

## Lateral Spreading Analysis Example

Memo-To-Designers 20-15, dated May, 2017 (MTD 20-15) presents the procedure for liquefaction-induced lateral spreading analysis for new and existing bridges. This procedure includes both geotechnical and structural analyses to be performed by Geotechnical Services (GS) and Structure Design (SD), respectively, in an interactive and, when necessary, iterative manner. The following is a step-by-step example of geotechnical liquefaction-induced lateral spreading analysis using the MTD 20-15 procedure.

### Background

Due to relatively low strength and high compressibility, soils susceptible to liquefaction are not generally considered suitable for supporting bridges on shallow or spread-footing type foundations. In California, most existing bridges located at such sites are likely to be supported on deep or pile foundations. The Department's current practice is to use deep foundations when soil liquefaction is predicted to occur at a structure support location.

Bridge support locations predicted to experience soil liquefaction due to the design ground motion may be susceptible to liquefaction-induced lateral spreading hazards. MTD 20-15 includes a procedure for liquefaction-induced lateral spreading hazard analysis for Seismic Design Criteria (SDC) compliant new bridges and, existing bridges for which the performance criteria are described in MTD 20-1.

These bridges are designed to have large displacement capacity against collapse when subjected to the Safety Level Evaluation (SLE) "Design Earthquake" as defined in MTD 20-1. That is, seismic analysis and design of these bridges are concerned with the stability or collapse, not serviceability, when subjected to the earthquake effects not exceeding those corresponding to the SLE "Design Earthquake." In other words, seismic design of these bridges are concerned only with large deformations. With regard to the ground, such large deformations are generally associated with soil-liquefaction. Thus, for seismic lateral stability of pile foundations, large lateral ground deformations generally associated with liquefaction-induced lateral spreading are of concern.

The scope of this example analysis is limited to the case of a typical pile-supported bridge abutment. It is assumed that the reader is familiar with the contents of MTD 20-15.

The lateral spreading analysis procedure included in MTD 20-1 utilizes the Newmark's Displacement Analysis Method (NDAM) to estimate liquefaction-induced lateral spreading displacements at bridge abutments. This method was developed by assuming a rigid block soil resting on the surface of a rigid base with the interface characterized by a perfectly plastic load (shear)-displacement (sliding) behavior. During a seismic shaking event, it is assumed that such rigid body when resting on the surface of an inclined rigid base can slide downward only. This sliding displacement occurs incrementally during those cycles of acceleration-time history with the peak acceleration greater than a threshold values. The threshold acceleration, termed as the yield acceleration, represents the magnitude of the input ground acceleration at which the rigid body yields in shear along the perfectly plastic interface. Due to the assumed perfectly plastic shear behavior, the NDAM cannot predict the pre-yield shear displacement of a flexible soil-mass, in particular those supported by flexible deep foundation, experience during a seismic shaking event.

The base or input motion is represented by the design acceleration-time history, which is designated herein as  $[a_{hg}(t)]g$ , where  $a_{hg}$  is the coefficient of horizontal ground (base) acceleration,  $t$  is the time and  $g$  is the acceleration due to gravity. For SLE design, the coefficient of the design horizontal peak ground acceleration, HPGA, is the absolute peak value of the base acceleration,  $|(a_{hg})_{max}|$ , referred to hereafter in this document simply as  $(k_h)_{max}$ . The parameter  $(k_h)_{max}$  is often referred to as  $a_{max}$  or  $k_{max}$  in the literature. It represents the design peak or maximum demand in a pseudo-static slope stability analysis or in the Newmark type rigid body displacement analysis.

The NDAM consists of two parts or steps:

- 1) *Limit Equilibrium Based Pseudo-Static Analysis for Lateral Sliding Stability:* A limit-equilibrium based pseudo-static lateral sliding stability analysis in which the seismic load effects on the soil mass are represented by a lateral inertial force,  $k_h W$ , acting in the direction (s) of the potential sliding. Here,  $k_h (=a_{hg})$  is the seismic horizontal acceleration coefficient, often referred to simply as the "Seismic Coefficient," and  $W$  is the weight of the soil mass being analyzed for lateral stability. The purpose of this analysis is to evaluate the value of  $k_h$  that results in a minimum factor safety against lateral sliding,  $FS_{min} = 1.0$  and the corresponding slip surface. For soil slopes, a  $FS_{min}=1.0$  implies full mobilization of the available shear strength or incipient yielding (in shear) of the soils along a certain slip surface. The corresponding  $k_h$  value is termed as the coefficient of (horizontal) yield acceleration, designated herein as  $(k_h)_y$ . It is often designated as  $k_y$  or  $k_{yield}$  in the literature.

2) *Permanent Sliding Displacement Analysis*: The NDAM assumed that any time during the ground shaking period the magnitude of the base acceleration  $[a_{hg}(t)]g$  acting in the opposite direction of potential sliding exceed the yield acceleration  $[(k_h)_y]g$  some finite amount of relative displacement between the soil-mass and the base occurs by sliding. Sliding stops soon after the magnitude of the base acceleration  $[a_{hg}(t)]g$  drops back to less than the yield acceleration  $[(k_h)_y]g$ . No relative sliding displacement occurs until the next cycle during which base acceleration  $[a_{hg}(t)]g$  exceeds the yield acceleration again  $[(k_h)_y]g$ . After each of these cycles, the soil-mass comes to a stop with respect to the base.

The total sliding displacement is the sum of the sliding displacements that occurs during the individual acceleration cycles. At the end of the ground shaking, the soil mass will come to a full stop. Therefore, the NDAM cannot predict any relative lateral displacement of the soil mass that occurs, if any, after the cessation of ground shaking.

As per MTD 20-15, using the coefficient of the yield acceleration,  $(k_h)_y$ , the design ground motions HPGA and the moment magnitude of the associated design earthquake  $M_w$ , use the method of Bray and Tavaslarou (2007) for Newmark's rigid body type sliding displacement to evaluate the liquefaction-induced lateral spreading displacement ( $\Delta$ ) at bridge abutments:

$$\Delta \text{ (inches)} = 0.3937 \text{ Exp}[-0.22 - 2.83 \text{ Ln}(k_h)_y - 0.333 \{ \text{Ln}(k_h)_y \}^2 + 0.566 \text{ Ln}(k_h)_y \text{ Ln(HPGA)} + 3.04 \text{ Ln(HPGA)} - 0.244 \{ \text{Ln(HPGA)} \}^2 + 0.278(M_w - 7)] \quad (1)$$

Note that the symbols  $k_y$  and PGA used in Bray and Tavaslarou (2007) are replaced with the symbols  $(k_h)_y$  and (HPGA), respectively.

A great deal of interaction between GS and SD is necessary to determine the need for and to perform the lateral spreading analysis. GS needs detailed information from SD on the proposed bridge and foundation design in order to develop a representative geometric model (profiles and cross sections) of the abutment ground to be analyzed for lateral spreading and ground displacements. GS needs the bridge design information and related data that SD is required to include with the request for a Foundation Report (FR) as per MTD 1-35 (Caltrans, 2008) and MTD 3-1 (Caltrans, 2014). For existing bridges, As-built plans that include the required abutment geometrics, foundation and other input parameters may be

useful, and sometimes sufficient, to perform an initial or preliminary liquefaction-induced lateral spreading hazard analysis.

### Step-By-Step Procedure

For this example, the coordinates of the bridge are assumed to be Latitude =  $34.065177^\circ$  and Longitude =  $-117.302619^\circ$ . The design ground motion parameters were evaluated based on a probabilistic seismic hazard analysis (PSHA) and correspond to a return period of approximately 1,000 years. The time-averaged shear velocity  $V_{s30}$  for the upper 100 feet soils at the site was estimated to be about 886 feet/sec (270 m/sec). Based on the procedure specified in the SDC and using the Caltrans ARS Online tool (Caltrans, 2018), the following SLE design ground motion parameters apply:

- Coefficient of Horizontal Peak Ground Acceleration, HPGA or  $(k_h)_{\max} = 0.8$
- Mean Earthquake Moment Magnitude for the design HPGA,  $M_w$  or  $M = 7.7$

*Step 1: Develop Geometric Model*

The geotechnical lateral spreading hazard analysis starts with the collection and review of the available site and bridge foundation design information needed to develop a geometric model of the abutment soil-wall-pile foundation system (Figure 1).

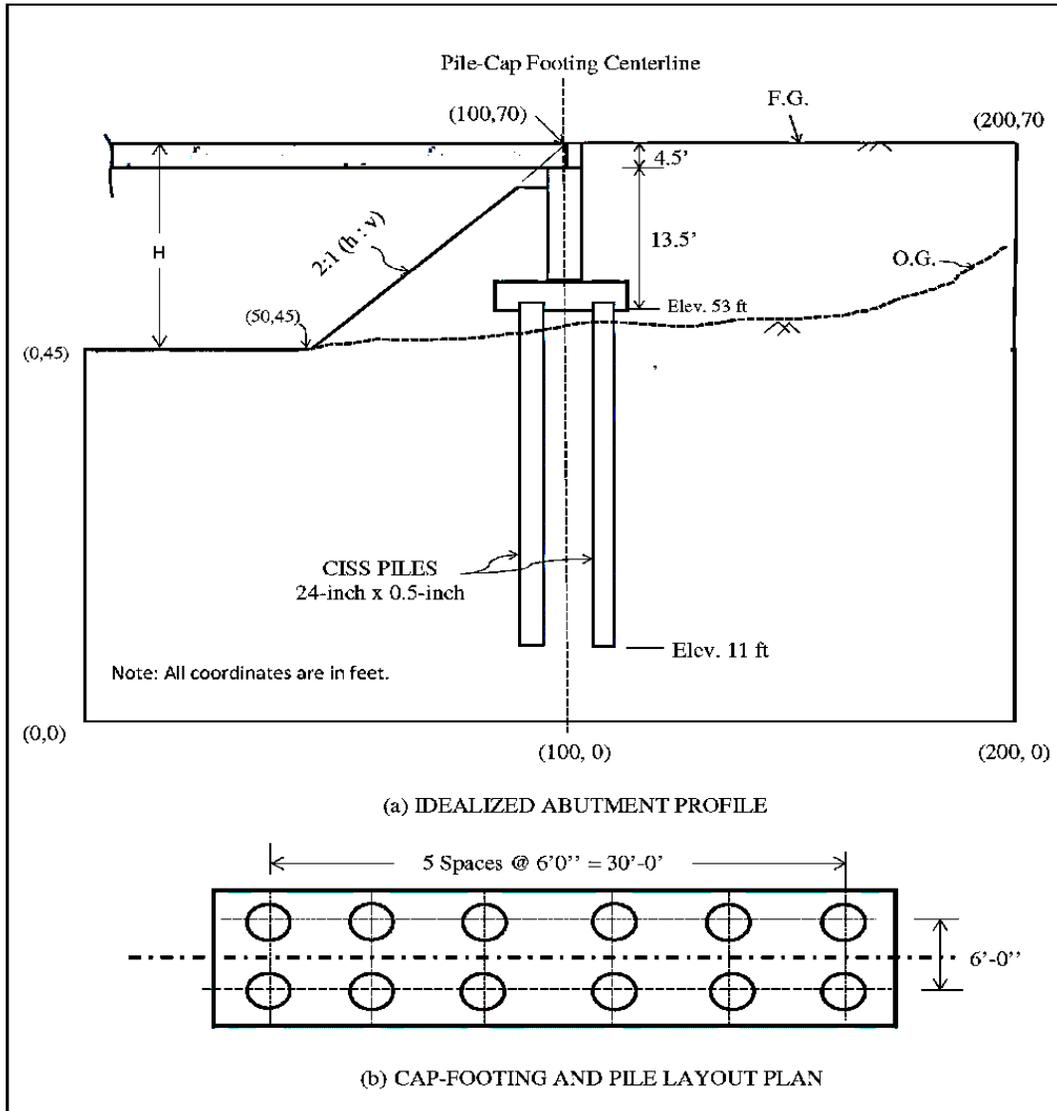


Figure 1. Idealized Geometric Model for Abutment 4

The bridge is a three-span structure with two seat-type abutments (Abutment 1 and 4) and two bent supports (Bents 2 and 3). For this example, liquefaction-induced lateral spreading analysis is performed for Abutment 4. Due to the presence of liquefiable soils, pile foundations are proposed to support the abutment.

Figure 1 shows the original grade (OG) and finish grade (FG) profiles, the proposed 2:1 (Horizontal:Vertical) abutment embankment end slope, the seat-type abutment supported by two rows of 24-inch diameter Cast-in-Steel Shell (CISS) piles. It also shows the proposed layout of the abutment piles.

With reference to Figure 1, the geometric model should extend laterally to the left and right a distance of at least 2H and 4H from the toe and top of the abutment end slope, respectively. Here H is the height of the abutment/embankment as shown in Figure 1. The soil profile should extend to a depth of at least 2H below the toe elevation of the abutment end slope or 3H from the top of the abutment embankment.

*Step 2: Develop Idealized Soil Profile with Design Soil Parameters*

Figure 2 presents an idealized soil profile at Abutment 4.

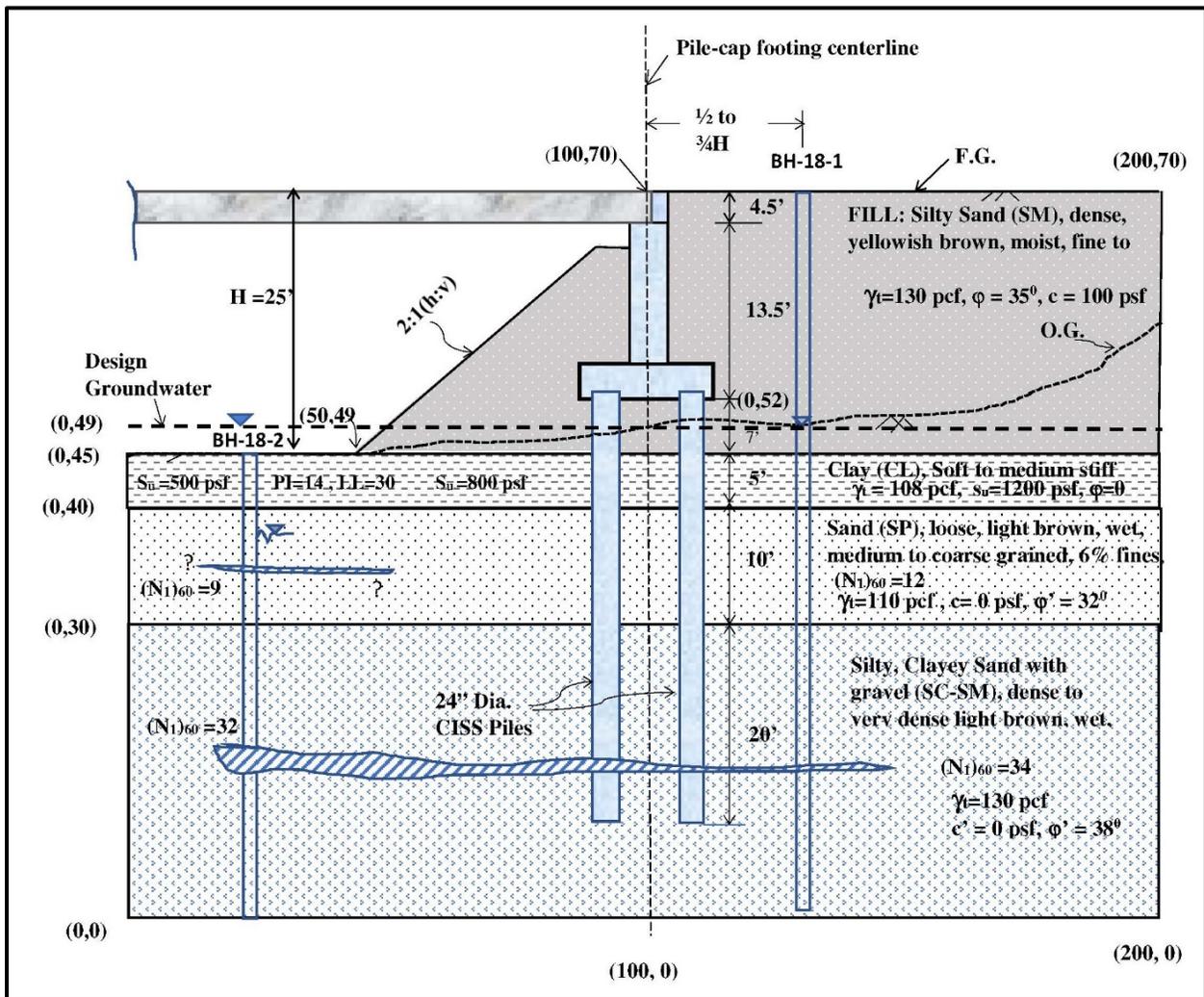


Figure 2. Idealized Geometric and Material Models for Analysis at Abut 4

Research conducted during the early phases of the project development process indicated the likelihood of the presence of potentially liquefiable soils and groundwater, both at relatively shallow depths at the proposed abutment locations. Based on this information, two borings were drilled near each abutment, as shown in Figure 2 for Abutment 4. These boring locations were selected to optimize the subsurface information necessary to develop subsurface soil models for a detailed seismic hazard evaluation, including pseudo-static slope stability analysis. For slope stability analysis, borings needed to extend to a depth of at least 2H below the bottom of the approach embankment fill. For the example case, deeper borings were needed for analysis and design of the foundation piles.

Since the presence of potentially liquefiable soil was anticipated, groundwater was measured and SPT blow counts were measured and interpreted in accordance with the ASTM Standard Test Method D6066. The upper portions of the borings were drilled using a hollow-stem auger until groundwater was encountered. Groundwater was allowed to stabilize prior to the depth measurements. Below groundwater, the borings were drilled using mud rotary methods.

Representative soil samples were tested in the laboratory to determine the soil types and parameters, in particular for the liquefiable soil layer(s). Design soil parameters were selected based on the field exploration and laboratory test results. The design soil parameters for each identified soil layer developed for all applicable loading and/or drainage conditions are presented in Figure 2. The majority of the soil profile and design soil parameters included in Figure 2 were obtained as part of the routine site characterization.

The following sections present brief discussions on the soil parameters needed specifically for the liquefaction-induced lateral spreading analysis.

The undrained residual shear strengths ( $S_r$ ) for the liquefied soils were based on the empirical correlation developed by Kramer and Wang (2015) and recommended in MTD 20-15 to evaluate  $S_r$  for use in the lateral spreading analysis:

$$S_r(\text{psf}) = 2116 \exp \left\{ -8.444 + 0.109(N_1)_{60} + 5.379 \left( \frac{\sigma'_{vo}}{2116} \right)^{0.1} \right\} \quad (2)$$

Where,

$(N_1)_{60}$  = Energy corrected and overburden pressure normalized SPT blow count. See "Liquefaction Module" of the Geotechnical Manual for the procedure to calculate  $(N_1)_{60}$ .

$\sigma'_{vo}$  = Initial in-situ effective overburden stress based on the ground surface and groundwater elevations applicable to seismic design (Extreme Event Limit State I).

The correction factor ( $C_N$ ) used in the evaluation of  $(N_1)_{60}$  was based on the effective overburden stress ( $\sigma'_{vo}$ ) corresponding to the ground surface and groundwater elevations

existing at the time SPT blow counts ( $N_m$ ) were recorded. Unlike liquefaction analysis, the SPT blow counts used in Equation (2) are not corrected for fines content.

Unless the piles are not capable of providing any nominal lateral resistance, the potential for significant lateral spreading displacement at piled bridge supports is considered low where the liquefiable soil layers within the depth of significance for lateral stability (i.e., 2H below the top of the abutment embankment) are characterized by  $(N_1)_{60} > 15$ . Soils with  $(N_1)_{60} > 15$  start to dilate (i.e., provide increased shear resistance) immediately after the onset of limited liquefaction, if any, and do not experience significant lateral displacements during earthquakes.

The undrained residual shear strength ( $S_r$ ) of the liquefied soils in Equation (2) is a function of both  $(N_1)_{60}$  and  $\sigma'_{vo}$ . Due to the variations in the ground surface profile,  $\sigma'_{vo}$  at a given elevation below the ground surface will vary laterally. There may also be lateral variations in the  $(N_1)_{60}$  values along the same elevation even for the same soil layer. In other words, within a single liquefied soil layer, the undrained residual shear strength ( $S_r$ ) may vary both vertically and laterally. This will also be true with regard to the undrained shear strength ( $S_u$ ) for cohesive soil layers. Therefore, such soil layers present within the depth of influence for lateral stability, should be divided into sufficient numbers of sublayers (vertically) and subzones (laterally) so that each subzone can be represented by an average value of the undrained shear strength parameter  $S_r$  or  $S_u$ .

For the example case, the clay as well as the liquefied soil layers in Figure 3 are divided laterally into three subzones and each zone is assigned a single value of the shear strength parameters  $S_u$  and  $S_r$ . The undrained residual shear strength ( $S_r$ ) values for three subzones of the liquefied soil are evaluated as follows:

(a)  $S_r$  for Zone L1-1:

This zone is located to the left of the toe of the abutment end slope with a flat ground surface at elevation 45 feet. The  $S_r$  value at the mid elevation (elevation 35 feet) of this zone is:

$$\begin{aligned} \sigma'_{vo} &= (5.0 \text{ ft}) (108 - 62.4) \frac{\text{lb}}{\text{ft}^3} + (5.0 \text{ ft}) (110 - 62.4) \frac{\text{lb}}{\text{ft}^3} = 466.0 \text{ psf} \\ (N_1)_{60} &= 9 \\ (S_r)_{L1-1} &= 2116 \exp \left\{ -8.444 + 0.109 (9) + 5.379 \left( \frac{466}{2116} \right)^{0.1} \right\} \text{ psf} \cong 125 \text{ psf} \end{aligned}$$

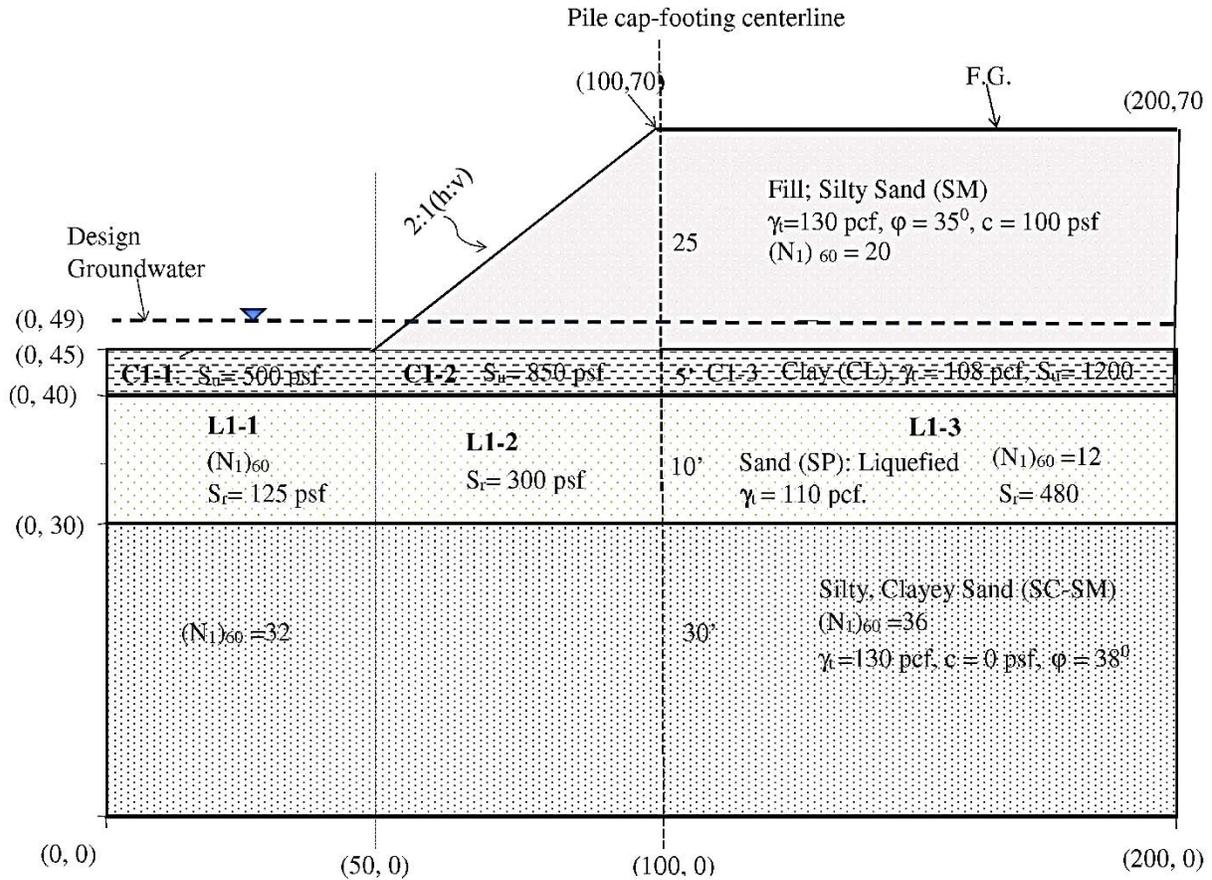


Figure 3. Soil Engineering Parameters for Liquefaction-Induced Lateral Spreading Analysis

(b)  $S_r$  for Zone L1-2

This liquefied soil zone is located underneath the steeply sloping ground surface between zone L1-1 and L1-3 (Figure 3). In the absence of any direct SPT blow count measurement ( $N_m$ ) within this zone, an assumed SPT ( $N_1$ )<sub>60</sub> value equal to the average SPT values for the Zones L1-1 and L1-3 was used in the  $S_r$  value. That is,

$$(N_1)_{60} = 20$$

$$\sigma'_{vo} = (130 \frac{lb}{ft^3})(12.5 - 4.0)ft + (130 - 62.4) \frac{lb}{ft^3} (4.0)ft + (108 - 62.4) \frac{lb}{ft^3} (5)ft + (110 - 62.4) \frac{lb}{ft^3} (5.0)ft = 1841.1 psf$$

$$\text{Average } (N_1)_{60} = (\frac{9+12}{2}) = 11$$

$$(S_r)_{L1-2} = 2116 \exp \left\{ -8.444 + 0.109 (11) + 5.379 \left( \frac{1841.1}{2116} \right)^{0.1} \right\} psf \cong 300 psf$$

(b)  $S_r$  for Zone L1-3

This zone is located to the right of the top of the embankment end slope (i.e., below the full height embankment zone). It is assumed that the abutment end slope surface extends upward to the intersection of the roadway Finish Grade (F.G.) surface and the transverse centerline of the abutment pile cap-footing (Figure 3). The  $S_r$  value at mid elevation (Elevation 35 feet) of this zone is:

$$\sigma'_{vo} = 130 \frac{lb}{ft^3} (21.0 \text{ ft}) + (130-62.4) \frac{lb}{ft^3} (4.0 \text{ ft}) + (108-62.4) \frac{lb}{ft^3} (5.0 \text{ ft}) + (110-62.4) \frac{lb}{ft^3} (5.0 \text{ ft}) = 3466.4 \text{ psf}$$

$$(N_1)_{60} = 12$$

$$(S_r)_{L1-3} = 2116 \exp \left\{ -8.444 + 0.109 (12) + 5.379 \left( \frac{3466.4}{2116} \right)^{0.1} \right\} \text{ psf} \cong 480 \text{ psf}$$

The undrained shear strengths ( $S_u$ ) for the three subzones of the clay layer are:

The laboratory measured  $S_u$  values for two representative samples, one each retrieved from boring BH-18-1 and BH-18-2 ( Figure 2 ) are 1200 psf and 500 psf, respectively. The average undrained shear strength for the Zone C1-2 is the average of the undrained shear strengths for the Zones C1-1 and C1-3.

$$\bullet (S_u)_{C1-2} = \left( \frac{500+1200}{2} \right) \text{ psf} = 850 \text{ psf}$$

The soil engineering parameters necessary for this analysis are summarized in Table 1.

Table 1. Idealized Soil Profile with Assigned Soil Parameters for Lateral Spreading Analysis and p-y Curves

Soil Layer No.	Top and Bottom Elevations (ft)	Soil Description	Zone <sup>(1)</sup>	(N <sub>1</sub> ) <sub>60</sub>	Total Unit Wt (pcf)	Shear Strength Parameters		p-y Parameters	
						Cohesion or Undrained Shear Strength, c, S <sub>u</sub> , or S <sub>r</sub> (psf)	Friction Angle, φ (degrees)	k <sub>s</sub> and/or k <sub>c</sub> (pci)	ε <sub>50</sub>
1	70-45	FILL: Silty Sand (SM), Medium dense, from fine-to coarse sand, trace clay, yellowish brown, from moist to wet.	-	20	130	c=100	35	k <sub>s</sub> =120/60 (above/below water table) k <sub>c</sub> =66	-
2	45-40	Silty Clay (CL), Soft to medium stiff, dark gray, wet (LL=30, PI =10)	C1-1	-	108	S <sub>u</sub> =500	φ <sub>u</sub> =0.0	k <sub>c</sub> =280 (Based on avg. S <sub>u</sub> =850 psf)	0.01
			C1-2	-		S <sub>u</sub> =850 <sup>(2)</sup>			
			C1-3	-		S <sub>u</sub> =1200			
3	40-30	Sand (SP), fine and medium, from loose to medium dense, 8-10 % fines, non-plastic, light brown, wet, (Liquefied Layer)	L1-1	9	110	S <sub>r</sub> = 125 psf	φ <sub>u</sub> =0.0	k <sub>c</sub> =100	0.02
			L1-2	11 <sup>(2)</sup>		S <sub>r</sub> = 300 psf			
			L1-3	12		S <sub>r</sub> = 480 psf			
4	30 to 0 <sup>(3)</sup>	Silty, Clayey Sand (SC-SM)with gravel (SM), dense, from fine to coarse sand, fine gravel, trace clay, grayish brown, wet	-	32-36	130	c=0	38	k <sub>s</sub> =125	-

Notes: (1) See Figure 3 for locations of soil layer zones, (2) Estimated, and (3) Soil profile for lateral spreading analysis should extend to a depth of at least 2H below the bottom of the slope. Here, H= Height of the embankment.

*Step 3: Perform GLE-Based Pseudo-Static Slope Stability Analysis*

The step involves a series of General Limit Equilibrium (GLE) based pseudo-static slope stability analyses performed using the models in Figures 1 through 3.

*Step 3.1: Select Lateral Spreading (Sliding) Mechanism*

Caltrans MTD 20-15 specifies a 2D wedge type lateral spreading mechanism. For a given value of the pile lateral resistance ( $R_{Tot}$ ) and seismic inertial loading ( $k_h$ ), a set of pseudo-static analyses (runs) are performed varying the slip surface to evaluate the minimum factor of safety ( $FS_{min}$ ).

MTD 20-15 imposes a number of important constraints on the horizontal as well as the vertical limits of the potential sliding mass when analyzing lateral spreading hazards (See Figure 4 of MTD 20-15 for details).

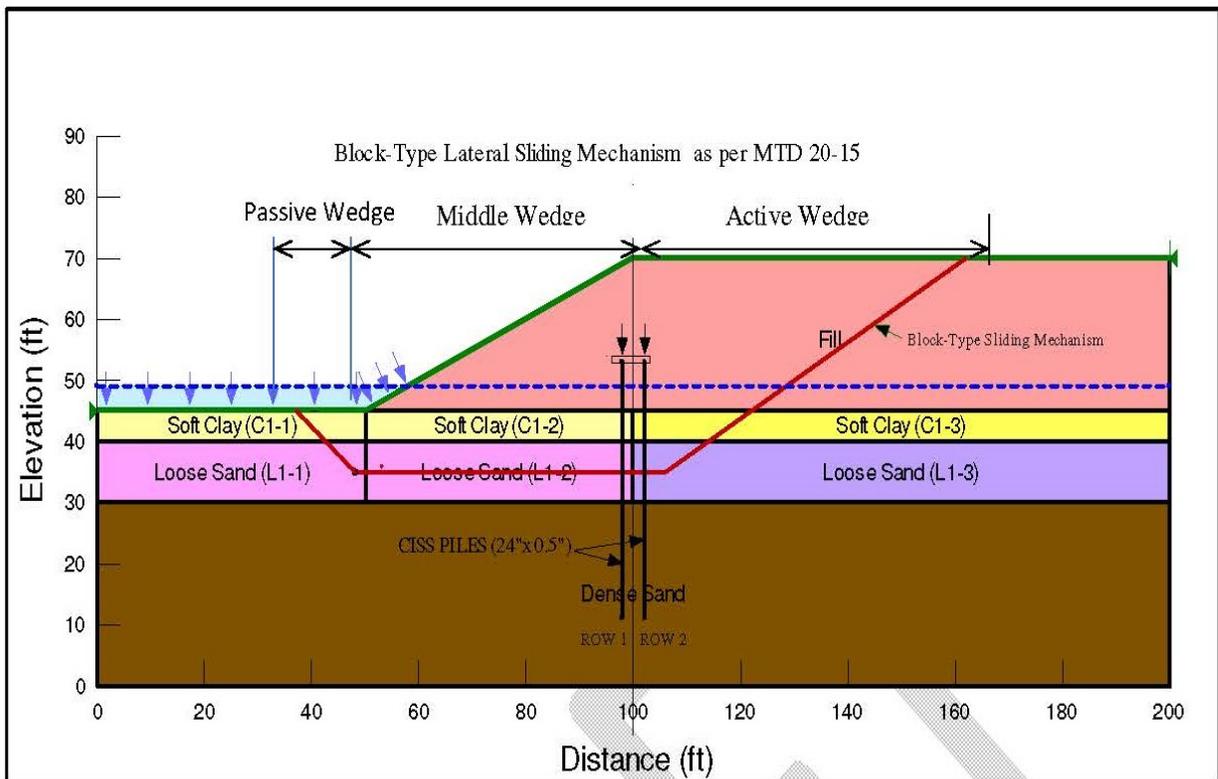


Figure 4. Digital SLOPE/W Model for the Example with Typical Wedge Type Sliding Mechanism

Figure 4 shows a typical geometry of the wedge-type sliding mass used in this example. The potential sliding consists of three wedges: (1) an Active Wedge, (2) a Middle Wedge, and (3) a Passive Wedge.

### *Step 3.2: Select Pseudo-Static Stability Analysis Method and Computer Software*

A GLE-based slope stability analysis procedure should be used for the evaluation of pseudo-static slope stability analyses of wedge-type and composite abutment soil-foundation sliding masses. Geo-professionals performing lateral spreading analysis should be thoroughly familiar with the computer software used to perform pseudo-static slope stability calculations. The results of such analyses must be verified as reasonable.

For this example, the GLE-based Morgenstern-Price procedure for slope stability analysis was used to perform all pseudo-static slope stability analyses using SLOPE/W (GEO-SLOPE, 2014). The Spencer procedure may also be used for this analysis.

### *Step 3.3: Develop Digital Slope Stability Model Including Pile Lateral Resisting Forces*

Develop a digital computer software model based on the idealized geometric, soil and foundation models shown in Figures 1 through 3. Figure 4 presents the digital model developed in SLOPE/W. To use pile lateral resistances as shown in Figure 3 of MTD 20-15, the abutment pile foundations were modeled in SLOPE/W as built-in 1-D vertical “reinforcement elements”.

As per MTD 20-15, the Structure Designer (SD) performs a soil-foundation interaction analysis at the abutment to determine the mobilized values of the total pile lateral resistance ( $R_{Tot}$ ) as a function of lateral ground displacement ( $\Delta$ ). See Figure 1(b) of MTD 20-15 for the definition and distribution pattern of the displacement parameter ( $\Delta$ ) along the pile. This information is sent to GS in the form of Curves 1 and 2 shown in Figure 5 of the MTD 20-15. GS uses this information to select the appropriate range ( $R_{Tot}$ ) of values to perform the geotechnical lateral spreading analysis in order to obtain Curve 3 shown on Figure 5 of the MTD 20-15.

In SLOPE/W, for each of the pile “reinforcement” element as shown in Figure 4, the input parameters for pile lateral resistances include:

- Pile length ( $L$ ), in unit of length
- Direction ( $\theta$ ), in degrees counterclockwise from the (horizontal) slip direction
- Shear force ( $V_c$ ) in unit of force per pile
- Shear force reduction factor ( $f_g$ ), dimensionless factor
- Pile spacing parameter,  $S$  (in unit of length measured in the direction normal to horizontal/slip direction), and
- Direction of the shear force (either parallel to slip or perpendicular to the reinforcement)

For N rows of piles (normal to the slip direction), N individual pile reinforcements, each representing a single row of piles in the normal direction to sliding, should be used in SLOPE/W. In a SLOPE/W analysis, the total pile lateral resistance  $R_{Tot}$  (force per unit width of abutment) may be equally divided between the N pile reinforcements or vice versa. That is, for each vertical pile reinforcement element shown in Figure 4:

$$V_c = \frac{(R_{Tot})S}{N} \quad (3)$$

For the example, there are two (2) rows of piles ( $N=2$ ). The input parameters for each pile in Figure 4, for a given value of  $R_{Tot}$  in kip/ft width of abutment, are as follows:

- Pile length (L) = 47 feet
- Direction ( $\theta$ ) =  $90^\circ$
- Pile spacing, S = 6.0 feet
- Shear force  $V_c = \left(\frac{R_{Tot} \times 6.0}{2}\right) = 3.0 \times R_{Tot}$  (kips per pile)
- Shear force reduction factor ( $f_g$ ) = 1.0
- Direction of the shear force = parallel to slip

*Step 3.4: Perform Pseudo-Static Slope Stability Analysis for  $R_{Tot} = 0.0$  and  $k_h = 0.0$*

Initial or preliminary forms of this analysis will usually be performed early in the project development, e.g., during the planning and preliminary engineering phases. The purpose of this analysis is to assess if the potential for liquefaction-induced lateral spreading, including flow type failure, exists at the site, and most significantly to determine the location of the potential sliding surface.

Perform a pseudo-static slope stability analysis for the abutment wall-soil-foundation system based on the digital model developed in Step 3.3 using:

- $R_{Tot} = 0.0$  kips/ft (or  $V_c = 0.0$  and  $S = 1.0$  as the input parameters)
- $k_h = 0.0$

The  $FS_{min}$  values obtained from this and the following step will provide a qualitative indication of soil lateral capacity available for the liquefied conditions to resist lateral sliding and valuable information on the potential sliding mass, including the locations of the critical slip surface and its intersection with the piles, and the extent of additional lateral support that may be needed from existing and/or new piles.

For example, a  $FS_{min} < 1.0$  indicates that, in the absence of any additional lateral support (e.g., from piles), the soil-mass analyzed is susceptible to liquefaction-induced lateral flow. Lateral flow involves large movements as stated in MTD 20-15. It can occur during ground shaking after soil liquefaction starts, and also immediately after the cessation of shaking. In the case of an existing pile-supported abutment, it also indicates that a re-evaluation of the lateral flow hazards will be necessary by including the additional lateral

capacity obtainable from the pile(s). In the case of new construction, it provides qualitative information on the need for, and the extent of, additional lateral supports such as piles or ground improvements. The lower the value of  $FS_{min}$  compared to 1.0, the higher is the likelihood for lateral spreading and larger ground displacement, requiring more extensive countermeasures.

For this example, a range of wedge-type slip surfaces were analysed using SLOPE/W by specifying a left and a right block of grid points (grid-block) to locate the intersection points of the slip planes associated with the three wedges as shown in Figures 4 and 5. The lower boundary lines for these grid-blocks were specified to be at the same elevation as the mid-point of the liquefied soil. This is to force the critical sliding surface within the middle wedge to be located at the mid-depth of the liquefiable soil layer as specified in MTD 20-15. Results of this analysis are presented in Figure 5.

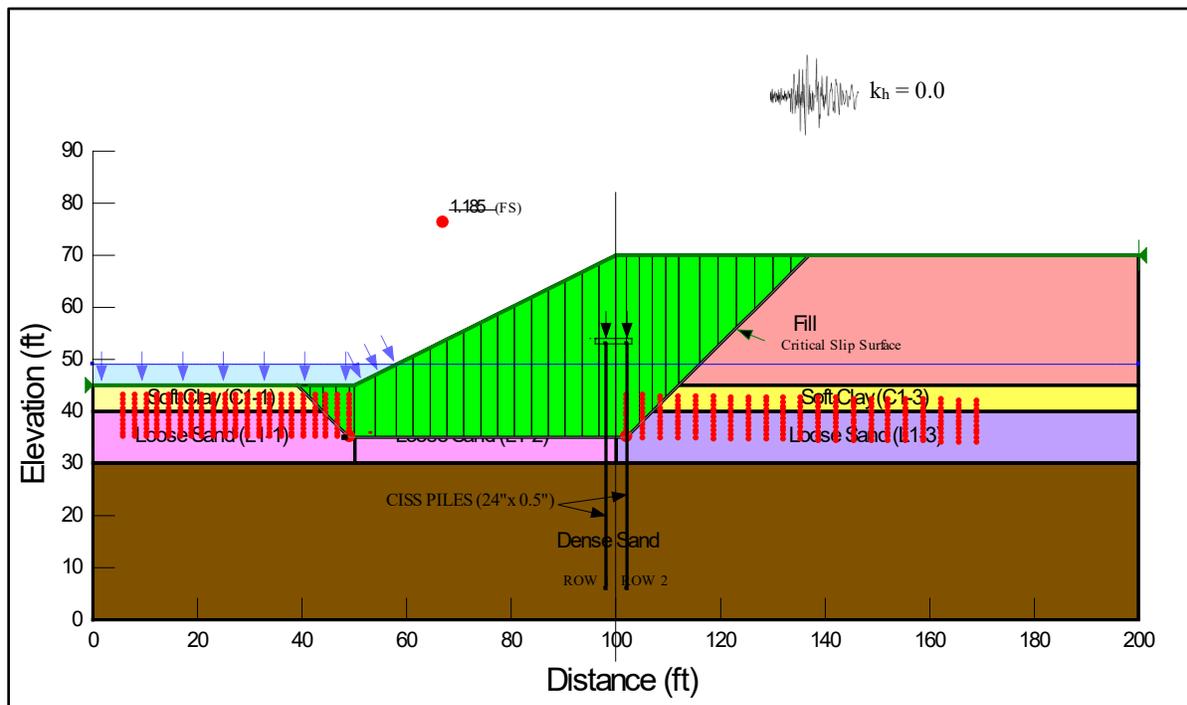


Figure 5. Results of Slope Stability Analysis for  $R_{Tot} = 0.0$  lbs/ft width and  $k_h=0.0$

As seen in Figure 5,  $FS_{min} = 1.18$  for these conditions. Therefore, this abutment is not considered susceptible to liquefaction-induced flow failure.

**Step 3.5: Perform Pseudo-Static Slope Stability for  $k_h = \text{HPGA}$  or  $(k_h)_{\max}$**

Determine if the available soil lateral capacity alone is sufficient to support the abutment wall-soil-foundation system. This step consists of performing a pseudo-static slope stability analysis for the following conditions:

- $R_{\text{Tot}} = 0.0$  kips/ft (or  $V_c = 0.0$  and  $S = 1.0$  as the input parameters)
- $k_h = \text{HPGA} = 0.8$

Results of this analysis are presented in Figure 6, which indicates a very low  $\text{FS}_{\min} = 0.2$ .

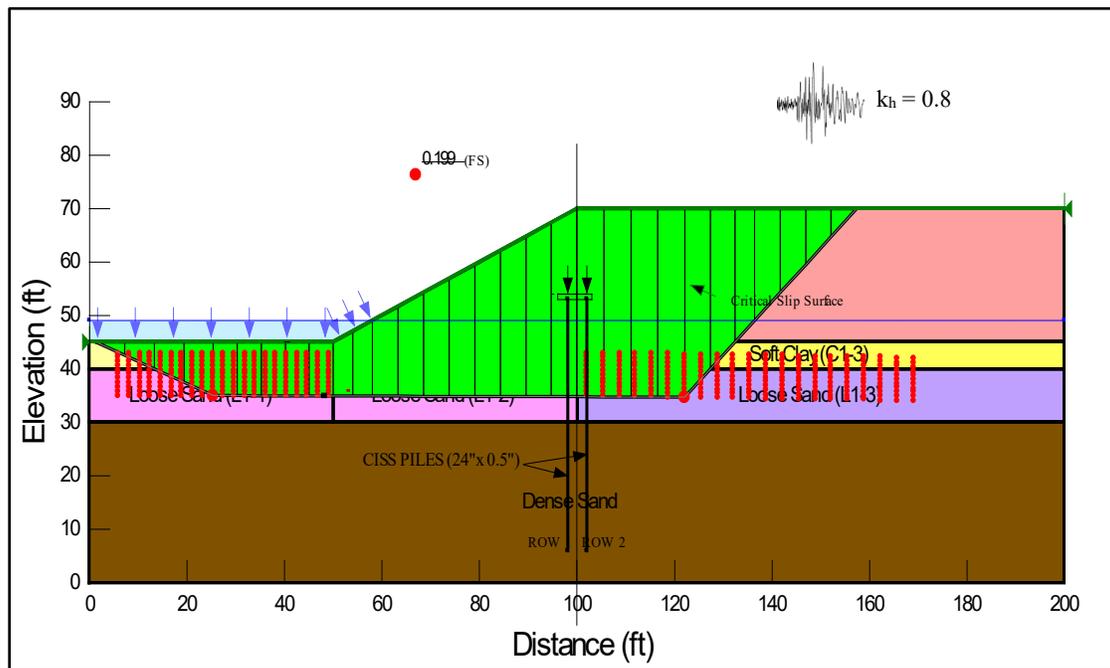


Figure 6. Pseudo-Static Slope Stability Analysis for  $R_{\text{Tot}} = 0.0$  lbs/ft and  $k_h = (k_h)_{\max}$

If this analysis indicates  $\text{FS}_{\min} \geq 1.0$ , the subject abutment will not be considered susceptible to lateral spreading hazards. No further lateral spreading analysis is necessary. Communicate this information to SD, and present the results and findings in the Foundation Report (FR).

Otherwise, continue with the lateral spreading hazard analysis.

**Step 3.6: Evaluate Coefficient of Yield Accelerations  $(k_h)_y$  for  $R_{total} = 0.0$**

Evaluate the coefficient  $(k_h)_y$  for the abutment without considering any contribution from piles to the lateral resistance, and determine the critical slip surface for the above conditions.

This step consists of performing a series of pseudo-static slope stability analyses with input  $R_{Tot}$  (or  $V_c$ ) = 0.0 into the digital model in Figure 4, and varying the input value for  $k_h$ . All other input parameters, except the limits of the specified left and right search grid blocks, remain the same. The lateral positions of the failure plane intersection are likely to vary with the input  $k_h$  values as well as the input  $R_{Tot}$  values. Therefore, it may be necessary to extend outward the lateral limits of the left and right search grid blocks.

As per MTD 20-15, the outer limits of the intersection of the active wedge sliding plane and the ground surface does not need to extend beyond a distance of 4H from the back of the abutment wall, where H is the height of the abutment.

Results of two typical slope stability analysis runs, for  $R_{Tot} = 0.0$  kips/ft and  $k_h = 0.05$  and 0.125 are presented in Figures 7(a) and 7(b), respectively. Notice the differences in the size and limits of the critical slip surfaces for the two different cases.

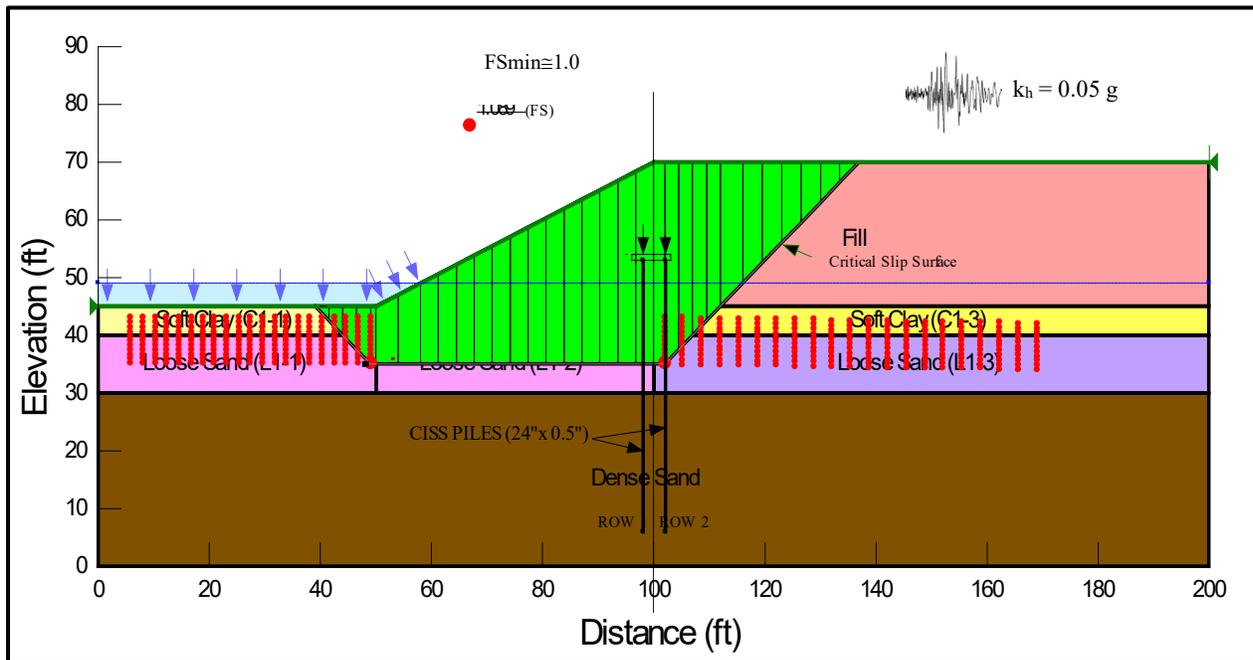


Figure 7(a). Results of Pseudo-Static Stability Analyses for  $R_{Tot}=0.0$  and  $k_h=0.05$

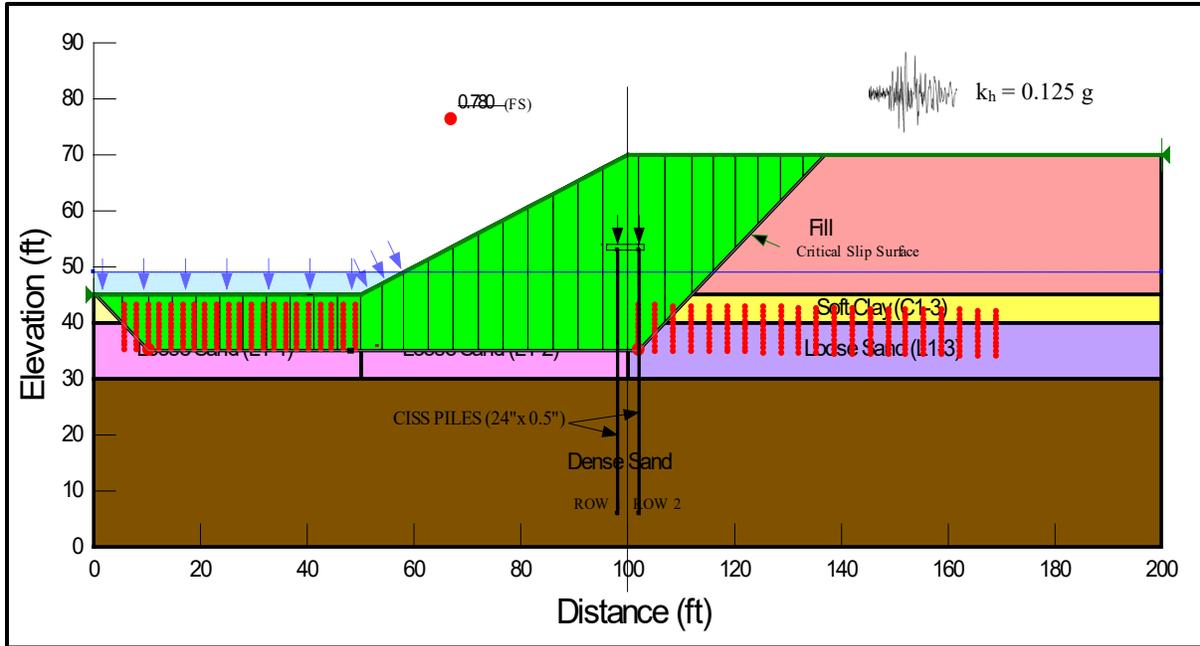


Figure 7(b). Results of Pseudo-Static Stability Analyses for  $R_{Tot}=0.0$  and  $k_h=0.125$

For each input value of  $k_h$ , the outcome of the pseudo-static slope analysis is a corresponding  $FS_{min}$  value. The  $FS_{min}$  values were obtained by repeating this analysis, varying the input  $k_h$  value while keeping the input  $R_{Tot}$  value the same ( $= 0.0$ , in this case). Results of this analysis are in Table 2 and plotted in Figure 8.

Table 2.  $FS_{min}$  Values for  $R_{Tot} = 0.0$  kips/ft and Different  $k_h$  Values

$R_{Tot}$ (or $V_c$ ) = 0.0 kips/ft	
$k_h$	$FS_{min}$
0.000	1.184
0.025	1.088
0.050	1.003
0.075	0.924
0.10	0.848
0.125	0.780
0.150	0.601

The  $(k_h)_y$  value for the current case (e.g.,  $R_{Tot}=0.0$ ) is evaluated from Figure 8 as equal to the  $k_h$  corresponding to a minimum  $FS_{min}=1.0$ . As shown by the arrows in Figure 8, for  $FS_{min}=1.0$ ,  $k_h=0.05$ . Thus,  $(k_h)_y = 0.05$  for  $R_{Tot}=0.0$ .

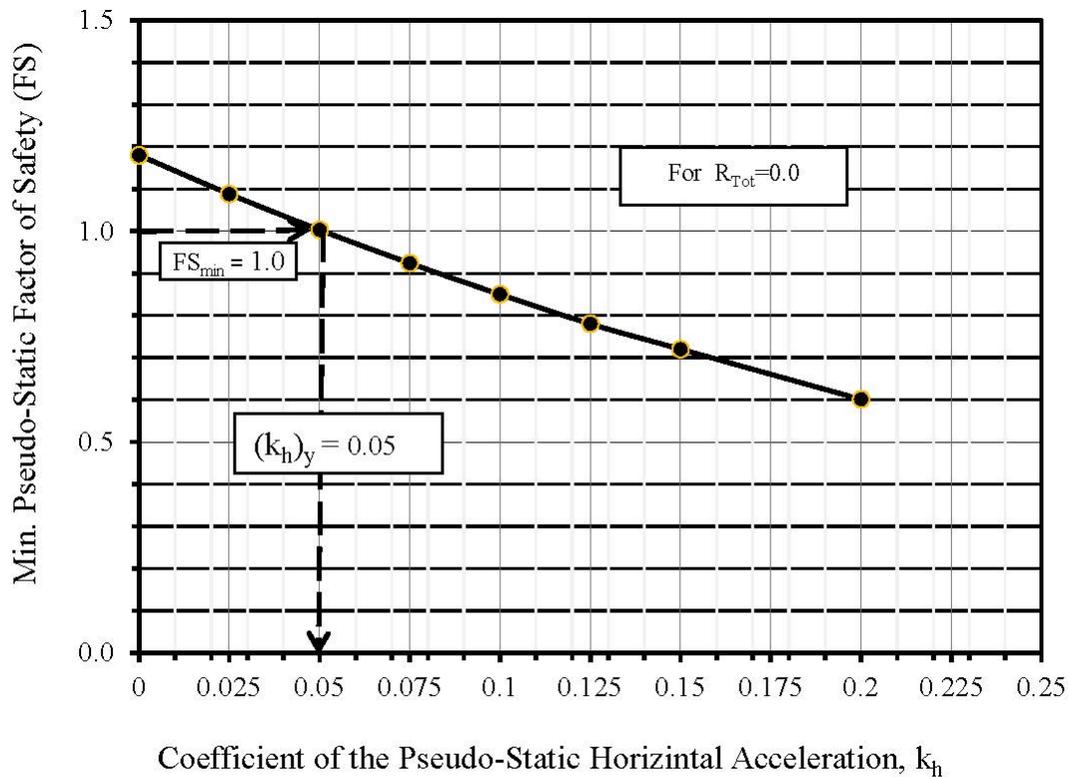


Figure 8. Coefficient of Horizontal Yield Acceleration  $(k_h)_y$  for  $R_{Tot} = 0.0$  kips/ft

**Step 4: Interim Communication with the Structure Designer**

Send SD the following information, and continue with the lateral spreading analysis:

- Table 1, including (if not provided already) the recommended liquefied soil profile and soil parameters necessary to perform p-y type laterally loaded pile analysis.
- Figure 7(a) which shows the locations where the piles intersect the critical sliding surface for  $k_h=(k_h)_y$ . SD will utilize this information in the soil-foundation interaction analysis as per MTD 20-15 to determine  $R_{Tot}$  as a function of the displacement ( $\Delta$ ).

*Step 5: Obtain/Select an Input Range and the Discrete  $R_{Total}$  Values*

Based on the geotechnical information provided in Step 4, SD will perform a soil-foundation interaction analysis for the proposed pile type, size, length and pile group layout shown in Figure 1. Based on this analysis, SD evaluates  $R_{Tot}$  values for a range of assumed displacement ( $\Delta$ ) values. Results are presented in Figure 9.

Based on Figure 9, the total ultimate or nominal pile lateral resistance per unit width of the abutment, designated here as  $(R_{Tot})_N$ , is about 28 kips/ft. A range of input  $R_{Tot} = 0.0$  to 60 kips/ft is considered adequate for further pseudo-static slope stability evaluation. The maximum value for input  $R_{Tot} = 60$  kips/ft is selected to be about two (2) times the  $(R_{Tot})_N$  in Figure 9.

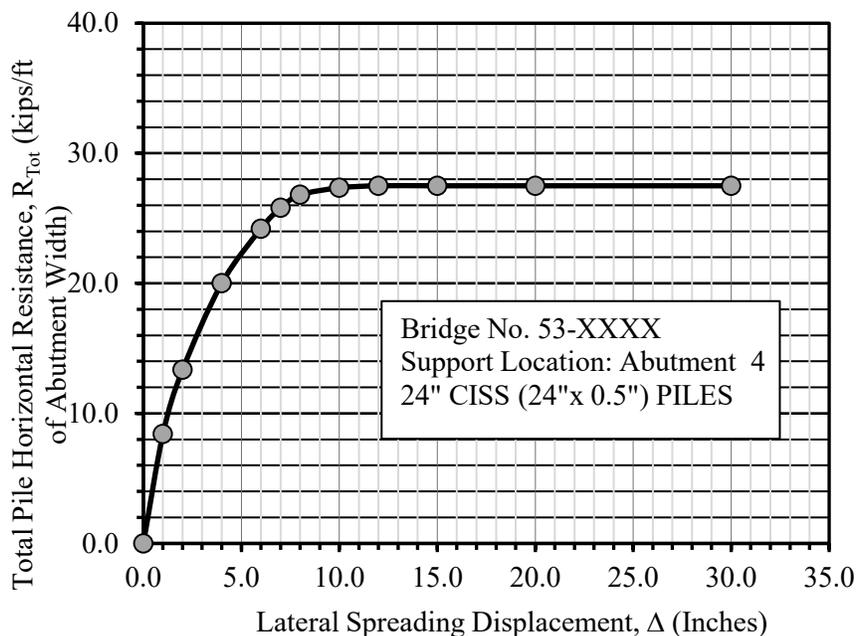


Figure 9. Plot of  $R_{Tot}$  Versus Lateral Spreading Displacement ( $\Delta$ )

Based on the above range of input ( $R_{Tot}$ ), a set of discrete  $R_{Tot}$  values included in Table 3 were selected for additional pseudo-static slope stability analyses. These numbers were selected to obtain a sufficient number of equally spaced discrete  $R_{Tot}$  values for the range of 0.0 to 60 kips/ft. The input  $V_c$  values in the SLOPE/W model are presented in Table 3.

Table 3. Selected Input  $R_{Tot}$  Values and the Corresponding  $V_c$  Values

$R_{Tot}$ (kips/ft)	$V_c$ (kips /Pile) <sup>1</sup>
0.0	0.0
5.0	15.0
10.0	30.0
20.0	60.0
30.0	90.0
40.0	120.0
50.0	150.0
60.0	180.0

Note 1: For two rows of piles and pile spacing parameter  $S=6.0$  ft.

*Step 6: Evaluate Coefficient of Horizontal Yield Accelerations  $(k_h)_y$  Each  $R_{Tot}$  Value*

Conduct pseudo-static slope stability analyses (similar to Step 3.6) for each  $R_{Tot}$  value in Table 3 to evaluate the corresponding yield acceleration. Inherent in this analysis for pile-pinning effects is the assumption that, for the soil-pile system, there exists a yield acceleration for each mobilized  $R_{Tot}$  value. Based on these analyses, plot a series of curves for each value of  $R_{Tot}$ , as shown in Figure 1, each similar to the one shown in Figure 8 for  $R_{Tot}=0.0$ .

The  $(k_h)_y$  value for each  $R_{Tot}$  are then determined from the corresponding plot in Figure 10 as equal to the  $k_h$  for  $FS_{min}=1.0$ . The  $(k_h)_y$  values are presented in Table 4.

Table 4. Yield Coefficient  $(k_h)_y$  for Various  $R_{Tot}$  Values

$R_{Tot}$ (kips/ft)	$(k_h)_y$
0	0.05
5	0.08
10	0.10
20	0.13
30	0.15
40	0.17
50	0.19
60	0.21

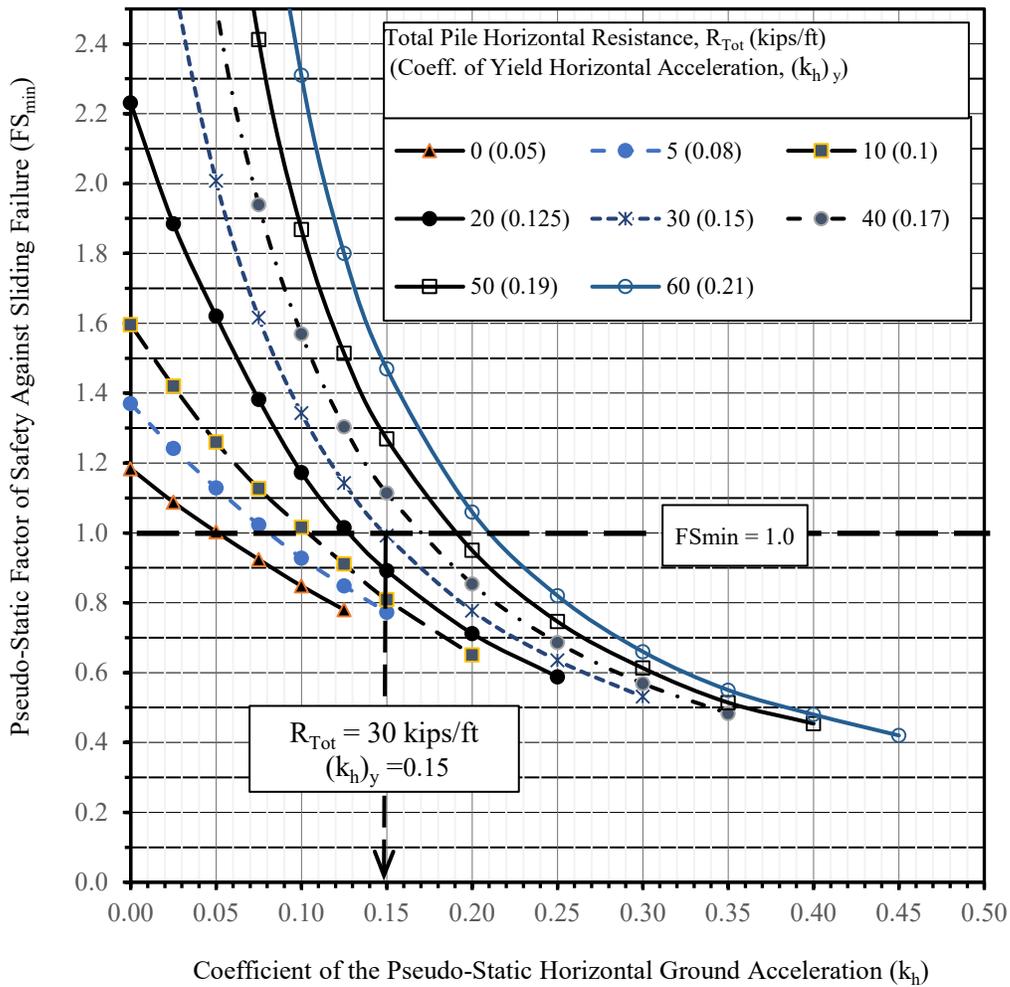


Figure 10. Plots of  $FS_{min}$  as a Function of  $k_h$  for Various  $R_{Tot}$  values

*Step 7: Determine Newmark Displacements as a Function of  $(k_h)_y$*

Calculate the median Newmark's rigid body type sliding displacement of the soil-mass due to the design ground motion for each  $(k_h)_y$  value in Table 4 using the empirical correlation (Eq. 1) by Bray and Tavasaru (2007),  $HPGA=0.8$  and  $M_w=7.7$ .

Results are presented in Table 5.

Table 5. Lateral Spreading Displacement ( $\Delta$ ) as a Function of  $R_{Tot}$

$R_{Tot}$ (kips/ft)	$(k_h)_y$	Displacement, $\Delta$ (inches)
0	0.05	172.8
5	0.08	102.2
10	0.10	75.6
20	0.13	50.9
30	0.15	40.2
40	0.17	32.4
50	0.19	10.4
60	0.21	8.6

**Step 8. Plot  $R_{Tot}$  versus Displacement ( $\Delta$ )**

Each  $(k_h)_y$  value in Table 5 and the corresponding value of the estimated median ground displacement ( $\Delta$ ), corresponds to a specific value of  $R_{Tot}$  (See Table 4). Plot the  $R_{Tot}$  versus Displacement ( $\Delta$ ) as shown in Figures 11 and 12.

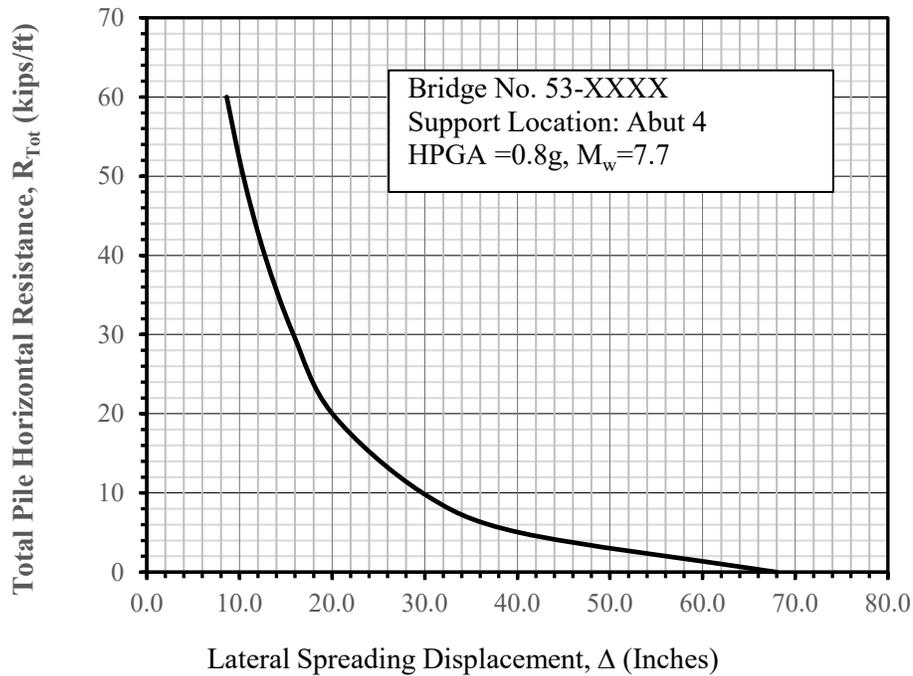


Figure 11 .  $R_{Tot}$  vs. Newmark Rigid Body Displacement ( $\Delta$ )

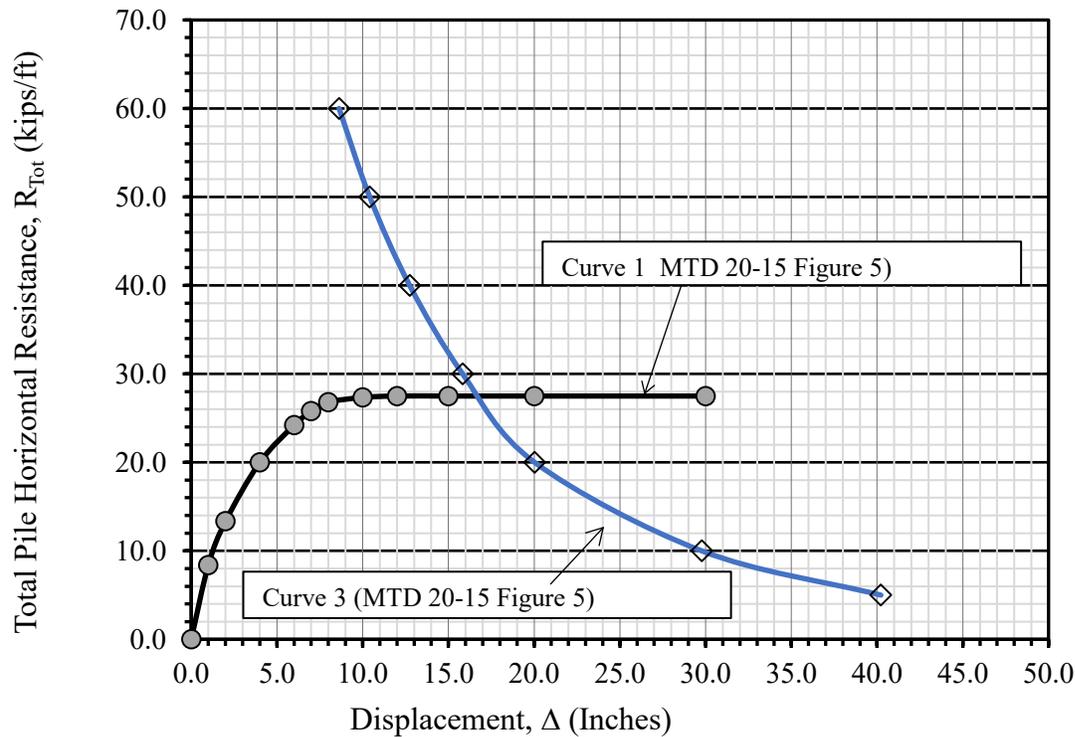


Figure 12. Interaction Curves (Corresponding to Fig. 5 of MTD 20-15)

**Step 9: Send Results of Lateral Spreading Analysis to Structure Designer**

Send SD the results of the geotechnical lateral spreading analysis presented in Table 5 and Figure 11 or Figure 12. The SD will use this information to plot Curve 3 in Figure 5 of MTD 20-15 with Curves 1 and 2. SD will determine the “compatible” liquefaction-induced lateral ground displacement ( $\Delta$ ) at the abutment from the intersection of Curve 2 and Curve 3 in Figure 5 of the MTD 20-15. SD will evaluate these findings and inform GS if the predicted compatible lateral spreading displacement is acceptable or design modifications and additional analysis will be needed.

**Step 10: Prepare and Submit Report**

Once SD confirms that no further geotechnical lateral spread analysis is necessary, report the findings to SD either by incorporating the above analysis into the Foundation Report or preparing a separate technical memorandum. A draft report may be submitted to SD for review and comments. Prepare and submit a final report by addressing the review comments, if any, from SD.

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