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Design of Landslide Stabilization Using the Strength Reduction Method and LRFD

Lucas O. Strom

David A. Varathungarajan

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16. Abstract

Current design methods for landslide stabilization with numerical modeling do not bridge the gap between the strength reduction method (SRM) and load and resistance factor design (LRFD), resulting in designers making assumptions about how LRFD design criteria (which were developed based on limit-equilibrium analyses) translate to design using numerical modeling, a significantly different design approach. It is unclear if these assumptions are representative, resulting in potentially overly conservative or aggressive designs. The purpose of this study is to evaluate the relationship between strength reduction factors used in the numerical analyses and slope stability factors of safety used for different limit states in the LRFD approach. In addition, the study evaluates the relationship between design parameters for stabilizing structural elements (namely shears, bending moments and deflections) as determined by the two approaches and provides preliminary recommendations on load factors to be applied to the SRM-determined shears and moments to obtain equivalent values as determined by LRFD.

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Executive Summary

Current design methods for landslide stabilization with numerical modeling do not bridge the gap between the strength reduction method (SRM) and load and resistance factor design (LRFD), resulting in designers making assumptions about how LRFD design criteria (which were developed based on limit-equilibrium analyses) translate to design using numerical modeling, a significantly different design approach. This study evaluates the relationship between design parameters for stabilizing structural elements (namely shears, bending moments and deflections) as determined by the two approaches and provides preliminary recommendations on load factors to be applied to the SRM-determined shears and moments to obtain equivalent values as determined by LRFD.

Whether following allowable stress design (ASD) or LRFD, slope stability problems in geotechnical engineering have traditionally been approached using limit equilibrium methods (LEM). SRM is an approach which uses numerical analysis to design landslide stabilization systems, requiring fewer assumptions than the LEM approach. In the SRM, a series of analyses are completed by which the model is run to equilibrium using soil shear strength parameters which are incrementally reduced by a strength reduction factor (SRF). The SRF is similar to the traditional global stability factor of safety (FS).

We analyzed a simplified case of a cantilever wall consisting of a row of drilled shafts embedded in a uniform soil profile using both LEM and SRM approaches. The LEM analyses were completed using the Shoring Suite software program, which calculates shears, moments, and deflections for an earth retention system based on input earth pressure loading. The LEM analyses considered nominal earth pressures and factored earth pressures following the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design specifications (AASHTO, 2020). The SRM analyses were completed using FLAC, a finite difference analysis software which considers the effects of soil-structure interaction and calculates mobilized earth pressure loading based on movements in the soil mass. The SRM analyses were run considering SRF values ranging from 1.0 to 1.5.

Based on the results of the cantilever wall analyses, we outline three proposed methods for applying load factors to the SRM-determined maximum moments, so that the factored moments would be equivalent to those determined by the LEM analyses.

Next, we completed SRM analyses of a failing embankment overlying a weak soil layer, to be stabilized using a similar drilled shaft design. We then applied the load factors from the three proposed methods to moments determined for the embankment case. We observed significant variation in the factored moment values among the three methods. At the end of the report, we

suggest aspects of the analysis that will require further study to calibrate the load factors and formally incorporate SRM into LRFD landslide stabilization design.

Given the uncertainty and variability in the LRFD-based SRM methods observed in this study, the authors suggest that practitioners consider both LRFD- and ASD-based methods for design of landslide stabilization structures.

1 Introduction

The use of deep foundation structures (e.g., drilled shafts, driven piles, auger piles) for lateral support of landslide stabilization, alone or in combination with earth retention structures such as ground anchors, has become well-established in geotechnical engineering practice. While traditional limit-equilibrium methods (LEM) remain common for designing landslide repairs, numerical methods can offer several advantages over LEM, when the complexity of numerical methods is justified by difficult project geometry or variations in subsurface properties.

For slope stability problems, of particular interest is the strength reduction method (SRM), a technique by which a slope stability factor of safety (FS) is determined in numerical analyses by reducing the shear strength of the soil by a strength reduction factor (SRF). The maximum stable SRF is equivalent to the FS value. Using this method, the critical failure surface for the slope is determined automatically, rather than by a specified slip surface search method or shape (e.g., circular, logarithmic spiral, block-specified). In addition, service-level slope deformations can be estimated, and in the case of a landslide stabilized by structural elements, the loads transferred from the ground to the structure as shears and bending moments can be estimated for structural design. The problem faced by transportation agencies and other project owners is that it is unclear how SRM-based landslide stabilization designs compare to stabilization designs using LRFD, which is based on LEM. Efforts to bridge the gap between SRM and LRFD rely on assumptions, but the degree of aggressiveness or conservatism of these is unclear.

The purpose of this study is to evaluate the relationship between strength reduction factors used in the numerical analyses and slope stability factors of safety used for different limit states in the LRFD approach. In addition, the study evaluates the relationship between design parameters for stabilizing structural elements (namely shears, bending moments and deflections) as determined by the two approaches and provides preliminary recommendations on load factors to be applied to the SRM-determined shears and moments to obtain equivalent values as determined by LRFD.

2 Background

2.1 Limit Equilibrium Methods

Two-dimensional analysis of slope stability by the limit-equilibrium methods of slices has endured largely unchanged since the 1960s (Fellenius, 1936; Bishop, 1955; Lowe & Karafiath, 1960, Morgenstern & Price, 1965; Spencer, 1967; Janbu, 1968). These methods vary in the types of equilibrium satisfied (moment, vertical or horizontal force, or a combination), the slip surface shape considered, and assumptions regarding interslice forces required to maintain force or moment equilibrium (Duncan, 1996). These methods remain in common practice because they are well-understood, produce results that are sufficiently accurate for a range of slope stability problems, and readily lend themselves to automation using computer programs (e.g., Geo-Slope International, 2021; Rocscience, 2022). These programs can model reinforcing elements (such as drilled shafts, piles, ground anchors, or soil nails) by incorporating external forces representing the structural element at the structure's location on the slope.

2.2 Numerical Methods

A drawback of the limit-equilibrium methods of slices is that they rely on assumptions regarding interslice forces, and the underlying assumption that a sliding soil mass can be divided into slices, itself contrived for the purpose of satisfying force and/or moment equilibrium. In addition, the methods of slices do not provide an estimate of service-level ground deformations. Griffiths and Lane (1999) outline several advantages of the numerical analysis approach over limit-equilibrium methods:

- Assumptions about the shape of the slip surface are not required, and the slope fails 'naturally' where the soil shear strength fails to resist driving shear forces.
- No assumptions are required to resolve interslice forces, because the sliding mass is not divided into slices.
- The numerical analysis can model service-level ground deformations.
- Progressive failure can be observed in the model prior to overall shear failure.

The authors demonstrate close agreement between the numerical and limit-equilibrium methods for several example problems with variations in slope angle, soil shear strength, groundwater, and surface water configurations.

2.3 Strength Reduction Method

The specific application of strength reduction is summarized in Dawson and others (1999), which cites previous research on the method and provides several example analyses using the explicit finite difference program FLAC. A series of analyses are completed while reducing the soil shear strength parameters by the equations:

$$\phi_{reduced} = \tan^{-1}\left(\frac{\tan\phi_{nominal}}{SRF}\right) \qquad (1)$$

$$c_{reduced} = \frac{c_{nominal}}{SRF} \quad (2)$$

In LEM slope stability analyses, the factor of safety is an output of the analysis, generally defined as the ratio of resisting to driving forces calculated while solving equations of force and/or moment equilibrium. In the numerical approach, the SRF value is an input to the analysis. The analyses are repeated with increasing SRF values until the model no longer converges (i.e., the normalized unbalanced force cannot be resolved to zero). The largest SRF value for which the model converges is equivalent to the FS for slope stability.

2.4 SRM for Structurally Stabilized Slopes

A research report completed for the Ohio DOT by Narsavage & Pradel (2021) provides a detailed evaluation of the SRM and compares the method to available LEM guidance for designing structurally stabilized slopes, such as the procedure in ODOT's Geotechnical Bulletin 7 (ODOT, 2020). As the authors discuss, an advantage of the SRM over LEM is the implicit consideration of soil-structure interaction during the SRM analysis; whereas LEM-based designs require the iterative use of multiple programs with more intermediate steps to achieve a similar result (in the case of the ODOT procedure, this includes use of a specialized software UASLOPE).

Gonzalez & Schaefer (2021) compare two analytical methods for design of landslide stabilization: an 'uncoupled' approach (sliding mass evaluated by LEM, structural elements evaluated by a p-y analysis) and 'coupled' approach (sliding mass and structural elements evaluated together by SRM), using three case histories with varying levels of instrumentation data available. Both the uncoupled and coupled approaches generally showed good agreement with the instrumentation data from the case histories. The authors concluded that while the LEM-based uncoupled approach provided more detailed results in better agreement with the field monitoring data for cases where abundant field information was available, the SRM was more effective for cases where field data were not readily available.

2.5 Study Approach

The literature on design of structurally stabilized slopes contains ample comparisons between SRM- and LEM-based designs and establishes the validity of SRM as an analytical technique. However, a design procedure incorporating SRM into LRFD design methods has yet to be developed; this requires analyses to calibrate the SRM results to the results obtained from established LEM procedures. We selected a simplified case of a cantilever wall to determine how the SRM results could be calibrated. The cantilever wall case eliminates many of the variables considered in landslide repair design (e.g., porewater pressures, complex slip surface geometry, three-dimensional effects) to establish a basis for comparison between SRM and

LEM. The cantilever wall case also provides a direct analogue for comparison to established LRFD design methodologies for an embedded retaining wall. After analyzing the cantilever wall case, we repeated the SRM analysis for the case of an embankment underlain by a weak soil layer, to examine how the calibrated SRM analysis might be applied to a landslide repair design.

The following sections present details of our analyses using both SRM and LEM for the cantilever wall case, with the goal of determining appropriate load and strength reduction factors to apply to SRM-derived shears, moments, and deflections to obtain the equivalent values as estimated using LEM-based LRFD procedures. The procedure is then applied to SRM analysis of the embankment on a weak layer case.

3 Cantilever Wall Case

We began with a simplified analysis of a cantilever wall in a uniform soil profile to use as a basis to compare the SRM and LEM approaches (referred to as the Cantilever Wall case). Design procedures for cantilever walls using LEM methods are well established and defined in the AASHTO LRFD Bridge Design Specifications (2020). The analyses primarily involved two computer programs:

- FLAC (Itasca Consulting Group, 2016) analysis to determine shears, moments, and deflections for a range of SRF values.
- Shoring Suite (CivilTech, 2006) analysis to calculate shears, moments, and deflections based on nominal and factored lateral earth pressures, for comparison with FLAC results.

3.1 Geometry and Soil Properties

The Cantilever Wall case was analyzed for a wall with a 15-foot exposed height, with the drilled shaft wall elements extending approximately 38 feet from the top of the cut (embedment equal to approximately 1.5 times the exposed height). The soil profile consisted of a uniform, granular soil. Groundwater was excluded from the analysis. Figure 3-1 shows a schematic of the analysis section geometry.

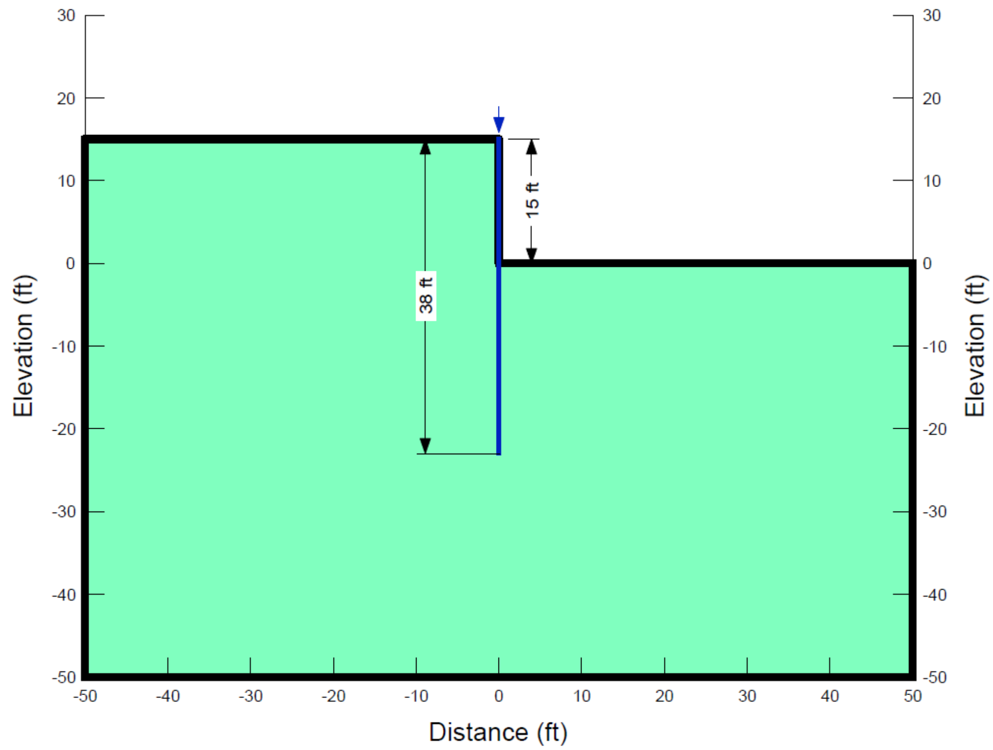

Figure 3-1: Schematic view of Cantilever Wall analysis section.

Soils were assigned a Mohr-Coulomb material model for both the LEM and SRM analyses. For the Cantilever Wall case, we assigned properties consistent with a medium dense, sandy soil. In addition to unit weight and shear strength, FLAC requires the input of elastic parameters (bulk modulus, shear modulus, and Poisson ratio). We selected modulus values based on correlations provided in the Unified Facilities Criteria (UFC) Soil Mechanics manual (U.S. Department of Defense, 2022). The elastic parameters are important for modeling behavior at small strains, whereas SRM analyses are concerned with how the soil mass behaves at the boundary between stability and failure (i.e., large strains). Therefore, the analysis is not especially sensitive to the selected values for the elastic parameters (Narsavage & Pradel, 2021).

Selected properties for the soils used in the Cantilever Wall analyses are summarized in Table 3-1.

Table 3-1: Soil properties used in Cantilever Wall analyses

Soil Unit	Unit Weight (pcf)	Friction Angle, ϕ (deg)	Cohesion Intercept, c (psf)	Bulk Modulus, K (psf)	Shear Modulus, G (psf)	Poisson Ratio, ν
Sandy Soil	125	30	0	3.33×10^5	1.54×10^5	0.3

deg = degrees; pcf = pounds per cubic foot; psf = pounds per square foot

3.2 Drilled Shaft Properties

We modeled the stabilizing structural elements as a single row of 3-foot diameter drilled shafts. We selected a lateral spacing of 3 shaft diameters (9 feet), as this is the generally accepted maximum spacing at which full arching develops (Broms, 1964). Models with greater lateral spacings must consider the potential for soil flow around the shafts. This would require a three-dimensional analysis, which involves considerably more effort than two-dimensional analyses for numerical modeling.

In addition to shaft cross-sectional area (a function of diameter) and spacing, FLAC requires the following inputs for the structural elements: density, elastic modulus, and second moment of area (i.e., moment of inertia). The moment of inertia is determined by the shape and dimensions of the shaft cross-section and the modulus is back-calculated from the bending stiffness, *EI*. The bending stiffness of drilled shafts is a non-linear function of bending moment and axial loading, and decreases significantly upon initial cracking. To obtain a reasonable single value for shaft bending stiffness, we completed a p-y analysis for a drilled shaft using the program LPILE (Ensoft, 2019), with zero axial thrust. Inputs to the program are shown in Table 3-2. The full text of the LPILE analysis is included in the supplemental material accompanying this report.

Table 3-2: LPILE inputs and drilled shaft information for Cantilever Wall p-y analysis

Parameters	Qty
Length (ft)	30
Shaft Diameter (in)	36
Concrete Compressive Strength (psi)	4,500
Cover Thickness (in)	4
Longitudinal Reinforcing Bar Size	#10
Longitudinal Bar Area (in^2)	1.27
Number of Longitudinal Bars	12
Transverse Reinforcing Bar Type	Spiral
Transverse Bar Size	#4
Number of Transverse Bars	28
Transverse Bar Spacing (in)	12
Reinforcing Steel Yield Strength (ksi)	60
Reinforcing Steel Elastic Modulus (ksi)	29,000
Gross Area (in^2)	1,017.9
Steel Area (in^2)	15.24
Reinforcement Ratio (%)	1.5
Moment of Inertia (in^4)	82,447

ft = feet; in = inches; ksi = kilopounds per square inch; psi = pounds per square inch; qty = quantity

After executing the program with the inputs shown in Table 3-2, LPILE produces a chart of bending stiffness versus moment, which is reproduced in Figure 3-2. We selected the value corresponding to the initial cracked condition to determine an elastic modulus value for the drilled shaft.

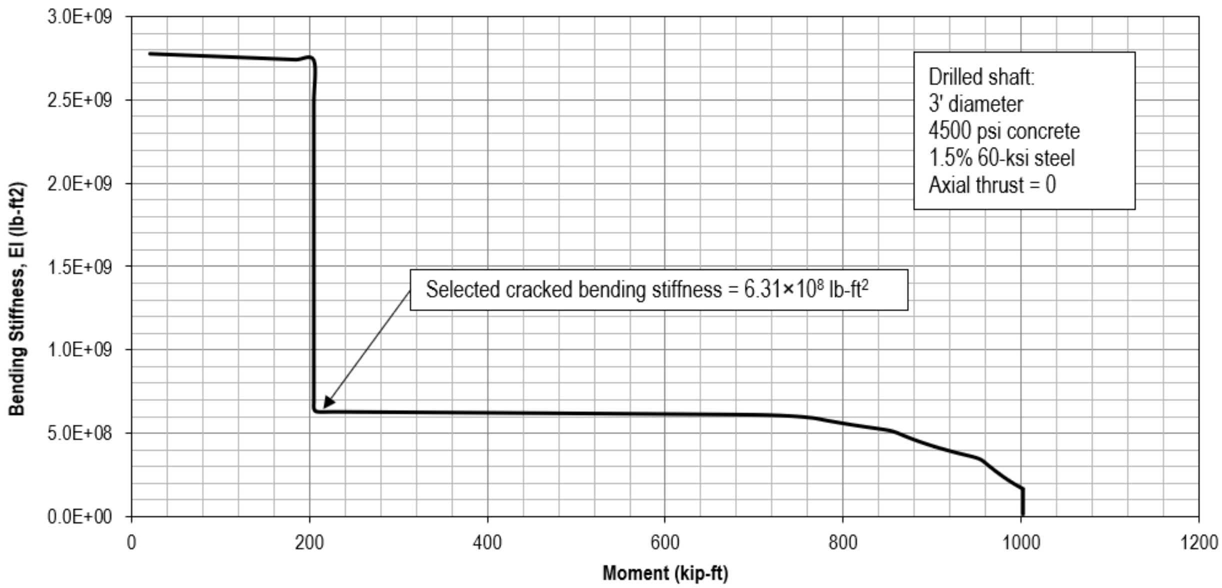

Figure 3-2: Non-linear bending stiffness – moment relationship for drilled shaft (from LPILE).

3.3 Limit Equilibrium Analyses

Shoring Suite is a widely used program developed to assist in design of earth retention systems for shoring applications, such as soldier pile walls, secant/tangent walls, slurry walls, tiebacks (ground anchors), or braced excavations. Based on earth pressure loading (and lateral components of any surcharges, if applicable) the Shoring module of the program calculates shears, moments, deflections, as well as the wall embedment required to satisfy force and moment equilibrium following guidance and requirements in:

- UFC Soil Mechanics Manual (U.S. Department of Defense, 2022),
- USS Steel Sheet Pile Design Manual (U.S. Steel, 1984), and
- FHWA-RD-75 (Goldberg and others, 1976)

We calculated nominal and factored earth pressures for the for the active and passive condition. Load and resistance factors for the strength limit state were used for the factored earth pressures. For the active earth pressure (driving side), we selected a load factor of 1.5 as indicated in Table 3.4.1-1 of AASHTO (2020). For the passive earth pressure (resisting side), we selected a resistance factor of 0.75 as indicated in Table 11.5.7-1 of AASHTO (2020). A plot of nominal and factored earth pressures versus depth is shown in Figure 3-3. These earth pressure distributions were used as part of the inputs to Shoring Suite.

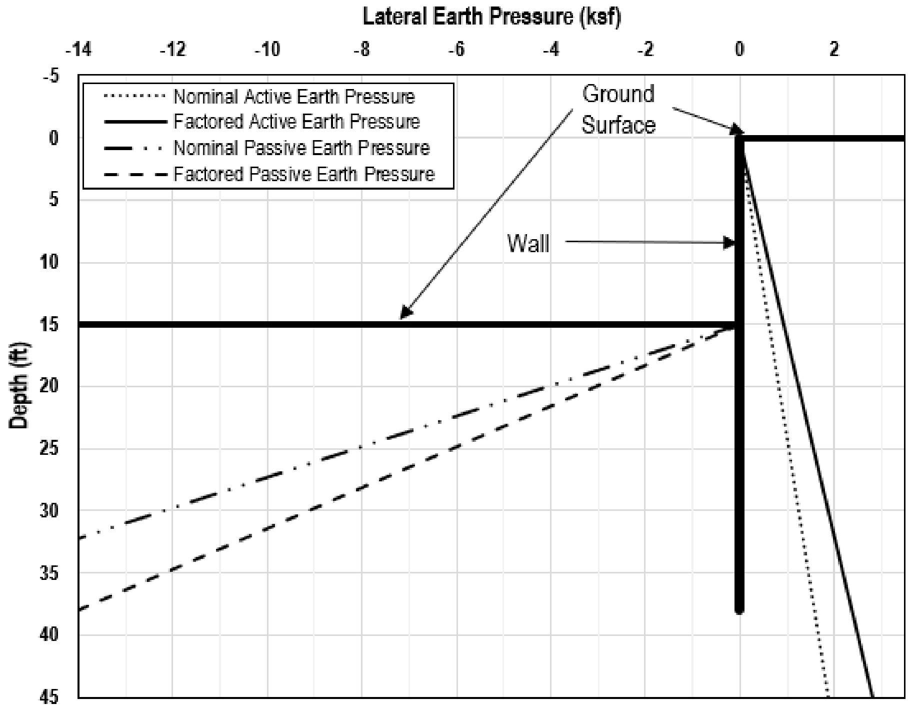

Figure 3-3: Schematic plot of nominal and factored lateral earth pressures.

Shoring Suite inputs include exposed wall height, wall type (drilled shaft, referred to in the program as 'Soldier Pile, Drilled'), pile diameter and spacing, arching parameters, active and passive earth pressure distributions, and parameters for the structural elements including elastic modulus and moment of inertia. Lateral support inputs (e.g., for bracing) were not used. An option is available to reduce the passive resistance by a factor of safety; we did not apply the reduction in our analysis because the input passive earth pressure was factored by the AASHTO-specified resistance factor, but this will be discussed as part of the comparison of results. Another option is available to fix the shaft embedment to a certain depth below the cut, but instead of fixing the embedment, we allowed the program to determine the embedment required to satisfy force and moment equilibrium.

A complete list of Shoring Suite inputs for the Cantilever Wall case, as well as the program output, are included in the supplemental material accompanying this report.

3.4 Finite Difference Analyses

We completed numerical analyses using the software FLAC version 8.0 (Itasca Consulting Group, 2016), a two-dimensional plane-strain, finite difference program specifically developed to model soil-structure interaction. The program represents soil/rock as a connected grid of zones assembled to represent the geometry of the problem being modeled. Each zone is assigned a constitutive model that represents the stress-strain behavior and failure criteria of the material being modeled. Mass, elasticity, and strength properties are assigned to each zone as input parameters for the associated constitutive model. Construction operations such as excavating, placing fill, and other changes in applied stresses are modeled by adding or removing zones or by applying forces at nodes. At each step of the modeling process, a system of linear equations is explicitly solved by evaluating changes in stresses and displacements at each of the nodes to obtain a condition of static equilibrium.

For the Cantilever Wall case, a mesh with a grid size of 1-foot squares was created to represent the soil mass. The left, right, and bottom sides of the grid are truncation boundaries which were set at a sufficient distance from the area of interest to avoid boundary effects. Nodes at the left and right boundaries were fixed against horizontal displacement, but free to move vertically (roller boundary condition). Nodes at the bottom boundary were fixed against both vertical and horizontal displacement (pinned boundary condition).

The modeling process implemented for the project consisted of the following steps:

1. A grid representing the soil mass was run to static equilibrium under gravitational acceleration to initialize in-situ stresses.
2. Structural elements were added to the model. The drilled shafts were modeled using "beam" elements using elastic parameters corresponding to a cracked 3-foot diameter drilled shaft. The properties of the structural elements are as previously discussed in Section 3.2.

   Interface elements were used to connect both the retained and excavated sides of the beam elements to the grid. The interface elements allow the grid to slide and separate relative to the beam elements (i.e., the interface element allows for modeling of potential loss of ground in front of the drilled shafts). The interfaces were assigned properties corresponding to adjacent soil properties in accordance with recommendations in the FLAC manual (Itasca Consulting Group, 2016). After adding the beam elements to the problem, the model was run to equilibrium.
3. To simulate the excavation for the cantilever wall, zones were incrementally removed until the excavation reached the final depth, with the model run to equilibrium after each increment. The end of this step represents the SRF = 1.0 case.
4. After the excavation was completed, displacements were reset to zero, and the soil strengths were incrementally reduced to determine the FS for the slope and wall system. The soil strengths were reduced using equations 1 and 2 described in Section 2.

   As the soil/rock strengths were reduced, we monitored shear forces, bending moments, and deflections of the drilled shafts in the model. Our analyses considered a design FS value of 1.3 for the strength limit state, but continued the strength reduction to a maximum SRF value of 1.5.

Text files containing the FLAC inputs for each step are included in the supplemental material accompanying this report. Figure 3-4 shows the analysis grid after step 3.

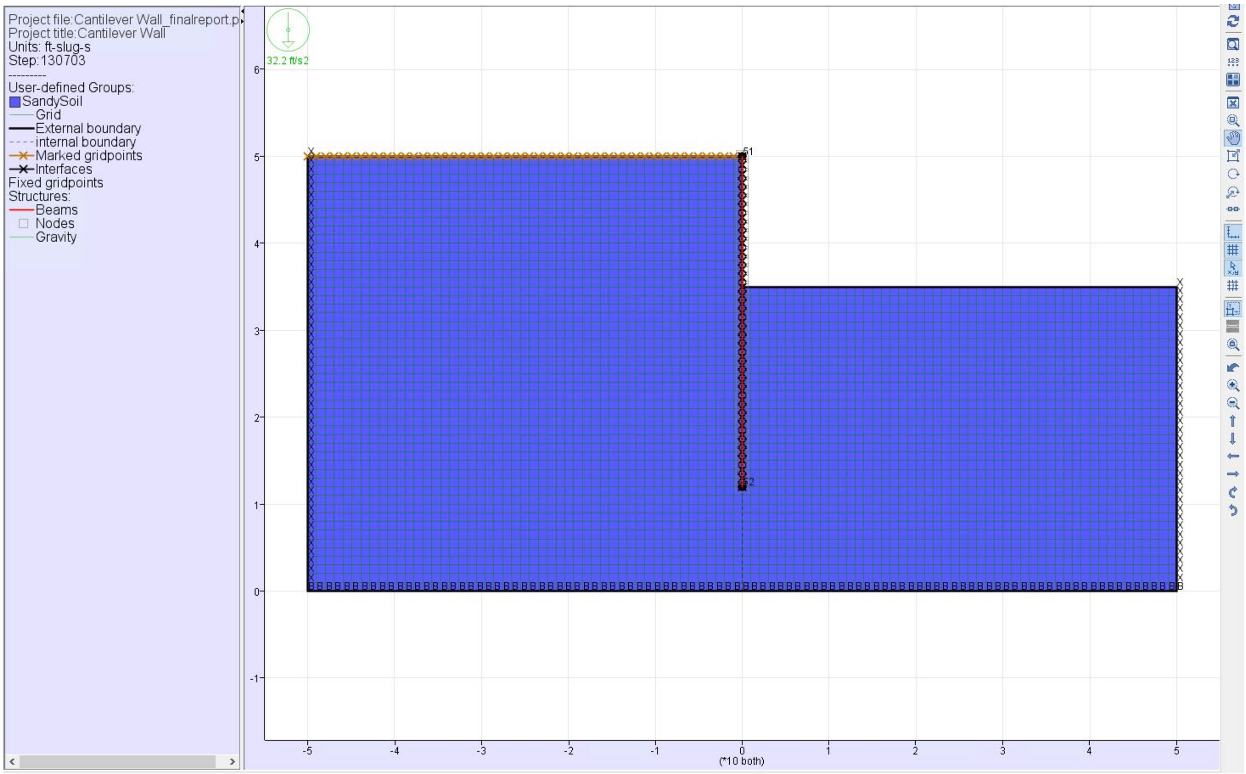

Figure 3-4: Analysis model for the Cantilever Wall case as viewed in FLAC, showing the grid, structural beam elements, and boundary conditions for the model.

3.5 Comparison of Analysis Results

The shears, moments, and deflections along the length of the drilled shaft are plotted for the SRM and LEM analyses in Figure 3-5.

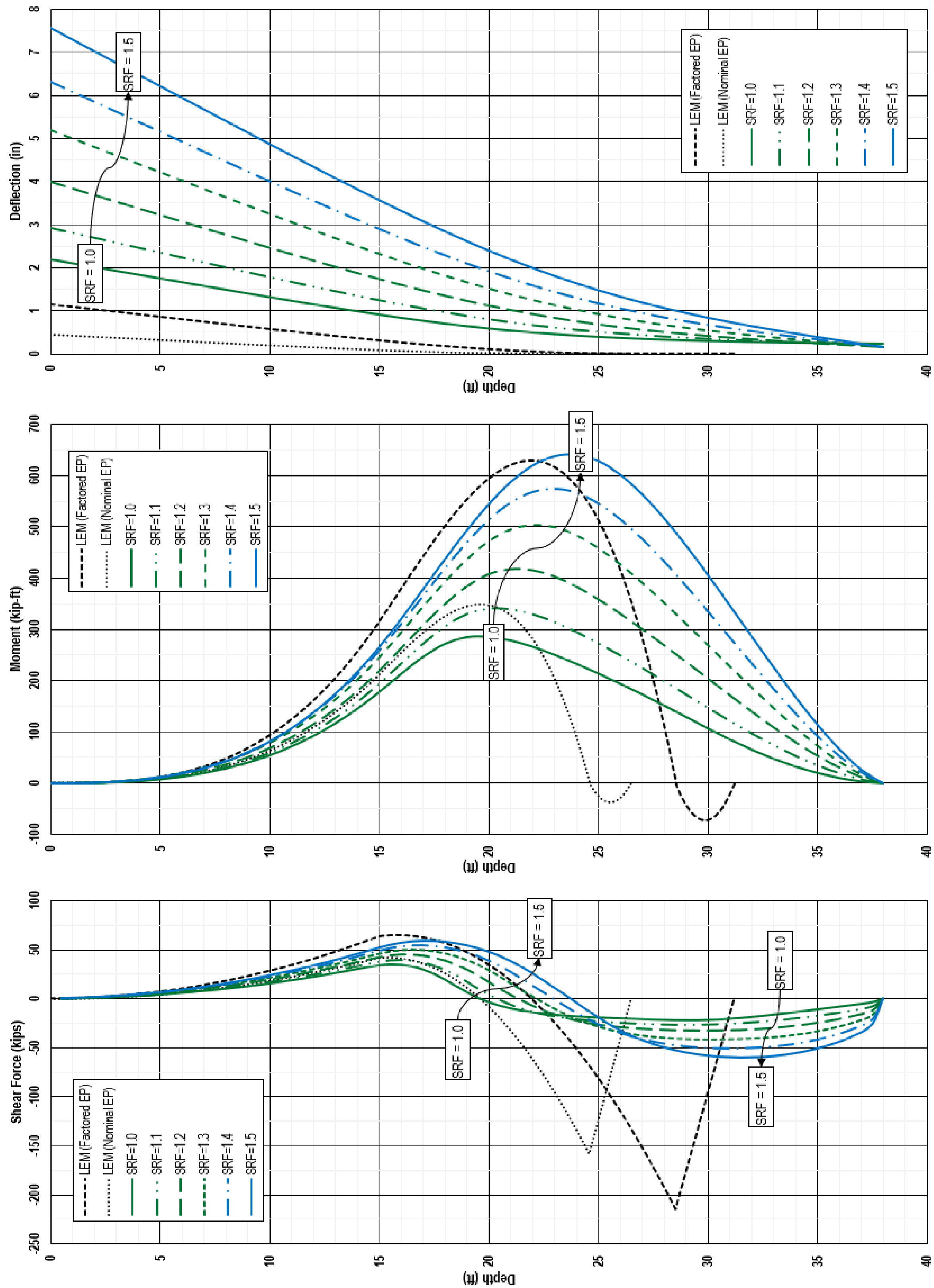

Figure 3-5: Shear, moment, and deflection plots along drilled shaft length for Shoring Suite (LEM) and FLAC (SRM) analyses.

Several observations can be drawn from the results:

- There are similarities in the shapes of the distributions of results, but significant variation between the LEM and SRM results in the maximum values (which are what would be used for design).
- The embedment depth recommended by Shoring Suite varies from the value assumed for the FLAC analyses, and varies depending upon whether the input earth pressures are nominal or factored.
- The shears determined from the LEM analyses show a sharp negative peak toward the bottom of the shaft. This appears to be a product of how Shoring Suite resolves force equilibrium for the earth pressures below the cut (i.e, the input passive pressure is not reduced to account for the displacement required to mobilize the full passive resistance, as is inherently considered in the FLAC analyses).
- The maximum moment value for the nominal earth pressure LEM analysis is similar to the maximum moment value for an SRF of 1.1.
- The maximum moment value for the factored earth pressure LEM analysis is greater than the maximum moment value for a SRF value of 1.3, and is closer to the value for a SRF of 1.5.
- The maximum deflection values for the LEM cases are significantly lower than the maximum deflection values for the SRM analyses.

The results of the SRM and LEM analyses depend strongly on the assumed earth pressures. The earth pressure distributions calculated in FLAC incorporate the effects of soil-structure interaction, whereas Shoring Suite uses the earth pressure distributions exactly as they are input to the program. Figure 3-6 shows FLAC-determined earth pressure distributions for SRF values of 1.0 and 1.3, plotted together with the earth pressure distributions input to Shoring Suite (as previously shown on Figure 3-2).

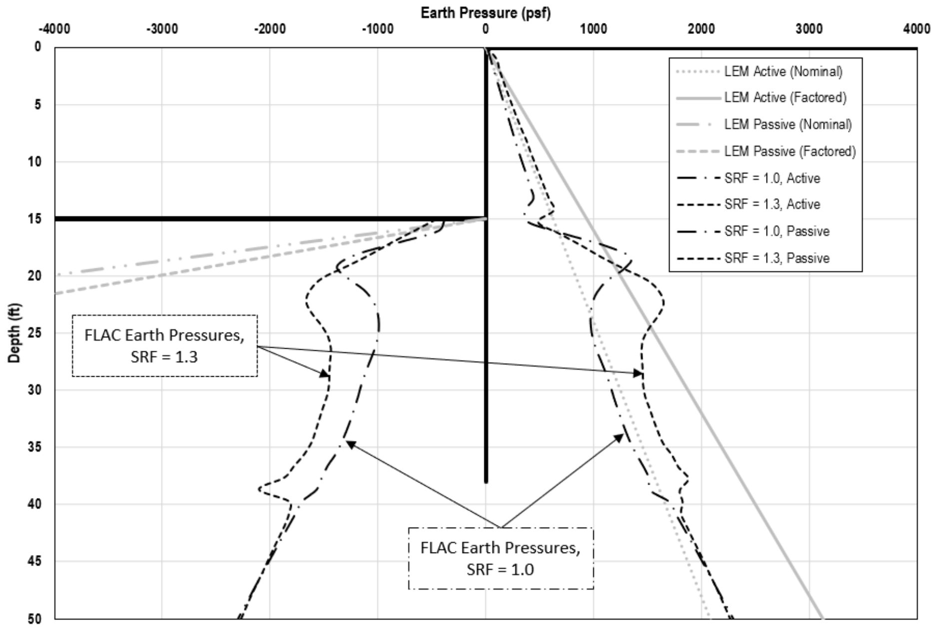

Figure 3-6: Comparison of earth pressure distributions used in the LEM and SRM analyses.

On the retained side, the FLAC earth pressures are closer to the active earth pressures used in Shoring Suite, but exceed the assumed active earth pressures where the wall rotates back into the retained zone. The earth pressures mobilized on the cut side of the wall in the SRM analysis are significantly lower than both the nominal and factored passive earth pressures assumed for the LEM analysis. In the SRM analysis, the movement of the drilled shaft into the cut side is not sufficient to mobilize the passive resistance values assumed in the LEM analysis. This is consistent with allowable stress design approaches for embedded walls, wherein a greater reduction is placed on the passive pressure (a factor of safety of 2 or higher, which is similar to a resistance factor of 0.5 or less, compared to the AASHTO-specified resistance factor of 0.75) to account for the deformation required to mobilize passive pressure. As discussed in Section 3.3, because our analysis followed LRFD specifications for reduction of passive resistance, we did not apply the additional reduction to the LEM passive earth pressure that would be typically used in an allowable stress design approach.

The results from each analysis approach can be compared by computing the 'result ratio,' i.e., taking the maximum value for a given parameter as determined in Shoring Suite and dividing by the maximum value of that parameter as determined in FLAC for a given SRF value. The following two subsections discuss the result ratios for the nominal and factored earth pressure LEM analyses.

3.5.1 Nominal Earth Pressures

Result ratios for the LEM analysis using nominal earth pressures versus SRF values are shown in Table 3-3, followed by a discussion of design implications for the service limit state.

Table 3-3: Result ratios for LEM with nominal earth pressures

Force, Moment, and Deflection Ratio	SRF Value 1.0	SRF Value 1.1	SRF Value 1.2	SRF Value 1.3	SRF Value 1.4	SRF Value 1.5
$V_{max(-)}$	7.08	6.04	4.85	3.79	3.11	2.61
$V_{max(+)}$	1.21	1.09	0.95	0.84	0.78	0.73
M_{max}	1.22	1.02	0.83	0.69	0.61	0.54
δ_{max}	0.21	0.16	0.11	0.09	0.07	0.06

NOTE:

Result ratios are computed by dividing the maximum parameter value from Shoring Suite (LEM) analysis by the maximum parameter value from FLAC (SRM) analysis.

$V_{max(-)}$ = maximum negative shear; $V_{max(+)}$ = maximum positive shear; M_{max} = maximum moment, δ_{max} = maximum shaft deflection

Based on these results, some observations can be drawn regarding the LEM analysis using nominal earth pressures:

- Because the nominal passive earth pressures used in the LEM analysis are much greater than the passive earth pressures mobilized in the SRM analysis, the deflections output by the LEM-nominal case are significantly lower than those predicted by the SRM analysis at a SRF value of 1.0.
- The maximum moment calculated in the LEM analysis is close to that calculated in the SRM analysis at a SRF value of 1.1.

3.5.2 Factored Earth Pressures

Result ratios for the LEM analysis using strength limit state factored earth pressures versus SRF values are shown in Table 3-4, followed by a discussion of design implications for the strength limit state.

Table 3-4: Result ratios for LEM with factored earth pressures

Force, Moment, and Deflection Value	SRF Value 1.0	SRF Value 1.1	SRF Value 1.2	SRF Value 1.3	SRF Value 1.4	SRF Value 1.5
$V_{max(-)}$	9.59	8.19	6.57	5.14	4.22	3.54
$V_{max(+)}$	1.83	1.64	1.44	1.27	1.18	1.10
M_{max}	2.21	1.85	1.518	1.25	1.10	0.98
δ_{max}	0.52	0.39	0.29	0.22	0.18	0.15

NOTE:

Result ratios are computed by dividing the maximum parameter value from Shoring Suite (LEM) analysis by the maximum parameter value from FLAC (SRM) analysis.

$V_{max(-)}$ = maximum negative shear; $V_{max(+)}$ = maximum positive shear; M_{max} = maximum moment, δ_{max} = maximum shaft deflection

Based on these results, some observations can be drawn regarding the LEM analysis using factored earth pressures, for the strength limit state:

- For typical wall and landslide repair design, the maximum moment is more likely than maximum shear to control the shaft design. Therefore, the focus of this discussion for the strength limit will be on the maximum moment.
- Based on the result ratios for the factored earth pressure LEM case, there are multiple possible ways to factor the SRM-determined maximum moment to match the LEM maximum moment.
  - Method A: The maximum moment from the factored LEM case is approximately 1.25 times the value determined in the SRM analysis at a SRF value of 1.3. Therefore, a load factor of 1.25 could be applied to the moments determined from a SRM analysis designed to a SRF value of 1.3.
  - Method B: AASHTO (2020) Table 3.4.1-2 specifies a load factor of 1.5 for loads due to horizontal active earth pressures. This load factor is approximately equal to the maximum moment result ratio of the factored LEM case to the SRM at a SRF value of 1.2. Therefore, the SRF of 1.2 could be considered comparable to the service limit. This same load factor could be applied to the service limit moments determined by the SRM analysis to obtain a LRFD-equivalent design moment.
  - Method C: The maximum moment from the factored LEM case matches that of the SRM analysis for a SRF value of 1.5. Therefore, it could be suggested that designers using SRM for a stabilization design use a target SRF of 1.5 instead of 1.3; however, this is likely not practical for active landslides, and the resulting slope movements would be too large and cause numerical instability.

The proposed load factors for deflections and moments at the service and strength limit states will require confirmation by additional analyses that more closely simulate a landslide problem.

4 Embankment on Weak Layer

The next step in the study was to complete a SRM analysis in a manner similar to the Cantilever Wall case, applied this time to the case of an embankment founded on a weak soil layer.

4.1 Geometry and Soil Properties

The Embankment on Weak Layer case was analyzed for a 40-foot-high embankment sloped at 2H:1V (horizontal to vertical). The soil profile consisted of a compacted embankment material, underlain by a 14-foot-thick layer of lower shear strength soil, underlain by a 'competent' layer of stronger soil down to the bottom of the model. Contacts between soil layers were horizontal, and groundwater was excluded from the analysis.

Similar material properties were used for the embankment material and 'competent' layer as for the soil used in the Cantilever Wall case. The shear strength of the weak layer was adjusted until the model showed a FS value of 1.0 for the unreinforced slope in SLOPE/W. The soil properties used in the analysis are shown in Table 4-1, and the SLOPE/W analysis used to calibrate the shear strength of the weak layer is shown in Figure 4-1. The results shown in Figure 4-1 are from an analysis using the Morgenstern & Price (1965) method of slices; analyses completed using the Bishop (1955) and Spencer (1967) methods produced FS values within 1 percent of the Morgenstern & Price result, with similar slip surface geometry.

Table 4-1: Soil properties used in Embankment on Weak Layer analyses

Soil Unit	Unit Weight (pcf)	Friction Angle, ϕ (deg)	Cohesion Intercept, c (psf)	Bulk Modulus, K (psf)	Shear Modulus, G (psf)	Poisson Ratio, ν
Embankment	130	30	50	5.00×10^5	2.31×10^5	0.3
Weak Layer	120	14	0	3.33×10^5	7.14×10^4	0.4
Competent	125	32	0	3.33×10^5	1.54×10^5	0.3

deg = degrees; pcf = pounds per cubic foot; psf = pounds per square foot

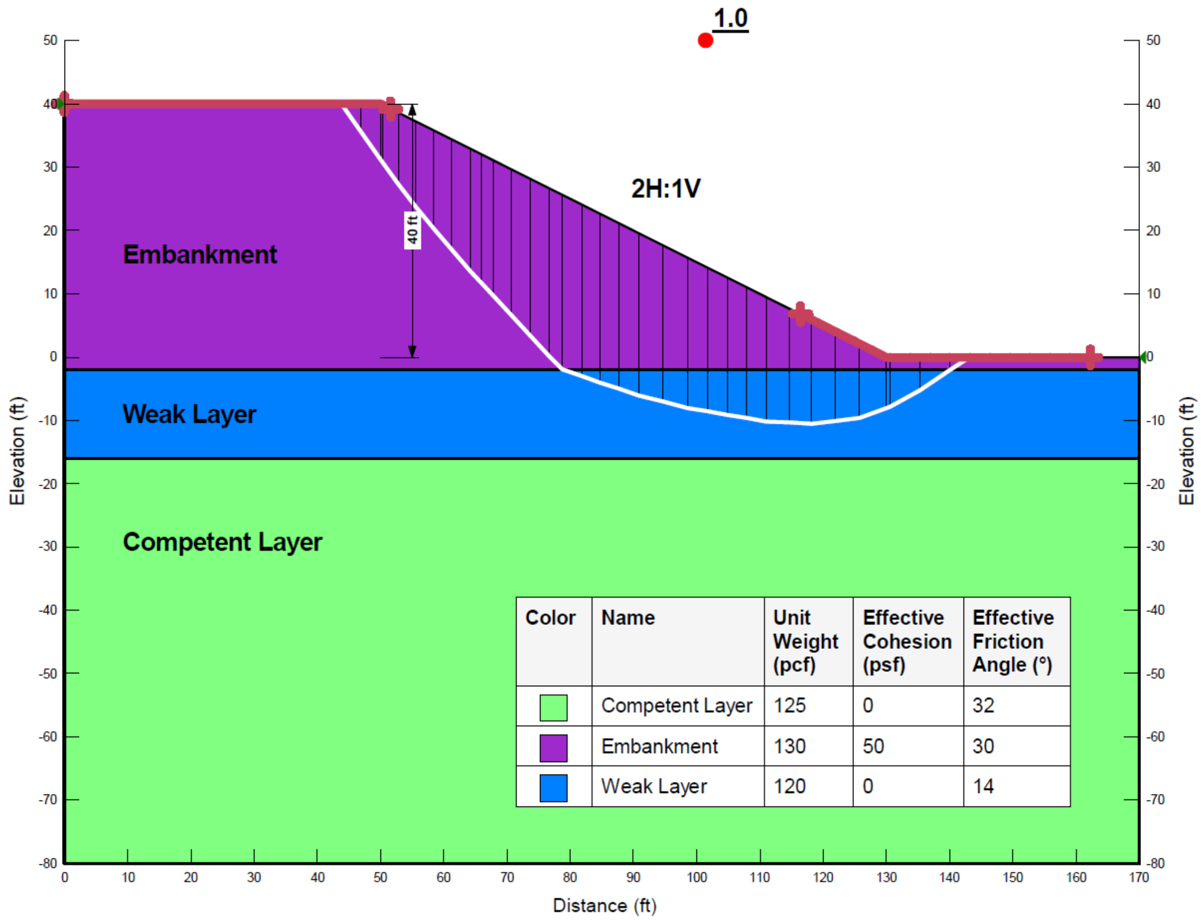

Figure 4-1: Schematic view of Embankment on Weak Layer geometry and back-analysis.

4.2 Drilled Shaft Properties

For the Embankment on Weak Layer case, we initially used drilled shafts with the same diameter, spacing, and bending stiffness as were used in the Cantilever Wall case. However, the higher loading from the embankment relative to the Cantilever Wall case resulted in excessive deflections and structural loads, so we adjusted the drilled shaft design by increasing the diameter to 4 feet and decreasing the lateral spacing to 2 shaft diameters. The shafts were placed within the lower third of the slope, which is typical for landslide repair design. The depth from the top of the shaft to the bottom of the weak layer was 30 feet, and the shaft embedment was set to 1.5 times this value, for a total shaft length of 75 feet. The LPILE inputs for the updated design are shown in Table 4-2, and the resulting plot of bending stiffness versus moment is shown in Figure 4-2.

Table 4-2: LPILE inputs and drilled shaft information for Embankment on Weak Layer p-y analysis

Parameters	Qty
Length (ft)	75
Shaft Diameter (in)	48
Concrete Compressive Strength (psi)	4,500
Cover Thickness (in)	4
Longitudinal Reinforcing Bar Size	#14
Longitudinal Bar Area (in^2)	2.25
Number of Longitudinal Bars	12
Transverse Reinforcing Bar Type	Spiral
Transverse Bar Size	#5
Number of Transverse Bars	28
Transverse Bar Spacing (in)	12
Reinforcing Steel Yield Strength (ksi)	60
Reinforcing Steel Elastic Modulus (ksi)	29,000
Gross Area (in^2)	1,810
Steel Area (in^2)	27
Reinforcement Ratio (%)	1.5
Moment of Inertia (in^4)	260,000

ft = feet; in = inches; ksi = kilopounds per square inch; psi = pounds per square inch; qty = quantity

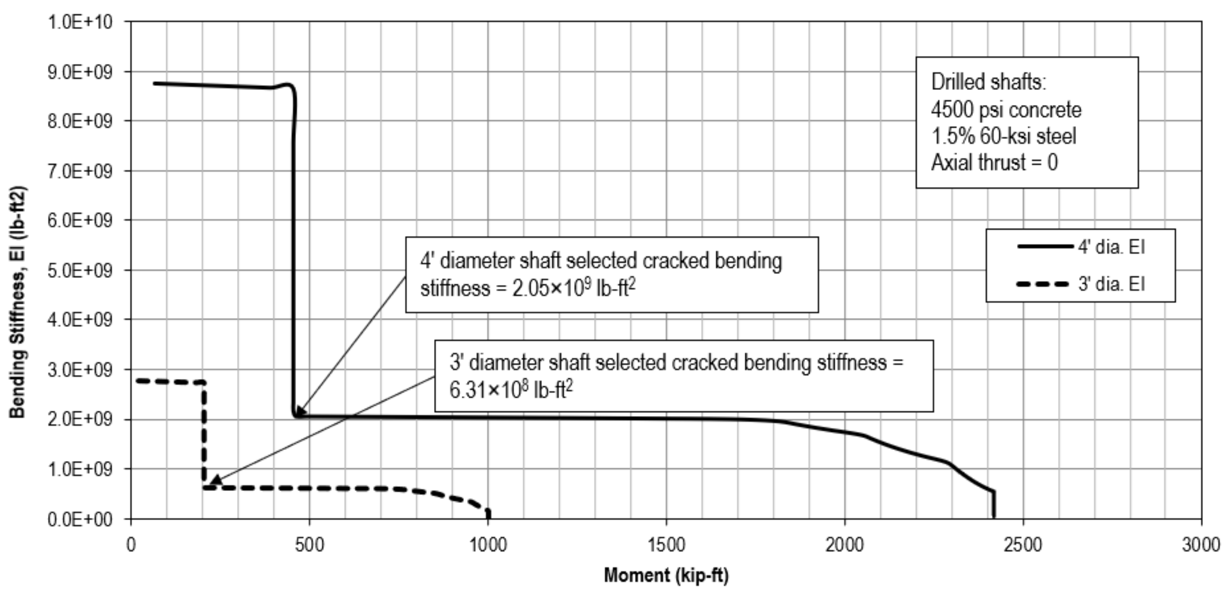

Figure 4-2: Non-linear bending stiffness – moment relationship for 4-foot diameter drilled shaft (from LPILE). The bending stiffness – moment relationship for the 3-foot diameter shaft used in the Cantilever Wall Case is included for comparison.

4.3 Finite Difference Analyses

For the Embankment on Weak Layer case, a mesh with a grid size of one-foot squares was created to represent the soil mass. Boundary conditions were set in the same ways as for the Cantilever Wall case. The modeling process implemented for the project consisted of the following steps:

1. A grid representing the soil mass was run to static equilibrium under gravitational acceleration to initialize in-situ stresses. This also represents the FS = 1.0 case (i.e., slope is failing). Figure 4-3 shows a plot of the maximum shear strain increments in the FLAC model after step 1 with the critical slip surface identified by the SLOPE/W analysis added in white. While the precise location of the slip surface differs between the two methods, the shape of the slip surfaces is consistent.

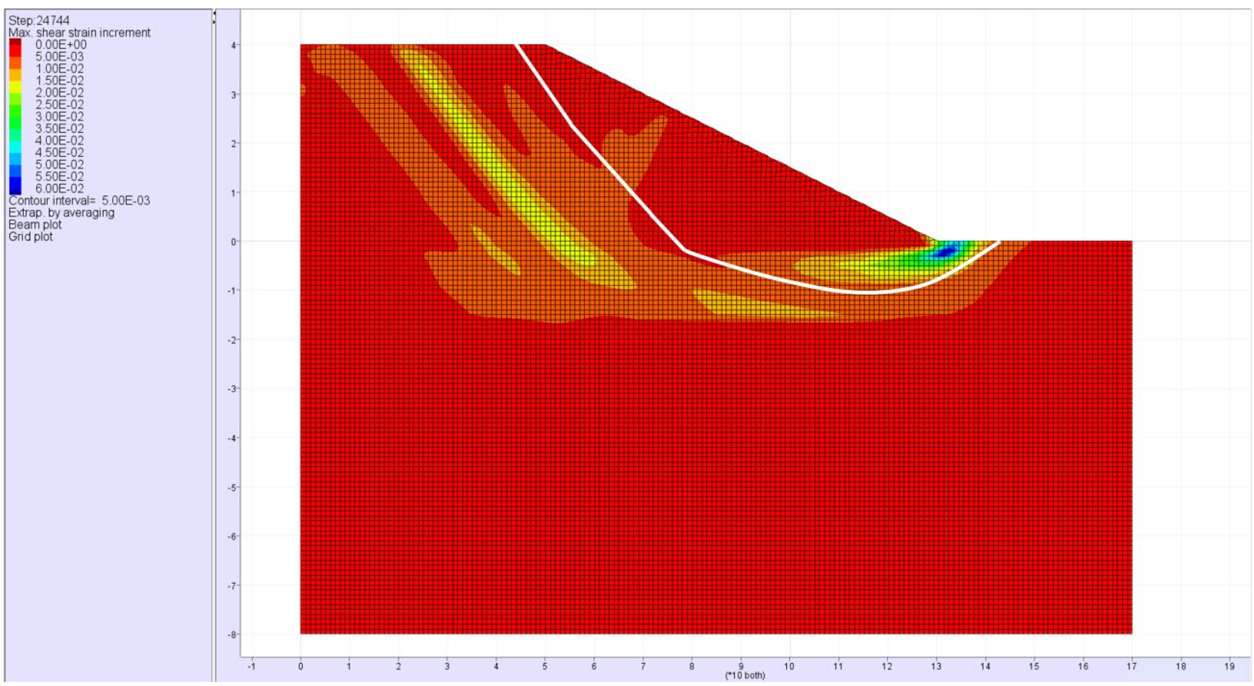

Figure 4-3: Plot of maximum shear strain increment for the Embankment on Weak Layer model following initialization, showing where shear movement has occurred in the model.

2. Structural elements were added to the model. The drilled shafts were again modeled using "beam" elements as described in Section 3.4. The properties of the structural elements are as previously discussed in Section 4.2. The beam elements were connected to the soil mass via interfaces. A weighted average of the interface friction for the three soil ways was used as a single value of for the full length of the interfaces. After adding the beam elements and interfaces, the model was run to equilibrium. This step represents the SRF = 1.0 case for the stabilization design.

3. After the beam elements were added, displacements were reset to zero, and the soil strengths were incrementally reduced to determine the FS for the slope and wall system. The soil strengths were reduced using equations 1 and 2 described in Section 2.

As the soil/rock strengths were reduced, we monitored shear forces, bending moments, and deflections of the drilled shafts in the model. As with the Cantilever Wall case, the model was analyzed with SRF values up to 1.5.

Text files containing the FLAC inputs for each step are included in the supplemental material accompanying this report. Figure 4-4 shows the analysis grid after step 2.

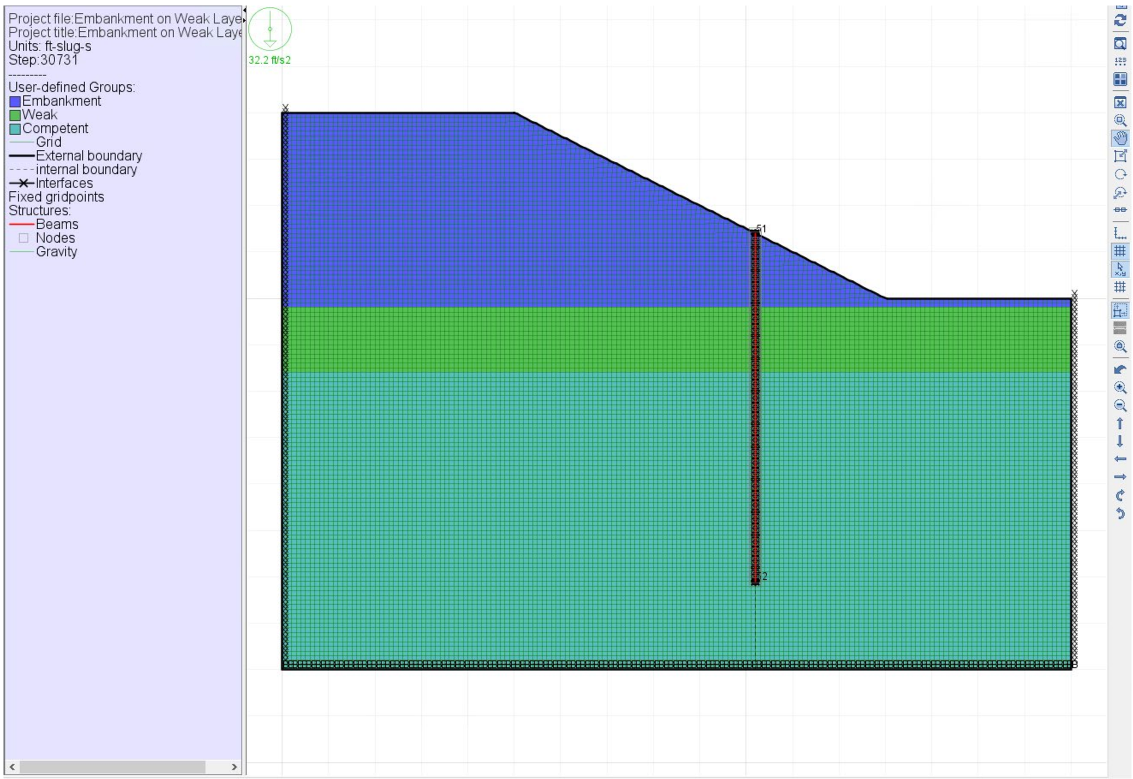

Figure 4-4: Analysis model for the Embankment on Weak Layer case as viewed in FLAC, showing the grid, structural beam elements, and boundary conditions for the model.

4.4 Analysis Results

The shears, moments, and deflections along the length of the drilled shaft are plotted for the SRM analyses in Figure 4-5.

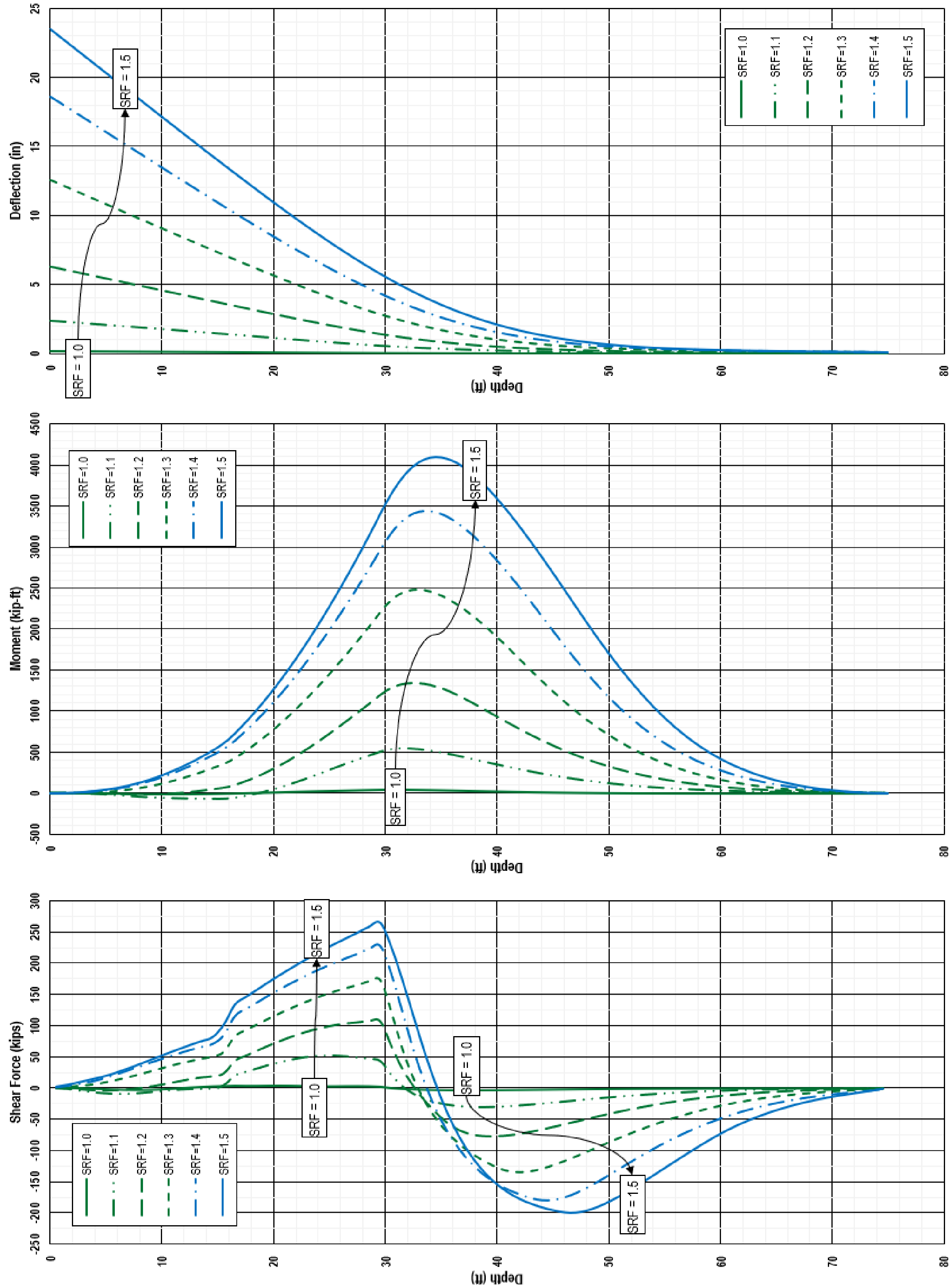

Figure 4-5: Shear, moment, and deflection plots along drilled shaft length for FLAC (SRM) analysis of Embankment on Weak Layer case.

These results show an increase in shears, moments, and deflections progressing with increasing SRF value that is similar to what was observed for the Cantilever Wall case. Some observations drawn from the results:

- There is a large increase in deflection at the top of the shaft between SRF values of 1.1 (approximately 2 inches) and 1.2 (approximately 6 inches), and at a SRF of 1.3 the maximum deflection increases to approximately 12.5 inches. These large movements may not be considered acceptable in an actual landslide stabilization project, depending on the purpose of the repair and the project's design criteria for movement.
- The initial LPILE analysis completed to establish the selected stiffness parameters indicated a nominal moment capacity of approximately 2,374 kip-ft, which is exceeded in this analysis for SRF values of 1.3 and greater. However, the drilled shafts were modeled elastically, i.e. structural yield was not possible.

In practice, these issues could be addressed by adjusting the shaft diameter, reinforcement, and spacing, or adding a secondary stabilizing structural element, such as ground anchors. For this study, we assumed shaft properties for both the Cantilever Wall and Embankment on Weak Layer cases to illustrate the analysis procedure rather than to complete a stabilization design.

4.5 Application of Design Procedure

The discussion in Section 3.5.2 includes three proposed methods to factor SRM-calculated maximum moments to equivalent maximum moments determined by a LEM analysis using AASHTO LRFD-factored earth pressures. Applying the three methods to the results of the Embankment on Weak Layer case:

- Method A: apply a load factor of 1.25 to the maximum moment determined for a SRF value of 1.3. The maximum moment at SRF = 1.3 was 2,476 kip-ft, so the LEM-equivalent factored design moment would be 3,095 kip-ft.
- Method B: apply a load factor of 1.5 to the maximum moment determined for a SRF value of 1.2. The maximum moment at SRF = 1.2 was 1,345 kip-ft, so the LEM-equivalent factored design moment would be 1,614 kip-ft.
- Method C: use the maximum moment determined for a SRF value of 1.5, without applying a load factor. The maximum moment at SRF = 1.5 was 4,088 kip-ft, so this would be the LEM-equivalent factored design moment.

4.6 Evaluation of Design Procedure

The three proposed methods for factoring SRM-calculated maximum moments used load factors calibrated based on the Cantilever Wall analyses to provide an equivalent LRFD-factored moment for drilled shaft design. However, the resulting moments show significant variation when applied to the Embankment on Weak Layer case. This may be due to one or more of several factors:

- The drilled shaft sizing, stiffness, and spacing used may not be sufficient to resist the earth pressure loading from the embankment. Factored moments using the proposed load factors from the Cantilever Wall case may show less variation for a stabilization designed to resist the increased Embankment loading.
- The load factors developed for a homogenous soil profile may not account for the effects of a multi-layered soil profile, particularly one with a weak layer.
- The load factors developed for the Cantilever Wall case may not account for the passive earth pressures against the drilled shaft along the bottom of the slope (above the horizontal ground surface), which may affect the loading on the shaft.
- The load factors developed for the Cantilever Wall case may not be applicable for the more significant ground deformations that occur in modeling of landslides.

Several considerations for further study are provided in the following section.

5 Recommendations for Additional Study

The analyses described in this report are limited to two cases using simplified geometry and subsurface conditions. In order to develop a formalized design procedure incorporating strength reduction methods to the LRFD design framework, the load factors and strength reduction factors described in Section 3 will need to be calibrated using cases considering more realistic landslide conditions. Potential test cases include:

- Modeling a landslide case involving a slope moving on an inclined residual-strength slip surface, rather than the horizontal weak layer analyzed in Section 4.
- Testing the application of strength reduction to the slip surface layer only, rather than all materials in the model, or utilizing different strength reduction factors on the slip surface than for other materials.
- Incorporating the effects of porewater pressures.
- Examining the relationship between LEM and SRM results for analyses incorporating stabilizing structural elements other than drilled shafts (e.g., ground anchors).
- Apply the design procedure to case histories of landslide stabilization projects with instrumentation data, such as inclinometer measurements of slope movements, strain gauge data for structural response of the stabilizing elements, or pressure cells to measure mobilized earth pressures. With this information, an evaluation could be made of how design moments developed using Methods A, B, and C compare to (1) the implemented design, (2) the measurements made of the constructed repairs, and (3) if the different design methods produce similar results for different landslide repair case histories.

If a consistent set of load factors cannot be calibrated after considering the test cases listed above, a possible alternative approach would be to design the stabilization system by incrementally reducing the soil strengths until the maximum moment in the shaft reaches the allowable moment capacity (as determined by an ASD approach). The SRF resulting in this condition could be considered the FS for the stabilized slope. While this design approach would not rigorously follow LRFD procedures, it may be preferable given the apparent uncertainty and variability in appropriate SRFs and load factors for use in landslide mitigation. Until further study and calibration of appropriate LRFD load factors and SRFs can be determined for landslide design using numerical modeling, the authors suggest that practitioners consider the three design methods presented herein in addition to an ASD approach.

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