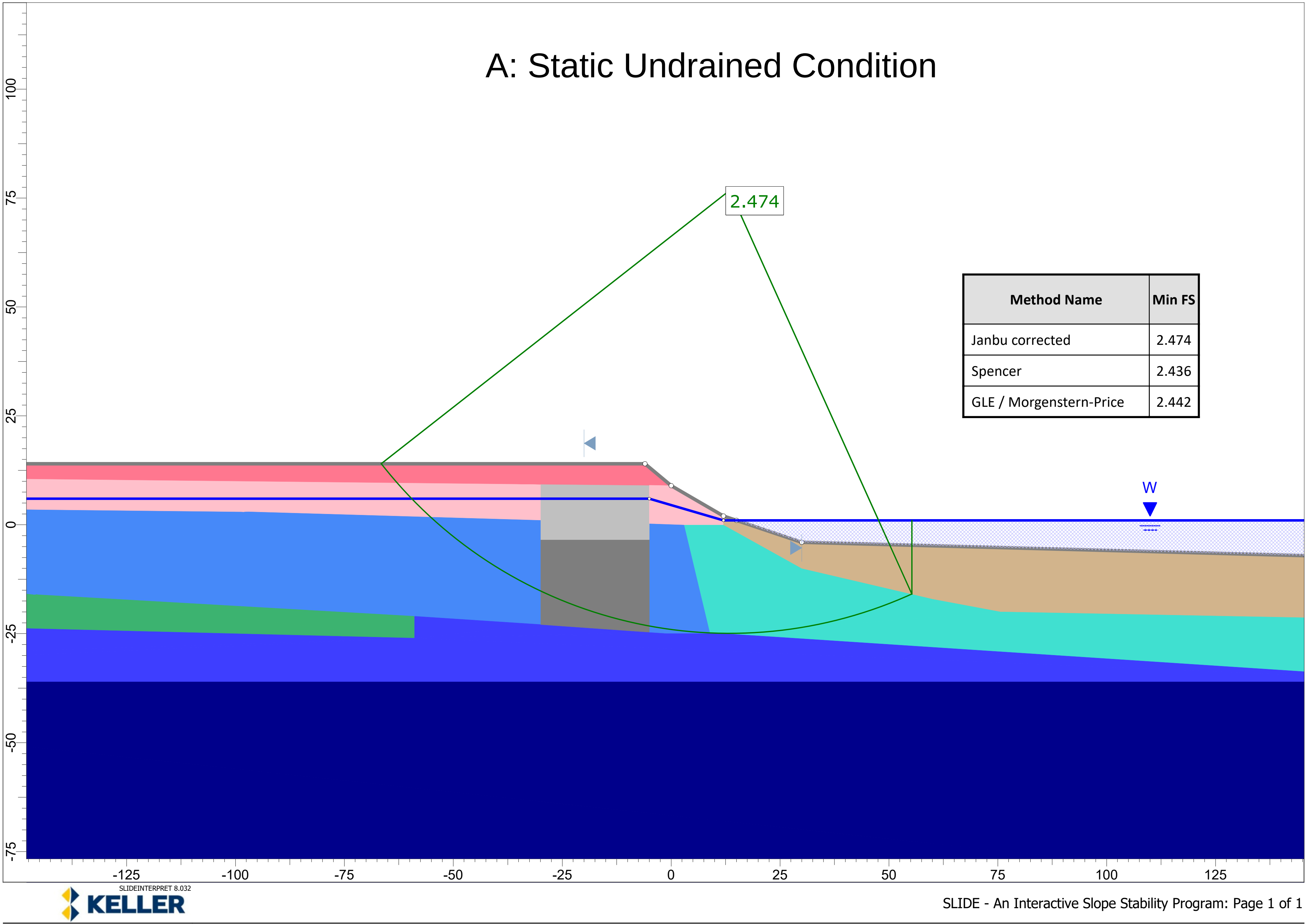
During the soil mixing production, Keller will review the wet soilcrete strength development as well as production cell mixing logs and may add additional soil mixing cells if the soilcrete strength is below the target average UCS values as listed above.

## 9. References

Bruce, Mary Ellen C., Ryan R. Berg, James G. Collin, George M. Filz, Masaaki Terashi, David S. Yang, and Sa Geotechnica. Federal Highway Administration design manual: Deep mixing for embankment and foundation support. No. FHWA-HRT-13-046. United States. Federal Highway Administation. Offices of Research & Development, 2013.

Filz, G. M., and E. Templeton. "Design guide for levee and floodwall stability using deep-mixed shear walls.”." New Orleans District and Hurricane Protection Office, US Army Corps of Engineers, Final Report Contract W912P8-07 (2011): 0031.

## Appendix A
## Slope Stability Results




| Method Name | Min FS |
|---|---|
| Janbu corrected | 2.474 |
| Spencer | 2.436 |
| GLE / Morgenstern-Price | 2.442 |

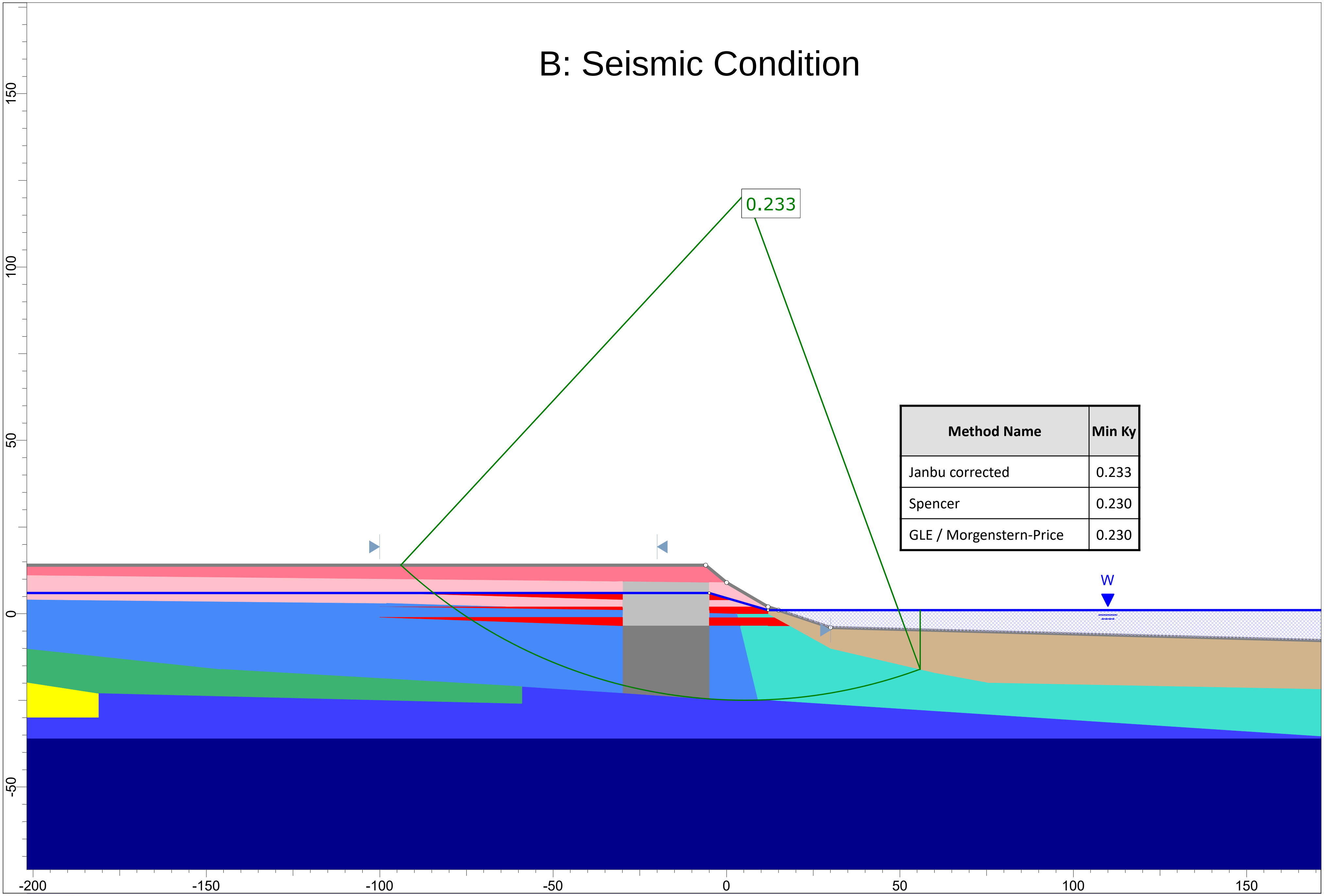
# Undrained Condition

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Undrained Cu | Undrained Cutoff |
|---|---|---|---|---|---|---|---|---|
| Engineering Fill (N) | | 130 | 130 | Mohr-Coulomb | 200 | 35 | | |
| Fill (E) | | 100 | 100 | Mohr-Coulomb | 100 | 30 | | |
| Bay Mud | | 95 | 95 | Undrained | 450 | | | |
| Bay Mud (Offshore) | | 95 | 95 | Vertical Stress Ratio | | | | |
| Clay 1 | | 120 | 120 | Drained-Undrained | | | 800 | 2000 |
| Clay 2 | | 120 | 120 | Vertical Stress Ratio | | | | |
| Clay 3 | | 120 | 120 | Vertical Stress Ratio | | | | |
| DSM-Fill | | 110 | | Undrained | 2500 | | | |
| DSM-Mud | | 110 | | Undrained | 3000 | | | |
| Merritt Sand | | 130 | 130 | Mohr-Coulomb | 200 | 41 | | |
| Recent Bay Deposits | | 84 | 84 | No strength | | | | |

-100 -200 -300 -400 -500 -600 -700 -800

-600 -500 -400 -300 -200 -100 0 100 200 300 400 500

# B: Seismic Condition




| Method Name | Min Ky |
|---|---|
| Janbu corrected | 0.233 |
| Spencer | 0.230 |
| GLE / Morgenstern-Price | 0.230 |

SLIDEINTERPRET 8.032

KELLER

# Seismic Condition

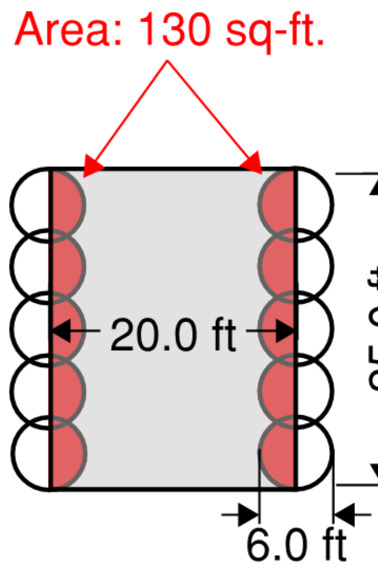
| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Cohesion (psf) | Phi (deg) | Undrained Cu | Undrained Cutoff |
|---|---|---|---|---|---|---|---|
| Engineering Fill (N) | | 130 | 130 | 200 | 35 | | |
| Fill (E) | | 100 | 100 | 100 | 30 | | |
| Bay Mud | | 95 | 95 | 450 | | | |
| Bay Mud (Offshore) | | 95 | 95 | | | | |
| Clay 1 | | 120 | 120 | | | 800 | 2000 |
| Clay 2 | | 120 | 120 | | | | |
| Clay 3 | | 120 | 120 | | | | |
| DSM-Fill | | 110 | | 2500 | | | |
| DSM-Mud | | 110 | | 3000 | | | |
| Merritt Sand | | 130 | 130 | 200 | 41 | | |
| Recent Bay Deposits | | 84 | 84 | | | | |
| Liquefied | | 110 | 110 | 100 | | | |

-100 -200 -300 -400 -500

-400 -300 -200 -100 0 100 200

# Appendix B
# Static Stability Check of DSM

## A: Soil Mixing Geometry and Properties




| | |
|---|---|
| $S_{dsm} := 20ft$ | Spacing between two SM panel |
| $B := 25ft$ | Width of SM Zone |
| $H_{dsm} := 33ft$ | Height of SM Zone |
| $\gamma_{dsm} := 110pcf$ | Total unit weight of SM Zone |
| $ARR := \frac{130ft^2}{S_{dsm} \cdot B} = 26 \cdot \%$ | Area replacement ratio of SM Zone |
| $b := ARR \cdot S_{dsm} = 5.2 \cdot ft$ | Equivalent width of SM panel |
| $W_{dsm} := B \cdot H_{dsm} \cdot \gamma_{dsm} = 90750 \cdot \frac{lbf}{ft}$ | Total weight of SM Zone |
| $UCS_{dsm} := 150psi$ | Design 28-day UCS for transverse DSM panels |
| $\tau_{liquefied} := 100psf$ | Liquefied soil strength above EL -3.5 |

$$\tau_{compU} := ARR \cdot \frac{UCS_{dsm}}{2} + (1 - ARR) \cdot \tau_{liquefied} = 2882 \cdot psf$$

Composite shear strength above EL -3.5

$$\tau_{soil4} := c_4 = 450 \cdot psf$$

Mid-point shear strength of soil Layer 4, refert to Figure B1

$$\tau_{compL} := ARR \cdot \frac{UCS_{dsm}}{2} + (1 - ARR) \cdot \tau_{soil4} = 3141 \cdot psf$$

Composite shear strength below EL -3.5

**1: Use composite shear strenght of 2500 psf above elevation -5**

2**: Use composite shear strenght of 3000 psf below elevation -5**

## B: Global Stability

**Refer to Section 4 of the submittal file**

## C: Check for Combined Overturning and Bearing Capacity

### C-1: Resultant Vetical Force

$H_w := 29.5ft$ Height of water pressure from bottom of Soil Mixing

$U := \gamma_w \cdot H_w \cdot B = 46020 \cdot \frac{lbf}{ft}$ Uplift water pressure at bottom of Soil Mixing

$N_R := W_{dsm} + Va - Vp - U = 37532 \cdot \frac{lbf}{ft}$

### C-2: Determine Location of Resultant Vetical Force

**Static Case:**

$$x'_N := \frac{P_p \cdot h_{Pp} + W_{dsm} \cdot \frac{B}{2} + Va \cdot B - P_a \cdot h_{Pa} - U \cdot \frac{B}{2}}{N_R} = 43 \cdot ft$$

Refer to Figure B1

**Seismic Case:**

$$x'_{SN} := \frac{P_p \cdot h_{Pp} + W_{dsm} \cdot \frac{B}{2} + Va \cdot B - P_a \cdot h_{Pa} - P_{Sa} \cdot h_{SPa} - U \cdot \frac{B}{2}}{N_R} = 41 \cdot ft$$

Refer to Figure B2

**Since the location of resultant vertical force is located outside of the soil mixing zone, the design is safe against overturning and bearing capacity failure in both static and seismic conditions, and no further analysis of this failure moe needs to be done (FHWA 2013)**

## D: Check for Sliding

**Static Case:**

$$\Sigma P_{static} := P_a - P_p = -45603 \cdot \frac{lbf}{ft}$$ Sum of Horizontal Forces

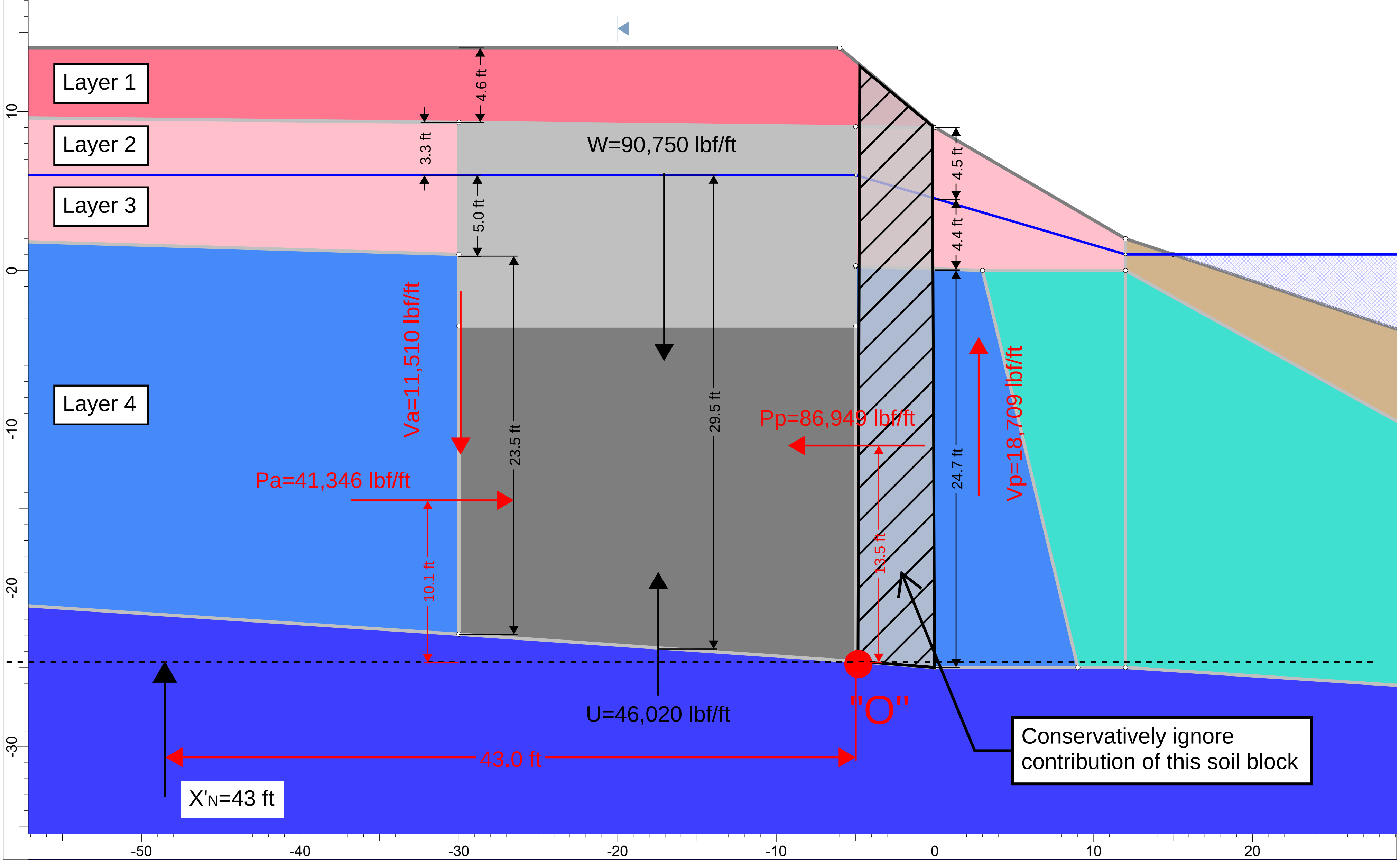
**The base shear of soil mixing will not be mobilized in static condition**

**Seismic Case:**

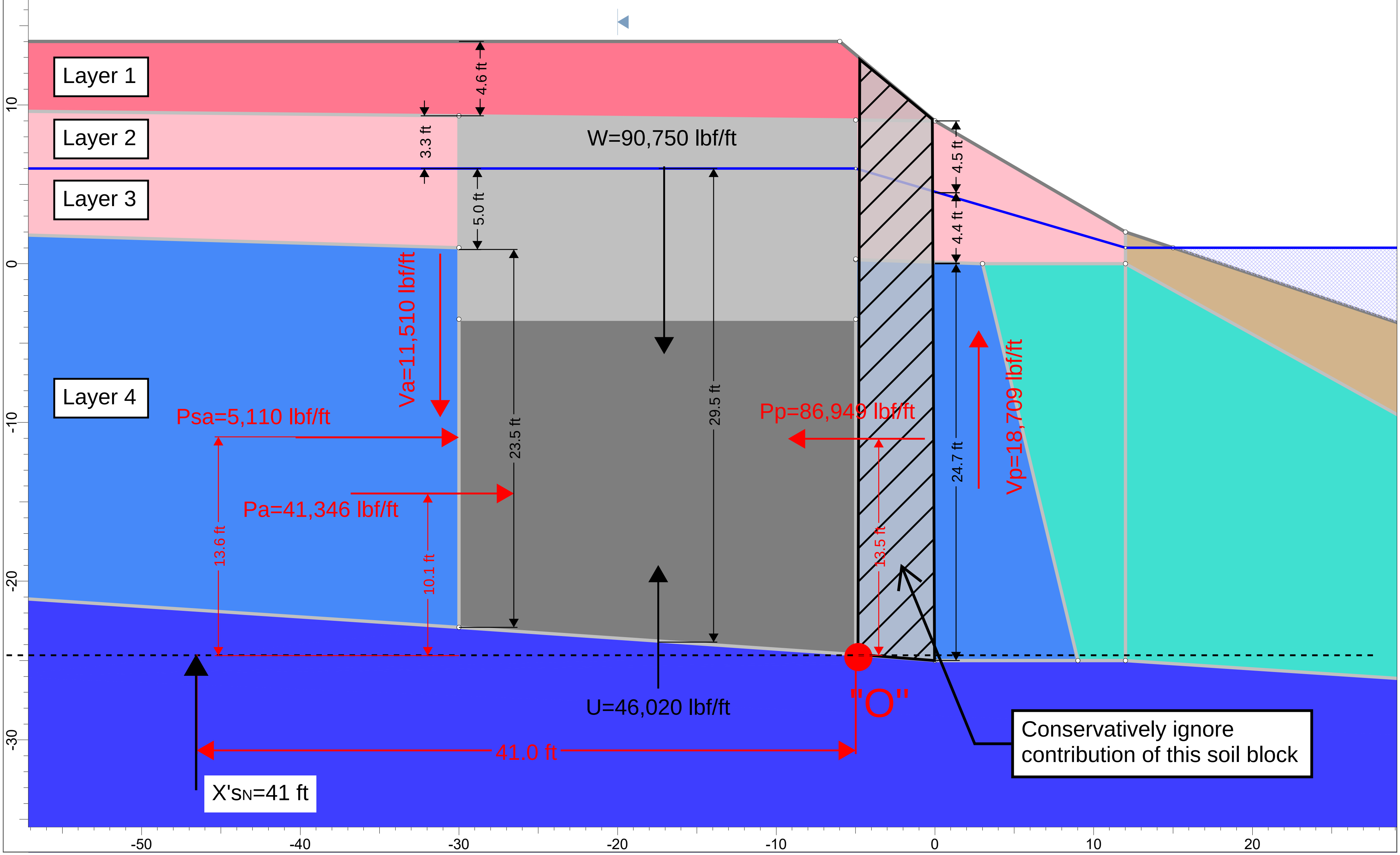
$$\Sigma P_{seismic} := \left(P_a + P_{Sa}\right) - P_p = -40493 \cdot \frac{lbf}{ft}$$ Sum of Horizontal Forces

**The base shear of soil mixing will not be mobilized in static condition**

# Figure B1: Combined Overturning and Bearing Capacity (Static)



# Figure B2: Combined Overturning and Bearing Capacity (Seismic)



## E: Extrusion of Liquefied Soil Between Soil Mixing Panels

Reference: Design Guide for Levee and Floodwall Stability using Deep-Mixed Shear Walls (2011)
FHWA: Deep Mixing for Embankment and Foundation Supprot (2013)

**The Liquefaction zone extends from EL. 6 to -3.5, refer to Figure B2 for geometry**

$H_e := 6ft - (-3.5ft) = 9.5 \cdot ft$ Thickness of soil clayer for extrusion check

$c_{soil} := \tau_{liquefied} = 100 \cdot psf$ Liquefied Shear Strength, refer to Figure B3

$\sigma_{vL} := 4.7ft \cdot 130pcf + 8.1ft \cdot 100pcf = 1421 \cdot psf$ Average value of the total vertical stress in soil layer to the LEFT of the DSM

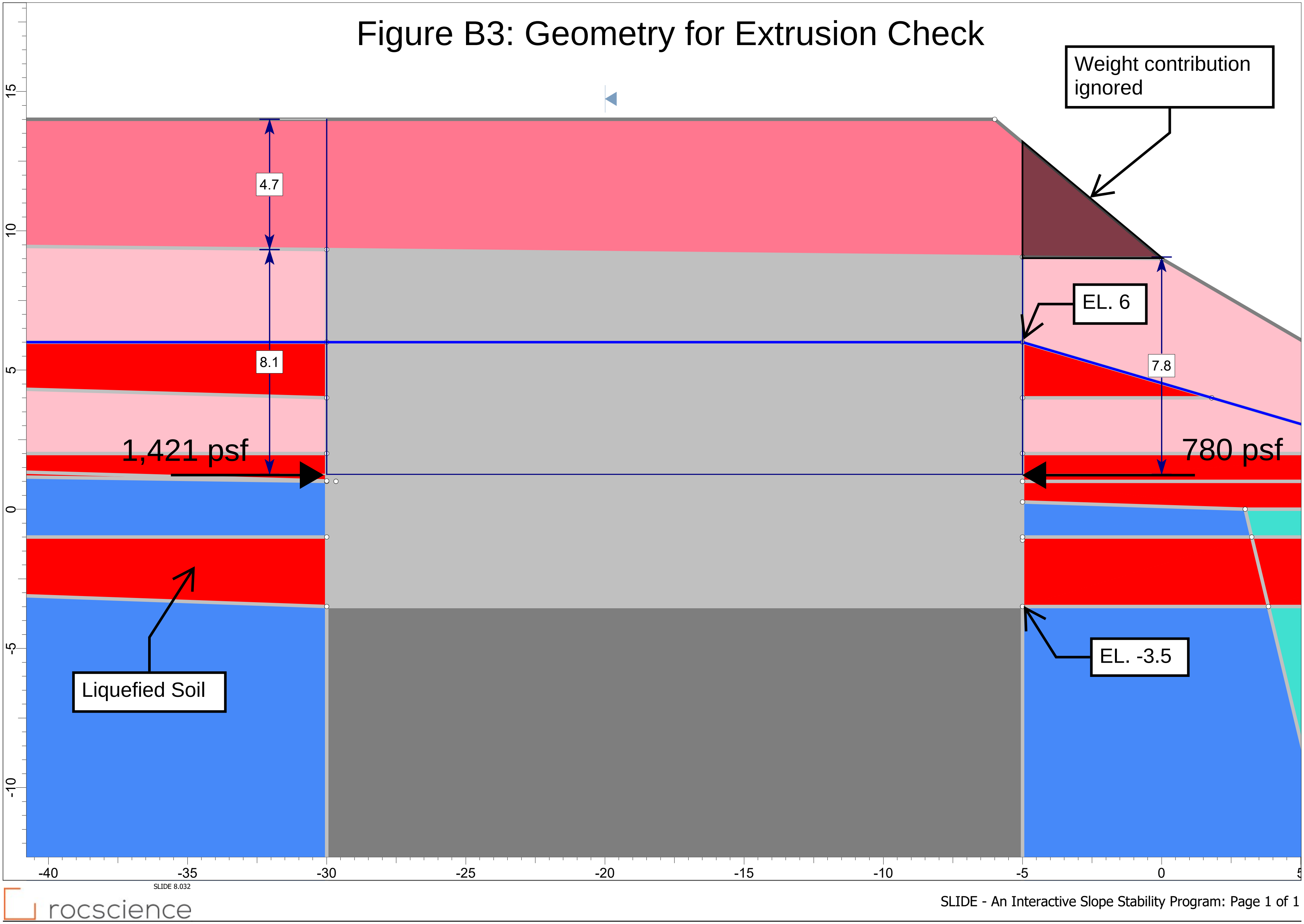
$\sigma_{vR} := 7.8ft \cdot 100pcf = 780 \cdot psf$ Average value of the total vertical stress in soil layer to the RIGHT of the DSM

$$F_{e1} := \frac{2 \cdot c_{soil} \cdot \left[2 + B \cdot \left(\frac{1}{S_{dsm} - b} + \frac{1}{H_e}\right)\right]}{\sigma_{vL} - \sigma_{vR}} = 2$$

$F_{e1} > 1.3$ **OK**

Note: According to Table 11 of FHWA (2013), suggested factor of safety against soil extrusion through shear panels is 1.3.

# Figure B3: Geometry for Extrusion Check



## Soil Properties:

$\gamma_w := 62.4\text{pcf}$

### Layer 1: Engineering Fill

$\gamma_1 := 130\text{pcf}$ $\phi_1 := 35\text{deg}$ $c_1 := 200\text{psf}$ $h_{L1} := 4.7\text{ft}$ $h_{R1} := 0\text{ft}$

$$k_{1a} := \tan\left(45\text{deg} - \frac{\phi_1}{2}\right)^2 = 0.271 \qquad k_{1p} := \tan\left(45\text{deg} + \frac{\phi_1}{2}\right)^2 = 3.69$$

### Layer 2: Existing Fill (above water)

$\gamma_2 := 100\text{pcf}$ $\phi_2 := 30\text{deg}$ $c_2 := 100\text{psf}$ $h_{L2} := 3.3\text{ft}$ $h_{R2} := 4.5\text{ft}$

$$k_{2a} := \tan\left(45\text{deg} - \frac{\phi_2}{2}\right)^2 = 0.333 \qquad k_{2p} := \tan\left(45\text{deg} + \frac{\phi_2}{2}\right)^2 = 3$$

### Layer 3: Existing Fill (below water)

$\gamma_3 := 100\text{pcf}$ $\phi_3 := 30\text{deg}$ $c_3 := 100\text{psf}$ $h_{L3} := 5\text{ft}$ $h_{R3} := 4.4\text{ft}$

$$k_{3a} := \tan\left(45\text{deg} - \frac{\phi_3}{2}\right)^2 = 0.333 \qquad k_{3p} := \tan\left(45\text{deg} + \frac{\phi_3}{2}\right)^2 = 3$$

### Layer 4: Bay Mud

$\gamma_4 := 95\text{pcf}$ $\phi_4 := 0\text{deg}$ $c_4 := 450\text{psf}$ $h_{L4} := 23.5\text{ft}$ $h_{R4} := 24.7\text{ft}$

$$k_{4a} := \tan\left(45\text{deg} - \frac{\phi_4}{2}\right)^2 = 1 \qquad k_{4p} := \tan\left(45\text{deg} + \frac{\phi_4}{2}\right)^2 = 1$$

## Active Earth Pressure on the Left Side of Soil Mixing

$\sigma_{Ah1top} := 0 - 2 \cdot c_1 \cdot \sqrt{k_{1a}} = -208 \cdot psf$ use 20 psf

$\sigma_{Ah1bot} := \gamma_1 \cdot h_{L1} \cdot k_{1a} - 2 \cdot c_1 \cdot \sqrt{k_{1a}} = -43 \cdot psf$ use 20 psf

$$\sigma_{Ah2top} := \gamma_1 \cdot h_{L1} \cdot k_{2a} - 2 \cdot c_2 \cdot \sqrt{k_{2a}} = 88 \cdot psf$$

$$\sigma'_{Ah2top} := \sigma_{Ah2top} = 88 \cdot psf$$

$$\sigma_{Ah2bot} := \left(\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2}\right) \cdot k_{2a} - 2 \cdot c_2 \cdot \sqrt{k_{2a}} = 198 \cdot psf$$

$$\sigma'_{Ah2bot} := \sigma_{Ah2bot} = 198 \cdot psf$$

$$\sigma_{Ah3top} := \left(\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2}\right) \cdot k_{3a} - 2 \cdot c_3 \cdot \sqrt{k_{3a}} = 198 \cdot psf$$

$$\sigma'_{Ah3top} := \sigma_{Ah3top} = 198 \cdot psf$$

$$\sigma_{Ah3bot} := \left[\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2} + \left(\gamma_3 - \gamma_w\right) \cdot h_{L3}\right] \cdot k_{3a} - 2 \cdot c_3 \cdot \sqrt{k_{3a}} + \gamma_w \cdot h_{L3} = 573 \cdot psf$$

$$\sigma'_{Ah3bot} := \left[\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2} + \left(\gamma_3 - \gamma_w\right) \cdot h_{L3}\right] \cdot k_{3a} - 2 \cdot c_3 \cdot \sqrt{k_{3a}} = 261 \cdot psf$$

$$\sigma_{Ah4top} := \left(\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2} + \gamma_3 \cdot h_{L3}\right) \cdot k_{4a} - 2 \cdot c_4 \cdot \sqrt{k_{4a}} = 541 \cdot psf$$

$$\sigma_{Ah4bot} := \left(\gamma_1 \cdot h_{L1} + \gamma_2 \cdot h_{L2} + \gamma_3 \cdot h_{L3} + \gamma_4 \cdot h_{L4}\right) \cdot k_{4a} - 2 \cdot c_4 \cdot \sqrt{k_{4a}} = 2774 \cdot psf$$

## Seismic Increment

$$\sigma_{SAh1top} := 0$$

$$\sigma_{SAh1bot} := h_{L1} \cdot \left(14 \frac{psf}{ft}\right) = 66 \cdot psf$$

$$\sigma_{SAh2top} := \sigma_{SAh1bot} = 66 \cdot psf$$

$$\sigma_{SAh2bot} := \left(h_{L1} + h_{L2}\right) \cdot \left(14 \frac{psf}{ft}\right) = 112 \cdot psf$$

$$\sigma_{SAh3top} := \sigma_{SAh2bot} = 112 \cdot psf$$

$$\sigma_{SAh3bot} := \left(h_{L1} + h_{L2}\right) \cdot \left(14 \frac{psf}{ft}\right) + h_{L3} \cdot \left(4 \frac{psf}{ft}\right) = 132 \cdot psf$$

$$\sigma_{SAh4top} := \sigma_{SAh3bot} = 132 \cdot psf$$

$$\sigma_{SAh4bot} := \left(h_{L1} + h_{L2}\right) \cdot \left(14 \frac{psf}{ft}\right) + \left(h_{L3} + h_{L4}\right) \cdot \left(4 \frac{psf}{ft}\right) = 226 \cdot psf$$

## Active Resultant Forces and Locations

$$P_{a2} := \frac{\sigma_{Ah2top} + \sigma_{Ah2bot}}{2} \cdot (h_{L2}) = 473 \cdot \frac{lbf}{ft}$$

Resultant force 2 in active side

Location of resultant force 2 relative to "O"

$$h_{Pa2} := \frac{\sigma_{Ah2top} \cdot h_{L2} \cdot \left(\frac{h_{L2}}{2} + h_{L3} + h_{L4}\right) + (\sigma_{Ah2bot} - \sigma_{Ah2top}) \cdot \frac{h_{L2}}{2} \cdot \left(\frac{h_{L2}}{3} + h_{L3} + h_{L4}\right)}{P_{a2}} = 30 \cdot ft$$

$$P_{a3} := \frac{\sigma_{Ah3top} + \sigma_{Ah3bot}}{2} \cdot (h_{L3}) = 1928 \cdot \frac{lbf}{ft}$$

Resultant force 3 in active side

Location of resultant force 3 relative to "O"

$$h_{Pa3} := \frac{\sigma_{Ah3top} \cdot h_{L3} \cdot \left(\frac{h_{L3}}{2} + h_{L4}\right) + (\sigma_{Ah3bot} - \sigma_{Ah3top}) \cdot \frac{h_{L3}}{2} \cdot \left(\frac{h_{L3}}{3} + h_{L4}\right)}{P_{a3}} = 26 \cdot ft$$

$$P_{a4} := \frac{\sigma_{Ah4top} + \sigma_{Ah4bot}}{2} \cdot (h_{L4}) = 38945 \cdot \frac{lbf}{ft}$$

Resultant force 4 in active side

Location of resultant force 4 relative to "O"

$$h_{Pa4} := \frac{\sigma_{Ah4top} \cdot h_{L4} \cdot \left(\frac{h_{L4}}{2}\right) + (\sigma_{Ah4bot} - \sigma_{Ah4top}) \cdot \frac{h_{L4}}{2} \cdot \left(\frac{h_{L4}}{3}\right)}{P_{a4}} = 9 \cdot ft$$

$$P_a := P_{a2} + P_{a3} + P_{a4} = 41346 \cdot \frac{lbf}{ft}$$

Totla resultant force in active side

$$h_{Pa} := \frac{P_{a2} \cdot h_{Pa2} + P_{a3} \cdot h_{Pa3} + P_{a4} \cdot h_{Pa4}}{P_a} = 10.1 \cdot ft$$

Location of total resultant force relative to "O"

Vertical shear force in active side

$$Va := \frac{\sigma'_{Ah2top} + \sigma'_{Ah2bot}}{2} \cdot \tan(\phi_2) \cdot h_{L2} + \frac{\sigma'_{Ah3top} + \sigma'_{Ah3bot}}{2} \cdot \tan(\phi_3) \cdot h_{L3} + c_4 \cdot h_{L4} = 11510 \cdot \frac{lbf}{ft}$$

## Seismic Increment

$$P_{Sa2} := \frac{\sigma_{SAh2top} + \sigma_{SAh2bot}}{2} \cdot (h_{L2}) = 293 \cdot \frac{lbf}{ft}$$

Resultant seismic force 2 in active side

Location of resultant seismic force 2 relative to "O"

$$h_{SPa2} := \frac{\sigma_{SAh2top} \cdot h_{L2} \cdot \left(\frac{h_{L2}}{2} + h_{L3} + h_{L4}\right)}{P_{Sa2}} \ldots + \frac{(\sigma_{SAh2bot} - \sigma_{SAh2top}) \cdot \frac{h_{L2}}{2} \cdot \left(\frac{h_{L2}}{3} + h_{L3} + h_{L4}\right)}{P_{Sa2}} = 30 \cdot ft$$

$$P_{Sa3} := \frac{\sigma_{SAh3top} + \sigma_{SAh3bot}}{2} \cdot (h_{L3}) = 610 \cdot \frac{lbf}{ft}$$

Resultant seismic force 3 in active side

Location of resultant seismic force 3 relative to "O"

$$h_{SPa3} := \frac{\sigma_{SAh3top} \cdot h_{L3} \cdot \left(\frac{h_{L3}}{2} + h_{L4}\right) + (\sigma_{SAh3bot} - \sigma_{SAh3top}) \cdot \frac{h_{L3}}{2} \cdot \left(\frac{h_{L3}}{3} + h_{L4}\right)}{P_{Sa3}} = 26 \cdot ft$$

$$P_{Sa4} := \frac{\sigma_{SAh4top} + \sigma_{SAh4bot}}{2} \cdot (h_{L4}) = 4207 \cdot \frac{lbf}{ft}$$

Resultant seismic force 4 in active side

Location of resultant seismic force 4 relative to "O"

$$h_{SPa4} := \frac{\sigma_{SAh4top} \cdot h_{L4} \cdot \left(\frac{h_{L4}}{2}\right) + (\sigma_{SAh4bot} - \sigma_{SAh4top}) \cdot \frac{h_{L4}}{2} \cdot \left(\frac{h_{L4}}{3}\right)}{P_{Sa4}} = 11 \cdot ft$$

$$P_{Sa} := P_{Sa2} + P_{Sa3} + P_{Sa4} = 5110 \cdot \frac{lbf}{ft}$$

Totla resultant seismic force in active side

$$h_{SPa} := \frac{P_{Sa2} \cdot h_{SPa2} + P_{Sa3} \cdot h_{SPa3} + P_{Sa4} \cdot h_{SPa4}}{P_{Sa}} = 13.6 \cdot ft$$

Location of total resultant seismic force relative to "O"

## Passive Earth Pressure on the Right Side of Soil Mixing

$$\sigma_{Ph2top} := 0 + 2 \cdot c_2 \cdot \sqrt{k_{2p}} = 346 \cdot psf$$

$$\sigma'_{Ph2top} := \sigma_{Ph2top} = 346 \cdot psf$$

$$\sigma_{Ph2bot} := \left(\gamma_2 \cdot h_{R2}\right) \cdot k_{2p} + 2 \cdot c_2 \cdot \sqrt{k_{2p}} = 1696 \cdot psf$$

$$\sigma'_{Ph2bot} := \sigma_{Ph2bot} = 1696 \cdot psf$$

$$\sigma_{Ph3top} := \left(\gamma_2 \cdot h_{R2}\right) \cdot k_{3p} + 2 \cdot c_3 \cdot \sqrt{k_{3p}} = 1696 \cdot psf$$

$$\sigma'_{Ph3top} := \sigma_{Ph3top} = 1696 \cdot psf$$

$$\sigma_{Ph3bot} := \left[\gamma_2 \cdot h_{R2} + \left(\gamma_3 - \gamma_w\right) \cdot h_{R3}\right] \cdot k_{3p} + 2 \cdot c_3 \cdot \sqrt{k_{3p}} + \gamma_w \cdot h_{R3} = 2467 \cdot psf$$

$$\sigma'_{Ph3bot} := \left[\gamma_2 \cdot h_{R2} + \left(\gamma_3 - \gamma_w\right) \cdot h_{R3}\right] \cdot k_{3p} + 2 \cdot c_3 \cdot \sqrt{k_{3p}} = 2193 \cdot psf$$

$$\sigma_{Ph4top} := \left(\gamma_2 \cdot h_{R2} + \gamma_3 \cdot h_{R3}\right) \cdot k_{4p} + 2 \cdot c_4 \cdot \sqrt{k_{4p}} = 1790 \cdot psf$$

$$\sigma_{Ph4bot} := \left(\gamma_2 \cdot h_{R2} + \gamma_3 \cdot h_{R3} + \gamma_4 \cdot h_{R4}\right) \cdot k_{4p} + 2 \cdot c_4 \cdot \sqrt{k_{4p}} = 4136 \cdot psf$$

## Passive Resultant Forces and Locations

$$P_{p2} := \frac{\sigma_{Ph2top} + \sigma_{Ph2bot}}{2} \cdot \left(h_{R2}\right) = 4596 \cdot \frac{lbf}{ft}$$

Resultant force 2 in passive side

Location of resultant force 2 relative to "O"

$$h_{Pp2} := \frac{\sigma_{Ph2top} \cdot h_{R2} \cdot \left(\frac{h_{R2}}{2} + h_{R3} + h_{R4}\right) + \left(\sigma_{Ph2bot} - \sigma_{Ph2top}\right) \cdot \frac{h_{R2}}{2} \cdot \left(\frac{h_{R2}}{3} + h_{R3} + h_{R4}\right)}{P_{p2}} = 31 \cdot ft$$

$$P_{p3} := \frac{\sigma_{Ph3top} + \sigma_{Ph3bot}}{2} \cdot \left(h_{R3}\right) = 9160 \cdot \frac{lbf}{ft}$$

Resultant force 3 in passive side

Location of resultant force 3 relative to "O"

$$h_{Pp3} := \frac{\sigma_{Ph3top} \cdot h_{R3} \cdot \left(\frac{h_{R3}}{2} + h_{R4}\right) + \left(\sigma_{Ph3bot} - \sigma_{Ph3top}\right) \cdot \frac{h_{R3}}{2} \cdot \left(\frac{h_{R3}}{3} + h_{R4}\right)}{P_{p3}} = 27 \cdot ft$$

$$P_{p4} := \frac{\sigma_{Ph4top} + \sigma_{Ph4bot}}{2} \cdot \left(h_{R4}\right) = 73192 \cdot \frac{lbf}{ft}$$

Resultant force 4 in passive side

Location of resultant force 4 relative to "O"

$$h_{Pp4} := \frac{\sigma_{Ph4top} \cdot h_{R4} \cdot \left(\frac{h_{R4}}{2}\right) + \left(\sigma_{Ph4bot} - \sigma_{Ph4top}\right) \cdot \frac{h_{R4}}{2} \cdot \left(\frac{h_{R4}}{3}\right)}{P_{p4}} = 11 \cdot ft$$

$$P_p := P_{p2} + P_{p3} + P_{p4} = 86949 \cdot \frac{lbf}{ft}$$

Totla resultant force in passive side

$$h_{Pp} := \frac{P_{p2} \cdot h_{Pp2} + P_{p3} \cdot h_{Pp3} + P_{p4} \cdot h_{Pp4}}{P_p} = 13.5 \cdot ft$$

Location of total resultant force relative to "O"

Vertical shear force in passive side

$$Vp := \frac{\sigma'_{Ph2top} + \sigma'_{Ph2bot}}{2} \cdot \tan\left(\phi_2\right) \cdot h_{R2} + \frac{\sigma'_{Ph3top} + \sigma'_{Ph3bot}}{2} \cdot \tan\left(\phi_3\right) \cdot h_{R3} + c_4 \cdot h_{R4} = 18709 \cdot \frac{lbf}{ft}$$

# Appendix C
# Seismic Slope Deformation

KNA has evaluated the seismic slope deformation using the method proposed by Bray and Travasarou (2007). Per procedure stated in Bray and Travasarou (2007), the Yield Coefficient (Ky) and Initial Fundamental Period (Ts) of the slope are computed as follows:

Ky = 0.23, refer to Table 2 and Figure C-2

$$T_s = \frac{4 \times H}{V_s} = 0.1956\ s \qquad \text{Equation C-1}$$

where Vs = 450 ft/s (Figure C-1) is the weight-average shear wave velocity within H = 22 ft based on the design Vs profile correlated from CPT results (CPT-3, 6, 16C, 17 and 19) and MASW Data (M-1, 2, 3, 4 and 5). Please refer to Figure C-2 for slope geometry and Ky summary.

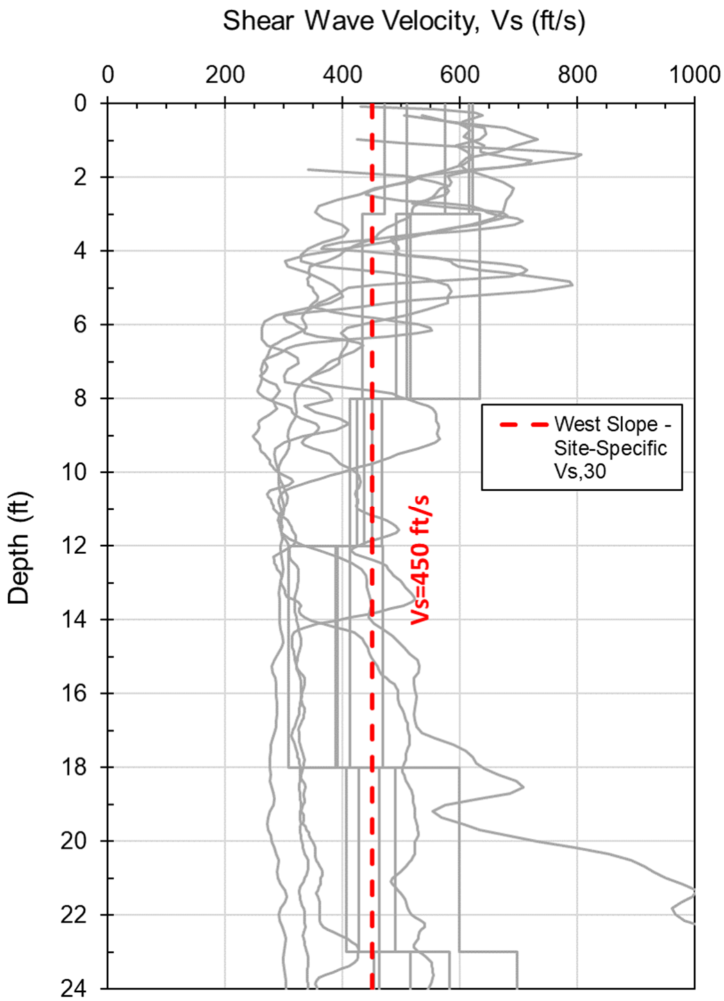



Figure C-1: Weighted-average Vs profile at West Slope

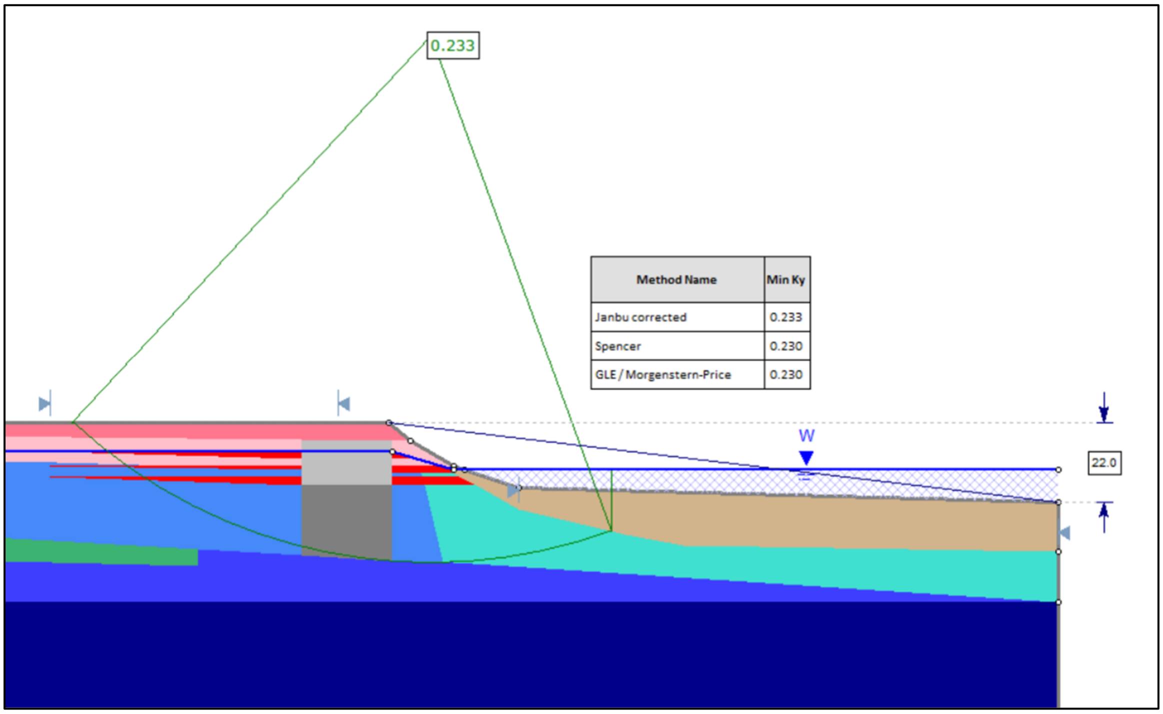


Figure C-2: Yield Coefficient (Ky) and West Slope Geometry

Based on $T_s = 0.1956\ s$, the Spectral Acceleration $S_a$ at $1.5T_s = 0.2933\ s$ can be computed as 1.052g per seismic mapped values provided by Rockridge Geotechnical, dated April 29, 2020, attached here.

The summary of seismic deformation computed based on Bray and Travasarou (2007) has been attached at the end of this section.

Bray, Jonathan D., and Thaleia Travasarou. "Simplified procedure for estimating earthquake-induced deviatoric slope displacements." Journal of geotechnical and geoenvironmental engineering 133, no. 4 (2007): 381-392.

# Alameda Marina - West Slope

**Latitude, Longitude: 37.77710324, -122.25095143**


| | |
|---|---|
| **Date** | 4/29/2020, 4:27:41 PM |
| **Design Code Reference Document** | ASCE7-10 |
| **Risk Category** | II |
| **Site Class** | E - Soft Clay Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.668 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.657 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.501 | Site-modified spectral acceleration value |
| $S_{M1}$ | 1.578 | Site-modified spectral acceleration value |
| $S_{DS}$ | 1.001 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | 1.052 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | D | Seismic design category |
| $F_a$ | 0.9 | Site amplification factor at 0.2 second |
| $F_v$ | 2.4 | Site amplification factor at 1.0 second |
| PGA | 0.645 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 0.9 | Site amplification factor at PGA |
| $PGA_M$ | 0.58 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 2.67 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 2.552 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 1.668 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.98 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.958 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.657 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.645 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $C_{RS}$ | 1.046 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 1.024 | Mapped value of the risk coefficient at a period of 1 s |

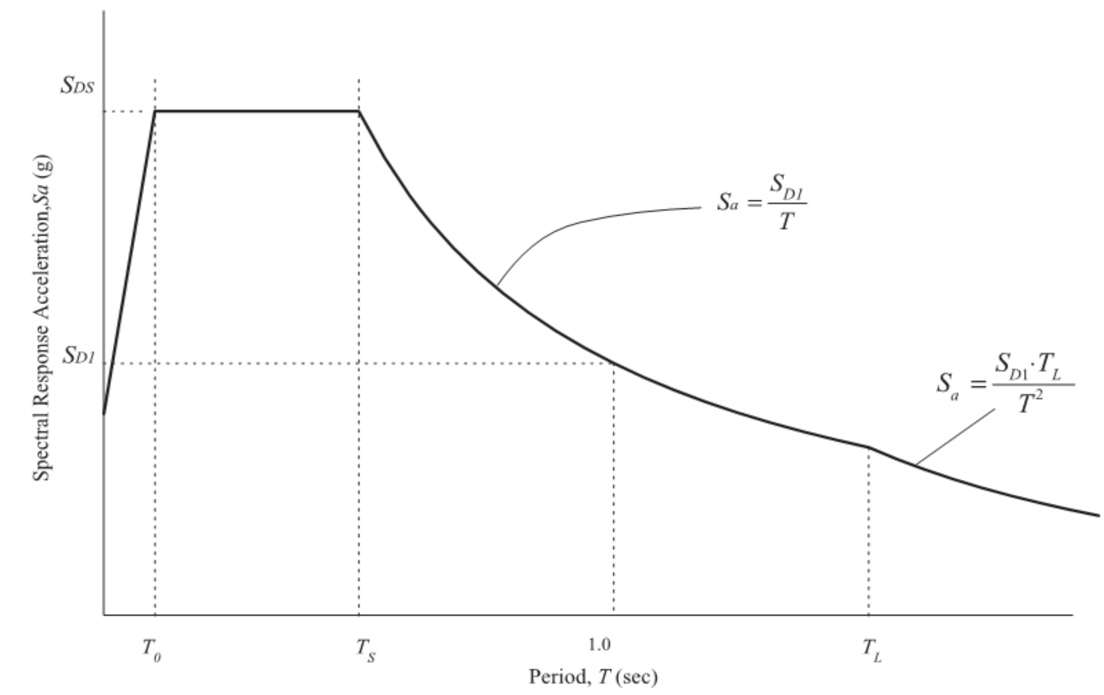



**FIGURE 11.4-1 Design Response Spectrum.**

| | | |
|---|---|---|
| **$S_{D1}$** | 1.001 | Input from Mapped Values |
| **$S_{DS}$** | 1.052 | |
| **$T_L$** | 8 | |

| | |
|---|---|
| **1.5Ts** | 0.2933 |

| | | |
|---|---|---|
| **$T_0$** | 0.190 | Computed Values |
| **$T_s$** | 0.952 | |
| **$S_a$ (1.5Ts)** | 1.052 | |

Used in slope deformation computation below

**Reference: Section 11.4.5, ASCE 7-10**

# Appendix C

**Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements**
by Jonathan D. Bray and Thaleia Travasarou
*Journal of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007*

***SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET***

| **Input Parameters** | | | |
|---|---|---|---|
| Yield Coefficient (ky) | 0.230 | | Based on pseudostatic analysis |
| Initial Fundamental Period (Ts) | 0.1956 | seconds | 1D: Ts=4H/Vs 2D: Ts=2.6H/Vs |
| Degraded Period (1.5Ts) | 0.2933 | seconds | |
| Moment Magnitude ($M_w$) | 7.3 | | |
| Spectral Acceleration ( Sa(1.5Ts) ) | 1.052 | g | |

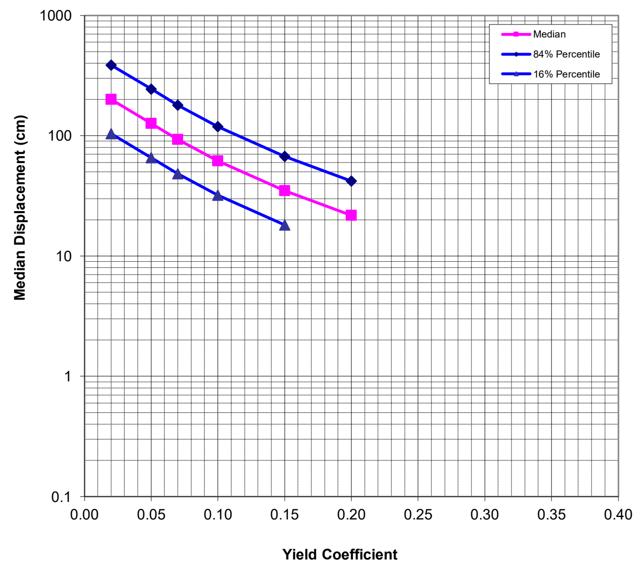
| **Additional Input Parameters** | | |
|---|---|---|
| Probability of Exceedance #1 (P1) | 84 | % |
| Probability of Exceedance #2 (P2) | 50 | % |
| Probability of Exceedance #3 (P3) | 16 | % |
| Displacement Threshold (d_threshold) | 5 | cm |

| **Intermediate Calculated Parameters** | | | |
|---|---|---|---|
| Non-Zero Seismic Displacement Est (D) | 17.05 | cm | eq. (5) or (6) |
| Standard Deviation of Non-Zero Seismic D | 0.66 | | |

| **Results** | | | |
|---|---|---|---|
| Probability of Negligible Displ. (P(D=0)) | 0.00 | | eq. (3) |
| D1 | 8.8 | cm | calc. using eq. (7) |
| D2 | 17.0 | cm | calc. using eq. (7) |
| D3 | 32.9 | cm | calc. using eq. (7) |
| P(D>d_threshold) | 0.97 | | eq. (7) |

**Dependence on ky**

| ky | P(D="0") | D (cm) | Dmedian (cm) | D-84% (cm) | D-16% (cm) |
|---|---|---|---|---|---|
| 0.020 | 0.00 | 200.6 | 200.6 | 386.6 | 104.0 |
| 0.05 | 0.00 | 126.7 | 126.7 | 244.3 | 65.7 |
| 0.07 | 0.00 | 93.0 | 93.0 | 179.4 | 48.3 |
| 0.1 | 0.00 | 61.8 | 61.8 | 119.1 | 32.0 |
| 0.15 | 0.00 | 35.0 | 35.0 | 67.4 | 18.1 |
| 0.2 | 0.00 | 21.9 | 21.9 | 42.1 | 11.3 |
| 0.3 | 0.01 | 10.3 | 10.2 | 19.7 | 5.2 |
| 0.4 | 0.07 | 5.6 | 5.3 | 10.5 | 2.4 |




**Notes**

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
7. Rigid slope is assumed for Ts < 0.05 s
8. When a value for D is not calculated, D is < 1cm
9. ky may be estimated using the simplified equations shown below.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

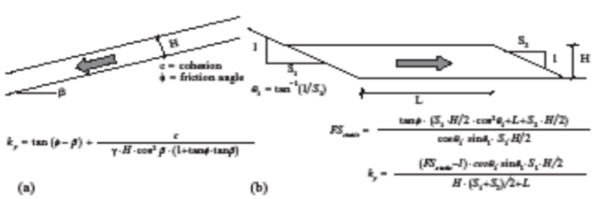



**Fig. 14.1.** Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

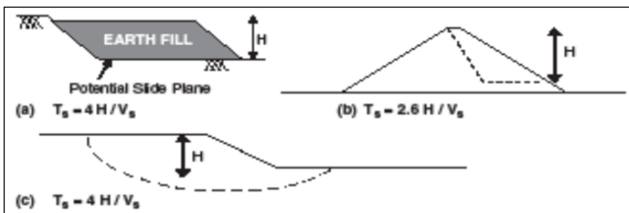



**Fig. 14.4.** Estimating the initial fundamental period of potential sliding blocks

Figures from Bray, J.D. (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, Vol. 6, pp. 327-353.