Back-analysis of Preexisting Landslides

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ABSTRACT

Drained residual shear strength is applicable to slopes containing preexisting shear surfaces and preexisting landslides. Laboratory ring shear testing by various researchers suggest the possibility of strength gain along preexisting shear surfaces which had previously obtained a residual strength condition. This paper presents back-analysis of two case histories that were used to suggest strength recovery to confirm test results developed herein and explain how the recovered strength may be useful in understanding slope behavior during the rest period and movement reactivation. These back-analyses are important to determine whether or not strength gain occurs and the practical significance, if any, of the strength gain in design of the remedial measures.

INTRODUCTION

Drained residual strength for soils along the shear surface is considered applicable for the analysis of reactivated landslides (Skempton, 1964). Initially a linear Mohr-Coulomb strength envelope for the residual strength, with an effective stress cohesion (c') and residual friction angle (ϕ'_r), was considered appropriate in the analysis of natural and manmade slopes (Skemption, 1964). Subsequently Chandler (1977) and Bromhead (1978) conclude that the relationship between shear stress (τ), and effective normal stress (σ'_n) is nonlinear (stress dependent) for these clays. Stark and Eid (1994) tested 36 natural soils and show that the residual strength envelope is stress dependent and recommend that a stress dependent failure envelope be used in stability analyses to model the effective stress dependent behavior of the residual strength. As a result, a stress dependent residual strength failure envelope in the back-analysis and design of slopes is gaining acceptance in practice (Stark et al., 2005).

D'Appolonia et al. (1967), Angeli et al. (1996), and Gibo et al. (1997) use case histories and laboratory test results to suggest that the strength along a preexisting shear surface in an old landslide can be greater than the residual strength because of strength recovery. Some laboratory studies, such as Ramiah et al. (1973), Angeli et al. (1996 and 2004), Gibo et al. (2002), Stark et al. (2005), Carrubba and Del Fabbro (2008), and Stark and Hussain (2010a and 2010b), also suggest that preexisting shear surfaces may gain strength during a period of no movement and exhibit a strength greater than the residual value upon initiation of movement. The recovered strength increases with rest time and is noticeable at $\sigma'_n \leq 100$ kPa which corresponds to shallow landslides (Gibo et al., 2002, Stark and Hussain, 2010a and 2010b). Back-analysis of two landslides that prior researchers suggest illustrate strength recovery was performed during this study to investigate strength recovery.

BACKGROUND AND FACTOR OF SAFETY THRESHOLDS

Only a well documented and well instrumented case history with few uncertainties can be used to study the possibility of strength recovery along preexisting shear surfaces because the increase in strength is not large. Existing literature on the start and stop of slope creep movement suggests that creep movement starts when factor of safety (FS) reaches a value at or near unity and stops when FS becomes greater than unity. Patton (1984) concludes from the study of the Downie landslide in British Columbia, Canada that when FS decreases due to a rise in groundwater level, slope creep begins when FS is about 1.03. Hutchinson (1988) uses two landslides, Sandnes in Norway and Sandgate in England, to conclude that movements due to slope creep became negligible when FS reaches 1.05. Observations by Patton (1984) and Hutchinson (1988) can be used to conclude that FS ≈ 1.10 may be sufficient to prevent slope creep and slide movement in natural slopes and landslides. These cases may be used to conclude that slope creep movement can start when FS < 1.05.

CASE HISTORIES FOR STRENGTH RECOVERY

Colluvium Slope in West Virginia

The concept of "healing" of a shear surface is described by D'Appolonia et al. (1967) in relation to a colluvial slope failure in Weirton, West Virginia. The slide occurred due to excavation of the toe of an ancient landslide for a steel plant expansion. The rock was weathered and highly jointed to a depth of about 12.2 m below the rock surface. At the time of the toe excavation, the slip surface was established from the presence of slickensides observed in excavations and boreholes. The slope was stable with no discernable landslide movement during the last several decades because no evidence of tension cracks, curved tree trunks, or displaced retaining walls, roadways, foundations or utility lines were observed.

The geometry of the old slide surface, which was also the critical failure surface after the toe excavation, is shown in Figure 1. The natural water content and the index properties show a marked increase near the colluvium-alluvium interface which suggest the soil comprising the old shear surface was of different composition than the overlying colluvium and may be composed of soil derived from the underlying claystones in the alluvium. The material present along the shear surface has LL = 51%, PL = 25%, CF = 55%, and natural water content of 26%.

The groundwater condition in the slide mass was established using data from 29 piezometers that were installed along the critical cross-section (see Figure 1). The drained strength parameters of the shear surface were established using consolidated drained direct shear tests and consolidated-undrained triaxial compression tests with pore pressure measurements on undisturbed and remolded block samples obtained from exposed slickensided surfaces of the slide surface. Peak strength parameters measured by D'Appolonia et al. (1967) from drained direct shear tests on intact/undisturbed slickensided specimens with a best fit linear relationship between shear and effective normal stresses yielded c' = 7.66 kPa and ϕ' = 20º and residual strength parameters of c' = 0 and ϕ'_r = 16º.

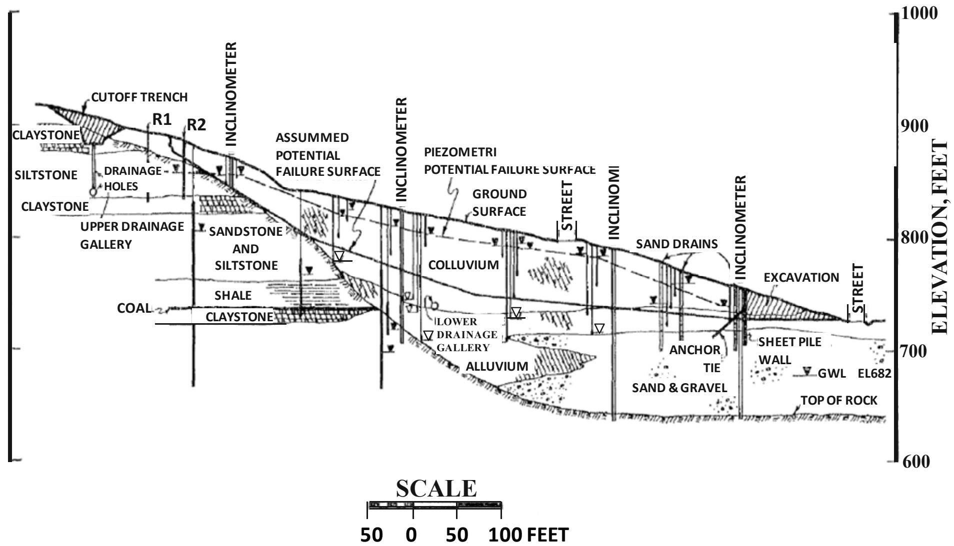

Figure 1. Cross section of slope showing instrumentation, excavation and potential failure surface (from D'Appolonia et al. 1967)

Back-Analysis by D'Appolonia et al. (1967)

D'Appolonia et al. (1967) performed stability analyses using four cross-sections including the one in Figure 1, the drained peak/healed and residual strength parameters, no toe excavation, measured pore-water pressures from 29 piezometers, and two stability methods, Morgenstern-Price (1965) and Ordinary Method of Slices (OMS). The stability analyses performed by D'Appolonia et al. (1967) using the Morgenstern-Price (1965) method and residual strength parameters, i.e., c' = 0 and ϕ'_r = 16º, yielded a FS = 1.03, whereas peak/healed strength parameters, i.e., c' = 7.66 kPa and ϕ' = 20º, yielded FS = 1.51.

Because the shear surface was formed by prior shear movements and no slope movement was occurring at the time of toe excavation, D'Appolonia et al. (1967) conclude that FS must have been greater than 1.03 so the shear strength is probably greater than the residual value. They suggest that if the residual strength was the maximum strength that could be mobilized at the time of the proposed toe excavation, i.e., FS = 1.03, the slope should have shown evidence of at least slope creep and any rise in groundwater in the past should have caused slide movement prior to toe excavation but it did not. In the upper Ohio River Valley region, colluvial slope movement usually occurs in the spring after a wet winter because a rise in groundwater can adversely impact marginally stable slopes. D'Appolonia et al. (1967) report about a 1.52 m rise in groundwater surface in the area during the spring which could have reduced FS to 0.95 and resulted in slide movement but no evidence of such movement was observed. Based on the stable slope and laboratory strength data, D'Appolonia et al. (1967) conclude that the strength on the shear surface prior to toe excavation was greater than the residual value due to healing by desiccation and/or natural cementation.

Current Back-Analyses Results and Discussion

The present back-analysis uses the cross-section shown in Figure 1 and slope stability software XSTABL (Sharma 1996). A specified noncircular failure surface and the phreatic surface shown in Figure 1 were used for the stability analysis. The analysis was performed using the Generalized Limit Equilibrium (GLE) method in XSTABL because it emulates the Morgenstern-Price (1965) method so the results could be compared with the D'Appolonia et al. (1967) results. The stability analysis for slope geometry, slip surface, phreatic surface shown in Figure 1, and linear residual strength parameters of $c' = 0$ and $\phi'_r = 16°$, yielded a FS = 1.03 which is in agreement with that calculated by D'Appolonia et al. (1967) which verified the input parameters (see Table 1). Stability analyses were also performed with a phreatic surface raised 1.52 m and various shear strengths as described below:

- **Stress Dependent Residual Strength Relationship**
  A stress dependent relationship between the residual shear strength of the shear surface material and σ'_n was developed using the empirical correlation by Stark et al. (2005) and updated by Hussain (2010) for LL = 51% and CF = 55%. Stability analyses performed using a stress dependent residual strength failure envelope and the slope geometry, phreatic surface, and slip surface shown in Figure 1, yielded a factor of safety of 1.12 instead of 1.03 computed by D'Appolonia et al. (1967) (see Table 1). Thus, modeling the stress dependent nature of the residual strength is significant in this case. A stability analysis performed using a phreatic surface raised by 1.52 m, yielded a FS = 1.03 instead of 0.95.

- **Stress Dependent Recovered/Healed Strength Relationship**
  D'Appolonia et al. (1967) measured peak, i.e., healed, strengths on undisturbed slickensided specimens in direct shear tests but reported linear peak strength parameters as discussed above. The drained peak shear stresses measured on undisturbed slickensided specimens by D'Appolonia et al. (1967) do not show a linear relationship but are actually stress dependent with $c' = 0$ (see Figure 2). Furthermore, the undisturbed slickensided specimens were tested only at $\sigma'_n \leq 100$ kPa. Thus, a nonlinear peak failure envelope was developed herein using the direct shear test results from D'Appolonia et al. (1967) and is shown in Figure 2. Stability analyses performed using the stress dependent recovered/healed strength failure envelope yielded a FS = 1.22 instead of 1.51 computed by D'Appolonia et

al. (1967). A stability analysis performed using a raised phreatic surface by 1.52 m, yielded a FS = 1.15 instead of 1.39 from D'Appolonia et al. (1967).

A stress dependent recovered/healed strength envelope was also developed using the ring shear strength recovery test results presented by Stark and Hussain (2010a) on silty clay from Esperanza Dam in Ecuador, with a similar LL, i.e., 55% (see Figure 2). This stress dependent recovered strength envelope was used in the stability analysis along with the verified slope geometry, phreatic surface, and slip surface shown in Figure 1 and yielded a FS = 1.20 with reported phreatic surface and a FS = 1.15 with raised phreatic surface by 1.52 m (see Table 1). These values are in agreement with the FS values obtained from a stress dependent failure envelope derived from the direct shear data presented by D'Appolonia et al. (1967) which verifies the ring shear strength recovery test results by Stark and Hussain (2010a).

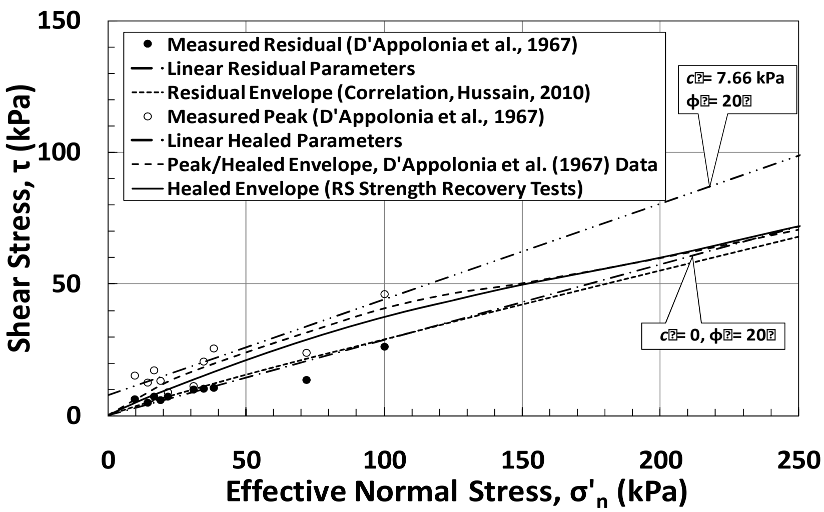

Figure 2. Stress dependent peak, recovered/healed, and residual strength failure envelopes for slip surface material of Weirton landslide, WV

The results of slope stability analyses using a stress dependent residual failure envelope from the empirical correlation suggest that the slope should have been marginally stable at the time of the toe excavation because the calculated FS was near unity (FS = 1.03) which is the same value computed by D'Appolonia et al. (1967) using a linear strength relationship. Thus, with slip surface material at residual strength (FS =1.03) some slope creep should have occurred in the recent past. Furthermore, a rise in the phreatic surface by 1.52 m with the slip surface material and residual strength conditions yielded a FS = 0.95 which should have resulted in slide movement or slope creep but it did not. These results suggest some strength recovery occurred.

Slope stability analyses using stress dependent recovered/healed strength failure envelopes established from direct shear tests by D'Appolonia et al. (1967) and ring shear strength recovery test results by Stark and Hussain (2010a), yielded FS = 1.19 with the phreatic surface shown in Figure 1 and FS = 1.12 with a raised phreatic surface by 1.52 m (see Table 1). Using recovered/healed strength even with a rise in phreatic surface of 1.52 m yielded a FS = 1.12 which was sufficient to prevent a creep movement. Stark and Hussain (2010a and 2010b) observed that the recovered strength reduces to the residual value with a small amount of shear displacement. Thus, the recovered strength may be useful in explaining slope creep behavior or slope stability prior to movement reactivation but it has little or no effect on the stability of the slope after the restart of movement. Because of the shallow depth of this slide, this case history also reinforces the conclusion by Stark and Hussain (2010a and 2010b) that strength recovery is possible in shallow landslides or shallow depths of deep-seated landslides.

Table 1. Summary of stability analysis results for Weirton Landslide, WV

Stress Parameters/Envelope	**FS for Two Phreatic Surfaces**	
	Observed	**Raised by 1.52 m**
D'Appolonia et al. (1967) Residual-Linear (c'=0, ϕ'_r=16º)	1.03	0.95
Residual - Stress dependent (from empirical correlation)	1.12	1.03
D'Appolonia et al. (1967) Peak/Healed - Linear (c' =7.44 kPa, ϕ'_r=20º)	1.51	1.39
Peak/Healed - Stress dependent	1.22	1.15
Recovered/Healed - Stress dependent (from ring shear strength recovery test results by Stark and Hussain (2010a))	1.20	1.12

Alver`a Landslide in Northeastern Italy

Angeli et al. (1996, 1999, and 2004) describe Alver`a landslide in the area of Cortina d'Ampezzo located in northeastern Italy. The landslide is frequently reactivated with several episodes of repeated movements since 1879. The most recent major reactivation occurred in 1945 (Angeli and Sivano, 2004). Subsequently another reactivation occurred in 1966 as a result of flooding that affected northeastern Italy. In 1989 a sophisticated landslide monitoring system was installed consisting of inclinometers, piezometers, and steel wire extensometers.

The main landslide is active and moving at a rate of several centimeters per year (Angeli et al., 1996 and 1999). The main slip surface identified by inclinometers is 18-25 m deep (see Figure 3). Geotechnical laboratory tests show significant differences between samples collected at different depths in the slope and those obtained from the main failure surface in a trial pit excavated in the lower part of the slide (Angeli et al., 1996, 1999, and Angeli and Silvano 2004). Mineralogical analyses performed on samples collected from the main failure surface show the material essentially consists of montmorillonitic clay. Index properties measured by Angeli et al. (1996) and Angeli and Silvano (2004) using main failure surface samples are LL = 69.3-99.1%, PI = 29.6-51.1%, CF = 56-71%, and drained residual friction angle, ϕ'_r = 9º -15.9º from ring shear tests by Angeli et al. (1996).

Angeli et al. (1996) Conclusions regarding Strength Regain in Alver`a Landslide

Angeli et al. (1996, 1999, and 2004) do not provide a back-analysis to reinforce their conclusions of strength gain. The upper and lower piezometric thresholds required to start and stop the slide movement established by Angeli et al.

(1996) are 0.4 and 1.3 m, respectively. Angeli et al. (2004) state "the lower threshold to stop the movement is compatible with the measured residual shear strength in conventional tests." But to restart movement a higher piezometric level was required which assumes no other change to the slope except shear strength and piezometric level. Angeli et al. (1996 and 2004) conclude that the longer the stationary period, the higher the piezometric level required to restart slide movement. These upper and lower piezometric thresholds for start and stop of the main landslide movement, respectively, were used to conclude that strength regain occurred on the preexisting slip surface (Angeli et al., 1996 and 2004). Strength recovery observed during laboratory direct and ring shear tests by Angeli et al. (1996 and 2004) is additional evidence of strength recovery and helps explain the landslide behavior. Angeli et al. (2004) also suggest that the application of the recovered strength to stabilization measures should be approached with caution because the recovered/regained strength is removed and reduced to residual with a small shear displacement.

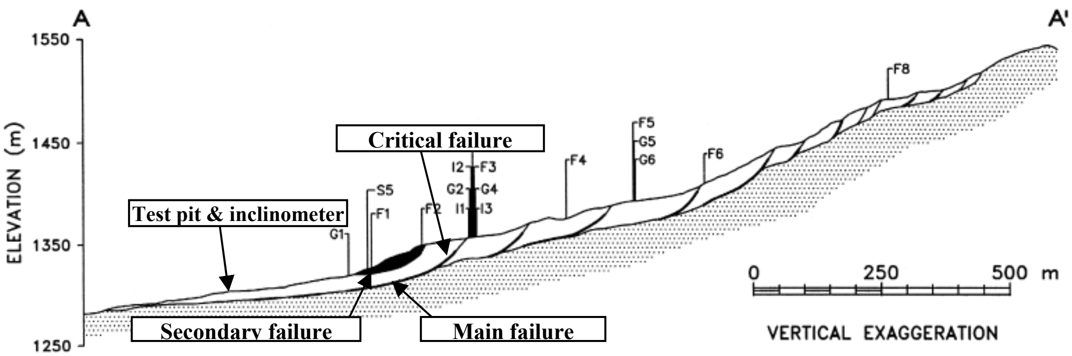

Figure 3. Cross-section of Alver`a landslide, Cortina d'Ampezzo, Italy (from Angeli et al. 1999)

Current Back-Analyses Results and Discussion

Back-analyses were performed herein using the Alver`a landslide cross-section in Figure 3. The well defined main failure surface (see "main failure surface" in Figure 3) in the lower portion of the landslide is considered the critical failure surface and used for the back-analysis. Any movement along this failure surface results in movement of the entire slide. Slope stability software XSTABL (Sharma (1996) and Spencer's (1967) stability method were used to back-calculate the drained friction angle for a factor of safety of unity (FS = 1.0). A noncircular failure surface was specified to match the critical failure surface. The slope material unit weight determined by Angeli and his coworkers, i.e., γ_{sat} =18.73 kN/m^3, was used for the back-analysis. Angeli et al. (1996 and 2004) report that the average depth of the groundwater surface (GWS) is about 0.8 m whereas the upper and lower thresholds to start and stop slope movement are 0.4 and 1.3 m, respectively. Considering the marginally stable landslide at an average GWS condition, i.e., below ground surface, an average ϕ'_{bc} = 13.6º yielded FS = 1.0 (see Table 2).

Stability analyses were also performed for the following five GWS depths: 0, 0.4, 0.8, 1.3, and 1.5 m, to investigate the sensitivity of the stability analyses to changes in GWS. Three types of stress dependent failure envelope were used in the back-analysis for each GWS condition, i.e., linear residual strength, stress dependent residual strength estimated from empirical correlation by Stark et al. (2005) and updated by Hussain (2010), and stress dependent recovered strength failure envelope estimated from ring shear strength recovery test results in Stark and Hussain (2010a) as discussed below:

- **Stress Dependent Residual Strength Relationship**
  Stark and Eid (1994 and 1997) and Stark et al. (2005) recommend using a stress dependent residual strength failure envelope in stability analyses. Therefore a stress dependent residual strength failure envelope was developed using the empirical correlation by Stark et al. (2005) and updated by Hussain (2010) for LL = 83% and CF > 50%. The stress dependent residual strength failure envelope shown in Figure 4 results in FS ≈ 1.0 for the average GWS condition of 0.8 m below ground surface and the other input parameters described above. The stress dependent residual strength failure envelope shown in Figure 4 was also used for the other four GWS conditions to calculate the corresponding FS values (see results in Table 2). These stability analyses yielded FS = 0.94 when GWS is at ground surface and FS = 1.05 when GWS is 1.5 m below the ground surface which is in agreement with the results computed using a linear residual failure envelope because the linear envelope is in agreement with the stress dependent envelope shown in Figure 4.

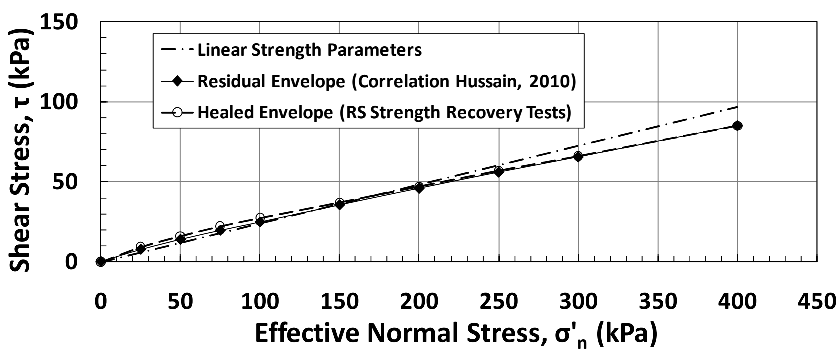

Figure 4. Stress dependent residual and recovered/healed strength failure envelopes for main slip surface material of Alver`a landslide, northeastern Italy

- **Stress Dependent Recovered/Healed Strength Relationship**
  Because the average value of liquid limit for the Alver`a landslide slip surface material is about 83% with CF > 50% and it contains montmorillonite, the main slip surface material can be compared with Madisette clay which was used to perform ring shear strength recovery tests by Stark and Hussain (2010a). Considering a stationary/rest period of less than 30 days as evidenced from Angeli et al. (1996 and 2004) and ring shear strength recovery test results for Madisette clay by Stark and Hussain (2010a), a stress dependent recovered strength failure envelope was developed (see Figure 4). This stress dependent recovered strength failure envelope was used in the back-analyses of all five GWS conditions. The results of the back-analyses using a stress dependent recovered strength failure envelope are shown in Table 2 and yield FS = 1.01 when GWS is at ground surface and FS = 1.12 when GWS is 1.5 m below the ground surface. The computed increase in FS as a result of strength

recovery/healing during the rest/stationary period may explain a higher piezometric level being required to restart slide movement.

A FS of about 1.05 is assumed to be the transition from no slope creep to slope creep based on Hutchinson (1988). Table 2 shows slope movement should stop with GWS 1.5 m below ground surface and stress dependent residual strength failure envelope because FS = 1.05. Table 2 also shows FS = 1.04 or ~1.05 with a stress dependent recovered/healed strength failure envelope and GWS of 0.4 m below ground surface, i.e., upper piezometric level required to restart movement. Thus, stability analyses performed using GWS at 1.5 m and residual strength may explain cessation of slide movement as does GWS at 0.4 m and recovered/healed strength because both FS values are about 1.05. Thus it is possible that some healing occurred depending on the actual pore pressures level in the field.

Table 2. Summary of stability analysis results for Alver`a landslide, Italy

Stress Parameters/Envelope	FS for Five Groundwater Surface Depths below Ground Surface				
	0.0 m	**0.4 m**	**0.8 m**	**1.3 m**	**1.5 m**
Residual - Linear ($c' = 0$, $\phi'_r = 13.6°$)	0.94	0.98	1.00	1.05	1.06
Residual - Stress dependent (from empirical correlation)	0.94	0.98	1.00	1.03	1.05
Recovered/Healed - Stress dependent (from ring shear strength recovery test results by Stark and Hussain (2010a))	1.01	1.04	1.07	1.10	1.12

SUMMARY AND CONCLUSIONS

Back-analyses of two case histories suggest strength gain may occur on a preexisting shear surface at shallow depths. The back-analyses illustrate the importance of using a stress dependent residual failure envelope for analyses and confirm laboratory testing that indicates strength recovery can occur at low effective normal stresses. Because the healed/recovered strength is removed after a small shear displacement and reduced to the residual value, it may be useful in explaining slope creep behavior or slope stability prior to reactivation but has no impact on slope stability after restart of movement. Therefore, the drained residual shear strength, not the recovered strength, should be relied upon in remedial design and design of slopes.

REFERENCES

Angeli, M-G., Gasparetto, P., Menotti, R. M., Pasuto, A., and Silvano, S., (1996). “A visco-plastic model for slope analysis applied to a mudslide in Cortina d’Ampezzo, Italy.” *Quarterly J. Engrg. Geol.* 29: 233-240.

Angeli,M-G., Gasparetto,P., and Bromhead,E., (2004). “Strength-regain mechanisms in intermittently moving slides.” *Proc. 9th Int. Symp. Landslides*, Rio de Janeiro, Brazil, Vol. 1: 689-696.

Angeli, M-G., Pasuto, A., and Silvano, S., (1999). “Towards the definition of slope instability in the Alver`a mudslide (Cortina d’Ampezzo, Italy).” *Geomorphology*, 30: 201-211.

Angeli, M-G., and Silvano, S., (2004). "Two cases of mudslides in different geological and climatic environments." *Proc. Int. Workshop, "Occurrence and Mechanics of Flow-Like Landslides in Natural Slopes and Earthfills",* Sorrento, Italy: 209-216.

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

Carrubba, P., and Del Fabbro, M., (2008). "Laboratory Investigation on Reactivated Residual Strength." *J. Geotech. Geoenviron. Eng.*, 134(3): 302-315.

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

D'Appolonia, E., Alperstein, R., and D'Appolonia, D. J., (1967). "Behavior of a colluvial slope." *J. Soil Mech. Found. Div.*, 93(4): 447-473.

Gibo, S., Inoue, H., Zhou, I., and Nakamura, S., (1997). "Stability analysis in the large moved landslide." *Science Bulletin of Faculty of Agriculture. University of Ryukyus* , No. 44 (1997/12), In Japanese with abstract in English: 275-280.

Gibo, S., Egashira, K., Ohtsubo, M., and Nakamura, S., (2002). "Strength recovery from residual state in reactivated landslides." *Geotechnique*, 52(9): 683–686.

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

Hutchinson, J. N., (1988). "Morphological and geotechnical parameters of landslides in relation to geology and hydrology." *Proc. 5th Int. Symp. Landslides,* Lausanne, Switzerland, 1: 3-35.

Morgenstern, N.R. and Price, V.E., (1965). "The Analysis of the Stability of General Slip Surfaces." *Geotechnique*, 15(1): 79-93.

Patton, F. D., (1984). "Climate, groundwater pressures and stability analyses of landslides." *Proc. 4th Intl. Symp. Landslides,* Toronto, Canada, 3: 43-59.

Ramiah, B.K., Purushothamaraj, P., and Tavane, N.G., (1973). "Thixotropic effects on residual strength of remoulded clays." *Indian Geotech. J.* 3(3): 189-197.

Sharma, S. K. (1996). "*XSTABL:* An integrated slope stability analysis program for personal computers." *Reference manual, Version 5*, Interactive Software Designs, Inc., Moscow, Idaho. 213 p

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

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, No. 1, pp. 11-26.

Stark, T. D., and Eid, H. T., (1994). "Drained residual strength of cohesive soils." *J. Geotechnical and Geoenvironmental Engrg.*, 120(5): 856-871.

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

Stark, T. D., and Hussain, M., (2010a). "Shear strength in preexisting landslides." *J. Geotechnical and Geoenvironmental Engrg.,* 136(7), 957-962.

Stark, T. D., and Hussain, M., (2010b). "Drained residual strength for landslides." *GeoFlorida2010,* GSP No. 199, ASCE, West Palm Beach/FL: 3217-3226.