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Division of Production Management
Office of Geotechnical Engineering

GB 7

Geotechnical Bulletin

DRILLED SHAFT LANDSLIDE STABILIZATION DESIGN

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This Geotechnical Bulletin (GB) is intended to provide guidance on design of drilled shafts for landslide stabilization. The most common method of remediation for an unstable existing slope consists of digging out the failed soil mass in a benched excavation and reconstructing the slope with compacted engineered fill, per GB2. However, in some situations with adverse slope geometry, limited right-of-way, or the failure of a river bank where the toe of the failure extends out into the bottom of the river, a structural solution must be employed. In this instance, drilled shafts can often be used to stabilize the existing unstable slope by embedding the shafts into a lower, stable stratum, and utilizing the mechanism of soil arching between the shafts to increase the nominal resistance against sliding to the point of stability. Where soil arching does not yield adequate resistance, a more robust structural solution such as a retaining wall may be necessary.

This bulletin contains guidance on all aspects of the design process, including landslide reconnaissance and exploration. This bulletin also contains a user’s guide for the University of Akron Slope Analysis Program, UA Slope 2.1, which is used in the Liang method analysis to determine the single shaft load for one row of evenly spaced shafts using soil arching for landslide stabilization. Additionally, this bulletin provides guidance for utilizing the program LPILE, developed by Ensoft, Inc., for design of drilled shafts reinforced with structural steel sections.

This bulletin and other information may be obtained from the Office of Geotechnical Engineering’s Web site (<http://www.dot.state.oh.us/Divisions/Engineering/Geotechnical/>). This Web site contains other ODOT Geotechnical documents and bulletins and has online copies of the Specifications for Geotechnical Exploration (SGE) and Geotechnical Engineering Design Checklists that are referenced in this bulletin.

A. SITE RECONNAISSANCE AND EXPLORATION

This section provides guidance for site reconnaissance, survey limits, exploratory drilling, and in-situ and laboratory testing of soil and bedrock for landslide remediation projects. Recommendations for installation of instrumentation, in the form of inclinometer casing and monitoring wells, are also provided. Subsurface exploration is a necessity for the analysis and design of a drilled shaft landslide stabilization solution. The analyses involved are quite rigorous, and the more data that is acquired; the more precisely the inputs can be estimated, and the more realistic the outputs will be.

1. Site Reconnaissance

a. Office Publication Search

Prior to making a site visit, the geotechnical engineer should endeavor to obtain as much knowledge about the geologic setting of the site as possible. A thorough search of various geologic, soil, and water resource publications can yield a large amount of information, which may provide greater insight to the probable causes and form of the landslide failure, and may immediately demonstrate which kinds of remediation options are possible and which kinds are not possible. See SGE Section 302.2, Office Reconnaissance, and the Geotechnical Engineering Design Checklists, Section II, Reconnaissance and Planning Checklist, for details and a list of publications recommended by the Office of Geotechnical Engineering.

b. Historical Geotechnical Data Search

In many cases, a landslide may have been explored in the past, one or more times. In some cases, a historical geotechnical exploration may have resulted in a remediation which subsequently failed. Such data can be helpful for planning a geotechnical exploration and for assessing feasible remediation alternatives. Historical boring logs add to the amount of available subsurface data, and mean that fewer new borings may potentially be required. Past recommended, designed, or constructed remediation schemes may also yield data which will be helpful in constructing a subsurface profile or in determining the reason for certain surface features.

Past project plans, either for the original construction of the roadway, a subsequent reconstruction, or for the construction of a remediation scheme, can also yield useful data. Cross sections will show both the historic “existing” ground profile, and the historic proposed ground profile, which can be compared to the existing ground line. This can show where soil or bedrock cuts or embankment fills were performed, and can help with determining which soils are fill as opposed to natural, virgin soils. Features from historical remediation schemes, such as rock buttresses, shear keys, benched excavations, and rock channel protection (RCP) can yield helpful data for the construction of a subsurface profile.

c. Site Visit / Drilling Reconnaissance

The geotechnical engineer should visit the site at least once, in order to gain a full understanding of the existing lay of the ground, evaluate the extent and severity of the landslide, and note surface features relevant to the geotechnical analyses. This site visit should also include a drilling reconnaissance, so that the engineer may decide

where drilling and installation of instrumentation should be performed, where there are obstacles to drilling access, and where drilling will be impossible. The engineer should also use this site visit to estimate the limits of the required site survey, and make notes on particular features which the survey should pick up.

The geotechnical engineer should make a sketch of the site, and should note all of the surface features which give evidence of the size, type, severity, and causes of the landslide. Figure 1 shows an example of a simple sketch of a typical landslide site, located along a river. We acknowledge that the toe of slope and limits of the slide are typically not known under water; however they are shown in this example for completeness. This example site will be used repeatedly throughout this bulletin.

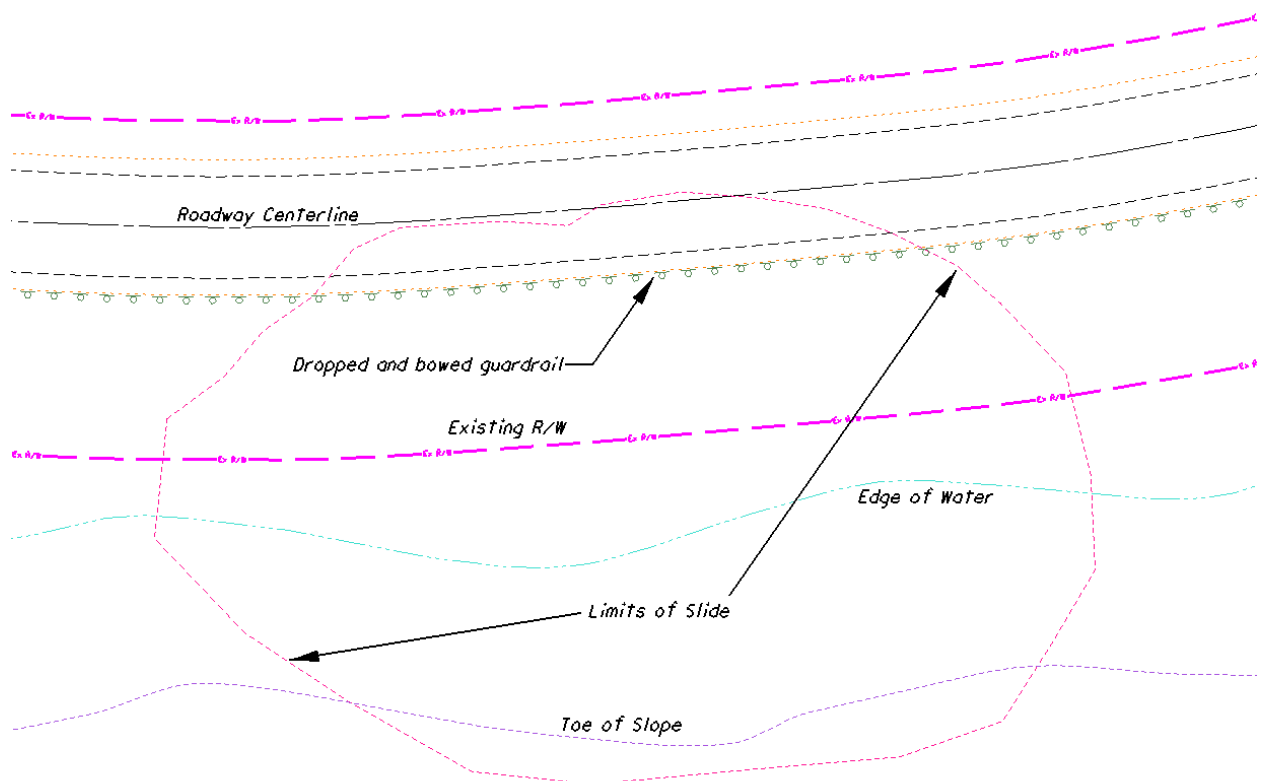


Figure 1: Example Landslide and Site sketch

The following is a list of typical landslide features which are often present and should be noted:

Landslide "Anatomical" Features:

- Head scarp
- Toe bulge
- Tension cracking
- Hummocky ground
- Bowed, leaning, or overturned trees
- Leaning or overturned utility poles
- Ditches, streams, and rivers, particularly if pinched by ground movement
- Erosion features

Surface seepage or wet, soft ground
Unusually verdant or wetland vegetation

Roadway features:

Longitudinal or transverse roadway cracking
Dropped or uneven sections of pavement
Deformed guardrail
Closed off or pinched ditches
Areas of pavement patching or “drag patching”
Apparent limits of roadway cut or fill
Rock cut slopes
Existing and likely future impact of the slide on the roadway and traffic

Evidence of past remediation:

Toe buttress
Driven piles, pipes, or posts
Retaining wall
Rock buttress or RCP
Drainage features

2. Site Survey

The geotechnical engineer should establish the limits of the site survey during or immediately after the site visit. The sketch of the site created by the engineer is often useful to identify the area and features which the survey should locate, and it may be helpful to provide the surveyor with a copy of this sketch.

The following are minimum guidelines on the limits of the area the site survey should capture:

- 100 feet beyond the limits of the failure area, in all directions
- 100 feet beyond the toe of the slope
- Both right-of-way limits

It may be necessary to extend the survey beyond these minimums in order to capture additional relevant features, or to set the site in a more general context. Also, if the slide is on a river bank or at the edge of some other body of water, it is preferable to obtain soundings of the bottom of the water feature, out beyond the “toe of the slope,” where the bottom levels off.

Once the site survey is completed, a plan and profile should be developed, showing land usage and all pertinent topographical and geotechnical features, including right of way lines. Cross sections at a 25-foot interval should be developed along the roadway centerline (or at right angles to the movement of the slide, if the slide is not nearly perpendicular to the roadway). These cross sections will be used to develop a subsurface profile of the soil and bedrock, and to develop models for the stability analysis and Liang method analysis to determine shaft loadings.

3. Subsurface Exploration Program

The geotechnical engineer must develop a subsurface exploration program which yields sufficient subsurface information to develop realistic subsurface profiles and models, but which is conservative and within reasonable limits for the available time and money. The

historical geotechnical data search may find historical site borings which will yield useful data for planning additional borings or reduce the number of necessary new borings.

a. Primary Borings

The most basic site exploration consists of drilling a single primary line of borings, in a cross section through the landslide. Enough borings should be performed such that the structure and slope of the soil strata and the bedrock surface may be determined. Figure 2 shows a plan view of the example landslide site again, with a proposed cross section of borings plotted through the slide. The following paragraphs will discuss the placement of the borings in this example.

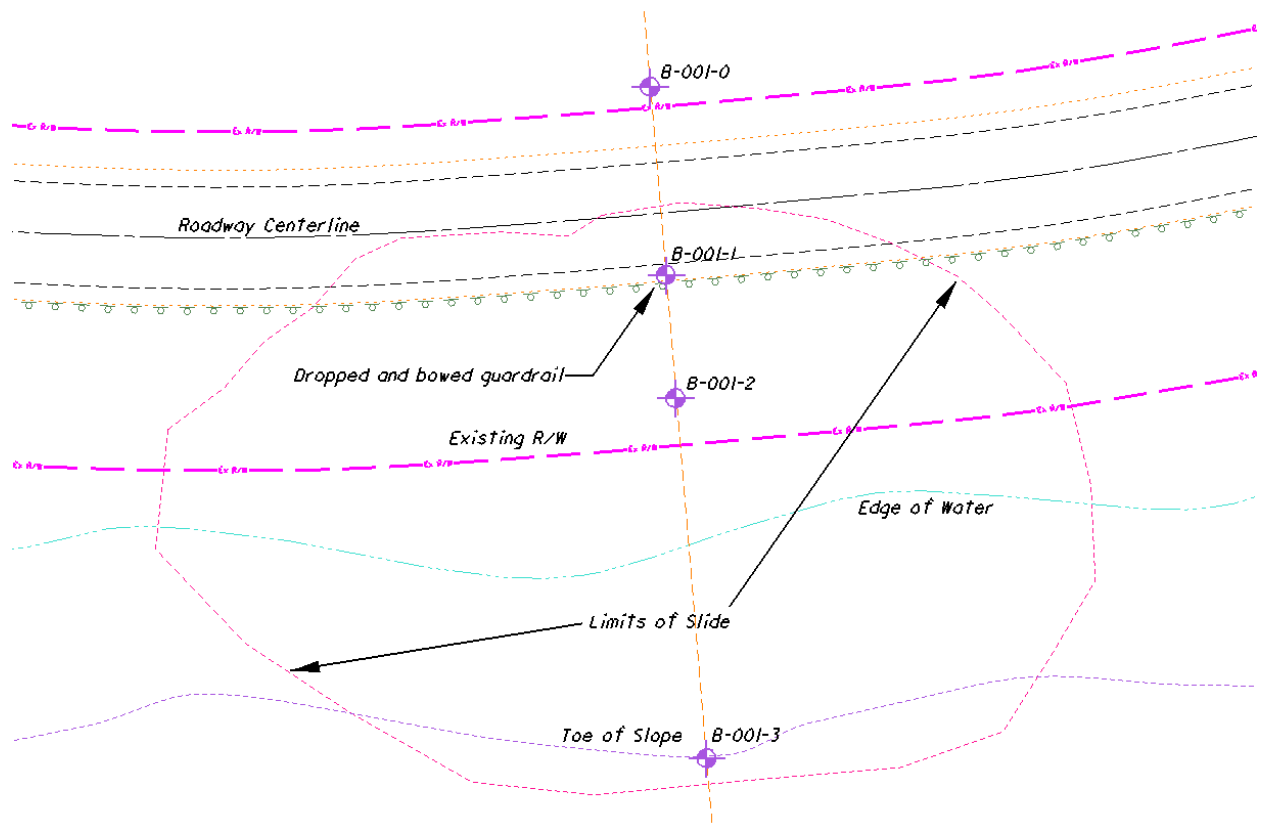


Figure 2: Example Primary Exploratory Boring Cross Section

Boring B-001-1 has been placed at the approximate crest of the slope, on the outside shoulder of the road, near the head of the failure. This is an important location for a boring, and is sometimes the only location a boring can be obtained, due to access difficulties. It should always be attempted to put a boring in this location, and to get it within the area of the slide, if at all possible. The further downhill this boring can be obtained, the better, although this could require removing guardrail.

Boring B-001-0 has been placed across the road and uphill from boring B-001-1. Often, the only locations at which borings will be possible are at the approximate locations of borings B-001-0 and B-001-1. The combination of borings B-001-0 and B-001-1 will allow a determination of the slope of the bedrock under the roadway and upper slide area. If no borings are possible further downhill, we may have to use the boring data

from these two borings and the existing ground surface to project and estimate a subsurface profile to the toe of the slide area.

Boring B-001-2 has been placed on the mid-slope. If there is a bench or nearly level area in the middle of the slope which is accessible to a drill rig, a boring should be attempted in this location. This will extend the knowledge of the subsurface information further down the hill, and allow a better projection or estimation of the subsurface profile. If it is possible to cut a path to the mid-slope with a bulldozer, with minimal disturbance to the slope, this may be attempted also. However, the engineer and driller should use caution with this method. It is possible to introduce further instability into the upper slide area, and quicken or worsen the failure, by cutting too deep of a bench into the side of the existing slope.

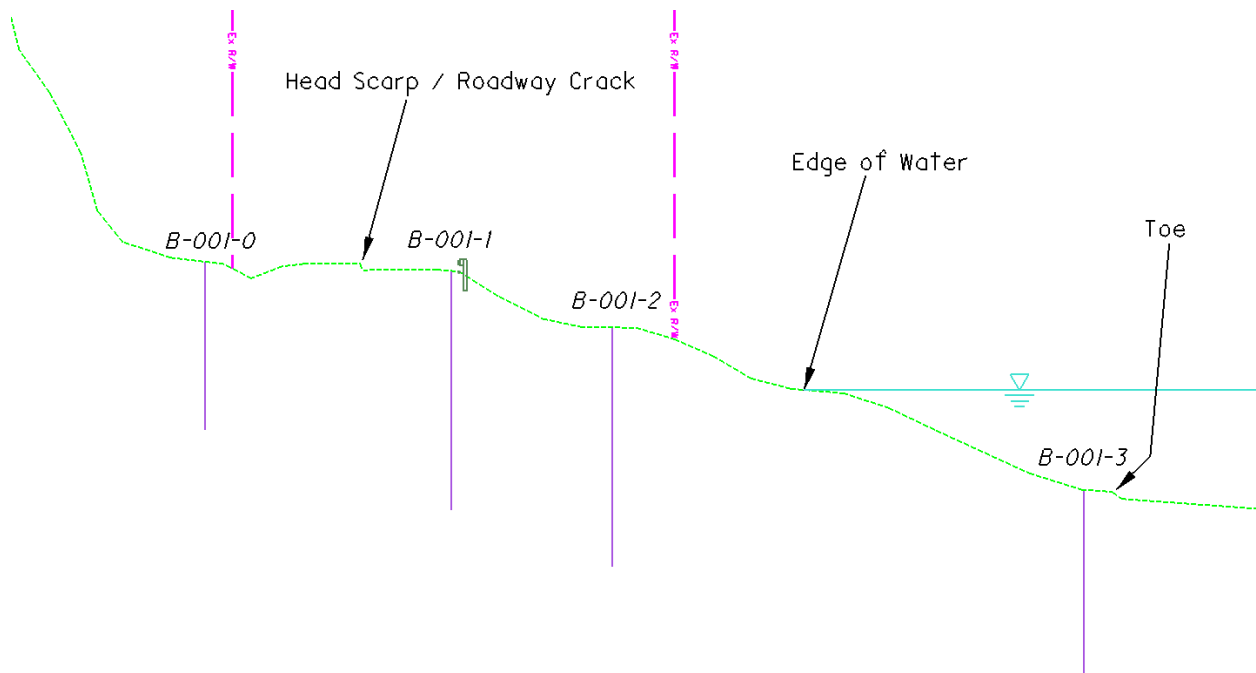


Figure 3: Example Exploratory Borings in Profile View

Boring B-001-3 has been placed at the toe of the slope. In this example, the boring is out in the river, drilled into the river bottom. If at all possible, a boring at the toe of the slope or toe of the slide area is very desirable to complete the subsurface profile. Often, however, this is not possible. Even when the slide is not on the bank of a river, the toe of the slope may be inaccessible to drilling equipment. If the slide extends into a body of water, a boring off of a floating platform, or barge, may be attempted at the toe. If a barge is not available, it may be possible to drill a boring at the edge of the water. Regardless, the more subsurface data which can be obtained along the cross section with the primary borings, the better. The proposed borings for this example are shown in profile view in Figure 3.

b. Secondary Borings

It may also be desirable to obtain additional borings up- or down-station (transverse to) to the primary boring cross section. These are often helpful to define the limits of the

slide area, to further refine the subsurface data, and to define the slope of the bedrock surface transverse to the direction of the slide. A better understanding of the top of bedrock across the site is especially helpful when planning the construction of drilled shafts.

If the landslide is very wide transverse to the direction of movement, or is composed of a number of smaller slides along a length of roadway, it is also prudent to drill more borings along additional “primary” boring cross sections. Each of these cross sections may be individually analyzed, to locate the most critical cross section or to further refine the remediation design. For additional guidance on boring location, see SGE Section 303.5.5, Boring Type C5, Landslide Borings.

c. Soil and Bedrock Sampling

When drilling and sampling for subsurface exploration of a landslide, continuous soil sampling should be performed, per SGE Section 303.5, Boring Type C, Geohazard Borings. Undisturbed (Shelby tube) sampling may be performed whenever soft or very soft cohesive soils are encountered, and should be performed at the depth of the shear failure surface, if this is known. If auger refusal or SPT refusal in bedrock is encountered, a core of the bedrock should be taken.

d. In-situ Testing / Instrumentation

Most soil samples will be obtained by split spoon with the Standard Penetration Test (SPT) method, which will give a rough estimate of the soil consistency or density. A pocket penetrometer reading should also be performed on cohesive soil samples, in order to obtain a second data point to determine the consistency and unconfined compressive strength. Exploration with Cone Penetrometer Testing (CPT) may also be performed in tandem with the drilling and soil sampling, in order to obtain a better estimate of the soil strength, and often to read the pore water pressure as well. In bedrock, a pressuremeter or dilatometer may be used to obtain an in-situ evaluation of the bedrock strength and stress-strain behavior.

Inclinometer casings are very often installed in landslide exploratory borings in order to determine the depth of the shear failure surface, and to obtain a better estimate of the rate and severity of the shear failure. The most advantageous location at which to install inclinometer casing is at the approximate center of the sliding mass. If the location of the head scarp and toe of the slide are known, determining the depth of failure at the center of the slide will give a good approximation of the shape of the entire failure surface. More than one inclinometer installation along the primary boring cross section can further refine the data. In the example shown in Figure 2 and Figure 3, inclinometer installations have been made at boring locations B-001-1 and B-001-2. Inclinometer installations in secondary boring locations or borings outside of the visible slide area are less critical, but may serve to define the size, shape, severity, and limits of the shear failure.

Ground water monitoring wells may also be installed in the landslide exploratory borings, to determine the approximate level of the water table. Elevated ground water is often a major contributing factor in landslide failures, and determining the shape and depth of the “static” ground water surface will aid in modeling the existing conditions.

Inclinometer installations and ground water monitoring wells should not be clustered in the same borings. If a side-by-side installation is desired, an offset boring should be drilled for the second instrumentation installation.

e. Laboratory Testing of Soil and Bedrock Samples

The laboratory testing program for a landslide exploration should generally be more rigorous than the programs for either a roadway subgrade exploration or a structure foundation exploration. Firstly, the number of soil samples is typically greater per boring, and additionally, we desire a greater refinement in the soil classification and shear strength determination. Classification testing of the soil aids in determining soil stratification for the subsurface profile. Moisture content testing aids in determining the ground water level. Undisturbed soil samples can be tested for unit weight and shear strength, which aid in determining engineering properties of the soils for the stability analysis modeling.

All soil and bedrock samples should be subjected to visual classification per SGE Section 602, and grouped by similar classifications into preliminary strata. At least one sample per stratum, per boring, should be subjected to mechanical soil classification per SGE Section 603. All undisturbed soil samples should be subjected to unit weight testing, and samples near the failure surface should be subjected to shear strength analysis by either unconfined compressive strength, unconsolidated-undrained (UU) triaxial compression, consolidated-undrained (CU) triaxial compression, or direct shear testing per SGE Section 604. The use of direct shear testing should generally be restricted to granular soil samples, as it tends to overestimate the friction angle for cohesive soils, and provides a very poor estimate of the cohesion. However, for soil samples from the bottom of a shear failure, where the shear surface is nearly horizontal, the direct shear test provides a fairly good, direct reading of the stress-strain behavior of the soil. Describe all bedrock samples per SGE Section 605. Intact bedrock cores may be subjected to unconfined compressive strength testing per SGE Section 606.

f. Boring Logs

Boring logs should be generated for every boring, per SGE Section 703.3, with visual descriptions of all soil and bedrock strata, and showing all available data from the exploratory borings, in-situ testing, and laboratory testing of the soil and bedrock samples. A Geohazard Profile (Landslide Exploration) should also be generated, per SGE Section 704, with graphic boring logs, and at least one cross section view per primary boring cross section.

B. SOIL AND BEDROCK PROFILES

A subsurface profile of the soil and bedrock should be generated for each primary boring cross section at which modeling and analyses are to be performed. Soil and bedrock information from the subsurface exploration and laboratory testing, as shown in the boring logs, should be combined with the cross sections from the site survey and any historical data to build as realistic a representation of the subsurface conditions as possible.

1. Identification of Soil and Bedrock Layers

Soil samples should be grouped across the subsurface profile into a finite number of discrete strata, so that modeling of the subsurface conditions can be performed for stability analyses. The graphical boring logs should be laid on top of the site survey cross section along the axis of the primary borings, or the cross section views from the Geohazard Profile may be used. The geotechnical engineer should analyze all of the soil samples, comparing factors such as visual and mechanical classification, color, plasticity, gradation, SPT blow count, water content, and undisturbed laboratory test results, in order to group these samples into logical units which will define soil strata. Pay attention to the depositional environment of the site, apparent areas of cut and fill, exposed bedrock faces, and past construction plans, to add to the boring data and further aid in identifying the various strata. Figure 4 shows the subsurface profile for the example problem introduced in Section A.

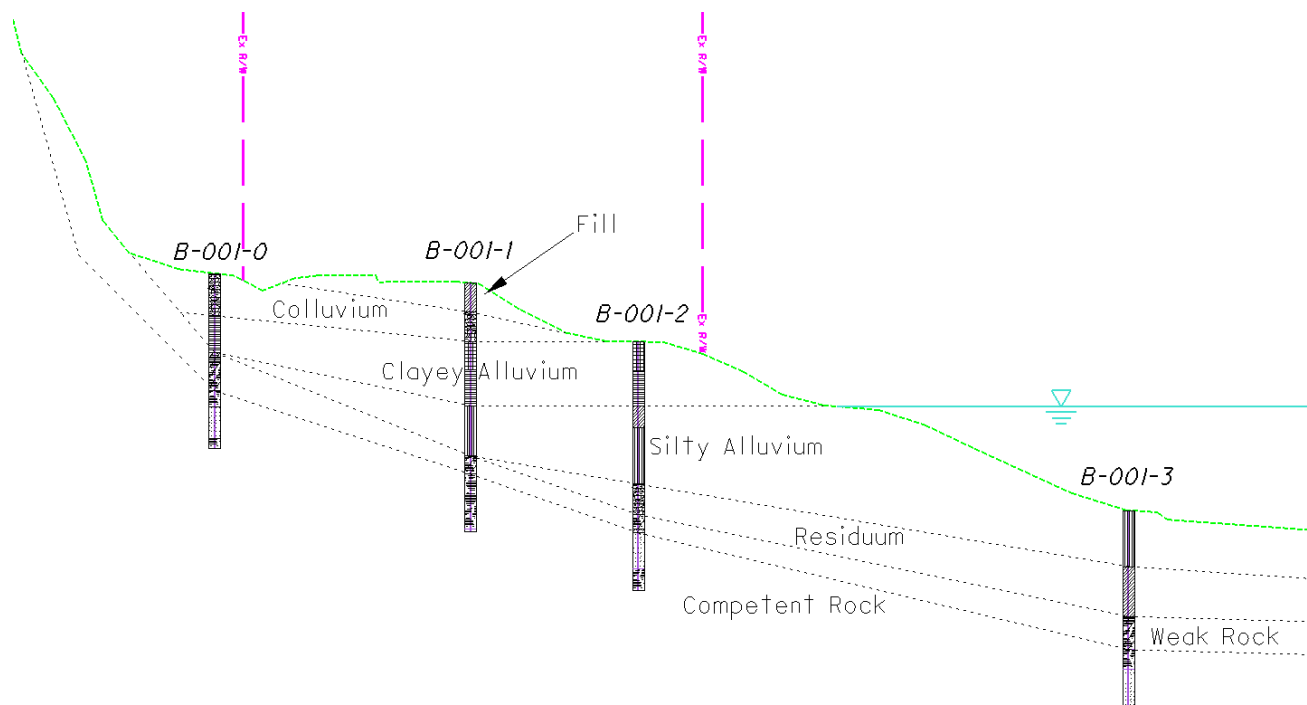


Figure 4: Example Soil and Bedrock Subsurface Profile

The top of bedrock should also be identified in each boring across the subsurface profile. A distinction should be made between weak bedrock, competent bedrock, and strong bedrock.

Generally, the top of weak bedrock will correspond with the depth at which SPT blow count refusal (greater than 50 blows per 6") is reached, but where exploratory borings can still be advanced by soil auger. This rock will typically have a relative strength of very weak to weak, with an unconfined compressive strength in the range of 300 psi to 1500 psi. Weak bedrock is often highly weathered or broken, with a low RQD. Weak bedrock, by this definition, is sometimes also called "Intermediate Geomaterial."

The top of competent bedrock will roughly correspond with the depth at which auger refusal is reached, and at which further bedrock sampling must be done by diamond-tipped core

bit. This rock will typically have a relative strength of slightly strong to moderately strong, with an unconfined compressive strength in the range of 1500 psi to 7500 psi. Competent bedrock is often slightly to moderately weathered.

Strong bedrock may be slow and difficult to core, and is important to note for constructability reasons. This rock will typically have a relative strength of strong to extremely strong, with an unconfined compressive strength in the range of 7500 psi or more. This rock is usually unweathered to slightly weathered.

2. Estimate Soil Engineering Properties

The engineering properties of the soil strata should be estimated in order to model the subsurface profile for stability analyses. These values can be directly interpreted from the results of undisturbed soil testing, or may be estimated with engineering judgment and experience using the results of soil classification testing and SPT blow counts.

Table 1 provides estimates for the unit weights of cohesive and granular (cohesionless) soils based on SPT blow count and depth of the soil sample. The values in Table 1 are based on the engineering experience of the author, and are useful as a first approximation for unit weight to be used in stability analyses, where unit weight testing of the soil has not been performed.

TABLE 1 – Typical Unit Weight Relationships for Various Soils

All unit weights in this table are expressed in pounds per cubic foot (pcf).

Properties for Cohesive Soils		Unconfined Compressive Strength q_u		Dry Unit Weight / Wet Unit Weight at Depth				
		tsf	psf	0-5 ft	5-10 ft	10-20 ft	20-40 ft	>40 ft
Consistency	Blow Counts N							
Very Soft	< 2	< 0.25	< 500	85/105	85/105	90/110	95/110	100/120
Soft	2 - 4	0.25 - 0.5	500 - 1000	90/105	90/110	95/115	100/120	105/125
Medium Stiff	4 - 8	0.5 - 1	1000 - 2000	95/110	95/120	95/120	105/125	115/130
Stiff	8 - 15	1 - 2	2000 - 4000	100/120	105/125	110/125	115/130	120/135
Very Stiff	15 - 30	2 - 4	4000 - 8000	105/125	110/125	115/130	120/135	125/140
Hard	> 30	> 4	> 8000	115/125	120/130	125/135	130/140	135/145

Properties for Granular Soils		Unconfined Compressive Strength q_u^*		Dry Unit Weight / Wet Unit Weight at Depth				
Density	Blow Counts N	tsf	psf	0-5 ft	5-10 ft	10-20 ft	20-40 ft	>40 ft
Very Loose	0 - 4			90/115	95/115	100/120	105/125	105/125
Loose	4 - 10			95/115	100/120	105/125	110/130	110/130
Medium Dense	10 - 30			100/120	105/125	110/130	115/135	115/140
Dense	30 - 50			110/125	115/130	120/135	120/140	120/140
Very Dense	> 50			115/130	120/135	125/140	125/140	130/150

* Granular (cohesionless) soils cannot, by definition, exhibit a meaningful value for unconfined compressive strength.

The angle of internal friction (ϕ) and cohesion (c) of the soils should be estimated by the geotechnical engineer as appropriate for a long-term (drained) stability analysis. Similarly to Table 1 for the unit weight, Table 2 provides estimates for the drained internal friction angle (ϕ) and cohesion (c) of cohesive and granular (cohesionless) soils based on SPT

blow count, consistency, and density. The values given in Table 1 and Table 2 are approximations, derived from SPT blow counts. It should be noted that the Standard Penetration Test yields highly variable results, and gives a poor approximation of the strength of cohesive soils, or soils which have a large amount of gravel or larger particles. These values provide a fair first estimate of the soil engineering properties, and should be altered as necessary by the geotechnical engineer, to fit the observed existing conditions, and the results of stability analyses.

TABLE 2 – Typical Strength Values for Various Soils

Properties for Cohesive Soils		"Typical" Long-Term Strength Values	
Consistency	Blow Counts N	Friction Angle (ϕ')	Cohesion (c')
Very Soft	< 2	12-18°	0-25 psf
Soft	2 - 4	18-20°	25-50 psf
Medium Stiff	4 - 8	20-22°	50-100 psf
Stiff	8 - 15	22-24°	100-150 psf
Very Stiff	15 - 30	24-26°	150-200 psf
Hard	> 30	26-28°	200-250 psf

Properties for Granular Soils		"Typical" Long-Term Strength Values	
Density	Blow Counts N	Friction Angle (ϕ')	Cohesion (c') (psf)
Very Loose	0 - 4	26-28°	
Loose	4 - 10	28-30°	
Medium Dense	10 - 30	30-34°	
Dense	30 - 50	34-36°	
Very Dense	> 50	38-40°	

3. Locate Ground Water Surface

The ground water surface should also be determined in the subsurface profile for representation in the stability model. In some instances, complex hydrogeologic conditions may exist, such that there is not one single ground water table with dry or moist soils above and saturated soils below. However, in most cases, a single ground water surface may be approximated. In the subsurface, the ground water surface may be located fairly accurately at single points through long-term observations with ground water monitoring wells. Short-term observations (made during drilling) are often inaccurate, due to low permeability limiting the rate of water level recharge in the open boring hole, caving of soils from the walls of the open boring hole displacing free water, and the use of drilling fluids. However, short term observations may give a clue about the range of depths at which the ground water surface lies, and sometimes, fairly accurate observations of the depth at which water was "first encountered" will be made. Water contents of the soil samples may also provide data to estimate the depth to the ground water surface.

Lastly, the geotechnical engineer should utilize knowledge of hydrogeology and subsurface flow to connect the ground water surface between known points. The ground water surface should intersect with free water at the ground surface, and should slope downwards with a realistic potentiometric surface, generally following the lay of the land. If bedrock is shallow, the ground water surface often coincides with the top of bedrock. Figure 5 shows the ground water surface in the subsurface profile for the example problem. It can be seen

that the ground water surface in this example intersects with the free water surface of the river at the bottom of the slope and with the roadway ditch at the top of the slope. Monitoring wells were installed at boring locations B-001-1 and B-001-2 to provide additional known points for the ground water level. The ground water surface has been interpolated between all of the known points, and estimated to the left of boring B-001-0.

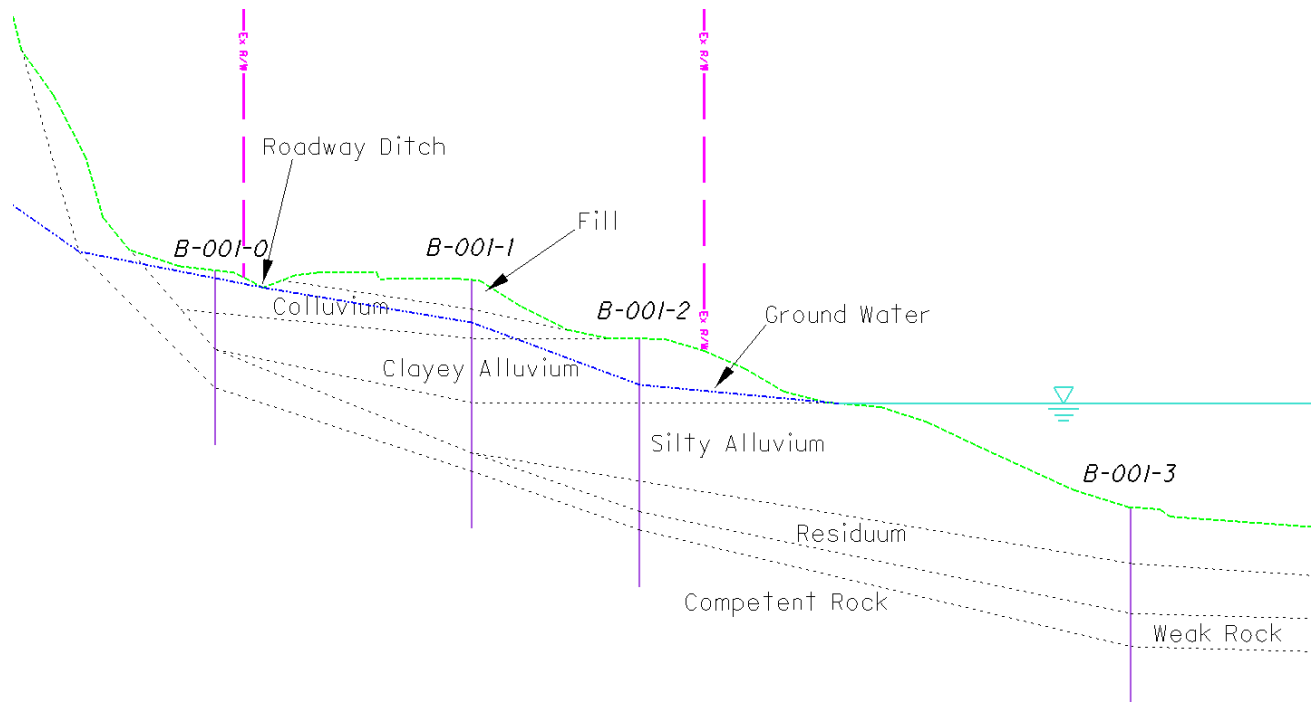


Figure 5: Example Ground Water Surface in Subsurface Profile

If the landslide was triggered or aggravated by a rapid drawdown event, or if a rapid drawdown event is likely to occur at the site in the future, the analyses should also be performed for a ground water surface in the rapid drawdown condition. For example, such an event occurred on the Ohio River, above the Belleville Lock and Dam, in January and February 2005, when runaway barges were caught in the flood gates of the dam. To construct a ground water surface for a rapid drawdown event, the normal pool elevation, the flood elevation, and the rapid drawdown elevation (if different than the normal pool) of the river should be known. River gaging stations along most major waterways provide a useful historical record of the river level during flood events. If flood data cannot be found, a conservative estimate is to assume the flood water reached the crest of the failing slope. The exact subsurface level of the ground water cannot be known, unless ground water monitoring wells were already in place before the flood event, and readings were taken during the flood, but nevertheless, it is possible to approximate a reasonable potentiometric surface, similarly to the normal ground water condition. In this case, however, the ground water surface will remain nearer to the existing ground surface, and will probably meet the ground surface somewhere in the mid slope. The ground water will typically continue at ground surface level down to the free water level below. Figure 6 shows an example of a rapid drawdown condition ground water surface in the subsurface profile.

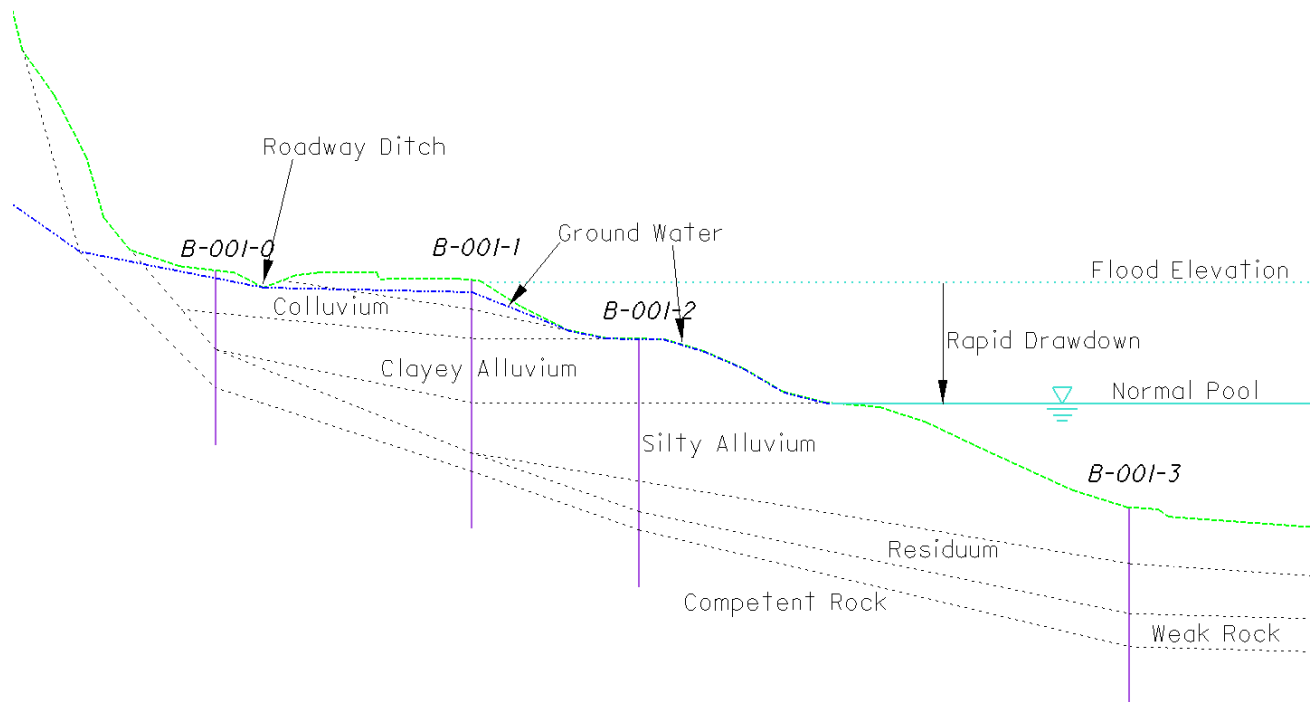


Figure 6: Rapid Drawdown Ground Water Surface in Subsurface Profile

4. Estimate Shear Failure Surface

Before conducting a stability analysis, the shear failure surface should be estimated by the geotechnical engineer, using engineering judgment and experience. The engineer should review the subsurface profile, inclinometer data, and landslide features noted in the site reconnaissance and site survey, and attempt to construct a realistic representation of the shear failure surface which fits with the available evidence. This failure surface is merely an approximation, based on the engineer's interpretation of the mode of failure, but it should provide a useful starting point, and will give a metric with which to compare the results of computerized stability analyses, to determine whether the outputs seem reasonable.

In constructing the estimated shear failure surface, make sure it intercepts the ground surface at the head scarp and toe bulge (if evident), and make sure it conforms to the inclinometer data showing the depth of shear failure. If the bedrock is steeply sloping beneath the hillside and relatively shallow compared to the size of the failure, consideration should be made that the failure surface probably intersects or travels along the top of rock. Highly weathered residual material at the top of the bedrock surface will often make up a thin, weak layer that provides a path of least resistance where a shear surface can develop.

If the toe of the landslide is in a river or other body of water, and is not visible from the ground surface, the engineer must use the other available data to project the failure surface out to its toe. Hopefully, soundings of the bottom of the waterway have been obtained. Depending on the detail and resolution of the soundings, the toe bulge may be apparent on the cross sections. Otherwise, if the toe of the slide is not readily apparent, consider projecting the failure surface out to the natural toe of the slope. Unless there is some bearing capacity failure which makes the underlying material too weak to hold up the

hillside above (unlikely in a natural setting where the slope has existed for a long time) the failure is likely to meet the ground surface at the natural toe of the slope.

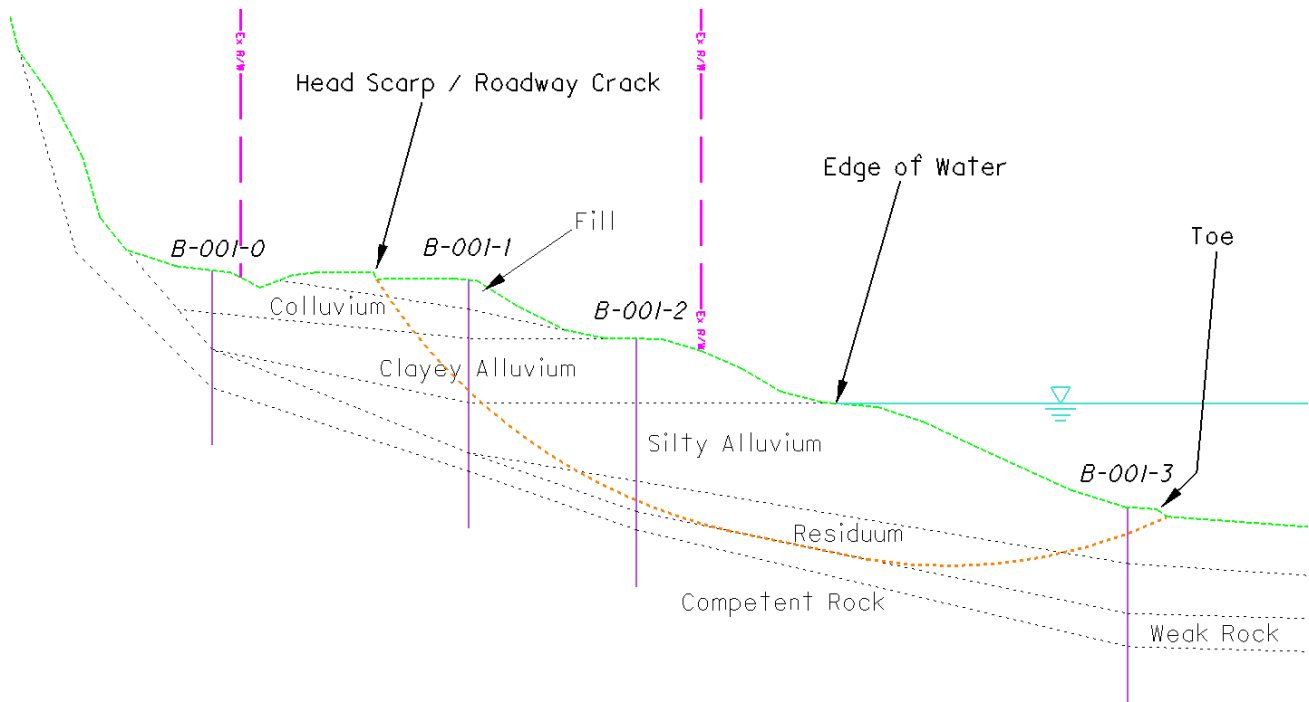


Figure 7: Estimated Shear Failure Surface in Subsurface Profile

Figure 7 shows the estimated shear failure surface for the example problem. The inclinometer at boring B-001-1 showed a shear failure at approximately 20 feet deep, near the bottom of the “clayey alluvium stratum,” and the inclinometer at boring B-001-2 showed a shear failure at approximately 28 feet deep, near the top of rock. The estimated shear failure surface has been connected between these two points and the head scarp visible at the ground surface, and has then been projected along the top of rock. Although a small bulge at the toe is evident, this is at the bottom of the river, and was not visible during the site visit. The bottom of the river was surveyed through soundings, but the resolution of this survey is not high enough to be sure of a toe bulge feature. Therefore, the estimated failure surface has been projected to the toe of the slope, where the river bottom levels off.

C. STABILITY ANALYSIS

A computerized stability analysis of the existing condition should be performed, using a model based on the subsurface profile developed along the primary boring cross section. The stability analysis model should initially use estimated shear strength and unit weight values for each of the subsurface profile strata similar to the recommended values from Table 1 and Table 2. These values will be refined during the analysis, through “back-calculation” of the engineering properties.

If the shear failure surface is estimated to travel along the top of rock, a new “soft rock” layer should be added along the surface of the top of rock, through which a shear failure surface will develop. Through experience with many slides of this type, we have found that there is often a

thin, weak boundary zone along the top of rock, with very low shear strength. This layer is often not identified in borings. OGE recommends representing this zone as a two-foot ± thick layer of very soft cohesive soil, with little to no cohesion, and an angle of internal friction, usually between 12° to 18°.

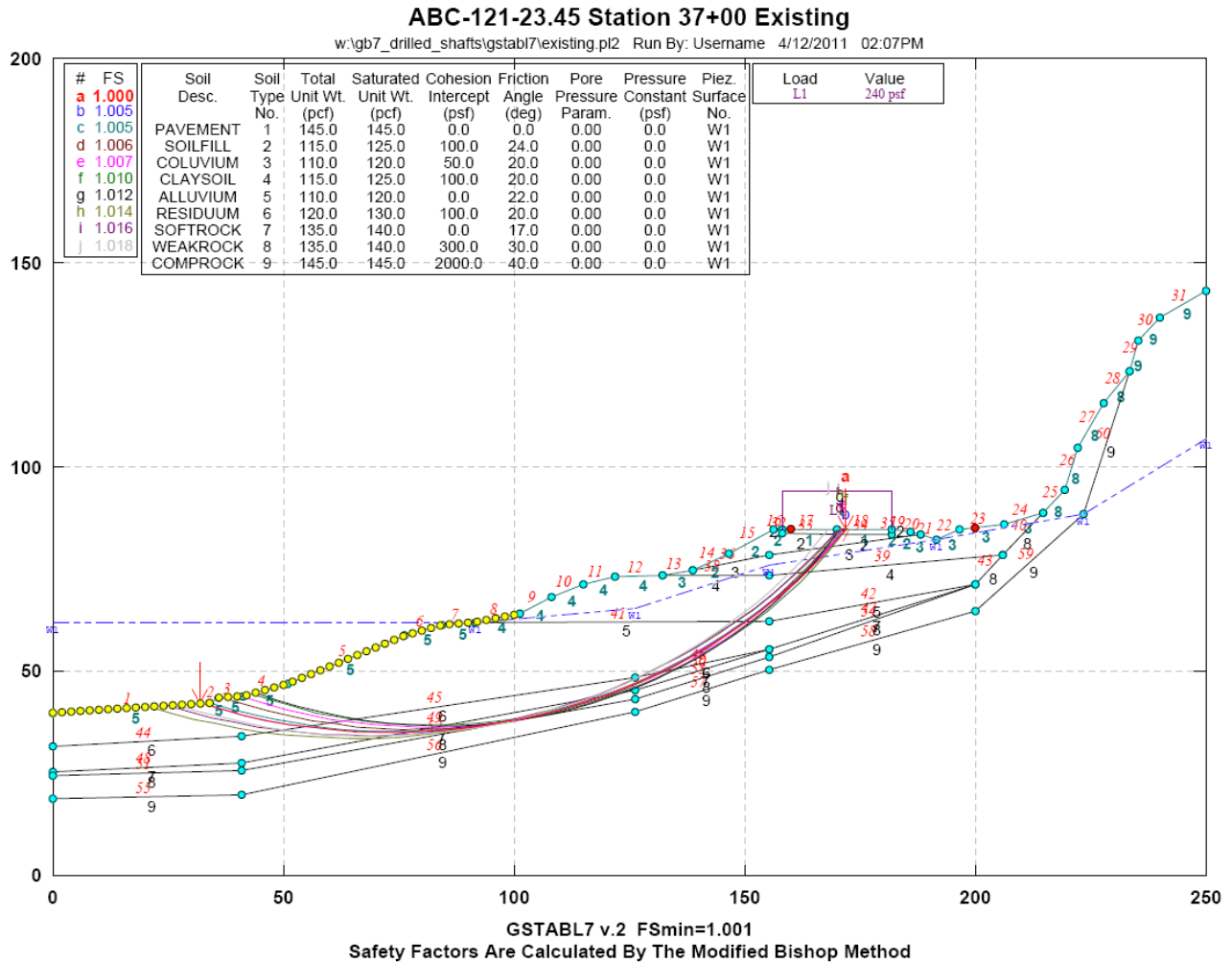


Figure 8: Computerized Stability Analysis

The calculated shear failure surface that is output by the analysis should have a Factor of Safety of 1.0, and should roughly coincide with the estimated shear failure surface or with the known points of shear failure, as given by the head scarp, inclinometer data, and toe bulge. If the initial run of the analysis does not meet these criteria, the engineering properties of the soil should be adjusted until the computerized analysis produces an output which conforms to these conditions. Figure 8 shows the output of the computerized stability analysis for the example problem in the existing condition utilizing GSTABL7 with STEDwin. Note that a thin “SOFTROCK” layer has been added to the top of the bedrock surface.

Do not include an artificially curved “layer” of residual failed soil which mimics the estimated failure surface. Such a layer will force development of the failure surface by the computerized analysis along the new layer. However, it will inaccurately predict the Factor of Safety for other

portions of the slope, and for a reconstructed or retained slope. This method of artificially forcing the failure surface also inaccurately predicts the loading on drilled shafts per the Liang method Analysis. We are interested in the reasons for the initial development of the shear failure, and in preventing similar such failures. Therefore, we wish accurate modeling of the entire slope and all strata for “pre-failure” conditions, so that we can see development of the failure surface along the expected path, and hopefully predict other likely failure surfaces, especially post-remediation. We acknowledge that the soil shear strength along the failure surface is lowered to residual strength values once the failure occurs and substantial movement has taken place. However, we would rather modify the strength of all of the soil strata and the level of the water table until an approximation of the initial failure conditions are achieved so that we can protect against any future such failures.

D. LIANG METHOD ANALYSIS

Dr. Robert Liang of the University of Akron published two papers, one in December, 2002, titled “Drilled Shaft Foundations for Noise Barrier Walls and Slope Stabilization,” and one in November, 2010, titled “Field Instrumentation, Monitoring of Drilled Shafts for Landslide Stabilization and Development of Pertinent Design Method,” as part of the results of a research project conducted for the Ohio Department of Transportation. The goal of this research was to develop a methodology for design of drilled shafts to stabilize unstable slopes. This research built upon the results of two earlier research projects by Dr. Robert Liang and Sanping Zeng, “Numerical Study of Soil Arching Mechanism in Drilled Shafts for Slope Stabilization,” and “Stability Analysis of Drilled Shafts Reinforced Slope.” These research projects used two-dimensional finite element modeling as well as centrifuge testing of physical models to study the effects of soil arching between pairs of equally spaced, laterally loaded drilled shafts in a single row. As a result of these research studies, Dr. Liang developed a mathematical model to predict the percentage of lateral load from the soil which is transferred to the shafts as opposed to the percentage which is passed between the shafts. The percentage of lateral load which passes between the shafts from the uphill soil mass to the downhill soil mass, the “Load Transfer Factor” (η), is a function of the soil cohesion (c), the soil angle of internal friction (ϕ), the drilled shaft diameter (D), the center-to-center spacing between the drilled shafts (S), the location of the drilled shafts on the slope (ξ) and the angle of steepness of the slope from horizontal (β). The two factors relating to shaft geometry, S and D , are typically expressed as a spacing-to-diameter ratio (S/D). We refer to this analysis methodology as the Liang method.

A computer program called UA Slope 2.1 was developed as part of the “Field Instrumentation, Monitoring of Drilled Shafts for Landslide Stabilization and Development of Pertinent Design Method,” research study. This program uses inter-slice forces from the method of slices for slope stability calculation, together with the results of the mathematical model for η , to calculate the force imposed on one shaft in a single row of equally spaced drilled shafts by a moving soil mass above an assumed soil shear failure surface. The failure surface is either estimated or determined by a stability analysis per Section C of this GB. The UA Slope 2.1 program requires as its inputs the geometry of the ground surface and soil strata, the geometry of the shear failure surface, the geometry of the phreatic surface, the physical properties (cohesion, friction angle, and unit weight) of each soil stratum, and the drilled shaft geometry (offset location, diameter, and spacing). A user’s manual for UA Slope 2.1 is attached as an appendix to this GB. The user’s manual fully describes how to input data and run the program, however, some of the major points will be reiterated and expanded upon in this section.

Firstly, it should be noted that the geometric origin for all data input in the UA Slope 2.1 program is at the top left, and the slope must always be represented from uphill on the left to downhill on the right. In other words, all geometric data is entered in X,Y coordinates, where 0,0 is at the top left, X increases from left to right, and Y increases from top to bottom.

Also, the geometric data for the UA Slope 2.1 program must be input in vertical slices. Each soil stratum (including the ground surface) is defined by points along its top, where it intersects each vertical slice. The maximum number of vertical slices is 20, and these include the boundaries at the right and left limits of the represented cross section. Therefore, depending upon the complexity of the geometry, the vertical slices must be chosen with care in order to represent each change in geometry along the top of each stratum. Strata must be input from top to bottom, and must be represented across the entire cross section, whether they extend all the way across or terminate somewhere in the middle. Wherever a layer terminates, the coordinates defining its top surface will be the same as those for the layer below – in other words its thickness will be zero. The maximum number of layers is 20. One extra layer – which does not count in the maximum number of soil layers – is always input, defining the bottom of the lowest layer. This bottom layer must be flat, with the same Y-coordinate all the way across the cross section.

The coordinates defining the ends of the shear failure surface must exactly meet with the ground surface, or the program will output an erroneous Factor of Safety. We recommend that vertical slices should be positioned at the end points of the shear failure surface, so that the ground surface coordinates and the shear surface end coordinates may coincide at these points. The points defining the shear failure surface do not otherwise have to be on the vertical slices, and points in-between vertical slices will usually need to be entered in order to properly define the shape of a curving failure surface. However, we recommend that wherever the failure surface intersects a vertical slice, this should be represented by an input point. The UA Slope 2.1 program allows a maximum of 50 points to define the shear failure surface.

Similarly, the points defining the phreatic surface do not have to be on the vertical slices, and it will often be advantageous to enter points in-between vertical slices to properly define the shape of the phreatic surface. However, the phreatic surface needs to extend across the entire cross section, from the first to the last vertical slices, which are the boundaries at the right and left limits of the represented cross section. The UA Slope 2.1 program allows a maximum of 50 points to define the phreatic surface.

We recommend that a geometric model of the analysis cross section be built in a CADD drawing, with each vertical slice drawn in, and all of the intersections between the slices and strata determined before attempting to input the data into the UA Slope 2.1 program. Building a geometric model will also allow determination of all of the points to define the shear failure surface and phreatic surface. We also recommend entering all the coordinates into a spreadsheet or similar matrix, to consult while entering the data into the UA Slope 2.1 program. This will limit the number of data entry errors.

Figure 9 shows the geometric model constructed for the example problem. This model has twelve vertical slices, including the left and right boundary limits. Note that the “Fill,” “Colluvium,” “Clayey Alluvium,” and “Residuum” layers do not extend all of the way across the

model. Where the “Fill,” “Colluvium,” and “Clayey Alluvium,” layers terminate on the right, their defining coordinates become the same as the ground surface for the rest of the width of the model. Where the “Residuum” layer terminates on the left, its coordinates become the same as the “Soft Rock” layer below. Note also that the geometric model constructed for UA Slope 2.1 is not as wide as the model constructed for stability analysis (as shown in Figure 3 through Figure 8; it is not necessary to make the UA Slope 2.1 model any wider than the limits of the shear failure surface).

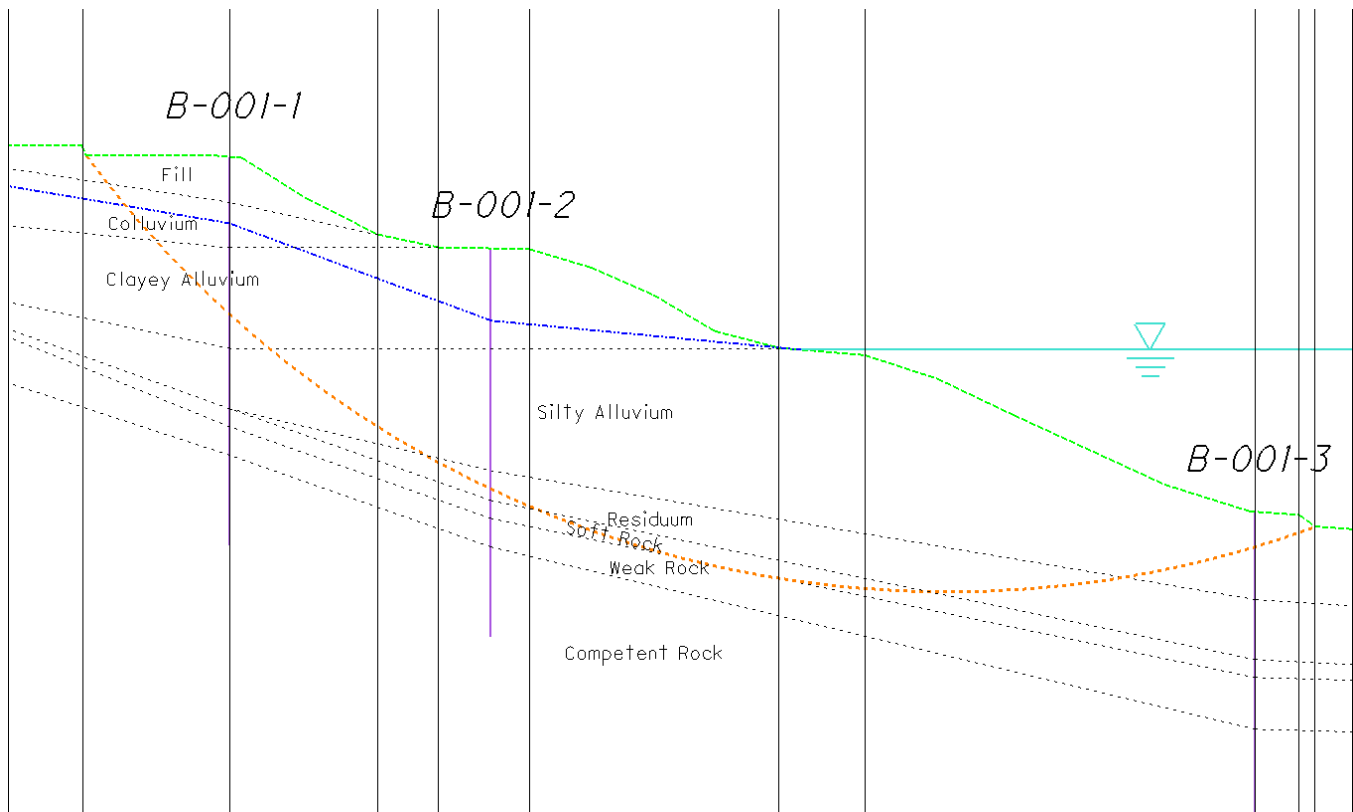


Figure 9: Geometric Model with Vertical Slices

The UA Slope 2.1 program requires the input of three engineering soil properties per stratum, cohesion, friction angle, and total unit weight. The cohesion and friction angle for each stratum should be approximately the same as those utilized in the computerized stability analysis. However, the unit weight must be approximated to a single value for each stratum. We recommend utilizing the moist unit weight for those strata entirely above the phreatic surface, and we recommend utilizing the saturated unit weight for those strata entirely below the phreatic surface. If the phreatic surface passes through a stratum, we recommend selecting a unit weight somewhere in-between the moist and saturated unit weights, based on the relative percentage of the stratum which is above or below the phreatic surface.

After the UA Slope 2.1 program is started for the first time, or if **File, New** is selected from the drop down menus, the Main Menu screen will open, with “UA Slope Program Version 2.1 – Untitled” displayed on the title bar. Figure 10 shows the UA Slope 2.1 program Main Menu. The Main Menu screen is the only screen in the UA Slope 2.1 program; all data is input on this screen, the program is executed from this screen, and program outputs appear on this screen. The program also has four drop-down menus, for file functions, program execution, program

options, and “Help” information about the program. The Options drop-down menu will display two small dialogue boxes for changing file paths (most of which should not be changed or the program will cease to function) and for Chart Options, which allows the user to change the colors used in the “Chart” graphic that appears in the upper right corner of the Main Menu screen. It should be noted that, by default, the program only has colors defined for 7 soil layers: if your model has more layers than this, the additional layers will change colors randomly every time the mouse scrolls over the Chart, which can become visually bothersome. Therefore, we recommend assigning colors to at least as many layers as you will have in your model.

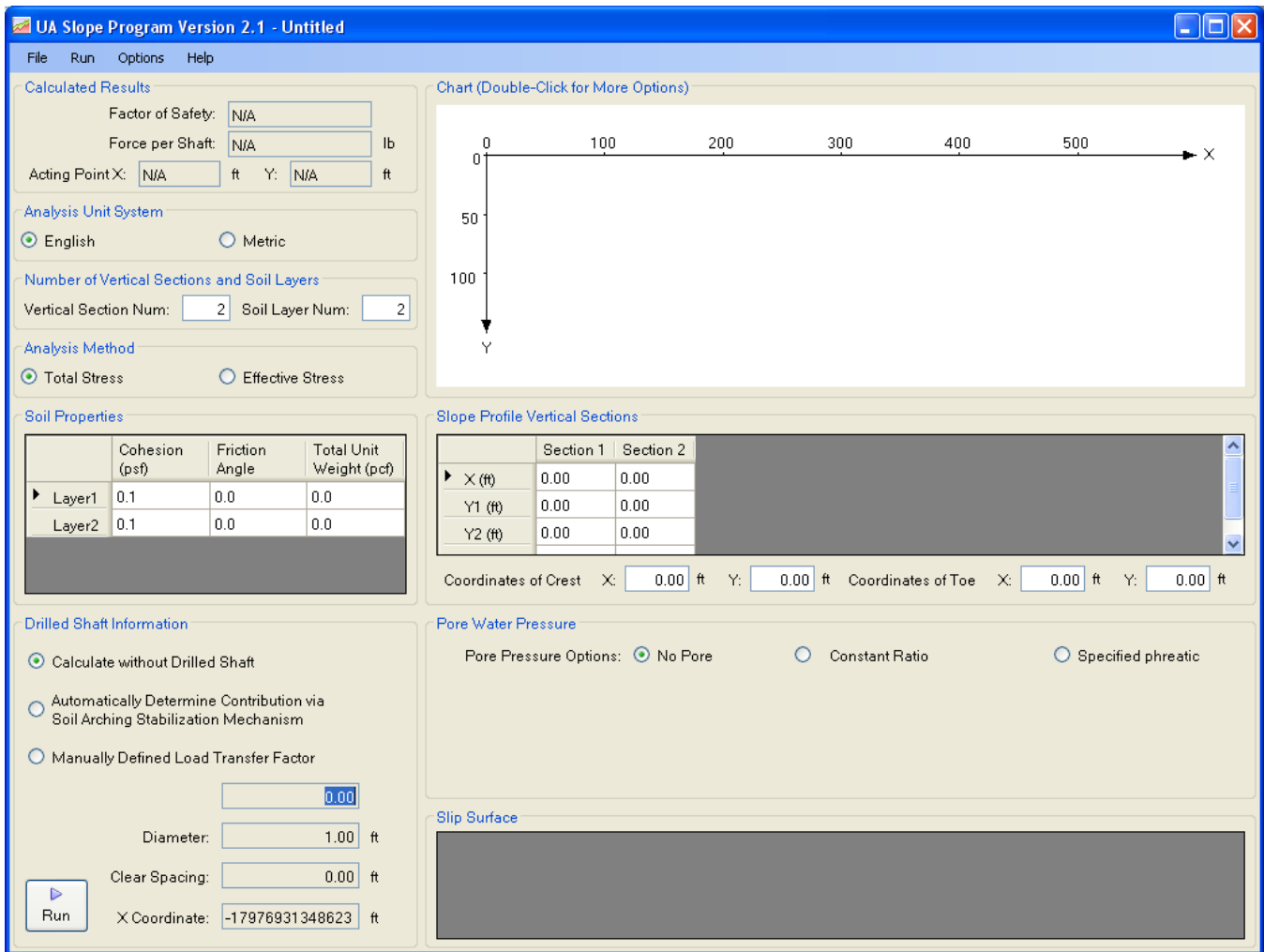


Figure 10: UA Slope 2.1 Program Main Menu

Before beginning to input data, the user must input the units of measurement (English or Metric), number of vertical sections, and number of soil layers. The user must also select the analysis method as either total stress method or effective stress method. We recommend using the effective stress method, as the load transfer factor equation coded in the program was based on effective stress concept, and is not calibrated or verified for total stress analysis. Pore Pressure Options, on the lower right of the Main Menu, allows the option to select “No Pore” pressure, “Constant Ratio” pore pressure, or “Specified Phreatic” surface. No pore pressure does not take into account pore water pressure effects, constant ratio pore pressure applies the same pore pressure effect to every slice in the method of slices analysis, and

specified phreatic surface allows the user to define a ground water table to set the effect of pore water pressure on the analysis. We recommend utilizing the specified phreatic surface, as this is the most realistic alternative.

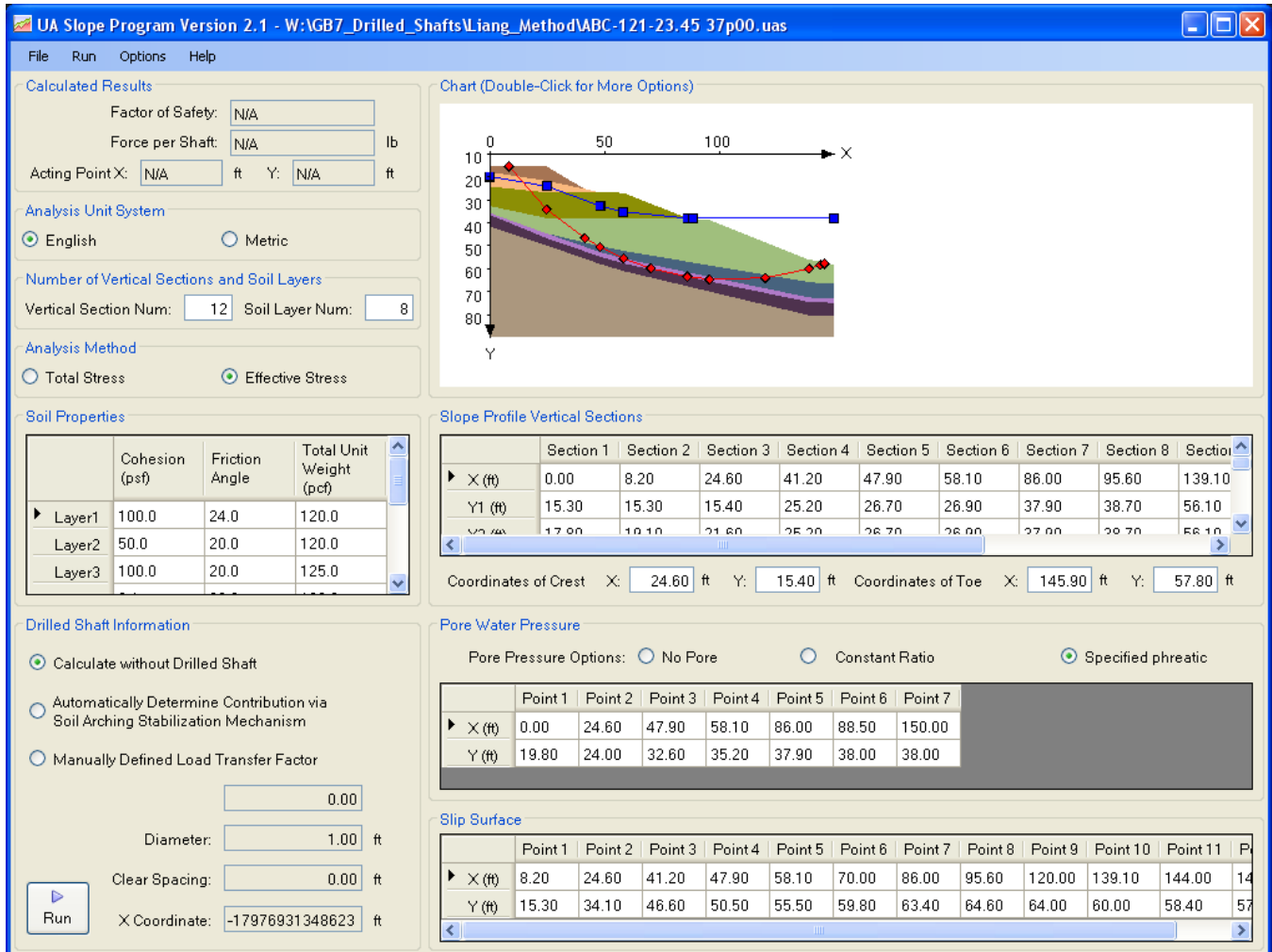


Figure 11: UA Slope 2.1 Program Main Menu with Problem Geometry

The problem geometry for the Slope Profile Vertical Sections, Specified Phreatic surface, and Slip Surface are input into spreadsheet-like grids. The X-coordinates are entered on the top row, and Y-coordinates are entered on each lower row, corresponding to each X-coordinate for each surface. The number of Slope Profile Vertical Sections and Soil Layers are specified as mentioned above on the upper right portion of the Main Menu screen, and their selection will automatically format the Slope Profile Vertical Sections grid for the appropriate number of entries. To select the number of points for the Specified Phreatic surface and Slip Surface, right-click with the mouse pointer in the box containing the grid, and select either Add Point, Set Points, or Delete Point. For the Set Points option, a second small dialogue box opens: the user should type the number of points in the dialogue box and then hit Enter on the keyboard. Soil Properties are entered similarly, in a grid on the left of the Main Menu, in which each row represents a single soil layer or stratum. Please note that the UA Slope 2.1 program will not allow soil cohesion values of zero (0) psf; which are typical for granular soils, and which we understand are also often used for cohesive soils in drained analyses. However, if the cohesion is set to 0, the load transfer factor equation coded in the program will always

evaluate η as “0,” invalidating the results of the analysis. The minimum cohesion value allowed in the UA Slope 2.1 program is 0.1 psf, and this should be used instead, whenever the user would normally select a cohesion of 0.

Please also note that although the data-entry grids in the UA Slope 2.1 program may look similar to spreadsheets, the data cannot be cut, copied, or pasted like a spreadsheet. Figure 11 shows the Main Menu screen with the problem geometry and soil properties entered for the example problem.

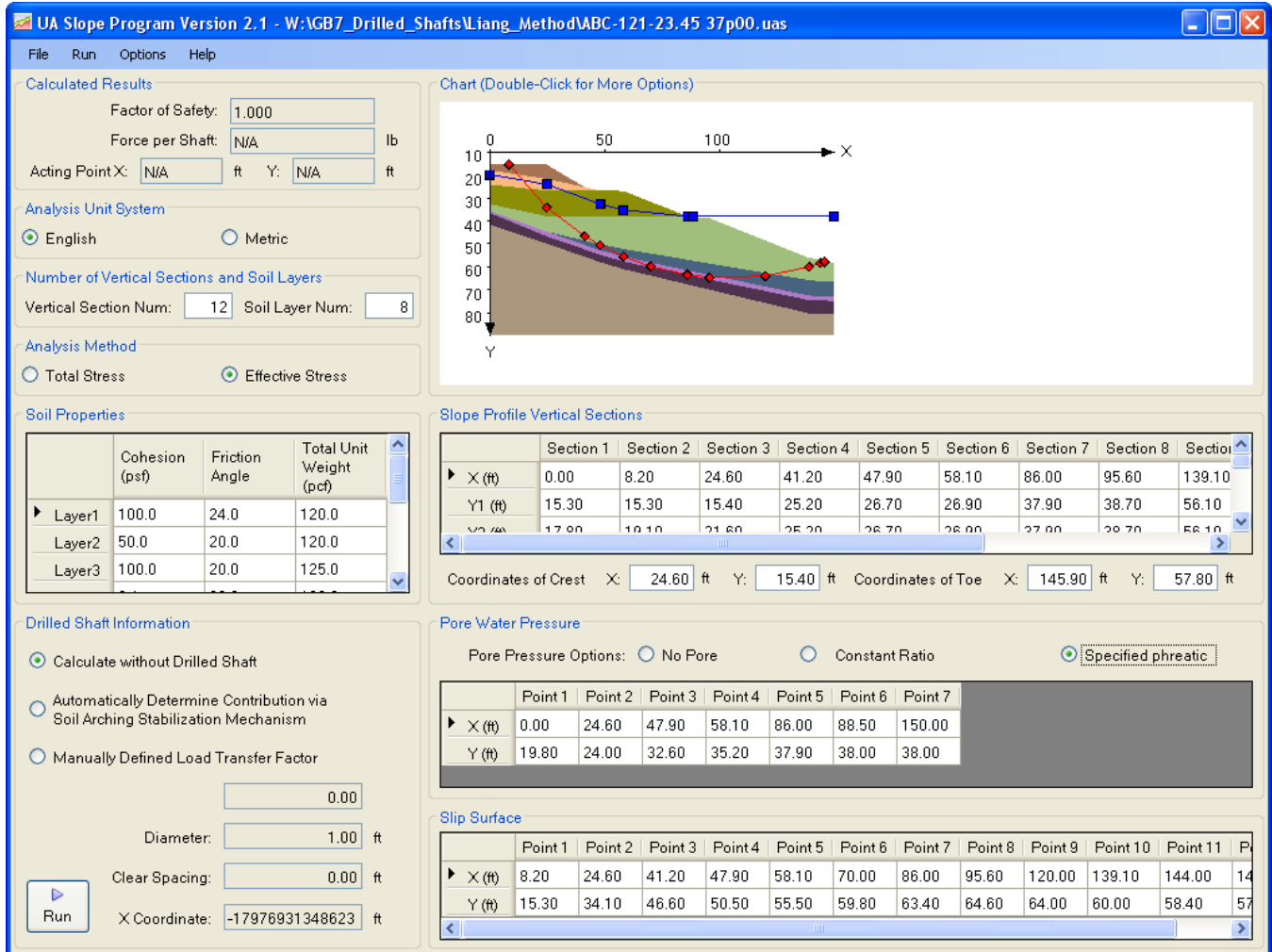


Figure 12: UA Slope 2.1 Program results without Drilled Shaft

Lastly, the user must define the Coordinates of [the] Crest of the slope and the Coordinates of [the] Toe of the slope. These coordinates have two purposes: first, to calculate the angle of steepness of the slope from horizontal (β); and second, to calculate the location of the drilled shafts on the slope (ξ). The program does not allow slope angles steeper than sixty degrees, since the load transfer factor equation coded in the program is not calibrated for steeper slope angles. Furthermore, if the crest and toe coordinates are not defined, ξ cannot be determined, and η will always evaluate as “0,” invalidating the results of the analysis. In the example problem, the Coordinates of Crest are at the outside edge of the roadway embankment, and the Coordinates of Toe are at the toe of slope, where the failure surface exits. Please note that the drilled shafts must be placed between the Coordinates of Crest and the Coordinates of

Toe, so ensure that these coordinates are to the left and right (respectively) of the part of the slope where drilled shafts might be installed.

The analysis should first be run without the drilled shaft effect (Calculate without Drilled Shaft), so that the Factor of Safety for the existing condition may be appraised and correlated with the Factor of Safety from the computerized stability analysis (this should be 1.0). If the initial run of the UA Slope 2.1 program does not have a Factor of Safety of 1.0, the soil properties or phreatic surface should be adjusted slightly until the Factor of Safety equals 1.0. Figure 12 shows the Main Menu screen with the Calculated Results of the analysis without the drilled shaft; the Factor of Safety of 1.000 is displayed at the upper left corner of the Main Menu.

To perform an analysis with the drilled shaft effect, the user must input the Drilled Shaft Information (geometry). Drilled Shaft Information consists of Diameter (D), Clear Spacing, and X Coordinate (location). The X Coordinate is the offset of the centerline of the single row of drilled shafts from the Y-axis (the uphill boundary slice). For this reason, it is helpful to make the left (uphill) boundary at an even-numbered offset from the roadway centerline or baseline. If the failure scarp does not extend beyond the roadway centerline, the roadway centerline can be used as the boundary slice: this allows the X Coordinate of the drilled shafts to be the same as the roadway offset of the drilled shafts. If the failure scarp does extend beyond the roadway centerline, some other reference point must be used, and the difference between the boundary slice location and the roadway centerline must be subtracted from the X Coordinate of the drilled shafts to get the roadway offset of the drilled shafts.

Please note that the drilled shaft Clear Spacing in the UA Slope 2.1 program is not the same as drilled shaft center-to-center spacing (S), as previously introduced. All other documentation for the Liang method uses S, the center-to-center spacing between the drilled shafts, to define drilled shaft spacing; however, the UA Slope 2.1 program utilizes the Clear Spacing *between* two adjacent drilled shafts to define drilled shaft spacing. For example, 3-foot diameter (D=3 ft) drilled shafts, with S/D=3, will have a drilled shaft center-to-center spacing of S=9 ft but will have a drilled shaft Clear Spacing of 6 ft.

Figure 13 shows the Main Menu screen with the Calculated Results of the analysis with the drilled shaft effect “Automatically Determine Contribution via Soil Arching Stabilization Mechanism;” in the Drilled Shaft Information on the lower left of the Main Menu screen, the drilled shaft Diameter is 3.00 ft, the Clear Spacing is 6.00 ft, and the X Coordinate is 50.00 ft. Note that in the Chart, in the upper right corner of the Main Menu screen, a representation of the drilled shaft (to scale with the cross section) is displayed. In the Calculated Results, in the upper left corner of the Main Menu screen, the Factor of Safety of 1.309, the Force per Shaft of 145079.432 lb, and the Acting Point of the resultant, X=50.000 ft and Y=43.349 ft are displayed. The X coordinate of the Acting Point is always the same as the X Coordinate of the drilled shafts, while the Y coordinate of the Acting Point is two-thirds of the distance from the ground surface to the defined Slip Surface (the program represents the horizontal earth pressure load as a triangularly distributed load with an intensity of 0 at the ground surface).

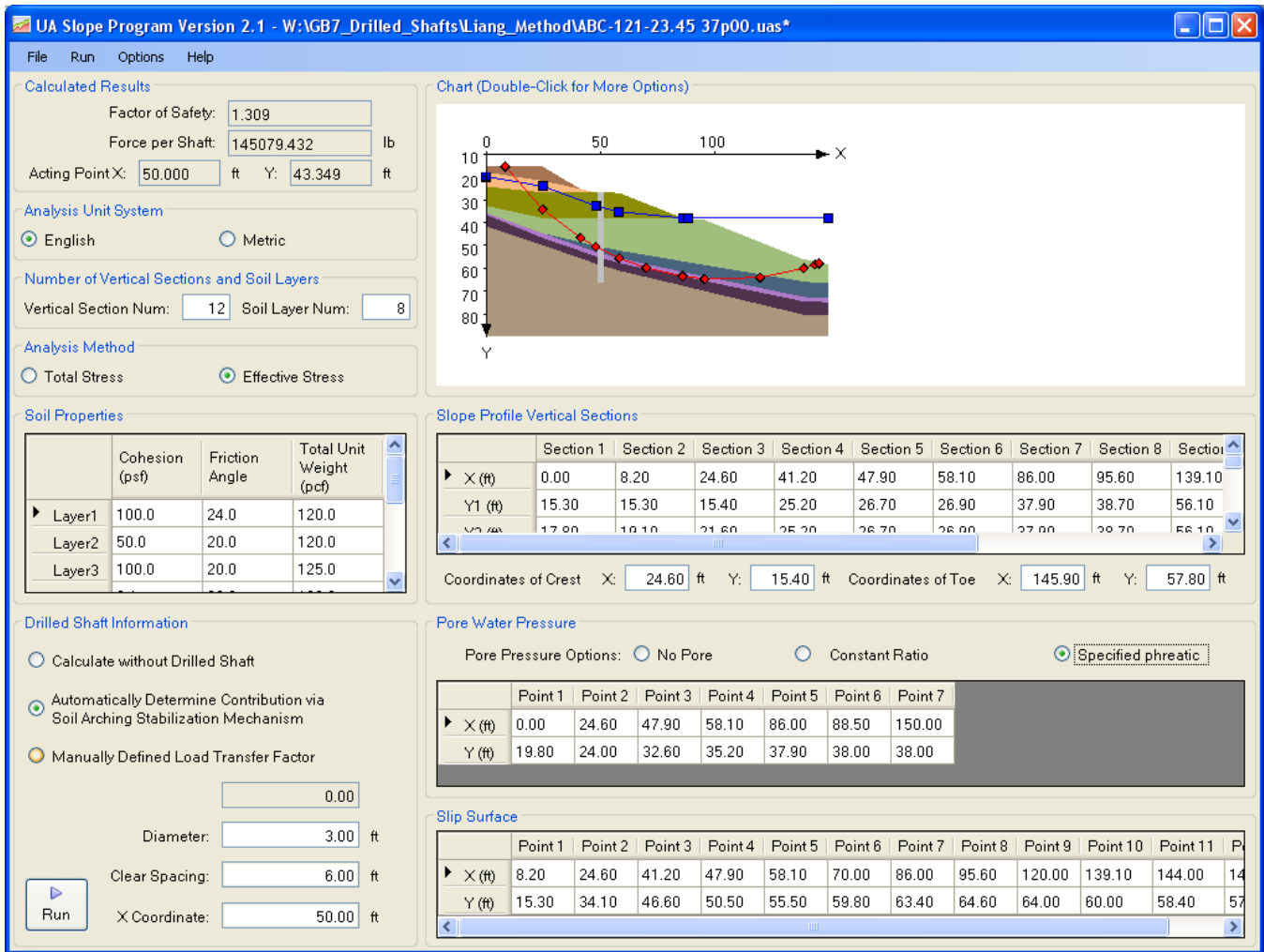


Figure 13: UA Slope 2.1 Program results with Drilled Shaft

Drilled shafts should be analyzed for a variety of geometric configurations of diameter (D) and spacing-to-diameter ratio (S/D). For each combination of D and S/D, the drilled shafts should be analyzed at varying offset locations (X Coordinate). Tables and graphs should be made for each geometric configuration of the drilled shafts, showing the relationship between drilled shaft location (X Coordinate) and Factor of Safety, and between drilled shaft location (X Coordinate) and Force per Shaft. If drilled shaft X Coordinate is not the same as offset from roadway centerline (if the Y-Axis in the UA Slope 2.1 program is not at roadway centerline), then the drilled shaft location should be adjusted to roadway offset in the graphs. Figure 14 shows an example of the Factor of Safety versus Offset graph for the example problem.

Note that in the example graph, the X-axis represents offset from roadway centerline, which in this case, is 10 feet less than drilled shaft location (X Coordinate) used in the UA Slope 2.1 program. Also, the diameter of the shafts is provided in inches, and shaft geometry is expressed as a spacing-to-diameter ratio (S/D). It can be seen that the Factor of Safety generally increases from 1.0 to a maximum somewhere near the middle of the failure, and then decreases back to 1.0 at the toe of the failure. Note also that the Factor of Safety decreases as the spacing between shafts increases, and as shaft size increases – as a larger shaft size at the same S/D ratio has a larger space between shafts.

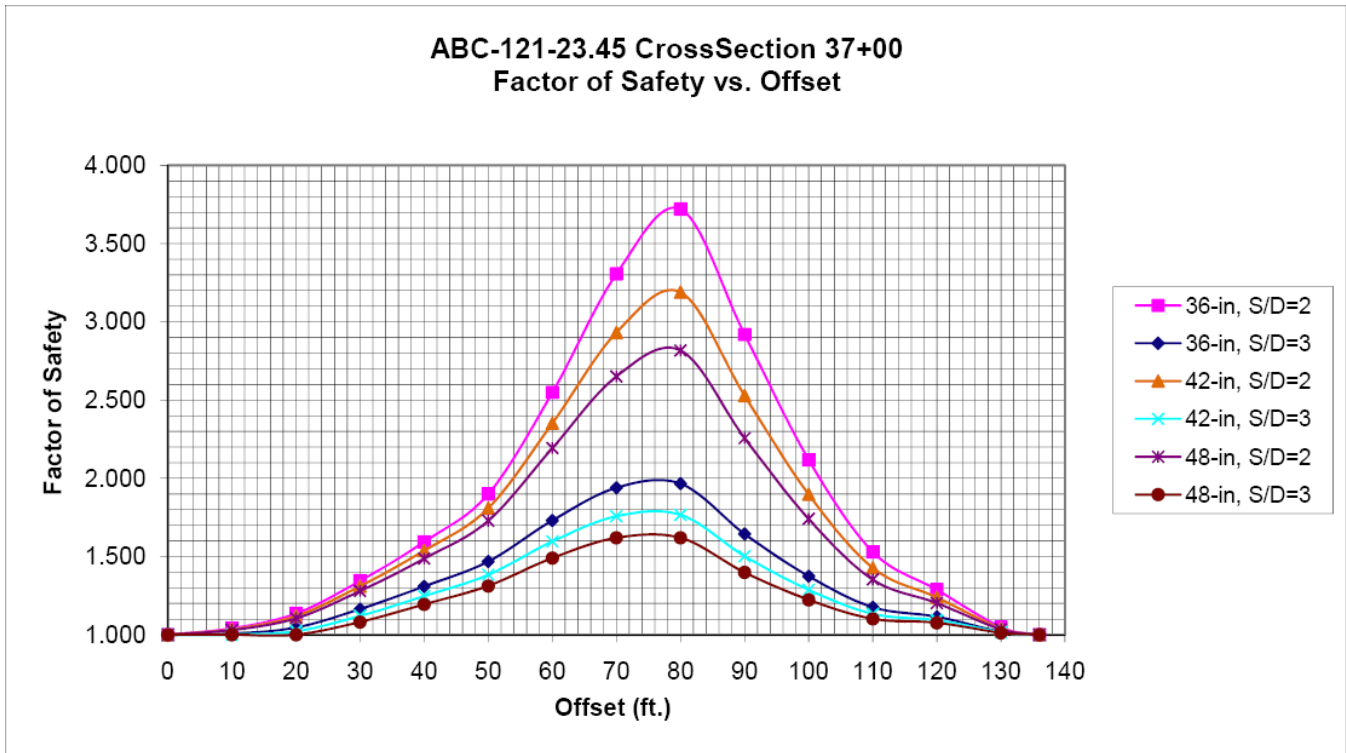


Figure 14: Factor of Safety vs. Shaft Offset from Roadway Centerline

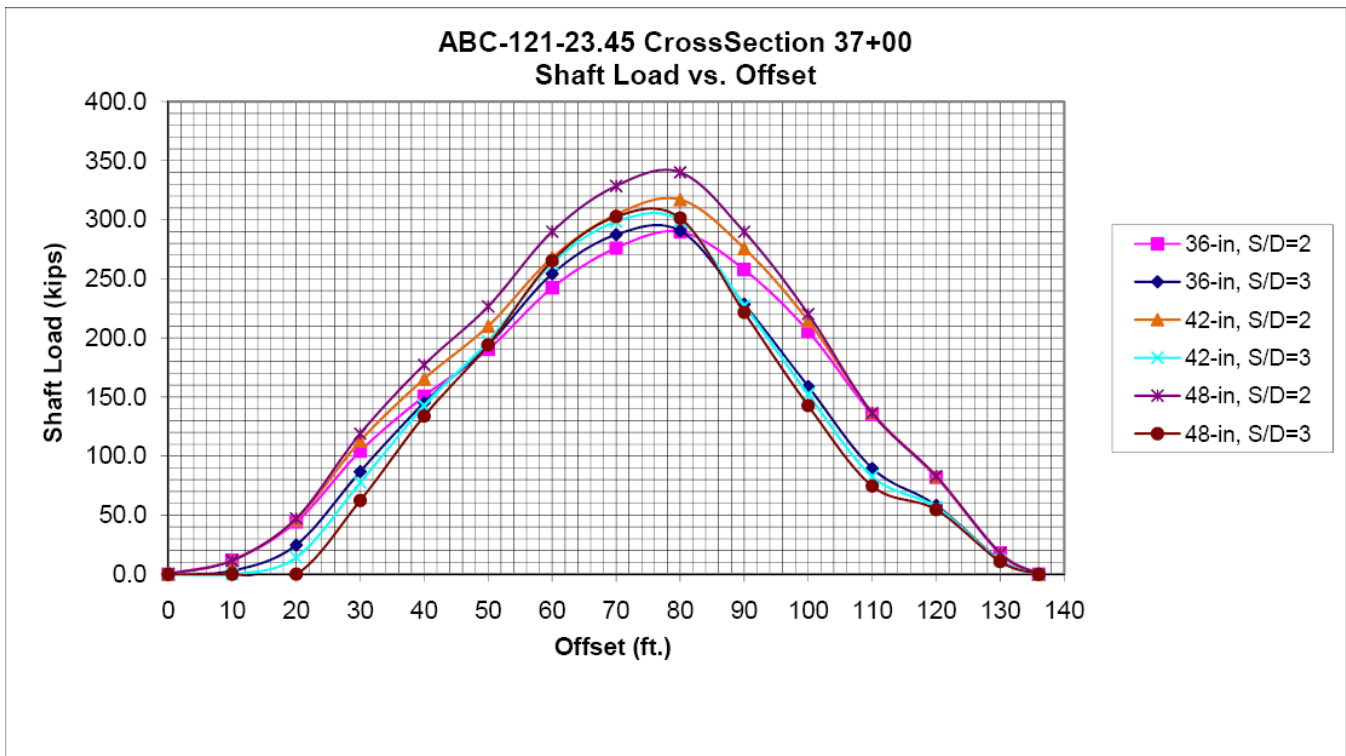


Figure 15: Shaft Load vs. Shaft Offset from Roadway Centerline

Figure 15 shows the associated Shaft Load versus Offset graph for the example problem. It can be seen from this graph that the shaft load tends to follow a similar trend to the Factor of

Safety, increasing from 0 to a maximum somewhere near the middle of the failure, and then decreasing back to 0 at the toe of the failure; this behavior is typical.

Based on the two sets of plots, a shaft size, geometry, and offset should be chosen which will maximize Factor of Safety, minimize shaft load, and conform to other constraints, such as right-of-way restrictions, and other issues which might affect constructability, such as slope steepness and waterways. The minimum Factor of Safety for slope stability required by ODOT is 1.3, therefore, an offset and geometry are typically chosen to meet this minimum Factor of Safety, and then the drilled shafts are assessed structurally, to determine if they are capable of resisting the predicted shaft load. In the example problem, drilled shafts of 36-inch diameter, with $S/D=3$ (9-foot center-to-center spacing) at an offset of 40 feet from roadway centerline (X Coordinate of 50 ft) had a Factor of Safety of 1.309, and were selected for design (see Figure 13 for the actual results of this analysis).

In some instances, it may be impossible to achieve the minimum Factor of Safety with a row of spaced drilled shafts, or right-of-way or constructability restrictions may mean that a sufficient offset to achieve the minimum Factor of Safety cannot be utilized. In this case, a drilled shaft wall may be necessary. This could take the form of a drilled shaft soldier pile and lagging wall, a "plug-pile" lagging wall, a tangent pile wall, or a secant pile wall.

Soldier pile walls consist of a row of drilled shafts spaced at typically a 4-foot to 8-foot center-to-center spacing. HP-section or W-section steel beam reinforcements (soldier piles) are inserted vertically into the shafts, with the webs of the steel sections placed parallel to the direction of the landslide movement. Structural concrete is poured into the shafts up to the bottom of the proposed depth of lagging. Low-strength grout is often poured on top of the structural concrete to finish filling the holes. The soil and grout is then excavated down to the top of the structural concrete, and then treated timber or precast concrete lagging panels are inserted in-between the steel beams, held in place by the flanges of the beams.

A plug-pile lagging wall consists of a row of structural drilled shafts spaced at a two shaft diameter center-to-center spacing ($S/D=2$). These structural shafts are reinforced with steel cages or steel beam sections, and then filled for their full length with structural concrete. Plug-piles of the same diameter are then drilled in-between the structural shafts, tangent to the structural shafts. The plug-piles are typically shallower (they usually do not penetrate bedrock), and serve the purpose of lagging. The plug piles are filled with structural concrete but are not steel reinforced.

A tangent pile wall is similar to a plug pile wall except that all of the drilled shafts are structural, steel-reinforced shafts. A row of structural shafts are drilled at two shaft diameter center-to-center spacing ($S/D=2$), reinforced, and back-filled with structural concrete. Additional structural shafts are then drilled in-between the existing structural shafts, tangent to the existing shafts. These in-between shafts are drilled to the same depth as the first set of shafts, reinforced the same, and also back-filled with structural concrete.

A secant pile wall is similar to a tangent pile wall, except that the spacing between piles is less than the diameter of the piles. First, a row of "secondary shafts," or unreinforced shafts, are drilled at slightly less than a two shaft diameter center-to-center spacing ($S/D<2$). 60 Inches center-to-center spacing is a typical spacing for 36-inch diameter secondary shafts, leaving 24

inches of space between shafts. The secondary shafts are then filled with concrete, and primary or “king” shafts are drilled between the secondary shafts, overlapping the secondary shafts by several inches. The primary shafts are steel reinforced and filled with structural concrete. The secondary shafts can be filled with structural concrete, but are sometimes filled with lean concrete in order to make the drilling for the primary shafts easier. Secant pile walls are used as retaining walls where a water-tight wall is required, but are not common for landslide stabilization, as they are difficult to construct, and do not necessarily have the same structural capacity as a tangent pile wall.

If the lateral earth loads are so high that a wall cannot be built to stand in a cantilever condition, any of the above wall types can be reinforced with grouted ground anchors (tiebacks) or with deadman anchors.

The UA Slope 2.1 program may also be used to calculate the loading on a drilled shaft retaining wall. For this case, the drilled shaft Diameter (D) can be set to anything equal to or less than the center-to-center spacing between reinforced structural shafts (S), and the Clear Spacing between shafts should be set to S-D. In this case, the Manually Defined Load Transfer Factor setting under Drilled Shaft Information should be used. This setting allows the Load Transfer Factor (η) to be explicitly defined; in the case of a wall, η should be set to zero (0), meaning that the drilled shafts take up the full load from the uphill soil slice, and no load is passed through to the downhill soil slice. The Factor of Safety calculated by the UA Slope 2.1 program should be ignored in the case of analysis for a wall. This Factor of Safety is for the *entire* defined shear failure surface, not for the wall structure. If we use a wall, we often assume that the slope below the wall will continue to fail, and that the Factor of Safety for the slope will continue to be marginal or inadequate. Factor of Safety for the wall is assessed through a structural analysis, and is separate from the Factor of Safety for stability of the slope.

E. LPILE ANALYSIS

Once the load on the shafts is determined, we need to determine the reaction of the shaft to the load, including the pile-head displacement, the shear and moment distributions, and determine whether the shaft is structurally capable of resisting the load. Any capable analysis software, such as COM624P, LPILE, or FBPIER may be used. ODOT OGE currently uses the program LPILE Plus Version 4.0, developed by Ensoft, Inc., therefore, the examples in this section will refer to LPILE.

1. Conversion of Force per Shaft to Distributed Shaft Load

The UA Slope 2.1 program calculates an unfactored horizontal earth pressure (EH) resultant load per shaft, however, for proper structural analysis of pile reaction, we need to convert this to a more realistic distributed load. We note that the actual load distribution is complex, and impossible to determine without direct measurement or back-calculation through measurement of displacements, however, we feel that a triangular load distribution is a close enough approximation of the actual condition to develop a realistic calculation of distributed shear, moment, and displacement in the drilled shaft.

The depth to the shear surface, depth to bedrock, and single shaft resultant force need to be determined at the chosen offset for the placement of the drilled shafts, based on the desired Factor of Safety predicted by the UA Slope 2.1 program. Utilizing these known

factors, we can calculate a triangular distribution of loading, from zero (0) at the ground surface to a maximum at the depth of the shear surface. This load is represented solely as a horizontal distributed load, with no vertical component, as this is a more conservative assumption, providing the maximum lateral loading. Research has shown that the vertical load component is either insignificant, or tends to provide a small amount of compression to the shaft, which marginally increases bending resistance. If using LPILE, the distributed load must be converted into units of pounds per inch (lb/in) of length along the drilled shaft.

We do not advocate the method of representing the load on the shaft as a single resultant point load, and “cutting off” the top of the shaft at the point of application of this resultant load. This method does not realistically predict either the shape or magnitude of shear and moment distributions, and cannot predict the displacement at the pile head.

2. Modification Factors for p-y Curves

If the drilled shafts are at a center-to-center spacing closer than about 3½ diameters, a reduction in the soil resistance p , for the p-y curve behavior of the soil, must be considered. The loss in capacity is due to soil-pile-soil interaction, and an overlap in the region of the soil that provides passive resistance to the deflection of the drilled shafts when placed in a closely-spaced group. This effect does not occur where piles are embedded in a relatively much stiffer material, such as bedrock or concrete, where the stress field effects are very limited, and the material does not deform substantially under the design loadings. Therefore, in LPILE, apply the p-multiplier from the ground surface (or artificially lowered ground surface) to the top of bedrock or to the bottom of the pile, whichever is shallower.

Reese, Isenhower, and Wang, “Analysis and Design of Shallow and Deep Foundations” (2006) publish an equation for the pile group p-reduction factor for a single row of piles placed side by side, $\beta_a = 0.64(S/D)^{0.34}$, for $1 \leq S/D < 3.75$, where $0.5 \leq \beta_a \leq 1.0$. This is an empirical relationship based on testing by a number of researchers in a number of different soil types. We recommend this equation for determination of the p-multiplier.

3. Soil Layers

LPILE will calculate a passive resistance, “Mobilized Soil Reaction,” for the soil mass in reaction to the lateral load imposed on the shaft. The actual passive resistance of the downhill soil mass in a landslide will be reduced due to translation of the soil mass away from the drilled shafts; however it will not usually become zero, as “full depth crack” theories assert. In order to model the loss of passive resistance of the downhill soil mass, we recommend three separate methods, depending on the type of drilled shaft retaining structure installation.

For the case of a row of spaced drilled shafts, in which the downhill soil mass must remain in place, and where soil arching and downhill load transfer are dependant mechanisms for retention of the uphill soil mass, we recommend load transfer reduction, based on the Factor of Safety (FS) of the existing slide plane, calculated by the UA Slope 2.1 program with the inclusion of the drilled shafts. Please note, that for this design case, the UA Slope FS must meet the minimum required Factor of Safety for an unreinforced slope ($FS \geq 1.30$), and the slope must have a steepness of 2H:1V or flatter, per GB2. In this case, the p-multiplier, as determined under “2. Modification Factors for p-y Curves” is reduced above the shear plane. The reduced p-multiplier is equal to $[(1-1/FS) \times \beta_a]$, or $[\beta_a - (\beta_a/FS)]$. For

example, if $S/D = 3.0$ and $FS = 1.301$, then $\beta_a = 0.930$, and the p-multiplier above the shear plane = $[0.930 - (0.930/1.301)] = 0.215$.

For the case of a retaining wall in which the downhill soil mass will be left as-is, and its stability analysis downhill of the proposed wall shows that the downhill soil mass does not meet the minimum required Factor of Safety for an unreinforced slope ($FS \geq 1.30$), the downhill soil mass must be considered unstable. In this case, we may assume that the downhill soil mass will continue to fail away from the wall, while the wall retains the uphill soil mass. In order to model the anticipated loss of passive resistance of the downhill soil mass, we recommend artificially lowering the ground surface in the LPILE analysis, completely discounting the passive resistance of the soil between the existing ground surface and the artificially lower ground surface. To do this, first determine the angle of steepness of the slope - downhill of the drilled shafts - from horizontal (β_{dh}), then determine the depth to the shear surface at the location of the drilled shafts (d_τ). For slopes of steepness from $\beta=0^\circ$ to 45° , lower the ground surface by an amount equal to $d_\tau \tan(\beta_{dh})$. For slopes of steepness $\beta=45^\circ$ or more, discount the entire soil mass from the actual ground surface to the depth of the shear failure surface. Model all soil layers below the artificially lower ground surface normally, per the LPILE user's manual.

For the case of a retaining wall in which the downhill soil mass will be regraded to a stable slope (lower at the base of the wall than behind the wall) set the ground surface in LPILE equal to the proposed regraded ground surface, and model all soil layers below the proposed ground surface as in the proposed condition.

4. Pile Length

The drilled shaft should be embedded in a solid stratum below the shear failure such that deflection at the pile head will be constrained to appropriate serviceability limits (see below under "8. LPILE Output" for details of the required serviceability limits). Ideally, the shaft should be embedded into bedrock to provide resistance to deflection. Regardless, the drilled shaft should extend a minimum of 10 feet below the shear surface. Total pile length should be chosen such that the distributed shear and moment at the bottom of the drilled shaft either "quiet to zero" or reach the second inflection point below the maximum value.

5. Steel Reinforcement

In the past, it has been common to reinforce concrete drilled shafts with steel reinforcing bar (re-bar) cages. However, due to the expense in time and money for labor to construct re-bar cages, and the relative fragility of these cages before they are embedded in the concrete shaft, it has recently been the practice to more commonly utilize steel HP-section or W-section beams as reinforcement. These steel sections are generally cheaper per pound of steel than re-bar cages, often require less weight of steel per length of drilled shaft, require no cost in labor for fabrication, and can be easily and quickly installed with little danger of distorting or otherwise damaging them during or before installation. However, we leave it up to the designer to choose whether to use a concrete shaft reinforced with steel re-bars or with a steel beam section. Alternately, the designer may design for both options, so that the contractor may choose which method to employ. Sometimes, we will want both options to be used simultaneously – for example, some historical projects have used primarily steel beam reinforcement, but used re-bar reinforcement for selective shafts in which research instrumentation was installed.

LPILE is capable of realistically representing many different configurations of steel re-bar reinforcement for a concrete shaft. However, due to the wide variety of steel beam sections, and the inherent variability of structural properties (especially for non-compact or slender steel sections), no attempt was made to program data for steel beams into LPILE. LPILE is able to analyze steel pipe piles, which have more regular structural properties; therefore, in order to analyze steel beam sections, we recommend calculating equivalent steel pipe pile sections by setting the pipe wall thickness the same as the beam flange thickness and calculating for a pipe pile with equivalent elastic section modulus (S_x). This approach has been verified by one of the developers of the LPILE software, William M. Isenhower, PhD., P.E.

Analyze the shaft as a steel pipe pile without concrete, although the steel beam section is actually embedded in a concrete shaft. We acknowledge that this is conservative, as it is generally recognized that concrete encased sections are restrained from both local and lateral buckling, and that the concrete stiffens the web and allows an increased shear resistance due to tension field effects. However, at present there is little research into the shear resistance of concrete encased steel sections. The AISC code addresses concrete encased steel sections and specifies that the shear resistance be based on the steel section alone. In the case of steel beam section reinforced shafts, the concrete exists primarily to transfer load to the steel member, and we are relying on the steel for shear and moment resistance. Although this produces a conservative design, we recommend this approach until more research is available into the behavior of composite sections. In the case of drilled shafts with steel sections used as soldier piles, we do not feel this approach is conservative, as a significant portion of the steel sections is exposed from the concrete to support the lagging.

6. Pile Cross Section Characteristics

When analyzing an equivalent steel pipe pile, select Circular Steel Pipe under the Cross-Sectional Shape in LPILE. Set the Outside Diameter of Pipe and Pipe Wall Thickness under Cross-Sectional Dimensions equal to the calculated dimensions for the equivalent pipe pile. However, in order to develop the proper reaction from the soil in LPILE, set the Diameter of the pile under Pile Properties equal to the nominal borehole diameter for the drilled shaft, rather than the diameter of the equivalent steel pipe pile. In order to obtain as realistic a reaction as possible from the pile, set the Moment of Inertia and cross-sectional Area under Pile Properties equal to the actual values of I_x and A_s for the steel HP or W beam section. Set the Modulus of Elasticity equal to that for a steel section alone (approximately 29,000,000 psi), not for a composite section.

The ground surface should be represented as level, not inclined. Inclination of the ground surface is usually used in LPILE to represent pile batter, and in any event, represents a reduction in the soil resistance near the ground surface, which is not relevant, as we are already discounting soil resistance near the ground surface.

When analyzing a steel re-bar reinforced concrete shaft, select Circular Reinforced Concrete Shaft under the Cross-Sectional Shape in LPILE. Set the Shaft or Pile Diameter under Cross-Sectional Dimensions equal to the nominal borehole diameter for the drilled shaft. Under Arrangement of Reinforcement, select the appropriate options to define the

proposed steel re-bar arrangement. Set the Moment of Inertia and cross-sectional Area under Pile Properties equal to the calculated values for a circle with a diameter equal to the nominal borehole diameter. Set the Modulus of Elasticity equal to that for the structural concrete of the shaft (approximately 3,824,000 psi for 4500 psi compressive strength concrete).

Unless there is a constructability concern which dictates a smaller rock socket diameter, the diameter of the drilled shaft should be the same over its entire length. The structural steel used in a steel reinforcing beam section (or equivalent pipe pile) should have 50 ksi yield strength. The steel used in re-bar reinforcement should be represented with 60 ksi yield strength. The structural concrete for a re-bar reinforced shaft should be represented with a 4500 psi compressive strength (concrete cylinder strength), Class S concrete, per CMS Item 524 Drilled shafts. If using a drilled shaft reinforced with a steel beam section, Class C concrete with a 4000 psi compressive strength may be used, per the Plan Note "DRILLED SHAFTS FOR SLOPE STABILIZATION;" which may be found in Appendix 1, attached to the end of this Geotechnical Bulletin.

If analyzing a steel re-bar reinforced concrete shaft, use LPILE Analysis Type 3, "Computations of Ultimate Bending Moment and Pile Response Using Nonlinear EI," which takes into account the cracking of the concrete section with deflection, and the resulting loss in stiffness. If analyzing a steel beam section reinforced drilled shaft, use LPILE Analysis Type 1, "Computations of Pile Response with User-Specified, Constant EI (Basic Modeling)" or Analysis Type 4, "Computations of Ultimate Bending Moment and Pile Response with User-Specified EI;" both of these analysis types use a constant beam stiffness, which is unaffected by deflection of the beam.

7. Pile-Head Boundary Conditions and Loading

At the ground surface, the pile should be free to move both laterally and rotationally. In LPILE, there are multiple options to define boundary conditions and loading at the pile head. The option "1 Shear [F] & 2 Moment [F-L]" should be selected, with a value of zero (0) input for both the shear and moment loading. This defines a pile which is free at the pile head, with a moment and shear which will decrease to zero at the pile head. All other options define rotational or displacement fixity of the pile head or define a deformation at the pile head.

Horizontal earth pressure (EH) loading on the pile should be represented as a triangular distributed load – as noted previously – with a value of zero at the ground surface (pile head), and a maximum at the depth of the shear surface. If the horizontal distance between the drilled shafts and traffic loading is less than or equal to half the depth to the shear surface at the location of the drilled shafts (d_τ), also apply an (unfactored) vehicular live load surcharge (LS) to the drilled shafts equal to two feet of soil with a unit weight $\gamma_s = 125$ pcf, per AASHTO LRFD Bridge Design Specifications, Section 3.11.6.4.

Run LPILE twice for each loading case; running analyses with unfactored loading for the Service (I) Limit State, to determine pile head deflection; and with factored loading for the Strength (I) Limit State, to determine the structural shear and moment capacity of the pile. For the factored Strength Limit State condition, use a load factor of $\gamma_{LS}=1.75$ for the

vehicular live load surcharge (LS) and a load factor of $\gamma_{EH}=1.50$ for the horizontal earth pressure (EH), per AASHTO LRFD Bridge Design Specifications, Section 3.4.1.

8. LPILE Output

After the computational analysis of the pile behavior is completed, there are several items which should be inspected immediately. LPILE produces plots of moment versus depth and shear force versus depth. For the factored Strength Limit State analysis, if either of these plots does not “quiet to zero” at the bottom, or reach the point of second inflection below the maximum value, the pile length will have to be increased. If it appears that several iterations may be required to determine the optimal pile length through incremental increases, it may be more efficient to analyze a pile which is known to be too long, and then cut down the pile length to the optimal point. Note that we do not recommend an embedment of less than 10 feet below the shear surface, regardless of the results of the shear and moment plots.

LPILE also generates a plot of lateral deflection versus depth and calculates a (maximum) deflection at the pile head. For the unfactored Service Limit State analysis, the maximum deflection at the pile head must be limited to 1% or less of the drilled shaft length above bedrock (if not embedded in bedrock, this is 1% of the total drilled shaft length); however, if the drilled shafts are to be installed within 10 feet of the edge of pavement, the deflection at the pile head must be limited to 2” or less. Use whichever serviceability limit requires the least deflection. If the drilled shaft deflects more than the required serviceability limit, we consider this to represent failure, and a stronger reinforcement or larger diameter drilled shaft will have to be selected and re-analyzed.

LPILE also provides maximum values for shear and moment in the shaft. Use these values, from the factored Strength Limit State analysis, in the structural analysis of the shaft, in order to determine if it is structurally capable of resisting the loading without failing in either bending or shear.

9. Geotechnical Resistance

A check of geotechnical resistance against overturning of the drilled shaft should also be performed per *FHWA GEOTECHNICAL ENGINEERING CIRCULAR NO. 10, PUBLICATION FHWA-NHI-10-016, DRILLED SHAFTS: CONSTRUCTION PROCEDURES AND LRFD DESIGN METHODS* (GEC 10), Chapter 12. A resistance factor of $\phi=0.67$ should be used for this analysis; therefore, multiply the Factored Load for the Strength Limit State analysis by $1/\phi$ (1.5 times). At this increased loading, the drilled shaft may fail structurally, even though it may have adequate resistance against moment and shear in the factored Strength Limit State analysis. However, the check of geotechnical resistance is not a consideration of the structural capacity of the drilled shaft, but of the geotechnical resistance of the soil and bedrock to resist excessive overturning movement of the drilled shaft. Therefore, a stiffer section should be analyzed than would be the case for Strength or Service Limit State analyses.

If using a steel beam section reinforced drilled shaft, maintain the Modulus of Elasticity equal to that for the steel section (approximately 29,000,000 psi). However, multiply the Moment of Inertia of the steel section (I_x) by the ratio between the Strength Limit State Factored Load and the Service Limit State Unfactored Load on the drilled shaft, divided by

the resistance factor above, $\phi=0.67$ (multiply I_x by from approximately 2.2 to 2.6 times). This should keep the bending behavior of the drilled shaft under the increased loading similar to that of the drilled shaft under unfactored loading. Any difference in deflection should be attributable to soil reaction.

If using a conventional re-bar reinforced concrete shaft, follow the procedure for Geotechnical Strength Limit State analysis as described in FHWA GEC 10, Chapter 12, Section 12.3.3.3.1.

If the drilled shaft deflects more than 10% of the nominal drilled shaft diameter ($D/10$) this is considered failure of geotechnical resistance against overturning. In this case, deeper embedment of the drilled shaft, a larger diameter drilled shaft, or stiffer reinforcement of the drilled shaft may be required.

F. STEEL BEAM SECTION DESIGN

After determining the Service Limit State lateral deflection of the shaft and the Strength Limit State moment and shear distributions for the single shaft load by analysis with an appropriate software package, such as LPILE, COM624P, or FBPIER, check that the shaft reinforcement is capable of resisting the calculated factored maximum moment and maximum shear force. This section is provided to give guidance for the design of steel beam W-sections or HP-sections as drilled shaft reinforcement, as this is not typical practice at this time. If designing a conventional re-bar reinforced concrete shaft, utilize the LRFD design procedures for laterally loaded drilled shafts, per FHWA GEC 10, Chapter 12 and Chapter 16.

At this time, ODOT is utilizing Load and Resistance Factor Design (LRFD) methods, per AASHTO LRFD Bridge Design Specifications, for the design of steel beam sections resisting shear and moment due to lateral earth loadings. Per FHWA Policy Memorandum Related to Structures, dated June 28, 2000, Load and Resistance Factor Design (LRFD) Specifications are required for all new culverts, retaining walls, and other standard structures on which States initiate preliminary engineering after October 1, 2010. It is no longer acceptable to use Load Factor Design (LFD) methods or Allowable Stress Design (ASD) methods.

We recommend using steel with a minimum yield stress (F_y) of 50 ksi (ASTM A709[M] grade 50) for beam sections used for landslide stabilization drilled shaft reinforcement. Per ODOT Bridge Design Manual, Section 302.4.1.1.C, ASTM A709[M] grade 36 is not recommended and is being discontinued by the steel mills.

1. Minimum Concrete Cover for Reinforcing Steel

Whether designing a drilled shaft reinforced with a steel reinforcing bar (re-bar) cage or with steel HP-section or W-section beams, ensure that the reinforcement can fit within the drilled shaft with the minimum required concrete cover per BDM 301.5.7 and CMS 509.04.B. The minimum concrete cover between soil and steel reinforcement for a drilled shaft of 4 feet or less in diameter is 3 inches. The minimum concrete cover between soil and steel reinforcement for a drilled shaft greater than 4 feet in diameter is 6 inches.

2. Load and Resistance Factor Design (LRFD)

For LRFD, use a load factor of $\gamma_{LS}=1.75$ for the vehicular live load surcharge (LS) and a load factor of $\gamma_{EH}=1.50$ for the horizontal earth pressure (EH), per AASHTO LRFD Bridge Design Specifications, Section 3.4.1. Use factored loading and resistance for structural capacity (flexure and shear) design of the steel beam section reinforcement. Use a resistance factor $\phi_f=1.00$ for flexural resistance and a resistance factor $\phi_v=1.00$ for shear resistance per AASHTO Section 6.5.4.2. Check the flexure resistance of the steel beam section according to AASHTO Section 6.10.8. Check the shear resistance of the steel beam section according to AASHTO Section 6.10.9. If the steel section is embedded in a concrete shaft, assume that it has continuous lateral bracing and transverse stiffening.

3. Iterative Design Process

Use an assumed steel section for LPILE (or other comparable software) analysis to determine the pile head deflection (unfactored loading) and distributed and maximum moment and shear (factored loading) for the beam. Check that the selected steel section is capable of resisting the calculated maximum moment and maximum shear per LRFD procedures, and check that the pile head deflection is less than the required serviceability limit (see Section E.8 above). If these requirements are not met, select a more capable steel section. If the minimum capable steel section will not fit within the selected nominal drilled shaft diameter with the minimum required concrete cover, a larger diameter drilled shaft will need to be considered. If the deflection, flexure, and shear requirements are greatly exceeded, consider selecting a lighter steel section (and possibly smaller diameter drilled shaft) to save cost.

Every time a new steel section or a new nominal shaft diameter are selected, recalculate the pile reaction with LPILE, and check the deflection and the flexure and shear resistance of the steel section per LRFD specifications.

G. SLOPE PROTECTION AND REGRADING

Once a structural solution has been designed to remediate the slope failure or retain the soil mass, either with a row of spaced drilled shafts or with a wall, we must ensure that critical parts of the remainder of the slope geometry will remain in place, so that the structural solution will not fail. Failure of the structural solution does not necessarily mean failure of the structural elements. Failure also includes any movement of the soil mass, independent of the structural fix, which compromises the integrity of the facility the drilled shafts are designed to protect.

If using a row of spaced drilled shafts, per the Liang method, if the slope below the shafts continues to move, or if a new instability develops below the shafts, this system is jeopardized. This solution depends on soil arching and a percentage of the load being transferred between the shafts, through to the downhill soil mass. This mechanism increases the Factor of Safety (the nominal resistance versus load) of the entire landslide by effectively decreasing the method of slices interslice force. However, it depends on the soil mass downhill of the shafts to support the remaining interslice force. If the soil mass downhill of the shafts is removed, the passive support of this soil is eliminated, and the soil mass uphill of the shafts will fail between the shafts, causing the entire system to fail. In this case, the drilled shafts could remain in place without suffering a structural failure.

However, if the structural solution is designed to depend on the passive resistance of the downhill soil mass below the shear failure surface to provide support for the structural elements, and that soil mass fails away – either through a new shear failure or by erosion – the structural elements of the system may fail. Furthermore, if a new shear failure develops uphill of the structural retention system, this is regarded as failure of the system, even though no failure of the structure occurs.

Therefore, it is important to protect the integrity of the remainder of the slope once a structural solution has been selected and designed. Perform slope stability analyses of the slope both uphill and downhill of the structure. If the soil mass uphill of the structure is found to have inadequate stability, the uphill slope may need to be regraded, may need reconstruction with special benched excavation and embankment backfill (see GB2), may require the addition of subsurface drainage, or the structure may need to be modified (either moved to a new lateral location or increased in height) to properly retain the uphill soil mass. If using unreinforced CMS Item 203 embankment fill material or natural soil, the uphill slope may not be regraded to steeper than 2H:1V. If using a Reinforced Soil Slope (RSS), the uphill slope may be steeper than 2H:1V. However, analysis of RSS stability is beyond the scope of this document.

If the slope downhill of the structure is found to have inadequate stability, other measures may have to be taken, depending on the type of structural solution. If using a row of spaced drilled shafts, the downhill slope must be stabilized by one of the methods recommended above, or the row of shafts must be moved further downhill, in order to increase the stability of the lower slope. If the shafts are moved, the shaft loading will change, and the entire system will need to be redesigned. Regrading or reconstructing the lower slope may not be practical, due to limiting geometry at the toe of the slope. If neither movement of the shafts nor regrading of the lower slope is able to successfully increase the stability, a wall will be necessary.

If a wall is used, failure of the downhill soil mass may not contribute to failure of the system overall, as long as the material into which the drilled shafts are embedded (usually bedrock) does not fail, and as long as the drilled shafts are capable of standing cantilever or are reinforced by tiebacks. If there is a chance that the material into which the drilled shafts are embedded may fail along with the downhill portion of the landslide, the drilled shafts should be designed for this condition, and embedment should be increased.

Regardless of structural stability of the drilled shaft reinforcing system, there may be other reasons, environmental, aesthetic, or otherwise, for which the downhill soil mass may need to be stabilized. In this case, the downhill slope may need to be flattened, and/or the soil may need to be cut to below the existing shear failure surface. If the slope failure is along the bank of a river or other stream, erosion control – typically CMS Item 601 Rock Channel Protection (RCP) – may be necessary at the base of the slope in order to protect against erosion undercutting the toe of the slope and inducing an additional instability. Also, if along a body of water which is prone to flooding, design the system to resist loadings under the “normal” condition, the 100-year recurrence flood condition, and the rapid-drawdown condition. In general, design the structural system for the worst-case conditions which may be anticipated during the life of the structure.

Appendix 1:
Plan Note “DRILLED SHAFTS FOR SLOPE STABILIZATION”

Designer Note: This note is for slope stabilization structures that are buried, i.e. they do not have any lagging above ground. The note assumes that the drilled shafts are designed to rely only on the structural steel member and that the concrete in the drilled shaft is only to fill the void and is not structurally significant.

DRILLED SHAFTS FOR SLOPE STABILIZATION

Item 524, Drilled Shafts, __" Diameter, Above Bedrock, As Per Plan

Item 524, Drilled Shafts, __" Diameter, Into Bedrock, As Per Plan

This work consists of furnishing and installing drilled shafts for slope stabilization structures. The drilled shafts are reinforced with structural steel members instead of reinforcing steel cages. Furnish and install the drilled shafts in accordance with CMS 524 except as modified and supplemented below.

Excavate the hole for the drilled shaft within 6 inches of the plan location. The design is based on a maximum depth from ground surface to bedrock of __ feet. If field conditions indicate greater depths, notify the Engineer for further evaluation.

Furnish structural steel members according to the plan requirements and conforming to ASTM A572, Grade 50. Do not field weld or splice structural steel members. Place the steel member within the hole so it is vertical and not inclined more than ¼ inch per foot. Center the steel member within the hole. Place the steel member so that the flanges are parallel to the centerline of the row of drilled shafts. Do not allow the orientation of the flanges to vary by more than 15 degrees. Support the steel member so that it does not move during concrete placement.

Use Class C concrete according to CMS 511. The Contractor may place concrete using the free fall method provided the depth of water is less than 6 inches and the concrete falls without striking the sides of the hole. Pouring concrete along the web of the structural steel member is acceptable.

Check the position, the vertical alignment and orientation of the structural steel member immediately after concrete placement. Make corrections as necessary to meet the above tolerances.

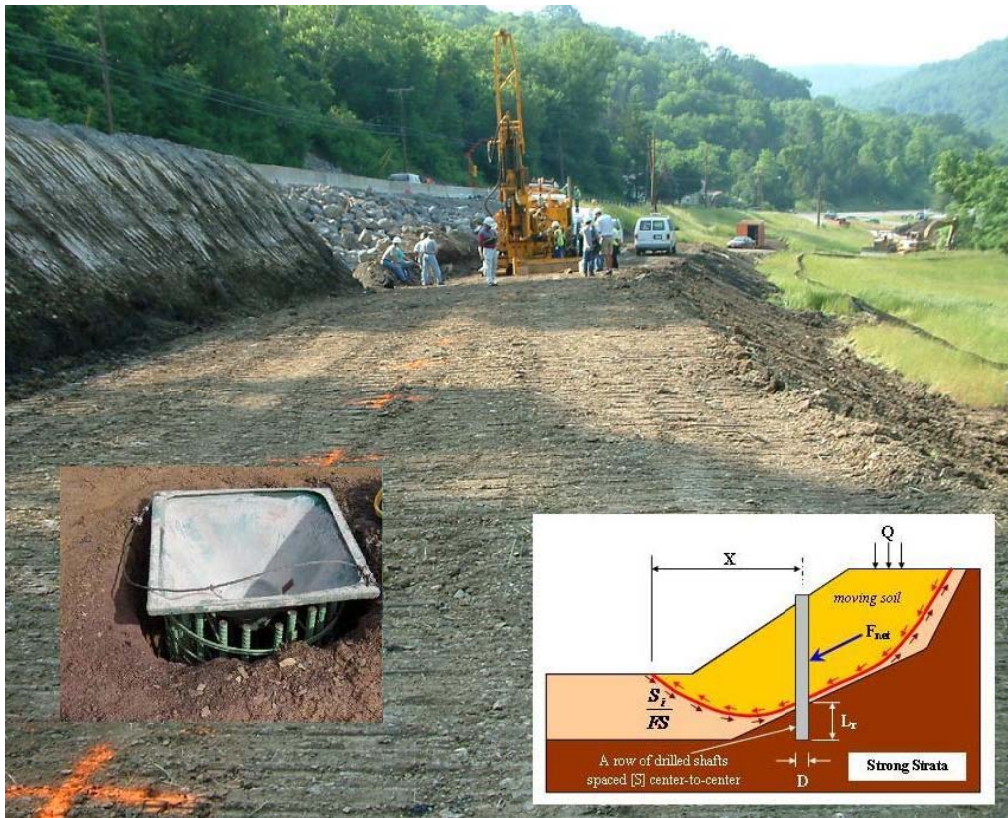
Method of Measurement: The Department will measure Drilled Shafts Above Bedrock, As Per Plan, along the axis of the drilled shaft from the existing ground surface to the top of bedrock, as determined by the Engineer. The Department will measure Drilled Shafts Into Bedrock, As Per Plan, along the axis of the drilled shaft from the top of bedrock to the bottom of the drilled shaft, as determined by the Engineer.

Appendix 2:
UA Slope 2.1 Program User's Manual



A User's Manual
for
UA SLOPE Program
Version 2.1

November 2010



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1. INTRODUCTION

The computer program UA SLOPE 2.1 is a modified version of a previous computer program UA SLOPE developed by Liang (2002) for analysis of a slope with or without a single row of spaced drilled shafts to determine geotechnical factor of safety (FS) and the net force on each drilled shaft. The basis of the theory and the validation of the program were presented in the main text of the report. Even though the interface of the UA SLOPE 2.1 program is user friendly and quite straight forward, this User's Manual was written to provide instructions on how to use the input interface for generating an appropriate input model for computer runs. Computations of the involved theories were fully coded into the computer program; therefore, there is no calculation task involved by the program users. Nevertheless, it is important to note that the users should possess fundamental knowledge of soil mechanics, foundation engineering, and slope stability analysis methods. More importantly, it is highly recommended that the users should consult the main text of this report to get a detailed understanding of the theories and equations used in developing this computer program, paying particular attention to the limitations and range of applicability of the program. The computer analysis results should be interpreted by experienced engineers to ascertain the reasonableness of the computational results. In general, this manual provides background information for the program. An illustrative example is used to help explain preparing input data for subsequent computer runs.

2. BACKGROUND

UA SLOPE 2.1 is a modified version of the previous program UA SLOPE by incorporating new equations to account for the arching effects in a slope/shaft system. A new user interface was developed in UA SLOPE 2.1 to enable quick and easy input of the parameters needed in the analysis. Extensive beta testing of the coded program was carried out by comparing UA SLOPE 2.1 results with other commercially available software, such as GSLOPE and STABLE Windows version, for slope stability computation of slopes without drilled shafts. As to the validation of the applicability of UA SLOPE 2.1 for analyzing FS and net force on the drilled shaft of a slope/shaft system, excellent comparisons between UA SLOPE 2.1 predictions and 3-D finite

element simulations using strength reduction techniques for about 50 cases were documented in the main report. The applicability of UA SLOPE 2.1 for realistic and complex slope geometries and soil profiles was confirmed with a case study of the ATH-124 load testing data combined with calibrated finite element simulation results. Thus, within the range of slope geometry, shaft dimension and spacing, and soil strength parameters, the UA SLOPE 2.1 program can be used with confidence for design of an optimized slope/shaft system that has an adequate geotechnical factor of safety of the system and the least construction cost. Structural design of the drilled shaft can be accomplished once the net force on the shaft is computed by UA SLOPE 2.1. The net force computed by the UA SLOPE 2.1 program is the net force on the portion of the drilled shaft above the failure surface. Therefore, this force needs to be assumed as a triangularly distributed load in the portion of drilled shaft above the failure surface in the LPILE analysis. An appropriate load factor based on the current AASHTO LRFD Bridge Design Specifications should be applied to obtain the factored load combinations for structural design of drilled shafts. However, the un-factored load combinations should be used to compute the shaft deflection for checking the serviceability of the structures under the working load conditions.

3. LIMITATIONS

The theoretical basis of the UA SLOPE 2.1 program in computing geotechnical factor of safety of a drilled shaft/slope system is the recognition of the reduced driving forces in the slope due to the load transfer from the soil to the drilled shaft through arching mechanisms. Therefore, the essential requirement for this theory to work is that arching would indeed develop in the field application and that this arching effect would be permanent. Some of the conditions that may prohibit the development of an arching condition in a slope/shaft system include, but not limited to, the following: (a) a liquefiable sand layer that has tendency to flow around the drilled shaft structures after liquefaction, (b) a very soft soil layer that may squeeze through the spacing between the drilled shafts, and (c) narrowly spaced or widely spaced drilled shafts where arching

could not develop, and (d) the stiffness of the drilled shafts is too small such that they may move along with the moving soil masses.

4 GETTING STARTED

The UA SLOPE 2.1 program contains drop down menus when it is opened in the windows environment. These four menus are labeled as “File”, “Run”, “Option”, and “Help”. The functionalities under the each menu are summarized in Figure 1, which also shows a logical flow of the data input process. As can be seen, the majority of input information is provided through the interface under the “File” menu, which allows for creating an input file through interactive input boxes. The input information needed for defining the problem slope pertains to slope geometry, soil profiles, soil properties, ground water conditions, and location of the slip surface. The input information needed for defining the drilled shafts includes diameter of drilled shaft, clear spacing between the adjacent shafts, length of drilled shaft, and the location of the drilled shaft. At the present time, only the effective analysis method is permitted in UA SLOPE 2.1 program. Once the input file is created, it should be saved before proceeding to the computation part of the analysis, which is done by the “Run” menu. The “Options” menu provides path and file name information as well as a chart option by which the color of soil layers, slip surface, and ground water location can be judiciously customized.

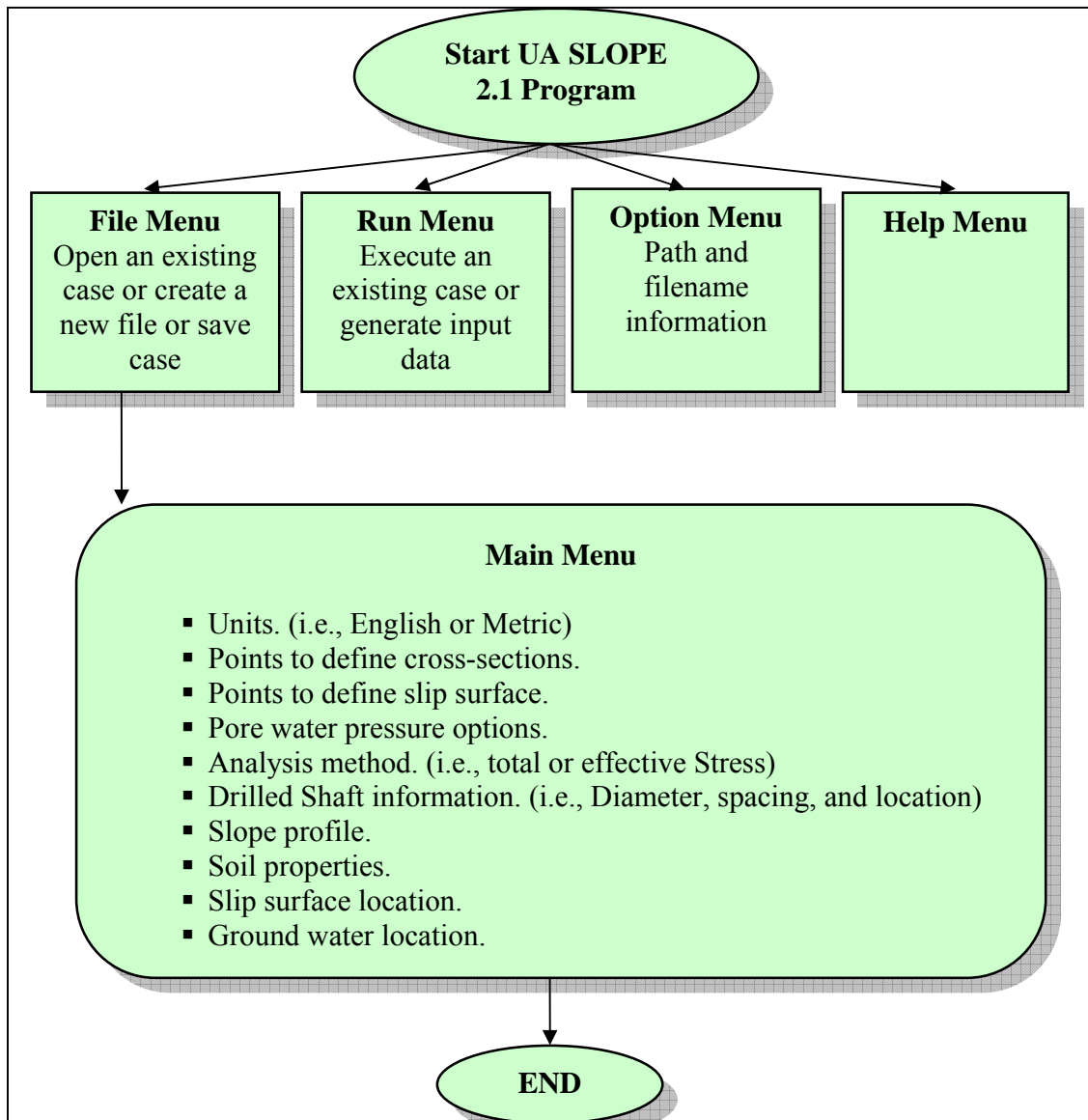


Figure 1: UA Slope Operational Flow Chart

4.1 Starting the Program

The program can be started by double clicking the UA SLOPE icon. The UA SLOPE 2.1 main menu will appear as shown in Figure 2 with the drop down menu under the “File” shown.

4.2 File Menu

The file menu option, like any Windows-based software, allows the user to open an existing input file, to create a new file, or to save one just created.

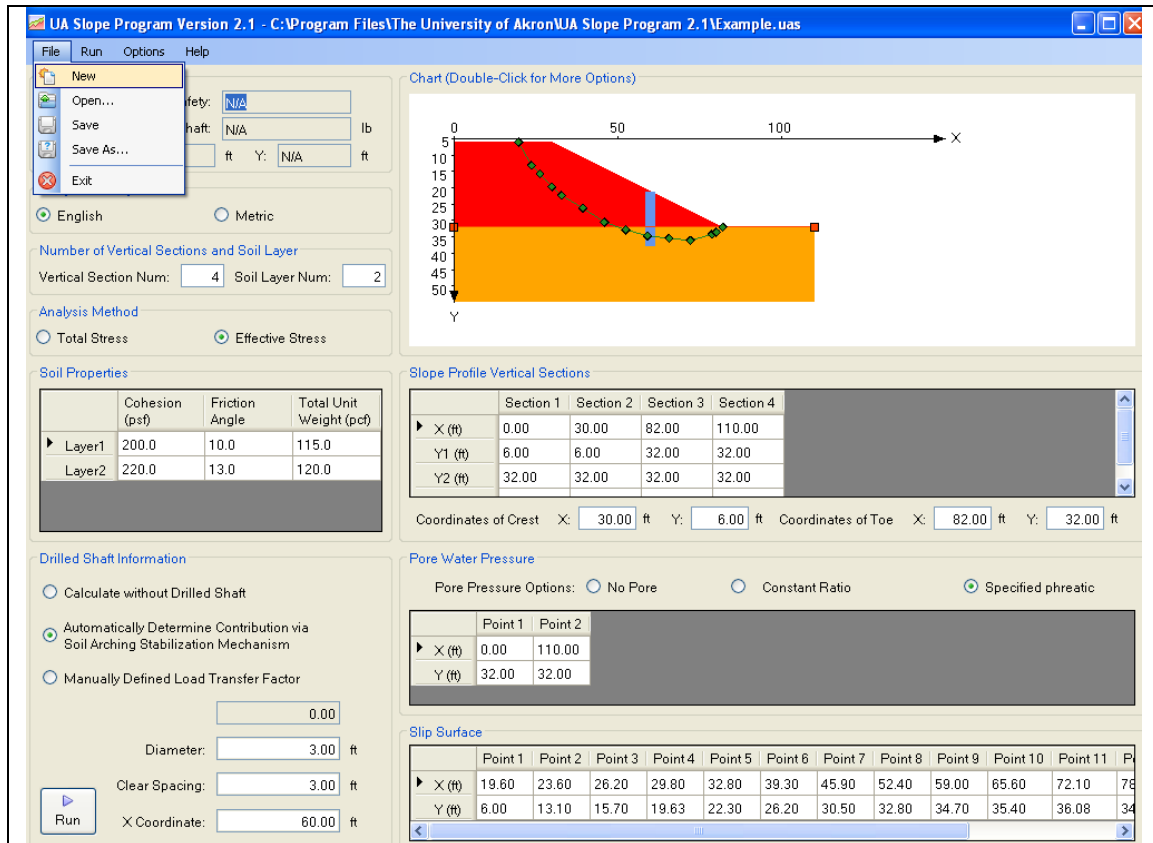


Figure 2: Main Screen showing the Pull Down under the File Menu

4.2.1 General Information

As depicted in Figure 1, the main menu is where the general information pertaining to the problem is provided through input boxes for creating a data file for the case. In the main menu page, as shown in Figure 3, on the left side of the window page, the user is required to provide basic information related to the project, the number of vertical section and number of soil layers used to define the slope soil profiles, units to be used in the analysis, selection between a total versus effective stress approach, input of soil properties for each soil layer, and input of drilled shaft information, such as diameter,

clear spacing between adjacent shafts, length, and location, and finally if drilled shafts will be considered in the slope stability analysis. There is also an option to manually input load transfer factor in lieu of the built-in semi-empirical equations presented in the report. On the right side of the window page, the graphic plot of the problem slope is shown once the coordinates defining the soil profiles are input. An input table is shown for typing in x and y coordinates of the points on each vertical line (section) for each vertical section used to define slope profile. Pore pressure as defined either by specified phreatic values or the constant ratio of the pore pressure. Once all input is completely provided, the data should be saved as a file for subsequent computation.

4.2.1.1 Unit

UA SLOPE 2.1 allows the user to choose between Metric and English units. The selected units will be used though out the analysis. Table 1 provides a summary of the commonly used units in both Metric and English units in UA SLOPE 2.1.

Table 1: Units used in UA Slope 2.1

Unit	Force [F]	Length [L]
Metric	kN	m
English	lb	ft

4.2.1.2 Number of Vertical Sections and Soil Layers

UA SLOPE 2.1 requires the user to input the total number of vertical lines (sections) and number of soil layers for defining the geometry, soil profile, and cross-section of the slope. To take advantage of this user friendly method of creating a cross section, a vertical line needs to be assigned to each breaking point of the boundaries of the slope and the boundaries between different soil layers. The program can only accommodate up to 20 soil layers, but requires a minimum of two soil layers. At each vertical line (section) with its x coordinate, the corresponding y coordinate for each intersecting point of the vertical line and the left to right lines (defining the soil layer boundaries and slope boundaries) needs to be provided.

4.2.1.3 Analysis Method

The program has the capability to run either total or effective stress analysis. However, as mentioned in the report, it is highly recommended that the effective stress approach be used, simply because the load transfer factor was based on effective stress concept.

4.2.1.4 Soil Properties

As shown in Figure 3, UA SLOPE 2.1 requires the user to input the effective shear strength parameters (i.e., cohesion $[F/L^2]$ and friction angle [Degrees]) and the total unit weight $[F/L^3]$ for each soil layer.

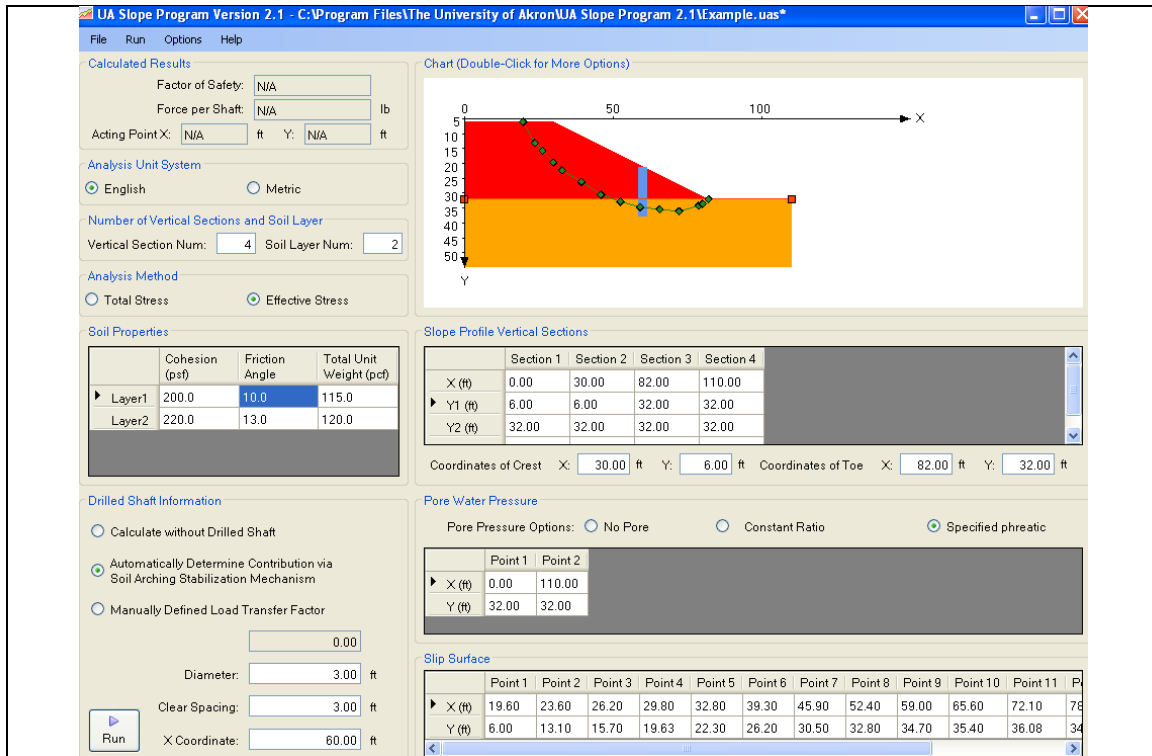


Figure 3: General Information Input Window

4.2.1.5 Drilled Shaft Information

In the drilled shaft information section, the user is required to input location of the shaft to be installed within the slope, shaft diameter, clear spacing between two adjacent shafts and location of the shaft. Also, the user can input the value of the load transfer factor directly using the option of manually input load transfer factor. The value of the load transfer factor can be anywhere between 0 and 1. This option allows for optimization of the drilled shafts size, shaft location and the spacing between the drilled shafts for a given unstable slope with a known slip surface to achieve the desired target FS of the slope/drilled shafts system (see Section 6 for more explanations).

4.2.1.6 Slope Profile Vertical Sections

The user is required to input the x and y coordinates defining the geometry and soil layers of the slope. The X_i value for each vertical line (section) should be entered in an ascending order from left to right. On the other hand, the Y_i value of each intersection

point on a particular vertical line should be entered in the order of from the top layer to the bottom one. The definition of the XY coordinate system used by the UA SLOPE 2.1 program is shown in Figure 4, in which the origin of the X-Y coordinates is at the upper left corner of the screen; the x-value increases as it moves to the right and the y-value increases as it moves down. It is very important that the slope should be represented in such a way that the slope is sloping down from left to right, as shown in Figure 4.

Also, the user is required to input the “Coordinates of the crest” and the “Coordinates of the toe”, these coordinate should be estimated based on the slope profile section and will be used to calculate the slope angle.

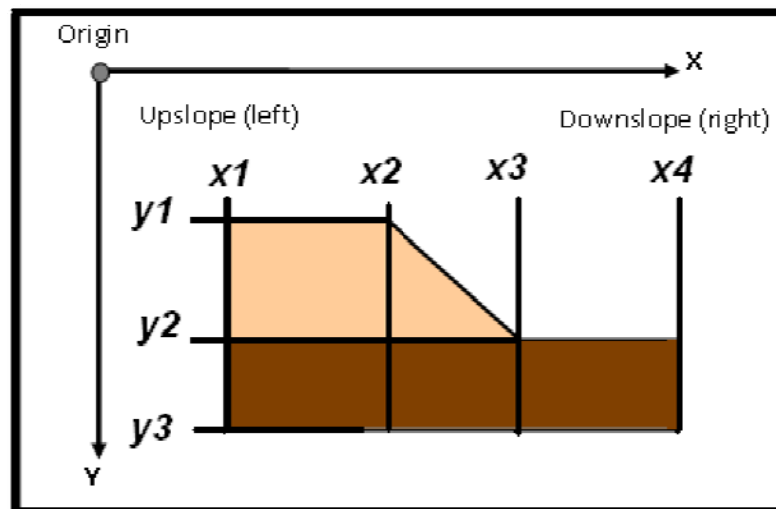


Figure 4: Definition of the x-y Coordinate System Used in UA SLOPE 2.1 Program

4.2.1.7 Pore Water Pressure

UA SLOPE 2.1 program allows the user to choose from three different pore water pressure options: (1) no pore pressure, (2) constant pore pressure ratio, and (3) specified ground water surface.

The ground water surface is defined by the X-Y coordinate of the points selected. The x-values should be input in an ascending order. As shown in Figure 5, the user can set number of point, to a maximum of 50 points, add or delete point by right clicking in the defined box in the pore water pressure part.

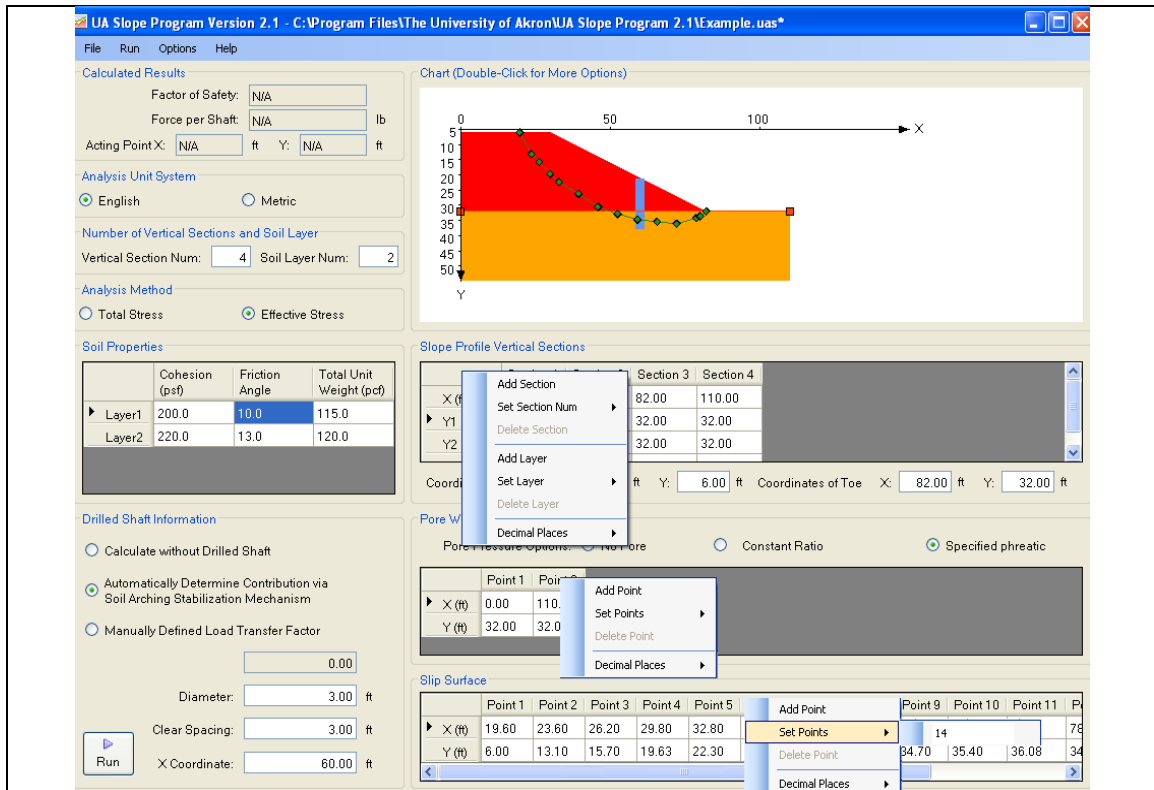


Figure 5: Set Number of Points in Defining Vertical Section, Ground Water Table and Slip Surface

4.2.1.8 Slip Surface

The user needs to input the total number of points, maximum 50 points, along the slip surface by right clicking in the slip surface box as shown in Figure 5. Once this is defined, then the x-y coordinates of each selected point should be typed in accordingly. Again, the x-values of the selected points should be input in an ascending order (i.e., in the direction from left to right).

4.2.1.9 Chart (Double-Click for More Option)

By double clicking on the chart in the “Chart” section, “Chart View” window appears and lets the user see the soil profile with or without “layers”, “pore pressure”, “slip surface”, and “shaft”. Also, there is an option for the user to do either “zoom in” or “zoom out” in this window.

4.3 Run Menu

The Run button, as shown in Figure 6, is used to execute an existing data file. The user can select whether to consider or not to consider the drilled shafts in the analysis. After inputting all the data the user need to either click on the run button or click on the execute under Run drop down, then by clicking the execute button, the results will appear in the “Calculated Result” textboxes in the same screen.

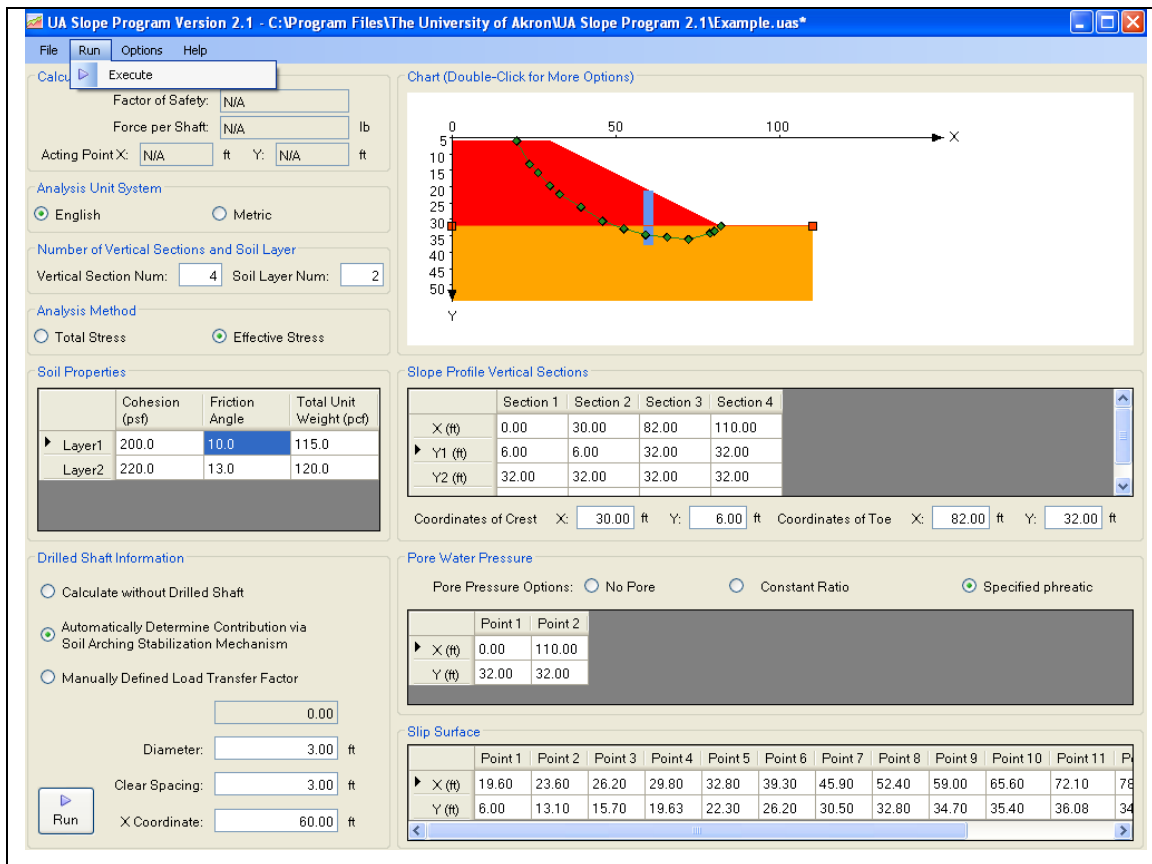


Figure 6: Run Menu

4.4 Options Menu

The Options menu provides information about path and file names. Also, as shown in Figure 7, there is a “Chart options” that the user can use to change the color of soil layers, slip surface, and ground water surface. By double clicking on the chart, the user will be

able to turn on or turn off the layers, slip surface, ground water surface, and to change the color of layers.

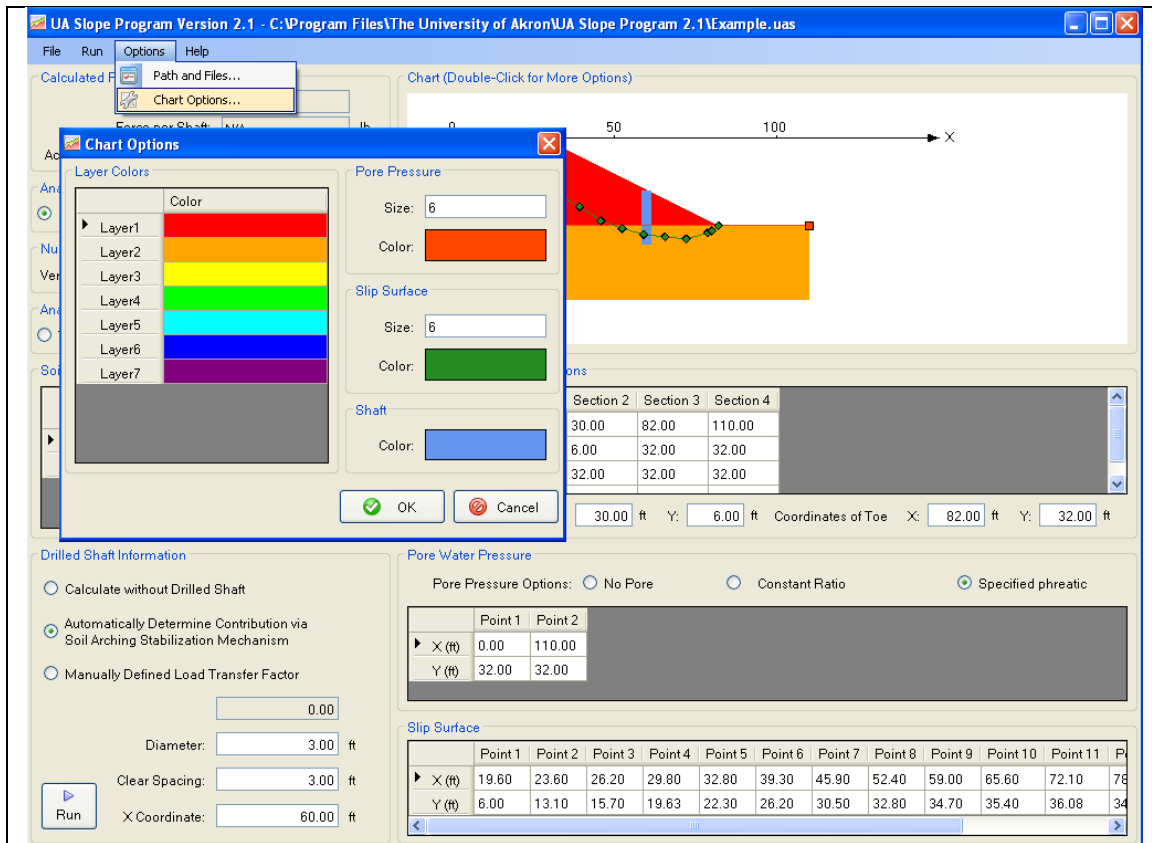


Figure 7: Chart Option in the Option Menu

5. ILLUSTRATIVE EXAMPLE

The primary purpose of this illustrative example is to show how to generate a data file for subsequent computation by the UA SLOPE 2.1 program. It is, however, not intended to demonstrate the design procedure.

5.1 General Description

The slope geometry for this illustrative example is shown in Figure 8. The slope is 26 ft high with a slope that is 2 horizontal to 1 vertical. The soil in the slope consists of two

different soil layers: the main slope body belongs to soil layer I and the firm stratum below the elevation of the toe belongs to soil layer II. The ground water table elevation is assumed as shown in Figure 8. The soil properties for the two different soil layers are summarized in Table 2. With the given slope geometry, soil profile, and soil properties, the factor of safety of this slope was calculated as 1.08. The example intends to show how the input file is generated and the typical analysis results. The readers should consult Chapter IV of the report for more detailed instruction on how to use the UA SLOPE 2.1 program to perform analysis and to finalize an optimized drilled shaft/slope system that satisfies the target geotechnical factor of safety and requires the least construction cost.

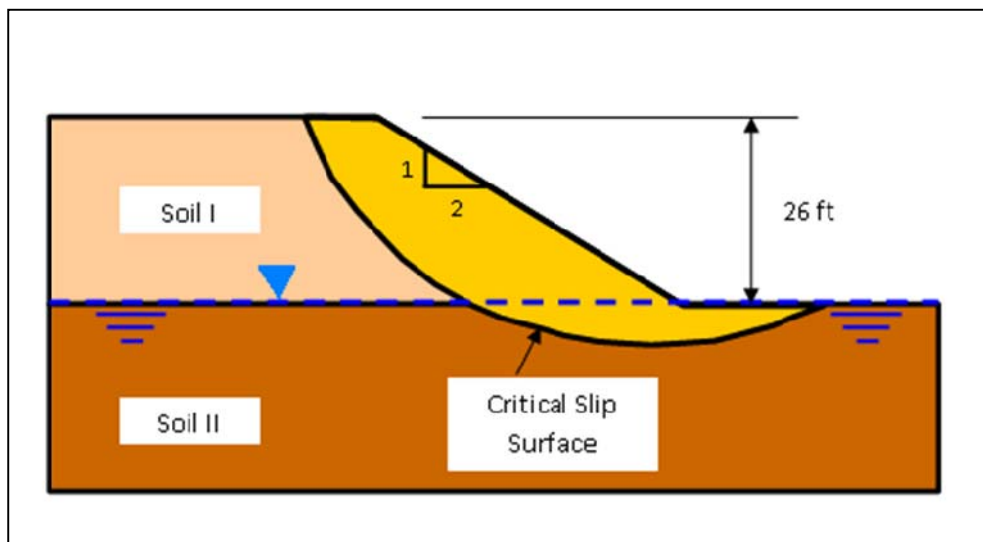


Figure 8: The Slope Geometry, Soil Profile, Slip Surface, and Ground Water Table for the Illustrative Example

Table 2: Soil Properties for the Two Soil Layers in the Illustrative Example

Soil	Cohesion (psf)	Friction Angle (Degree)	Unit Weight (pcf)	Soil Modulus (psf)
I	200	10	115	100000
II	220	13	120	100000

5.2 UA SLOPE 2.1 Program

To see the input file, the user can open the file name “Example1” from the provided UA SLOPE 2.1 installation disc. This example is further explained below.

5.2.1 General Information, Slope Geometry and Slip Surface Specifications

Please refer to Figure 9 in reading the following discussions.

- English units are used for this analysis. (i.e., Force = lb and Length = ft)
- From Figure 8, four vertical sections are used to define boundaries in the example.
- There are two soil layers; i.e., soil layer I for soils in the slope body and soil layer II for the soil in the foundation soil
- A total of fourteen points along the identified slip surface are entered to represent the location of the slip surface.
- The ground water table is defined by two points.
- The effective stress approach is used in this analysis.
- As a starting point, the drilled shaft is selected at $X = 60$ ft. The initial selection of drilled shaft dimensions for trial design is as follows: drilled shaft diameter = 3 ft, clear spacing between adjacent drilled shaft = 3 ft, and length of drilled shaft = 35 ft.
- The x-coordinates for the four (4) vertical sections to represent the slope profile should be input in an ascending order in the first row of the grid provided for the slope profile specifications as shown in Figure 9. The y- coordinates for each layer from top to bottom should be input corresponding to each x- coordinate entered. (See Figure 9 for the x and y values used in the analysis)
- From Table 2, material properties are input in the grid provided for the soil properties. The first row represents the material properties for the first (top) soil layer, and the second row represents the second (lower) soil layer, and so on.
- Once the input data is completely filled in, the data should be saved as a file before proceeding with computation. It is a good practice to run the option of “calculate without drilled shaft” first, so that the accuracy of the defined slope geometry, soil profile, and soil properties can be ascertained. Figure 9 shows the computed FS of the defined slope problem as 1.08. After a satisfactory run with an option of

without the drilled shafts, then the program can be run with an option to consider the drilled shafts. The factor of safety and the net force on the drilled shaft for the shaft/slope system will appear in the “calculated result” box in the same screen, as can be seen in Figure 10.

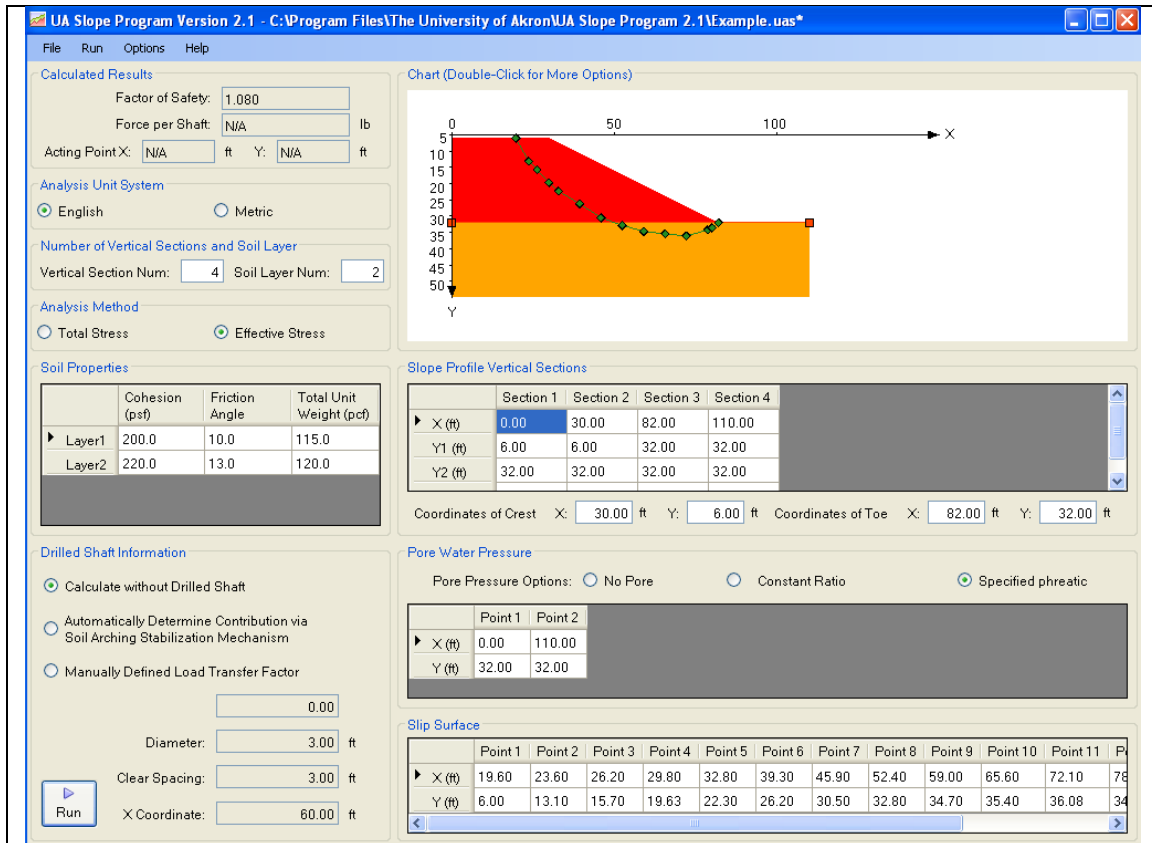


Figure 9: Input File of General Information, Slope Profile, and Soil Property for the Illustrative Example

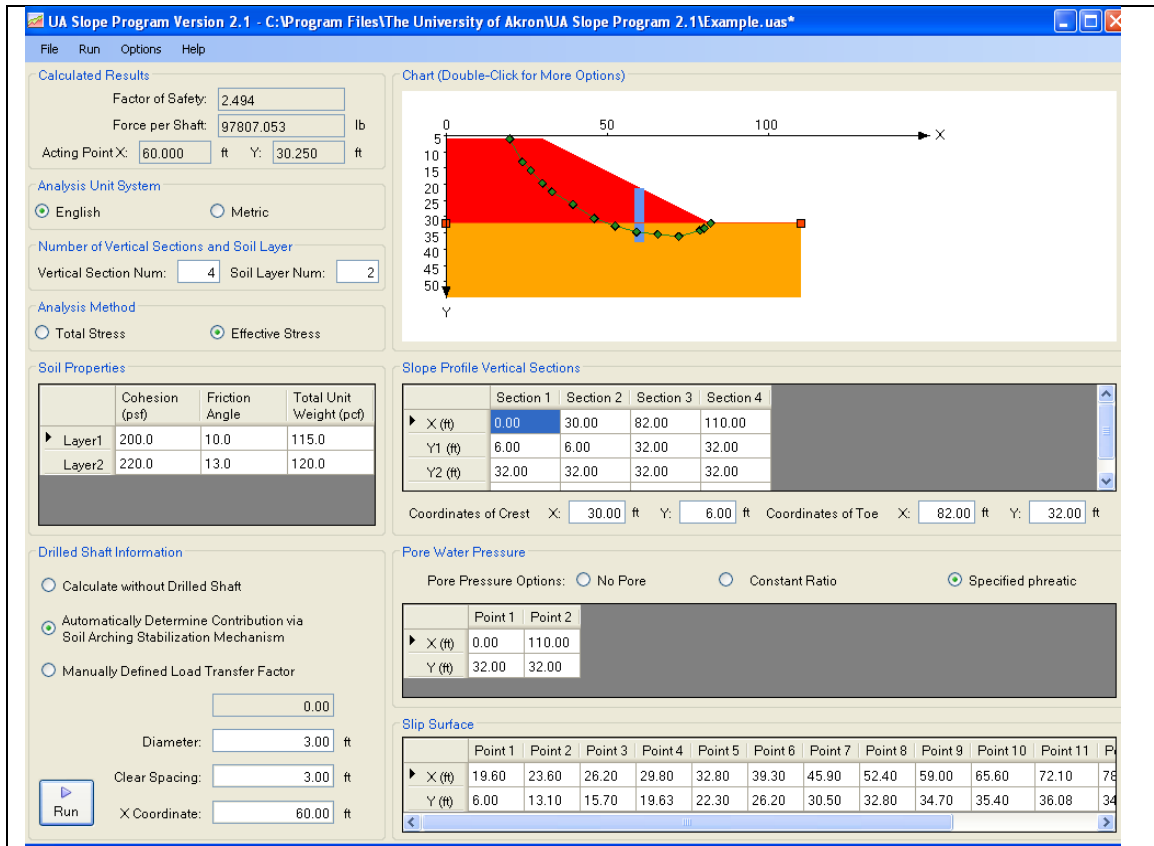


Figure 10: Output Result When Considering the Drilled Shaft Option

The user can proceed in the analysis by trying different drilled shaft diameters, spacings, and locations so that all design requirements and constructability issues can be evaluated. For example, several runs were carried out with the shaft diameter = 3 ft and clear spacing between shafts = 3 ft, while varying the location of the drilled shafts from X= 30 ft to X = 80 ft with 10 ft increments. The resulting factor of safety and the net force on the shaft for each considered shaft location are tabulated in Table 3 and plotted in Figure 11.

Table 3: Values of Factor of Safety of a Shaft/Slope System for Different Shaft Locations

X (feet)	FS	Force per shaft (kips)
30	1.15	9.1
40	1.4	40
50	2	81
60	2.4	97
70	1.7	62
80	1.1	4.14

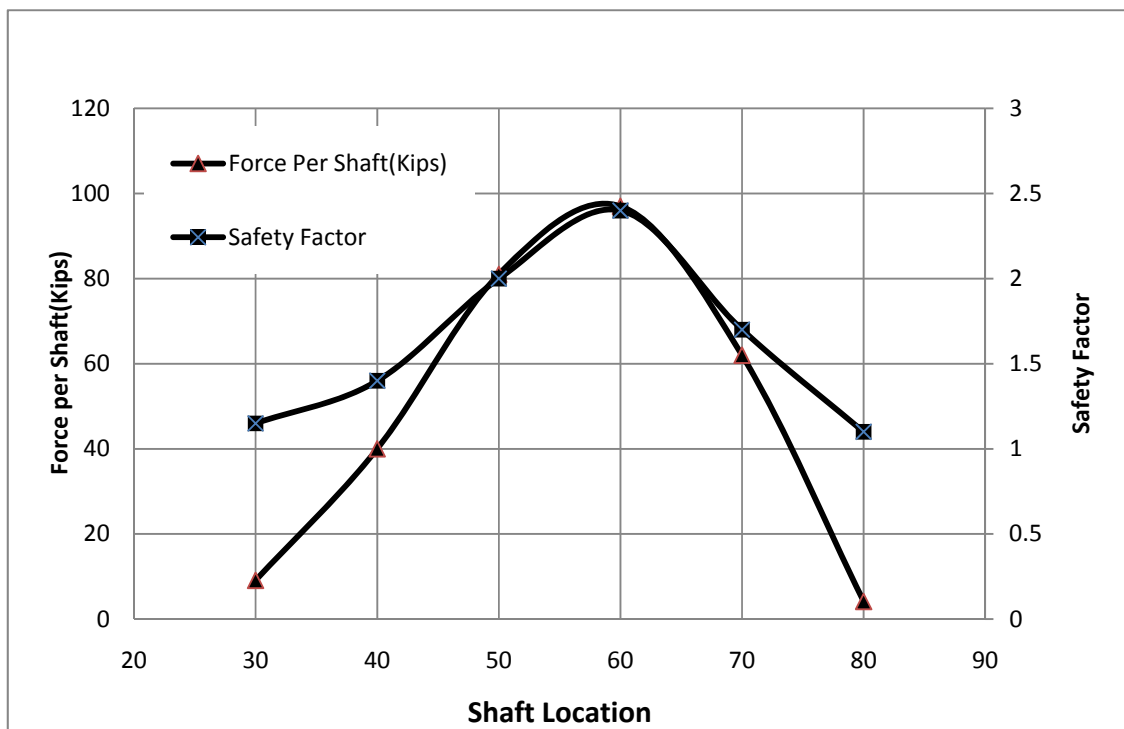


Figure 11: Plots of Factor of Safety and Net Shaft Force versus Drilled Shaft Location for the Illustrative Example

6. Step by step design methodology using manually input load transfer factor

Using manually input load transfer factor allows for optimization of the drilled shafts size, shaft location, shaft fixity (the necessary rock-socket length), and the spacing between the drilled shafts for a given unstable slope with known slip surface to achieve the desired target FS of the slope/drilled shafts system.

DESIGN EXAMPLE

The slope geometry, soil layers, slip surface and description of the layer for this example is exactly the same as it was shown in the previous section.

1. The stability of the existing condition of the slope, as it was shown in Figure 9, was examined and the factor of safety FS_0 was found to be 1.08.
2. Specify a target factor of safety to achieve with the installation of drilled shaft. Here in this example, choose FS_{target} to be 1.5.
3. Specify the possible locations of drilled shafts where it can be placed within the slope. For an illustration purpose, assume there is no constructability issue and the possible drilled shaft locations are between $X = 35.2$ ft and $X = 76.8$ ft.
4. Assume a shaft diameter $D = 3$ ft and shaft location of $X = 60$ ft, then, ξ is calculated as follows:

$$\xi = \frac{82 - X}{82 - 30} = \frac{82 - X}{52} = 0.42$$
5. Using the option of manually input the load transfer factor in the UA SLOPE 2.1 program, determine FS for several different η . ($0 < \eta < 1$)
6. Repeat Steps 3 – 5 for several drilled shaft locations. Consider an increment of 5 ft for the drilled shaft location.
7. For ξ and D selected in Step 4 and the results obtained from Step 5, create η -FS diagram as shown in Fig. 12 for several drilled shaft locations.
8. Determine the target load transfer factor (η_{target}) from η -FS diagram for several shaft locations corresponding to FS_{target} .

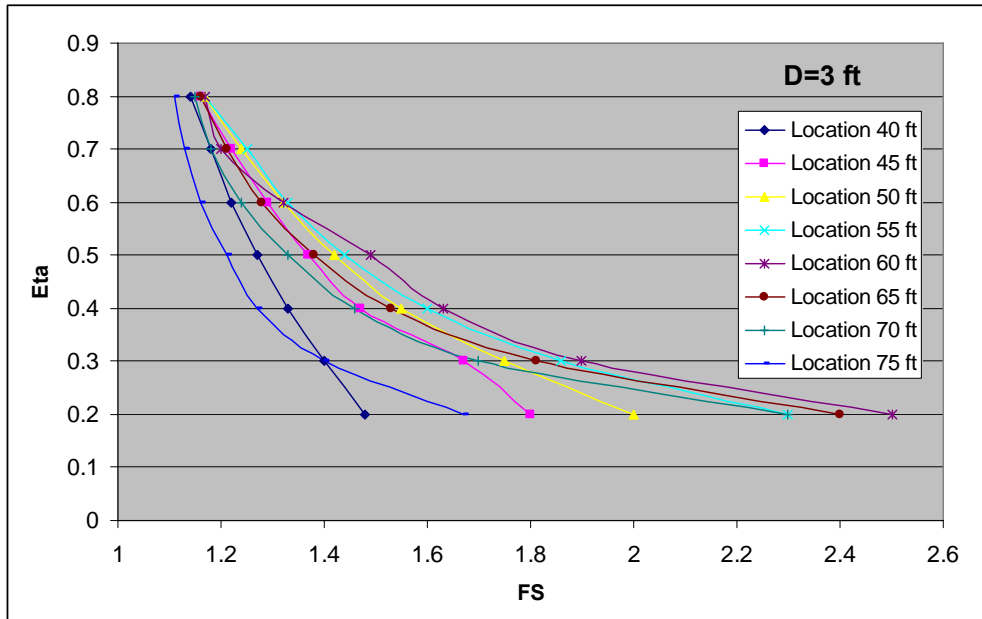


Fig. 12. η -FS diagram for the illustrative example at various drilled shaft locations
(D = 3 ft)

9. Plot the target load transfer factors (η_{target}) versus shaft locations (X) as shown in Fig. 13.
10. From Fig. 13, the optimum location of the drilled shafts is at X = 55 ft (i.e., $\xi = 0.52$) where the maximum target load transfer factor is found to be 0.46. (maximum eta is selected to get the minimum force on the drilled shaft.)

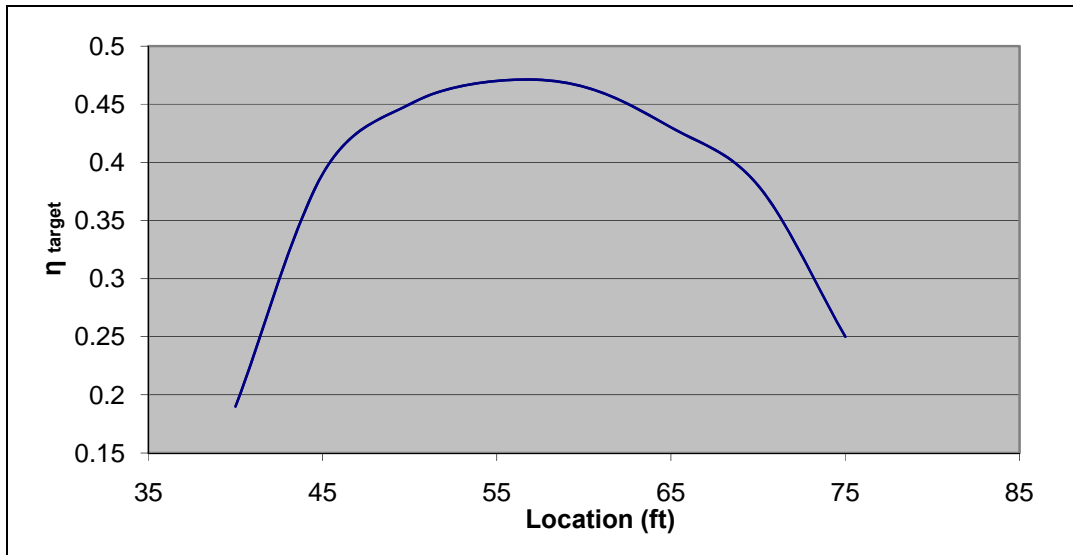


Fig. 13. η target vs. shaft location (X) for the illustrative example (D = 3 ft and $F_{\text{Starget}} = 1.5$)

11. For $\eta_{\text{target}} = 0.46$, $D = 3$ ft, and the location of the shaft = 55 ft, the spacing to diameter ratio (ξ_s) can be determined as 3.35 from the equation below to yield the targeted load transfer factor of 0.46. This ratio gives a clear spacing of 7.05 ft for $D = 3$ ft.

$$\eta = -0.272C^{0.153} (\tan \beta)^{-0.429} (-1.17 + 1.114 \frac{S}{D}) (e^{(-0.578 \tan \phi)}) (0.065 + 0.876D) (-0.252 + 0.61\xi - 0.57(\xi^2))$$

12. Input the above determined design parameters, including D, clear spacing, and shaft location into the UA SLOPE 2.1 program to find the net force on the shaft, as shown in figure 14. As can be seen, the UA SLOPE 2.1 run returns the FS = 1.51 and the net force = 81.9 kips.

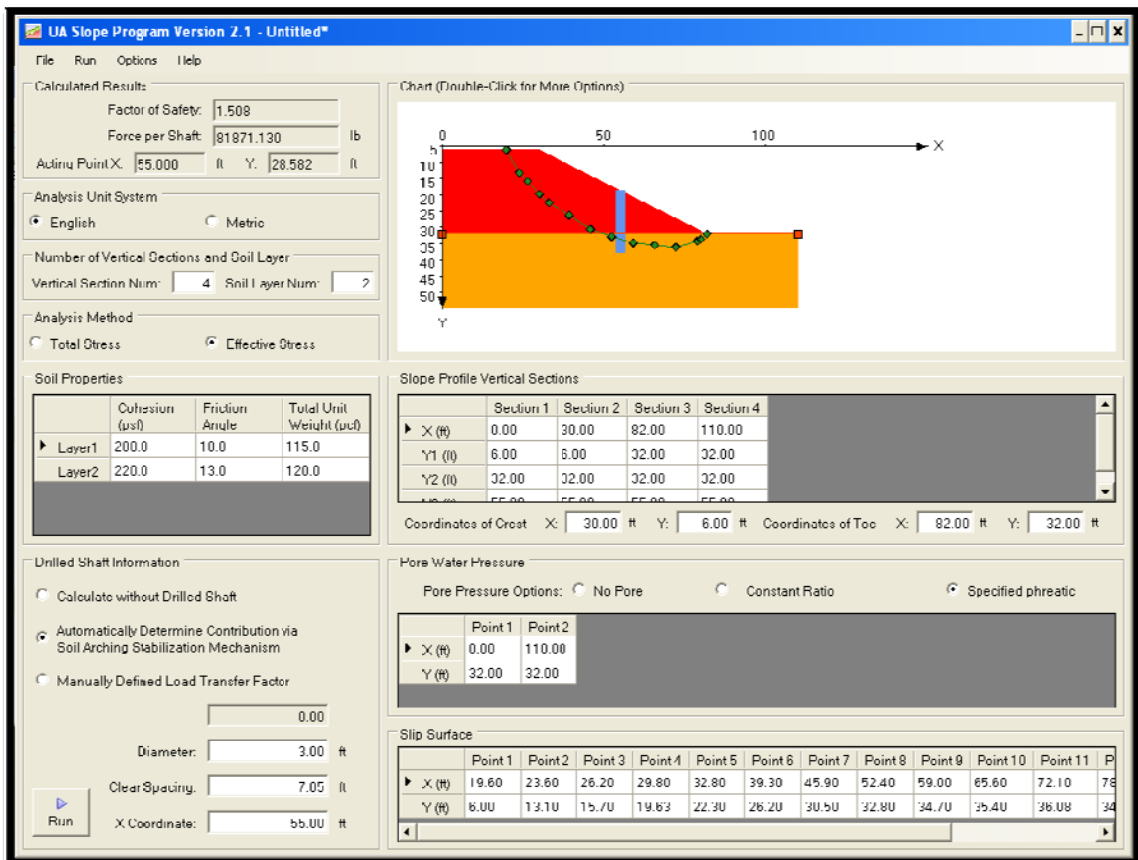


Fig. 14. Calculated FS and net force corresponding to the selected design parameters of shaft diameter, clear spacing, and shaft location

13. If desired, different diameters of the shaft can be tried by repeating Steps 4 to 12.

7 GENERAL COMMENTS

- Minimum number of soil layers should be two (2) layers, while the maximum number is twenty (20) soil layers.
- Only one single row of spaced drilled shafts can be considered.
- The point of origin for the coordinate system used by UA SLOPE 2.1 is at the upper left corner.
- The down slope direction of the slope should be defined by the convention, i.e., from left to right.
- Always make sure the end points of the slip surface (i.e., the initiation and exit points) match with the specified boundaries of the slope.
- Always start the analysis without considering the drilled shaft on the slope in order to confirm that correct input was provided in creating the input file.