Mechanisms of Strength Loss in Stiff Clays[a]

Discussion by Robert W. Day,[3] Member, ASCE

The authors' paper on the failure of the San Luis Dam, in California, was very interesting. The authors have concluded that failure of the upstream slope of the dam occurred as a result of deformations that developed when the reservoir was filled and emptied, eventually reducing the shearing resistance of the slope wash (a lean to fat clay) to the residual value. The authors also state that with a sufficient number of cycles of filling and emptying, the cumulative displacement could eventually become large enough [approximately 10 in. (25 cm)] to reduce the shearing resistance of the slope wash to its residual value. Given this magnitude of movement, should not the inclinometers at the crest of the dam have recorded the slope movement with each cycle of filling and lowering of the reservoir?

Fig. 2 shows a cross section of the dam and the location of the slide plane determined from inclinometer readings. The slide plane is about 600 ft (180 m) long. Of this length, about 200 ft (60 m) of the slide plane is in the compacted clay core, 200 ft (60 m) of the slide plane is in the slope wash, and the final 200 ft (60 m) of the slide plane is in a compacted clayey-gravel fill located near the toe of the dam. Thus, when considering the material along the slide plane, the slope wash is confined on both sides by compacted clayey material. If failure of the San Luis Dam requires the slope wash to be in a residual state [i.e. 10 in. (25 cm) of cumulative displacement], then should not the compacted clayey fill on both sides of the slope wash also have to approach a residual state? Using the authors' residual values for the clayey compacted materials ($c' = 0$; $\phi' = 15 - 20°$) as well as those for the slope wash ($c' = 0$; $\phi' = 15°$) would produce a factor of safety well below 1.0 for the rapid-drawdown condition. Is it possible that the authors have used too high values for the compacted clayey soils and too low values (i.e. residual values) for the slope wash when calculating a factor of safety of 1.0 for the rapid drawdown of the reservoir?

Discussion by Fernando H. Tinoco,[4] Member, ASCE

The authors provided a very interesting case history in a highly plastic clay that is a topic of great interest to soil engineers. The paper touches directly or indirectly on several key issues, some of which are discussed herein.

The effective stress peak failure envelope on samples of stiff clay is nonlinear, as shown by Bishop et al. (1965) on undisturbed specimens of London clay and by Lambe et al. (1981) on Amuay clay. Bishop et al. (1971) showed that highly preconsolidated clay loses strength after the peak, with increasing displacement along the plane of sliding, and reaches residual strength when displacement is very large.

Lambe et al. (1981) indicated that the development and increase of perched water above the Amuay fat clay triggers slides by: (1) Decreasing effective stress and strength; (2) decreasing the strength line due to deformations in the Amuay clay from cyclic filling and emptying of the oil in the reservoir; and (3) increasing shear stress.

The Amuay clay presents a very complex stress history that includes sedimentation, erosion, desiccation, cliff forming, reservoir construction, development of perched water, cyclic loading, cracking, and large deformations. Lambe et al. (1981) presented different nonlinear strength envelopes for the following stages in the stress history of the Amuay clay: (1) Cliff formed; (2) perched water (intact slope); (3) perched water (cracked slope); and (4) large deformations (residual). The highest strength corresponds to cliff formation, and the strength line decreases sequentially in stages 2–4. The strength lines were obtained from results of testing undisturbed samples at different locations with stress history compatible to the appropriate stage (1, 2, 3, or 4). Lambe et al. also indicated that residual strength depends upon the plasticity of the clay, and plotted a nonlinear strength envelope for a plasticity index (PI) equal to 20% and a linear strength envelope for PI = 50%.

Stability analyses of five slides in Amuay clay by Lambe et al. (1981) showed that for the slides to occur the mobilized shear strength of the clay had to be larger than its residual strength. They concluded that the shear strength to be selected depends upon the stage of the clay just prior to the landslide; the pore pressures they used corresponded to pressures existing prior to a landslide. They measured high pore pressures within the Amuay clay after a landslide and discussed the possibility that these developed by deformations occurring during a landslide.

Mechanisms of strength loss in stiff clay may be identified as: (1) Reduction or elimination of high negative pore pressures; (2) generation of internal swelling force; and (3) continuous deformation after shear stress equals strength of element. Strength loss due to elimination of high negative pore pressure is affected by reduction in effective stress, but the strength envelope is not altered. The generation of internal swelling force, caused by soaking, lowers the strength envelope; its magnitude may be large enough to reduce shear strength to almost zero by inducing change in the structure of the clay for very low total normal stress. Tinoco (1981) proposed that the internal swelling force be included in the stability analysis of cuttings in London clay. He provided a simple method to take into account the strength loss due to displacement on the sliding surface.

The stresses plotted on Fig. 10 represent the loading conditions of: (1) End of construction; (2) reservoir full; and (3) drawdown. The authors did not explain whether they represent a point element or average values on the sliding surface; and the shear stresses are drawn above the residual-strength line, contrary to the results of back-analysis.

Research to determine the magnitude of pore pressure at the start of a landslide developed by displacements of deformations occurring along a potential sliding surface is necessary to validate the results of back-analyses.

Appendix. References

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[3]Chf. Engr., American Geotech., 5764 Pacific Ctr. Blvd., Ste. 112, San Diego, CA 92121.

[4]Visiting Prof., Dept. of Civ. and Envir. Engrg., Florida Int. Univ., University Park, Miami, FL 33199.

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Closure by Timothy D. Stark,[5] Associate Member, ASCE, and J. Michael Duncan,[6] Fellow, ASCE

The writers appreciate the comments of both Day and Tinoco.

Day suggests that a residual strength condition also may have developed in the compacted clay core (zone 1) and the compacted clayey-gravel fill (zone 3) because a large part of the slide plane passes through these materials (Fig. 2). The clay fraction of the slope wash is 60%, the zone 3 material has a clay fraction of 25%, and the clay fraction of the zone 1 material is 40%. The clay fraction is defined as the percentage finer than 0.002 mm by weight. Skempton (1985) concluded that if the clay fraction is greater than or equal to 50% the clay particles will undergo a "sliding" shear behavior, and the residual strength is likely to be considerably less than the peak strength. If the clay fraction is less than 25%, the soil will undergo a "turbulent" shear condition, and the residual strength will not be significantly less than the peak strength. If the clay fraction lies between 25% and 50%, the soil will undergo a "transitional" shear behavior.

The strength tests showed that the residual strength of the slope wash is considerably less than the peak strength, which is in good agreement with Skempton's conclusions. However, the clay fraction of the zone 3 material is only 25%, therefore it will probably exhibit little or no strength loss during reservoir operations. Since the clay fraction of the zone 1 material is 40%, some strength loss might be expected. However, the finite-element stress analysis showed that the highest shear stresses occurred in the slope wash and thus the deformations in the zone 1 material due to the reservoir operations were probably small.

Thus the writers agree with Day in principle that it would be appropriate to reduce the strength of the zone 1 and zone 3 materials to the residual value. However, the reduction in strength from the peak to the residual would be small in comparison with what appears to have occurred in the slope wash.

The slope inclinometers near station 135 + 00 were installed after the slide occurred and thus measurements of the slope movement during the annual reservoir operations were not recorded. However, the cracks observed in the crest roadway between 1978 and 1981 were an indication of the movements that led to the loss of strength and the occurrence of the slide.

The stresses plotted in Fig. 10 represent the average shear and normal stresses in the slope wash during the reservoir operations. The stress state for the reservoir drawdown-condition plots above the residual-strength envelope, and the stress state for the reservoir full condition plots on the residual envelope.

[5]Asst. Prof. of Civ. Engrg., 2217 Newmark Civ. Engrg. Lab MC-250, Univ. of Illinois at Urbana-Champaign, 205 N. Mathews Ave., Urbana, IL 61801.
[6]Univ. Distinguished Prof., Virginia Polytech. Inst. and State Univ., Blacksburg, VA 24061.

Appendix. Reference

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Bearing Capacity of Auger-Cast Piles in Sand[a]

Discussion by Joram M. Amir,[2] Fellow, ASCE

The technique of auger-cast piling is applicable to many foundation situations, and the author made an important contribution to the understanding of the problems involved in the construction stage of these piles.

The following comments may shed some more light on both the analytical methods presented by the author and on his conclusions.

Presumably, all tests reported made use of hydraulic jacks to measure the loads. This procedure is inherently inaccurate, often giving an error of 20% (*Canadian* 1985). A typical error of, say, 10% is therefore reasonable to expect.

In the determination of the pile capacity, the author uses the 10% rule and, where inapplicable, Chin's (1970) method. Both methods are arbitrary, lack a theoretical basis, and may give results widely apart [e.g., 319 tons for Chin's method versus 211 tons according to the 10% rule (Fig. 3)]. For a mean value of 265 tons this gives a standard deviation of 54 tons, or a coefficient of variation of 20%. Thus, the combined error in the ultimate loads reported may be on the order of 30%.

The use of a bilinear variant of Chin's method to separate the shaft friction from the total load is rather problematic: In the writer's experience, the points on the initial part of the load-settlement curve show too much scatter w in Chin's coordinates to enable the drawing of any straight line. Assuming that such a line can be drawn, it is uncertain whether it will yield the correct shaft friction; theoretically it can even produce friction values that are higher than the total load at 10% settlement.

Disregarding this possibility, and assuming that this method does provide the value of the shaft friction, the coefficient of variation must be on the same order of that for the total load (30%).

The large errors involved in the shaft friction values are reflected in the β-values suggested by the author (Fig. 4). According to these values, for a vertical effective stress increasing linearly with depth the total friction on a 40 ft pile ($\beta = 0.5$) is somewhat lower than that of a 28.4 ft pile ($\beta = 1$). If, as assumed in (7), the vertical effective stress has a limiting value at 6–10 pile diameters, the result for all pile lengths is even more paradoxical: The longer the pile, the less total skin friction!

The author calculates the ultimate point resistance by subtracting the skin friction from the total ultimate load. This is mathematically correct, but one must not forget that in this case the variance of the result is a sum of the variances of the total and skin friction capacities. For the example given in

[a]February, 1991, Vol. 117, No. 2, by William J. Neely (Paper 25516).
[2]Pres., Found. Control Inst., P.O. Box 1425, Ramat-Hasharon 47100, Israel.