Proceedings of the

International Conference on Geotechnical Engineering

November 5-6, 2010
Lahore, Pakistan

Editors:

Sohail Kibria
Hamid Masood Qureshi
Arooj Mahmood Rana

Pakistan Geotechnical Engineering Society (PGES)

In Association With

National Engineering Services Pakistan (Pvt.) Ltd.

University of Engineering & Technology Lahore, Pakistan

Associated Consulting Engineers (Pvt.) Ltd. Lahore, Pakistan

Back-Analysis Procedure for Landslides

M. Hussain
College of Civil Engineering Risalpur (National University of Sciences and Technology), Pakistan (manzoor7@yahoo.com)
T. D. Stark
University of Illinois at Urbana-Champaign, USA (tstark@illinois.edu)
K. Akhtar
University of Illinois at Urbana-Champaign, USA (kakhtar2@illinois.edu)

ABSTRACT Back-analysis of a landslide is performed to estimate the mobilized shear strength of the problematic layer. Back-analysis can be an effective procedure for estimating the mobilized shear strength because it avoids the problems associated with laboratory and in-situ testing of the problematic layer. The mobilized strength can be used to assess landslide causation and design remedial measures. Even though the factor of safety is known (unity), back-analysis can still involve uncertainties, e.g. failure surface, slope geometry, pore-water pressures, use of tension cracks, shear strength and unit weight of other materials, and accuracy of the stability method. This paper investigates the influence of some of these factors on the back-analysis of landslides using case histories. The results of this investigation are used to develop guidelines for the back-analysis of landslides.

INTRODUCTION

The back-analysis is usually a more effective method of estimating the shear strength of the problematic layer than laboratory testing. At a minimum, the back-calculated strength can be used to verify the laboratory measured strength. Even though back-analysis usually yields a better shear strength estimate than laboratory tests, there are uncertainties in a back-analysis. Some of these uncertainties have been discussed by Leroueil & Tevenas (1981), Duncan and Stark (1992), Stark and Eid (1998), Gilbert et al (1998), Tang et al (1999), and Deschamps and Yankey (2006). Some of the uncertainties that influence the back-calculated shear strength are engineering properties of the other materials in the cross-section, slope geometry at the time of failure, phreatic surface and porewater pressures present at the time of failure, effect of rainfall, location of failure surface, and existence of tension cracks. Thus, experience and judgment should be used in selecting the input parameters and assumptions for the materials involved in the landslide. However, even experienced engineers sometimes make incorrect assumptions for the back-analysis which impacts the results. This paper presents the importance of the various input factors involved in a back-analysis using case histories and uses the results to develop guidelines for conducting a landslide back-analysis.

BACK-ANALYSIS PROCEDURE FOR LANDSLIDES

Based on the uncertainties with a back-analysis described in the subsequent section, a comprehensive back-analysis procedure for landslides was developed during the present study. The back-analysis procedure includes:

1) Understanding the subsurface conditions (type of soils/materials, thickness and shear strength of various layers, ground water surface, surface water sources, external water sources, porewater pressures, slope geometry, tension cracks, etc.).
2) Defining representative cross-sections that are located and oriented parallel to the direction of maximum movement. The cross-sections should include all relevant materials and structures.
3) Defining the type and location of the failure surface based on ground surface observation, slope inclinometer data, and subsurface features.
4) Selecting the appropriate stability method and software for the back-analysis.
5) Varying the shear strength of the problematic/weak layer until the factor of safety equals approximately unity (FS≈1.0) to determine the back-calculated shear strength of the problematic layer.
6) Comparing the back-calculated shear strength parameter (ϕ') with the results of laboratory strength testing on representative samples to ensure agreement.
7) Comparing the back-calculated shear strength parameter (ϕ') with empirical correlations, such as presented by Stark et al. (2005a) and improved by Hussain (2010), to ensure agreement. If the landslide is a first-time landslide, i.e., no slide occurred previously at the site, empirical correlations for the fully softened shear strength should be used to verify the back-

calculated shear strength depending on the level of progressive failure that contributed to the landslide. In a first-time slide, the back-calculated shear strength should be at or slightly below the measured or estimated fully softened strength. If the landslide is a reactivation of a prior slide, laboratory testing or an empirical correlation for the residual strength should be used to verify the back-calculated strength. The back-calculated residual shear strength should be in agreement with the measured or estimated residual shear strength.

8) If the back-calculated shear strength is not in agreement with the appropriate empirical correlation and/or laboratory strength testing, there could be an error in the back-analysis. This should be investigated by checking the input parameters and the entire process should be repeated until the back-calculated shear strength is in agreement with the appropriate empirical correlation and/or laboratory strength testing.
9) Reconduct back-analysis with revised parameters and verify results are in agreement with empirical correlations and test results.

UNCERTAINTIES IN BACK-ANALYSIS

A landslide back-analysis is performed by making some assumptions on the failure conditions present along the slide mass/slope at the time of failure and assuming input parameters which yield a factor of safety of unity (FS = 1.0). A major difference between a design analysis and a back-analysis is that conservative assumptions in a design analysis are unconservative in a back-analysis. For example, assuming a higher phreatic surface is conservative in a design analysis but it is unconservative in a back-analysis because it results in a larger back-calculated shear strength which may over predict mobilized shear strength along the shear surface. Therefore, making reasonable assumptions on the conditions present along the shear surface at the time of sliding is important. Some of the uncertainties in a back-analysis are discussed herein which include slope geometry, material properties, phreatic surface and porewater pressures present at the time of failure, effect of rainfall, location of failure surface, and existence of tension cracks.

Geometry and Material Properties at Time of Failure

To perform a back-analysis, it is necessary to know the slope geometry at the time of the landslide to estimate the driving forces. The slope geometry can be obtained from prior topographic surveys or aerial photographs. Satellite images before and after the slide can also be used to determine slope geometry prior to failure.

The site stratigraphy, i.e., material types and thicknesses, can be obtained from borings and/or insitu tests. Large diameter borings, e.g., 0.6 to 1.0 meter in diameter, can be used to obtain more representative soil samples than conventional borings, better determine soil stratigraphy, estimate bedding strike and dip, and locate the failure surface by inclinometers and/or lowering professionals down in the borings.

Applicable Shear Strength

The shear strength is defined as the maximum value of shear stress that the soil can withstand. The time and conditions under which water is able to flow into or out of a soil mass determines whether a drained or undrained analysis should be performed (Duncan and Wright, 2005). Drained strength is the applicable soil strength when the soil is loaded slowly enough so excess pore pressures, i.e., pore pressures that exceed the hydrostatic value, are not induced by the applied loads. In the field, drained conditions result when loads are applied slowly to a mass of soil, or where they persist for a long enough time that the soil can drain the excess porewater pressure (Duncan and Wright, 2005). Rise in the groundwater surface by rainfall results in a greater hydrostatic pressure but not an undrained condition. Therefore, the back-analysis of such case histories which involve changes in groundwater surface due to rainfall or climatic conditions should utilize a drained analysis.

Because landslides are common phenomena in cohesive soils subjected to rainfall, drained shear strengths considered are the residual and fully softened which are discussed below.

Drained Residual Shear Strength

The drained residual shear strength of cohesive soils is applicable to new and existing slopes that contain a preexisting shear surface (Skempton, 1964 and 1985 and Stark et al., 2005a). Excess pore pressures usually are not generated along a preexisting shear surface and thus a drained stability analysis should be used. Stark et al. (2005a) recommend using c' of zero in a back-analysis because it provides agreement with laboratory measured residual shear strength and the residual strength is defined. Tiwari et al. (2005) also confirm that the back-calculated residual friction angle with $c' = 0$, agreed well with experimental results.

Although various researchers since Skempton (1964) conclude that back-calculated shear strength of reactivated landslides is in agreement with the drained residual shear strength of the slip surface material, most of the researchers use a linear relationship between the shear strength and effective normal stress. Stark and Eid (1994) use two case histories of reactivated landslides to conclude that the back-calculated shear strength of reactivated landslide is in agreement with the drained residual shear strength obtained from torsional ring shear testing using a stress dependent strength failure envelope. The shear strength data and empirical correlation presented by Stark et al. (2005a) and improved by Hussain (2010) show the residual friction angle is stress dependent which is in agreement with conclusions of Chandler (1977), Bromhead (1978) and Stark and Eid (1994 and 1997). Thus, the drained residual shear strength should be considered for the back-analysis of reactivated landslides as suggested by Skempton (1964 and 1985) but a stress dependent strength failure envelope should be used in the back-analysis instead of using a linear relationship especially for high plasticity soils.

Drained Fully Softened Strength

The drained fully softened shear strength of cohesive soils is also an important parameter in evaluating the stability of slopes that have not undergone previous sliding. In case of a first-time slide, the fully softened shear strength governs the failure (Skempton, 1970). However, Stark and Eid (1997) show that the strength mobilized in first-time slides can be less than the fully softened strength if part of shear surface contains some slickensides or preexisting shear surface. Stark and Eid (1997) also suggest using a stress dependent failure envelope for fully softened shear strength of cohesive soils. Stark et al. (2005a) recommend that c' equals zero be used for back-analysis of first-time landslides unless a first-slide occurred in overconsolidated material so c' can be greater than zero.

Skempton (1964) back-calculated the average strength parameters of c' = 6.7 kPa and ϕ'_r=18° for the problematic portion of both Brown and Blue London clay using a factor of safety of unity (FS = 1.0), a circular slip surface, and Bishop's (1955) stability method for the Northolt landslide. Duncan and Stark (1992) back-analyzed the same landslide using a noncircular slip surface obtained by joining known points along the failure surface (Slip Surface 'E' in Fig 1) and using Spencer's (1967) stability method which resulted in a back-calculated linear shear strength parameters of c' = 0.72 kPa and ϕ'= 25°.

Because the Northolt landslide involves a first-time slide, the fully softened strength should have governed the failure instead of the residual strength (Skempton, 1970). In the current back-analysis of Northolt landslide in London clay, a noncircular failure surface, 'E' as shown in Fig 1, and Spencer's (1967) stability method were used. The Northolt landslide was selected for back-analysis herein because it is a well documented case and was used by Skempton (1964) using a linear relationship between shear and effective normal stresses. Back-calculated fully softened strength linear parameters for the Northolt landslide c' = 0 and ϕ'=24.4° yielded FS = 1.0. The stress dependent fully softened strength failure envelope developed from the fully softened strength empirical correlation presented by Stark et al. (2005a) and improved by Hussain (2010) as shown in Fig 2 also yielded FS = 1.0.

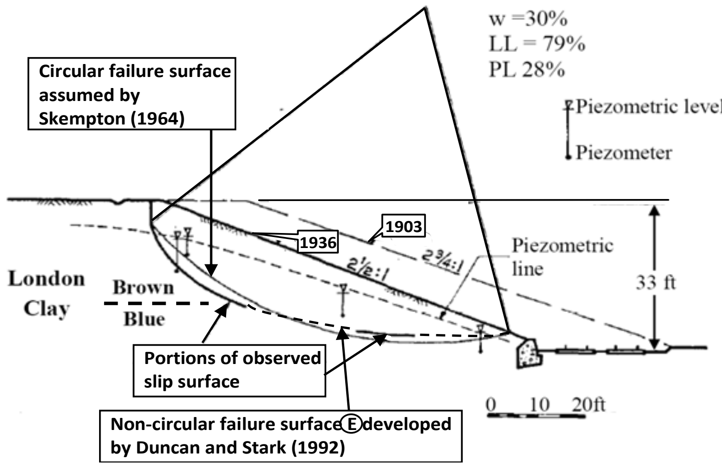

Fig. 1 Cross-section and failure surface used by Skempton (1964) and Duncan and Stark (1992) (from Skempton, 1964 with non-circular failure surface by Duncan and Stark, 1992).

Thus, the drained fully softened shear strength should be considered for the back-analysis of first-time landslides as suggested by Skempton (1970 and 1985) but a stress dependent strength failure envelope should be used in the back-analysis instead of using a linear envelope because the field fully softened strength is stress dependent.

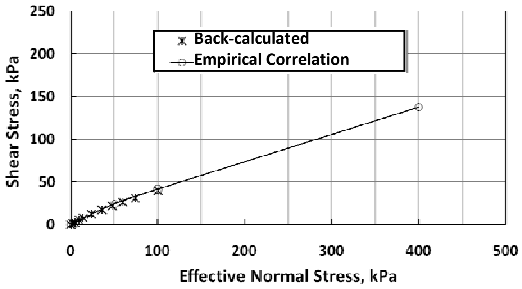

Fig.2 Stress dependent fully softened strength envelope for London clay developed from empirical correlation presented by Stark et al. (2005a) and revised by Hussain (2010).

Phreatic Surface and Pore Pressures

Determination of the phreatic surface at the time of the landslide is important because the porewater pressures affect the effective stress acting on the failure plane. The effective stress in turn affects the back-calculated shear strength of the problematic layer. Hussain (2010) shows that a variation in phreatic surface by 1.52 m results in a 8-10% change in FS for Weirton landslide in West Virginia and Alver`a landslide in Italy. Thus, assuming a higher phreatic surface than occurred in the field will result in a higher back-calculated shear strength parameters (ϕ') for FS = 1.0. Assuming the porewater pressures higher than at failure is unconservative because it results in back-calculated strengths that are too high (Duncan and Stark, 1992). This is contrary to design analyses where a higher level of porewater pressure or phreatic surface is usually conservative. If there is large uncertainty in the phreatic surface or porewater pressure condition, it may be easier and more reliable to back-calculate the phreatic surface rather than the ϕ'.

Effect of Rainfall and Pore Pressures

Rainfall contributes to the triggering of landslides by infiltrating the slope cover, which causes a rise in the groundwater surface and an increase in the hydrostatic porewater pressures along the failure surface. Rainfall is a common cause of landslides and in particular shallow landslides (Wieczoreck, 1996). Shallow landslides in soils and weathered rock are often generated in steep slopes during intense parts of a storm (Wieczoreck, 1996).

In the case of deep-seated landslides, movement usually occurs when a cumulative rainfall threshold is reached. Stark et al. (2005b) studied the effect of rainfall in triggering a deep-seated landslide along a highway cutslope in the San Francisco Bay area that resulted in distress to a single family residence. It was determined that movement of the large slide mass usually occurred near the end of a heavy rainy season, because time was required for the groundwater surface to rise sufficiently in response to the increased rainfall. Review of the rainfall record revealed that at least 890 mm of rainfall had to occur to re-initiate movement in that landslide. If the cumulative rainfall was less than 890 mm/year, no distress was observed in the residence.

Therefore, two separate thresholds for each area need to be established:

a) Intense rainfall thresholds resulting in shallow landslides triggering, and
b) Progressive/cumulative rainfall thresholds resulting in deep-seated landslides.

Locating Actual Failure Surface for Back-Analysis

Determining the location of the failure surface along which failure occurred is important for defining the size of the slide mass, causation, and back-calculation of the mobilized shear strength parameters in the problematic layer. Slope inclinometers are important in determining the actual failure surface location however they are frequently installed after the slide and may not show sufficient movement. A back-analysis should only utilize the failure surface along which failure has occurred in the field because the shear strength parameters mobilized along that surface are the parameters being sought. If a different failure surface is used, the shear strength parameters required for equilibrium will be different. Thus, the actual failure surface should be specified in a back-analysis. Hutchinson (1983) explains various methods for locating the failure surface in moving and stationary landslides which include; inference from earlier ground surface movements, direct observation of slip surface in excavations and large diameter boreholes, porewater pressures recorded, contrast in material properties above and below the slip surface, and inclinometer data. This section uses three case histories to illustrate the importance of actual failure surface in the back-analysis.

1) In the back-analysis of the Northolt landslide in London Clay, inclinometer data was reported by Skempton (1964). Connecting the shear surface identified in each inclinometer resulted in a reasonable estimate of the actual failure surface (see Fig.1). In the back-analysis, a circular slip surface which passes through or close to the inclinometer data was used by Skempton (1964) to model the actual failure surface. Skempton (1964) may have used a circular slip surface

because no standard method for a noncircular slip surface was available at that time. Duncan and Stark (1992) in a back-analysis of the same case use a circular and the observed non-circular failure surface (see Fig.1) and show that the back-calculated factor of safety for the circular failure surface is slightly greater (FS=1.02) than that back-calculated for the observed failure surface (FS=1.0). Thus, available inclinometer data can be used to establish a circular or noncircular slip surface and the best representation of the field slip surface should be used for the back-analysis.

2) Stark et al. (2005b) use a back-analysis of a 70 m deep-seated landslide along a highway cutslope in the San Francisco Bay area to show that available information/data is appropriate and yields the best estimate of the actual failure surface. More importantly all of the information/data, e.g. inclinometer data and observed toe of the slide, should be used to estimate the actual failure surface and no information should be omitted. Fig.3 presents a cross-section for this cutslope. In this case all of the available information/data was used to develop the deep-seated failure surface shown in Fig.3. Another analysis for this project omitted the slope inclinometer installed by the State Highway Department at the slope toe that showed a failure surface at a depth of 10 m (30 ft.). This resulted in a shallow failure surface on the cutslope face for the back-analysis and developing a rainfall induced failure mechanism instead of a toe excavation mechanism that implicated the Highway Department. In addition to omitting the inclinometer data, the fact that the inclinometer was "sheared off" at a depth of 10 m (30 ft.) at the slope toe was not considered. As a result, the other analysis concluded that the periodic heave of the highway pavement was caused by expansive soils, differential settlement due to a transition from natural to fill material, and/or poor pavement construction instead of the toe of a deep-seated landslide. This might be a reasonable hypothesis if the inclinometer at the cutslope toe had not been sheared off at a depth of 10 m (30 ft.). Because the inclinometer was sheared off, the proposed failure surface must pass through the inclinometer and terminate in the highway where pavement heave was observed as shown in Figure 3. The figure also shows that a deep bedrock failure surface can incorporate/explain the heave of the highway pavement, the sheared inclinometer, the distress of the residence, the failure surface found in large diameter borings, BA-2, near the residence, and the sheared inclinometer installed adjacent to the residence. A shallow failure surface along the cutslope does not explain all of the observed movement and should not be used for the back-analysis. All available inclinometer and borehole data must be used in the back-analysis to establish the actual failure surface instead of searching a critical surface using some software.

3) In the third case, the use of slope inclinometers and interpretation of the subsurface data was confirmed using the numerical model FLAC Stark et al. (2010). Between 1988 and 1989, a housing development with about 50 units was completed on an undeveloped hillside near Novato, California and is referred herein as the Knolls. An 11 unit housing development was constructed upslope of the Knolls and is referred herein as the Vista. Only 7 of the 11 Vista lots

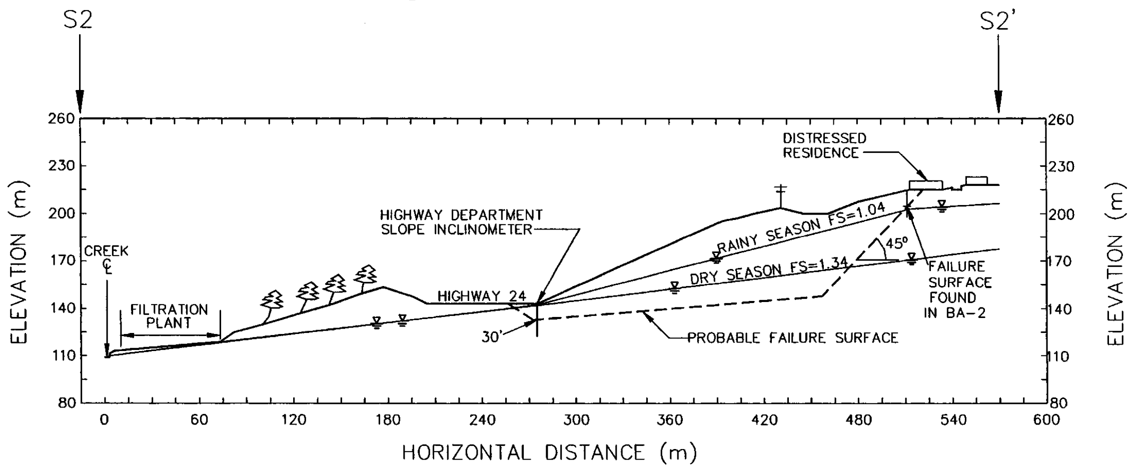

Fig.3 Post-excavation stability analysis for cross section critical cross-section (from Stark et al., 2005b).

were developed at the time of the 1996 landslide. Fig.4 presents an aerial view of these housing developments, the subsequent upslope development referred to as the BC Development, and an outline of the slide mass. Only a portion of the housing units in the Knolls and Vista development are shown in Fig.4. A landscape screen fill with a height of at least 22 m above the adjacent natural terrain and a length and width of about 165 and 80 m, respectively, was constructed just downslope of the BC Development (see Fig.4). The volume of the landscape screen when fill placement ceased was approximately 76,600 cubic meters of soil created from the BC Development.

Various geotechnical engineers employed for the BC Development drilled almost 80 borings across the site and none of the borings exceed a depth of about 15 m within the slide limits shown in Fig.4. The slope inclinometers installed in the Visa development after homeowner complaints of damage show the depth of sliding to be 40 to 45 m. Thus, none of the initial borings drilled within the slide limits were deep enough to uncover the problematic serpentinite. As a result, the designers probably were not aware of the weak layer underlying the site although the serpentinite is outcropping at numerous locations across the project site. Nine of the fifteen slope inclinometers installed after the initial report of distress provided useful information on location of the failure surface but the other six are either too shallow or outside the slide limits shown in Fig.4 and do not provide direct information on the location of the actual failure surface. Each of the nine useful inclinometers show only one slide plane at depths ranging from 5 m near the landslide toe to 40 m near the middle of the slide mass. The failure surface in Fig.5 (see dashed line) was developed by connecting the location of shear movement in the inclinometers, following the various material types, and passing the failure surface through the cracks observed at the top of the landslide and the housing distress observed at the landslide toe. Slope stability software was used to search for the critical failure surface between points of known location, e.g., inclinometer location.

The installation of inclinometers to a depth below the failure surface is important to locate the failure surface, however, inclinometers installed at shallower depths should also be considered because the actual failure surface must be below these inclinometers. The presence of nine useful inclinometers and surface observation of movement did not clearly define the entire failure surface. Fig.5 shows a large gap

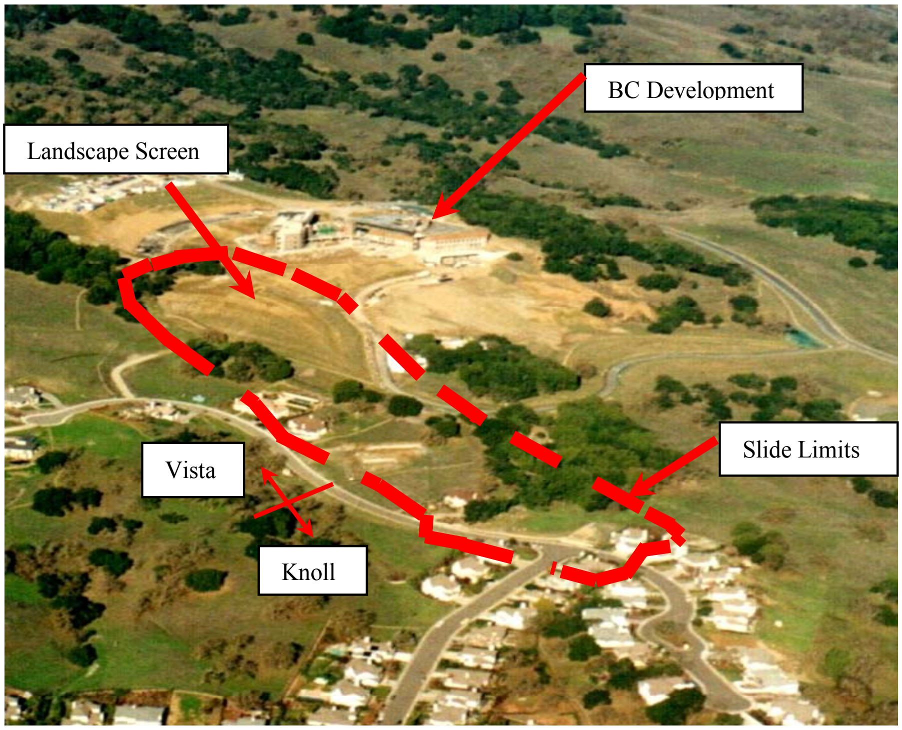

Fig. 4 Aerial view of housing developments, BC Development, and an outline of the slide mass.

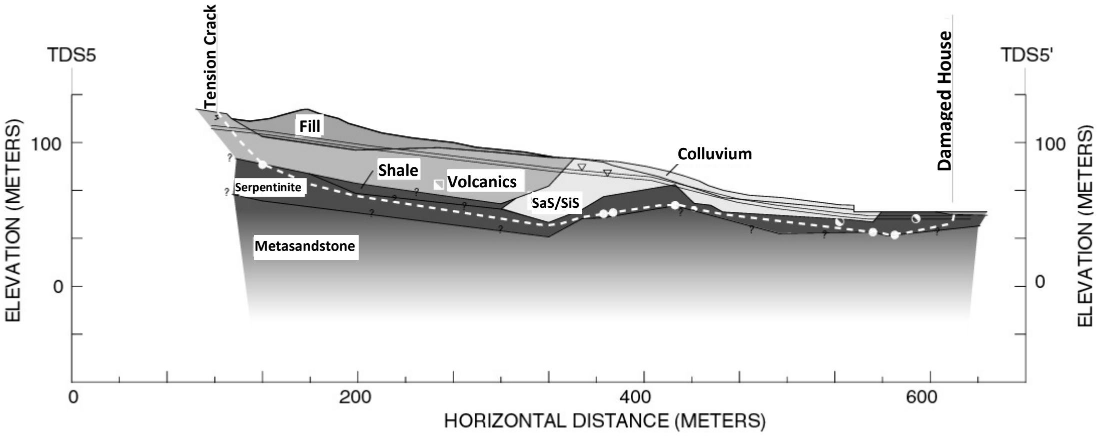

Fig. 5 Cross-section through the western portion of the landslide after surficial grading and placement of the landscape screen.

in inclinometers from under the fill to under the sandstone or a horizontal distance of about 130 m to about 370 m. Limit equilibrium analyses were then used to search for the failure surface in this range of horizontal distance to locate the depth in the serpentinite that yielded the lowest factor of safety. This failure surface (see Fig.5) resulted in the lowest factor of safety and the best combination of the observed movement and inclinometer data. To clarify the location of the failure surface in the serpentinite in this area, the numerical model FLAC was used to confirm the location of high shear stresses within serpentinite to verify the actual failure surface shown in Figure 5.

In summary, a common problem is searching for the critical failure surface during a back-analysis instead of forcing the failure surface to pass through known points of movement, e.g., sheared inclinometers and observed surface features. It is proper to conduct a search for the failure surface that yields the lowest back-calculated friction angle between the inclinometers and the observed surface and subsurface features. This can be accomplished by fixing the failure surface in the slope stability software at the location of the inclinometers and the observed features and allowing the software to search the critical failure surface between these fixed points.

Tension Crack in the Back-Analysis

Landslides usually involve cohesive materials and global failure is usually preceded by formation of a tension crack at the top of the slope which usually delineates the extent of the initial slide mass. Usually the opening of a tension crack(s) is followed by sliding along a well-defined failure surface unless the slope is quickly stabilized (Terzaghi et al., 1996). Once the tension crack is formed, all strength along the failures surface through the crack is lost (Duncan and Wright, 2005). If a tension crack develops, shear resistance is only developed along the length of the slip surface below the tension crack depth at the time of sliding. Thus including the depth of the tension crack in the failure surface is likely to result in an overestimate of the back-calculated shear strength of the problematic soil if there was no tension crack. Materials with a high cohesive strength are sensitive to the assumption of a tension crack in a back-analysis and this sensitivity increases when the tension crack is partially or completely filled with water (Deschamps and Yankey, 2006). Thus a tension crack should be incorporated in the back-analysis.

CONCLUSIONS

The back-analysis of landslides involves many uncertainties. Therefore, experience and judgment should be used in selecting the input parameters and assumptions for the back-analysis. The assumptions made in the back-analysis have different effects than those made in the design and even experienced engineers may make incorrect assumptions for the back-analysis. Based on the study presented herein, the following conclusions are drawn:

- A stress dependent fully softened and residual strength envelope should be used in the back-analysis of first-time and reactivated landslides, respectively.
- Intense/short duration and progressive/ cumulative rainfall thresholds should be established for landslide areas to determine the movements of shallow and deep-seated landslides, respectively.

- All available inclinometer and piezometric data must be used in the back-analysis of landslides to minimize uncertainties.
- Back-analysis procedure suggested during the present study should be used to avoid and/or minimize uncertainties in the back-analysis.

REFERENCES

Bishop, A.W. (1955). "The use of the slip circle in the stability analysis of earth slopes." *Geotechnique,* 5(1), 7-17.

Bromhead, E. N. (1978). "Large landslide in London Clay at Herne Bay, Kent." *Quarterly. J. Engng. Geol. and Hydrology*, 11, 291-304.

Chandler, R. J., (1977). "Back analysis techniques for slope stabilization works: a case record." *Geotechnique,* 27(4), 479-495.

Deschamps, R., and Yankey, G., (2005). "Limitations in the back-analysis of strength from failures." *J. Geotech. Geoenviron. Eng.*, (132)4, 532-536.

Duncan, J. M., and Stark, T. D., (1992). "Soil strengths from back analysis of slope failures." *Proc., Stability and Performance of Slopes and Embankments II*, ASCE, New York, 890-904.

Duncan, J.M., and Wright, S.G., (2005). "Soil Strength and Slope Stability." *John Wiley and Sons,* New York.

Gilbert, R. B., Wright, S. G., and Liedtke, E., (1998). "Uncertainty in back analysis of slopes: Kettleman Hills case history." *J. Geotech. Geoenviron. Eng.*, 124(12), 1167-1176.

Hussain, M., (2010). "Analysis and behavior of preexisting landslides." *Ph.D. Thesis,* University of Illinois at Urbana-Champaign, USA, 308 p.

Hutchinson, J. N., (1983). "Methods of locating slip surfaces in landslides." *Bulletin of the Assoc. of Engng. Geol,* 20, 235-252.

Leroueil, S., and Tavenas, F., (1981). "Pitfalls of back-analysis." *Proc., 10th Int. Conf. on Soil Mechanics and Foundation Engineering*, Balkema, Rotterdam, Netherlands, vol. 1, 185-190.

Skempton, A. W., (1964). "Long term stability of clay slopes." Fourth Rankine Lecture, *Geotechnique*, 14(2), 77-101.

Skempton, A. W., (1970). "First time slides in overconsolidated clays." *Geotechnique,* 20(3), 320-324.

Skempton, A. W., (1985). "Residual strength of clays in landslides, folded strata and the laboratory." *Geotechnique*, 35(1), 3-18.

Spencer, E., (1967). "A method of analysis of the stability of embankments assuming parallel inter-slice forces." *Geotechnique,* 17(1), 11-26.

Stark, T. D., and Eid, H. T., (1994). "Drained residual strength of cohesive soils." *J. Geotech. Geoenviron. Eng.*, 120(5), 856-871.

Stark, T. D., and Eid, H. T., (1997). "Slope stability analyses in stiff fissured clays." *J. Geotech. Geoenviron. Eng.*, 123(4), 335-343.

Stark, T. D., and Eid, H. T., (1998). "Performance of three-dimensional slope stability methods in practice." *J. Geotech. Geoenviron. Eng.*, 124(11), 1049-1060.

Stark, T. D., Choi, H., and McCone, S., (2005a). "Drained shear strength parameters for analysis of landslides." *J. Geotech. Geoenviron. Eng.* 131(5): 575-588.

Stark, T. D., Arellano, W. D., Hillman, R. P., Hughes, R. M., Joyal, N. and Hillebrandt, D., (2005b). "Effect of toe excavation on a deep bedrock landslide." *J. Performance of Constructed Facilities*, 19(3), 244-255.

Tang, W. H., Stark, T. D., and Angulo, M., (1999). "Reliability in back analysis of slope failures." *Soils and Found.*, Japanese Geotechnical Society, 39(5), 73-80.

Terzaghi, K., Peck, R. B., and Mesri, G., (1996). "Soil mechanics in engineering practice." 3rd ed. John Wiley and Sons, New York.

Tiwari, B., Brandon, T. L., Marui, H., and Tuladhar, G. R., (2005). "Comparison of residual shear strengths from back-analysis and ring shear tests of undisturbed and remolded specimens." *J. Geotech. Geoenviron. Eng,* ASCE., 131(9), 1071-1079.

Wieczoreck G. F., (1996). "Landslide triggering mechanisms." *In: Turner AK, Schuster RL (eds) Landslides investigation and mitigation, Special Report 247,* Transportation Research Board, National Research Council, Washington, D.C., 76-90.