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Residual Strength and Landslides in Clay and Shale

By Herbert L. Noble[1]

Until recently, the shear strength of overconsolidated clay, claystones, and shales in landslides was a mystery. Ordinary shear tests on samples of such materials from slides gave such high shear strengths as to indicate that the slides were impossible. In 1964, Skempton (8) showed that the strength remaining in laboratory samples after large shearing displacements corresponded closely with the computed strength from slides. A sample tested in a direct shear device shows increasing resistance with increasing displacement up to some peak value. With further displacement the resistance becomes smaller, until a nearly constant resistance persists regardless of the magnitude of displacement. This value of resistance is termed "residual strength."

This paper has two objectives. One is to present comparisons of laboratory measured residual strengths with those backfigured from three major landslides in the western United States. The second objective is to demonstrate how residual strength applies in practice to correcting active landslides by drainage, unloading, and buttressing.

Testing

The residual shear tests reported here were performed using a technique described by Kenney (6). This method, which applies to very fine grained soils, is a modification of the direct shear test. In this test, a slurry of remolded clay or shale is smeared on a porous stone in a layer about 0.25 in. (6.4 mm) thick and then consolidated under a vertical load for 18 hr to 24 hr. Following

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[1]Chf. Engrg., Materials & Substructures, Div. of URS/The Ken R. White Co., Denver, Colo.

consolidation, a shearing displacement is applied at the rate of about 0.01 in. (0.25 mm) per hr. After about 0.1-in. (2.5-mm) displacement, the shearing load is reversed in direction. About 10 to 15 reversals of shear are required before the shearing load falls to a constant value.

In performing the shear test with reversals, the writer used two 1-7/8-in. (48-mm) diam stones held in bronze retainers and mounted in a lever-type consolidation test frame with a horizontal roller bearing. With vertical load on the specimen, the upper stone was displaced laterally by a motor-driven unit with a gear reduction to give the desired low speed. A proving ring measured the shear resistance. Direction of shearing displacement was reversed without removing the vertical pressures.

The effect of repeated reversals is to orient the clay crystals parallel to the failure plane. However, a drawback to the ordinary direct shear test is that the area being sheared is constantly changing. A torsional or ring-shear device overcomes this disadvantage. Bishop, et al. (3) have described such a ring-shear device and have given the results of a large number of residual shear tests on both undisturbed and remolded samples from overconsolidated clays. Their data confirm Kenney's contention that the results obtained using slurried samples are practically the same as those obtained using undisturbed samples after large displacements. A great advantage to using slurried specimens is that undisturbed samples are not required; thus, test samples may be obtained from a Standard Penetration spoon, or by coring or any other method that does not mix one soil with another. It is also possible to use samples that have been in storage for many years, as was done by Bishop, et al. (3) in the case of the Panama Canal-Cucuracha Slide.

Bishop considers the direct shear test with reversals to be an empirical test. However, this paper presents evidence to show that, even though empirical, the test is still accurate enough for practical engineering applications.

Stability Analysis

In analyzing the stability of the three slides reported herein, the writer used the "General Slope Stability Analysis" developed by Bell (1). Bell's method permits the analysis of potential slip surfaces of any physically reasonable shape and soil layers of varying nature may be considered. The ground surface may be irregular and there can be a tension crack up-slope. Bell's method "may be regarded as an extension of the friction circle method for homogeneous sections previously described by Taylor." The Bell method gives safety factors that are slightly higher than given by the Bishop method of slices and substantially higher than given by the traditional Swedish slip circle.

Landslides are rarely approximated by circular slip planes. They are usually long in relation to their depth, and planar rather than circular. Since the depth of sliding was known in the three cases reported herein, the configuration was fixed and it was not necessary to search for so-called critical planes. Pore pressures reflected the elevation of the water table above the sliding planes.

Pipe Organ Slide

The massive Pipe Organ landslide is located on a mountain-side above the Beaverhead Valley in southwestern Montana and includes about 9,000,000 cu

yd (6.9×10^6 m^3) of material. Several cased holes in the slide have sheared off at depths ranging from 110 ft (33.5 m) to 150 ft (45.8 m). The overall length of the slide is about 2,000 ft (610 m) and its vertical height about 300 ft (92 m) (see Fig. 1).

TABLE 1.—Pipe Organ Slide

Data (1)	Case 1 (2)	Case 2 (3)	Case 3 (4)	Case 4 (5)
Shear strength				
Tangent of friction	Variable	0.18	0.12	0.24
Cohesion, in pounds per square foot (kilonewtons per square meter)	0	0	630 (30.2)	300 (14.4)
Computed Factors of Safety				
During sliding	1.00	1.00	1.03	1.48
With drainage by gravity wells	1.12	1.17	1.14	—
With key and buttress	1.34	1.40	1.36	—

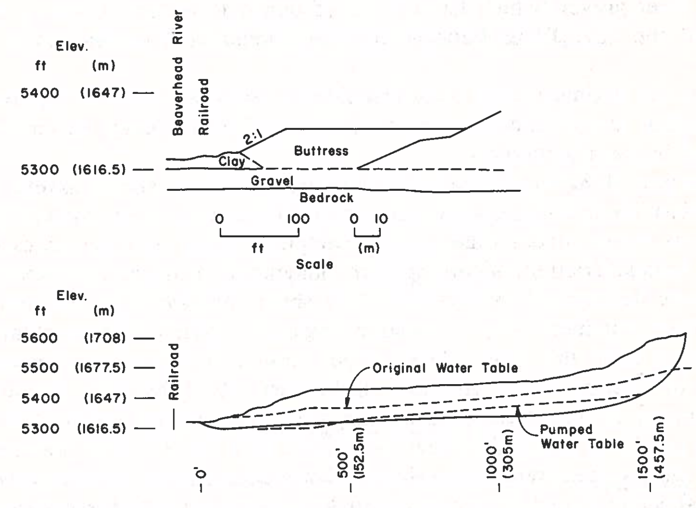


FIG. 1.—Cross Section, Pipe Organ Slide

The modern slide originated in a larger ancient slide that extends to the top of the mountain. The modern slide was triggered by a cut made at the toe for change in alinement of a railroad. For about 1 yr this slide moved at an average rate of 2 in. (51 mm) per week as the cracks and scarps progressed uphill from the toe.

The average depth to the water table during active sliding was about 55 ft (16.8 m), and it has been shown that this water was perched on the impermeable base of the shear zone. The progressive nature of the slide formed a series of pockets that prevented flow of ground water from one pocket to another.

The seat of the slide is in very stiff, green clay in which a heterogeneous rubble of rhyolite, tuff, and large basalt boulders float. This Tertiary colluvium is underlain by bedrock of the Mission Limestone formation.

A total of 40 borings, wells, and test pits were made in this landslide. Laboratory tests were performed on small undisturbed samples and large-scale tests were made in situ with a specially devised shear box enclosing a specimen 1 ft (0.3 m) square. A number of index tests, mineralogic, hydrologic, and ground-water pumping tests and chemical analysis have been performed.

Stabilization.—Movement of the Pipe Organ slide was stopped in 1967 by installing five deep vertical wells which were pumped continuously. These wells yielded about 30,000 gal (110 m^2) per day. Removal of this water lowered the water table in the slide by 20 ft to 30 ft (6.1 m to 9.2 m) and increased the computed safety factor to about 1.12. Since installation of these wells, there has been virtually no movement of the slide material. In 1970, the wells were converted to gravity drainage by drilling them deeper into pervious, nonsaturated limestone bedrock lying below the slide material. The system is working satisfactorily and the water table has declined even more.

Railroad tracks are located beside the toe of the slide and railroad officials consider the 1.12 safety factor obtained by drainage to be inadequate. Consequently, additional stabilization is planned in the form of a keyway extending into the river gravel (which lies on top of quartzite bedrock) with a buttress on top of the key. TThis buttress and key should increase the safety factor to 1.34.

The gravity drainage wells were installed for $65,000. Beyond this there will be a nominal annual expense for inspection and maintanance to ensure their continued proper functioning.

Comparison of Shear Strength Assumptions.—Table 1 shows safety factors computed for the Pipe Organ slide, using three different sets of strength properties. The factors shown under Case 1 were computed using varying values for the angle of residual friction, according to the magnitude of normal pressure. These values were determined by the laboratory shear reversal tests. Factors under Case 2 were obtained using a constant angle of residual friction obtained by backfiguring from the slide. Those under Case 3 are based on an assumed 7° angle of friction and a cohesion of 630 psf (30.2 kN/m^2). These values were selected by backfiguring from the slide. The values under Case 4 are from the results of the in situ-shear tests made on a 1-ft (0.3-m) square section.

Fig. 2 shows the strength envelopes obtained from the laboratory reversal residual shear tests and from the 1-ft (0.3-m) square in situ direct shear tests. Using the lowest in situ strength envelope gave a computed safety factor of 1.48 during sliding, a value almost 50% too high, despite the fact that the clays at locations chosen for testing were highly sheared and contained numerous, nearly horizontal slickensides.

The reason for using the large size shear box for in situ tests was to include as many such natural defects as possible. The maximum displacement possible with the in situ shear device was about 1 in. (25 mm). To prevent build-up

of excess pore pressure, each in situ test was continued for about 1 week. The direction of shear was reversed only three or four times; with the excessive time required, it was not practical to continue making reversals until a true residual value was obtained. At the time of the in situ tests, it was not yet known that the friction angle of the slide material decreased with increasing normal stress. In the tests, relatively light normal loads were used. It would be very difficult to build such a large area testing device which would not bind under a normal force of 15,000 lb (66,800 N). The large in situ device, as used in these tests, was not very effective for obtaining residual shear strength.

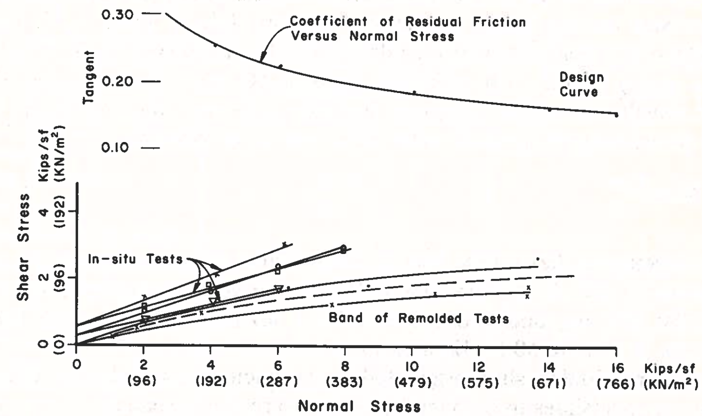


FIG. 2.—Shear Strength, Pipe Organ Shear Tests

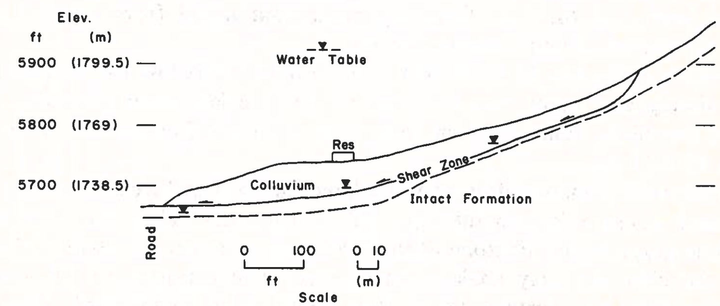


FIG. 3.—Cross Section, Golden Slide

The reason for the lower safety factor after drawdown by pumping under Case 1 (variable angle of friction) is that most of the shear zone is more than 100 ft (30 m) below the surface. At this depth, the effective normal stresses exceed 10,000 psf (480 kN/m^2) and at such high pressures, the tangent of the angle of friction drops to low values. Therefore, the increase in normal pressure (due to lowered water table) does not contribute much additional strength in the shear zone. In a shallow slide, however, the opposite would be true.

The fact that the laboratory values of residual shear strength (shown by the "design curve") when used with the known slide plane give a calculated safety factor of 1.0 is evidence of the validity of the residual shear concept. It is an extremely remote possibility that a curved envelope would fit by mere chance.

Case 3 (with cohesion and friction) approximates the situation fairly well, but it is hard to believe that there would actually be a remaining cohesive component of strength after displacement of 12 ft (3.66 m). Several authorities agree that a cohesive intercept of zero is the best assumption in an active slide (4,5,8,9).

Case 2, that with a constant angle of friction, gives nonconservative results because it overestimates the strength deep in the slide (where the normal stresses are high) and thus gives too great an effect for drawdown of the water table.

The present safety factor of the Pipe Organ slide is about 1.15, increased to this value as a result of improved drawdown by the gravity wells. The limiting value of safety factor with complete removal of water from the slide is 1.36. It is not feasible to obtain complete drainage however, since hydraulic barriers prevent interchange of ground water from one zone to another.

Golden Slide

The Golden slide lies on the side of South Table Mountain above Clear Creek near Denver, Colo. It contains about 500,000 cu yd (380,000 m^3), and is about 900 ft (280 m) long and about 260 ft (79.3 m) high (see Fig. 3). The depth of the slide is 40 ft to 50 ft (12 m to 15 m).

The modern Golden slide originated on an ancient landslide and is only one of numerous landslides in the vicinity. A cut to realine a road at the toe triggered the recent slide, and abnormally heavy rains probably also played a part. The rate of slide movement reached a maximum of 1 in. (25 mm) per day, both horizontally and vertically. The total displacement (for the modern slide) has been at least 3 ft (0.9 m). Cracking progressed uphill from the toe rapidly during the period of maximum movement.

The water table was 25 ft to 40 ft (7.62 m to 12 m) below the surface during active sliding and there was evidence of an artesian water condition in the head of the slide. Pumping from four drain wells has lowered the water table approx 10 ft (3 m).

The colluvium consists of overconsolidated fat clays and weathered clay shale, with basalt rocks afloat in the clay. The substratum below the slide is very hard, blue-gray, clayey siltstone of the Cretaceous Denver Formation, that from which the clay and clay shale were derived. The rim-rock of the mountain above the slide is a Tertiary basalt flow about 50 ft (15 m) thick. This basalt forms a reservoir that collects and conducts water into the slide zone.

Eleven borings have been made in this slide, and a number of index tests, X-ray diffraction, residual shear, triaxial tests, and unconfined compression tests were performed.

Stabilization.—In 1957, while cracks at the head of the slide were still progressing uphill, it was decided to unload the head of the slide as it was defined at that time. A cut about 20 ft (6 m) deep involving about 100,000 cu yd (77,000 m^3) was made. Material taken from the top of the slide was placed at the toe against a retaining wall which penetrated into underlying bedrock of the Denver Formation. At the time, it was decided that ground-water drainage was not practicable.

Movement continued, however, and cracks appeared beyond the limits of excavation uphill. This, plus the subsequent discovery that the slide was not as deep as originally believed, invalidated the original stability analysis which had led to the recommendation for unloading the head. With benefit of "hind-sight," it now appears that the safety factor was improved by only about 1% by unloading.

Drainage was tried next. In 1960, horizontal drains as long as 350 ft (107 m) were installed in the slide and vertical wells were drilled and pumped. This

TABLE 2.—Golden Slide

Data (1)	Case 1, using residual shear strengths (2)	Case 2, using triaxial test results (3)
Shear Strength		
Tangent of friction	0.26	0.38
Cohesion	0	0
Computed factors of safety		
During sliding	1.01	1.47
With excavation from head of slide	1.02	1.49
With excavation and drainage	1.11	1.62

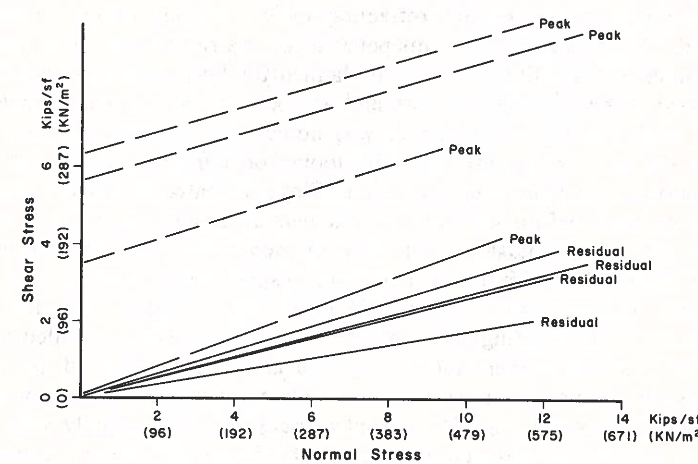


FIG. 4.—Shear Strength, Golden Slide

drainage was successful and, as far as can be determined, there has been no subsequent movement. Pumping rates vary seasonally, but the average total is about 250 gal (0.95 m^3) per day for the group of four wells.

In 1972, the writer became involved in reevaluating this slide for a highway project. By that time, the role of residual strength was gaining recognition. The writer benefitted from this and also from more accurate data on the depth of the shearing zone. This latter was provided by shearing-off of three of the drain wells at a depth of about 45 ft (13.7 m) below the surface.

Shear Strength.—Laboratory residual shear values were used with the Bell method to evaluate the stability of the Golden slide (see Fig. 4). The analysis showed a safety factor of 1.01 for conditions which existed during sliding and 1.02 for the slope after unloading but while cracks were still progressing uphill. The present safety factor is calculated to be 1.11, reflecting the improvement due to drainage. (The preceding values are shown under Case 1, Table 2.) There are no new cracks and the old ones have been obliterated. Judging from the absence of cracking and the general appearance of the slide area, the present calculated value is reasonable.

Peak strengths in triaxial compression (determined by others) give the safety factors shown under Case 2 of Table 2. These factors are about 50% too high even when the lowest individual test result is used in the computations.

Thrall 3

An incipient earth movement, attributed in part to the release of tectonic stress, the Thrall 3 slide occurred on the flank of the Manastash anticline which runs for 60 miles across central Washington. During excavation for a highway cut, a shearing zone began to develop at a depth of 130 ft (39.6 m) and roughly 1,500,000 cu yd (1,450,000 m^3) of earth were near failure. The incipient slide has a height of about 270 ft (82.4 m) and a length of about 960 ft (293 m).

Geology in the vicinity of the Manastash anticline is complex, with several hills and knobs on the north flank reflecting ancient folding. It was impossible to avoid them all and the highway cuts penetrated, at a right angle, what appears to be a recumbent fold. Buttresses were planned for both outboard backslopes of the divided highway and were installed as soon as the cut was completed. At several stages during excavation it was noted that horizontal offsets of 1 in. to 2 in. (25 mm to 51 mm) were developing on curving contacts between various members of the claystone formation. Slope indicator wells were installed at the top of slope before the cut was excavated, and others were installed later at the toe and at midslope. The rate of movement indicated was about the same in most wells with a total maximum displacement of about 4 in. (100 mm). Offsets, indicator movements, and small cracks uphill were the only manifestation of a developing slide. After the buttresses were installed at the toe, slope indicator movement rates decreased appreciably, but did not cease [see Fig. 5(*a*)]. A plot of slope indicator movement versus increasing depth of excavation is shown in Fig. 5(*b*). Displacement increased rapidly with each increment of excavation as the cut neared 150 ft (45.8m), indicating approaching failure. The development of a distinct shear zone at depths of about 130 ft (39.6 m) appears in Fig. 5(*c*).

A depth of about 120 ft (36.6 m) of badly fractured basalt and sandstone, dipping at constantly changing angles, overlies the overconsolidated green claystone of the Ellensburg Formation and the contact is convoluted by tectonic movement. The seat of the developing slide is in the claystone, but the shear zone passes upward to the surface through the fractured basalt and sandstone. The water table is far below the shearing zone.

Testing Program.—Ten deep borings were made, and samples were collected

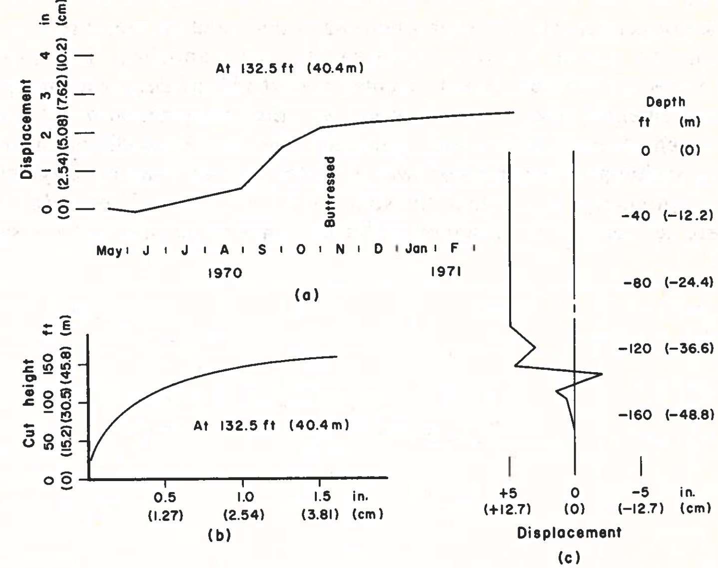
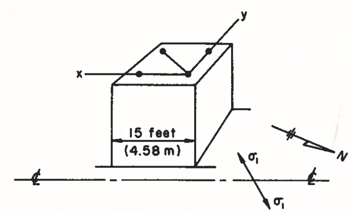


FIG. 5.—Displacements, Thrall 3

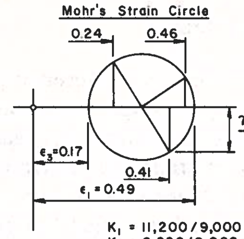


E = 2 x 10^6 psf = Young's Mod (95.8×10^6 N/m^2)
ν = 0.25 = Poisson's Ratio
ϵ = strain
σ = stress
γ = shear strain
K = State of Stress

Line	Original (Length – cm)	Change (Length – cm)	Unit Strain
y–y	305	1.25	.0041
x–x	305	0.74	.0024
x–y	432	1.98	.0046

$$\gamma_{xy} = 2\epsilon_B - \epsilon_x - \epsilon_y = 2(0.46) - 0.24 - 0.41 = 0.27\%$$

Mohr's Strain Circle



$$\sigma_1 = \frac{E}{1-\nu^2}(\epsilon_1 + \nu\epsilon_3) = \frac{2\times10^6}{1-0.25^2}(.49+.25(.17)) = 11{,}200 \text{ psf } (536 \text{ KN/m}^2)$$

$$\sigma_3 = \frac{E}{1-\nu^2}(\epsilon_3 + \nu\epsilon_1) = \frac{2\times10^6}{1-0.25^2}(.17+.25(.49)) = 6{,}200 \text{ psf } (297 \text{ KN/m}^2)$$

$$\sigma_2 = \text{overburden} = 9{,}000 \text{ psf } (431 \text{ KN/m}^2)$$

K_1 = 11,200/9,000 = 1.25 say 1.3
K_2 = 6,200/9,000 = 0.7

FIG. 6.—Strain Island and Computation of Residual Stresses and State of Stress, K Thrall 3

for ordinary shear tests, residual shear tests, mineralogical analysis, and laboratory index tests.

On this project, an elaborate program was conducted to measure the state of stress in the ground at other points on the same anticline. This program included excavation of full-scale test cuts to 80 ft (24 m) deep with numerous field measurements. One unique concept was the instrumentation of "strain islands," each 15 ft (4.58 m) square, in the claystone. A modification of the overcoring technique, this method was resorted to because the claystone is not competent enough to maintain the state of stress in a small core. Reference points were set below the surface of each island in a rectangular pattern similar

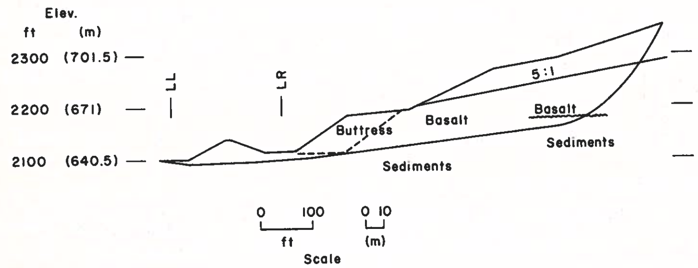


FIG. 7.—Cross Section, Thrall 3 Slide

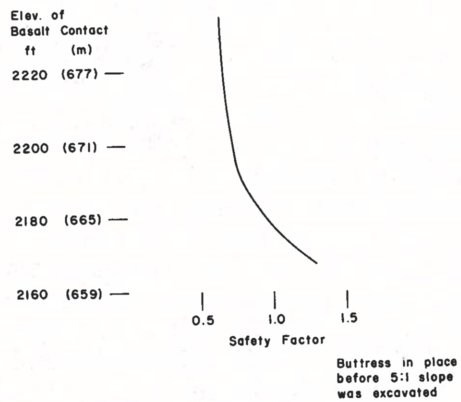


FIG. 8.—Safety Factors, Thrall 3

to the standard 45° strain rosette. Distances between the reference points were measured using an optical plummet and reading an engineer's scale taped to a level rod. Changes in reference point spacings as excavation proceeded around the strain island indicated the strains caused by relief of horizontal residual stresses. These strains were then analyzed by Mohr's strain circle to obtain principal strains. The modulus of elasticity required to convert strain to stress was obtained from pressure meter tests in bore holes drilled vertically and horizontally. The computations are summarized by Fig. 6.

It was discovered that the state of stress (ratio of horizontal to vertical stress)

is 0.7 in the direction parallel to the axis of the anticline and 1.3 at right angles to the axis. This information was used in a finite element analysis to predict the stresses that would develop in the ground when even greater cuts were made.

At Thrall 3 the major principal horizontal stress is probably about 1.3 times the vertical, but is oriented differently because the axis of the small fold does not parallel the Manastash anticline. Shearing strain and displacements calculated from the finite element analysis were valuable in judging whether the cumulative

TABLE 3.—Thrall 3 Slide

Data (1)	Case 1 (2)	Case 2 (3)	Case 3 (4)
Shear strength			
Tangent of friction	0.11 and 0.70	0.23	0.36
Cohesion	0	0	0
Number of layers	2	1	1
Computed factor of safety			
Buttress in place	1.05-1.10	1.07	1.66
Buttress in place and head and crown sloped at 5:1	1.16-1.20	1.37	2.14
Horizontal bench 200 ft wide	1.06-1.11	1.11	1.74

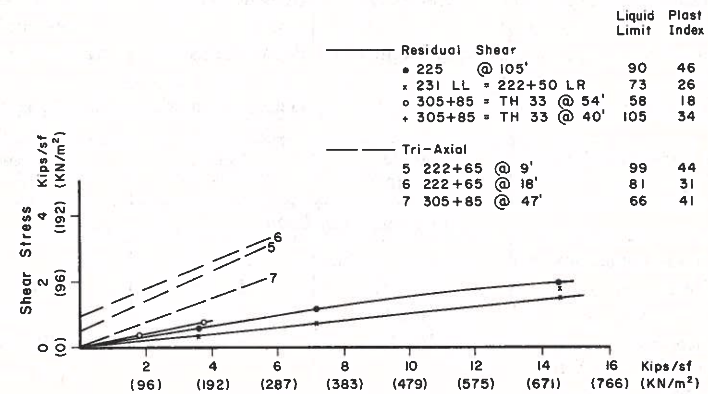


FIG. 9.—Shear Tests, Thrall 3

movements indicated by slope indicators were reaching dangerous levels.

Stabilization.—The buttress installed immediately after excavation bears on claystone at the toe of the slope since there is no competent stratum within a reasonable depth. After buttressing, the indicators showed that the slide plane shifted to a level below the buttress. To avoid the development of a major slide, the head and crown were then unloaded by excavating about 40 ft (12 m) deep on a cut sloped at 5:1 as shown in Fig. 7. This cut virtually removed the top of the hill. The cut slope has now been stable for about 1 yr, although

TABLE 4.—Summary of Data

Data (1)	Pipe organ slide (2)	Golden slide (3)	Thrall 3 slide (4)
(a) Field Data			
Average maximum depth, in feet (meters)	110-150 (33.6-45.8)	40-50 (12-15)	130 (39.6)
Depth—how determined	Borings sheared	Wells sheared	Slope indicators
Volume, in cubic yards (cubic meters)	9,000,000 (6,900,000)	500,000 (380,000)	1,500,000 (1,150,000)
Height, in feet (meters)	300 (91)	255 (77.8)	270 (82.4)
Length, in feet (meters)	2,000 (610)	900 (270)	960 (293)
H/L	0.15	0.28	0.28
Origin	Ancient slide	Ancient slide	Folding
Trigger	Toe undercut	Toe undercut	Toe undercut
Rate of movement	2 in./week (50 mm/week)	1 in./day (25 mm/day)	1/2 in./month (13 mm/month)
Magnitude of movement	12 ft (3.66 m)	3 ft (0.90 m)	2 in.-4 in. (50 mm-100 mm)
Progressive failure	Yes	Yes	No
Ground-water depth, in feet (meters)	55 (16.8)	25 (7.6)	Dry
Slide material, primary	Green clay	Brown clay and shale	Green and black claystone
Slide materials, secondary	Tuff, ryholite rubble	Basalt rocks	Fractured basalt
Parent formation	Tertiary	Denver formation clay shale	Ellensburg formation claystone
Single or composite layers	Single	Single	Composite
Number of borings	40	11	10
(b) Laboratory Data			
Typical index properties			
Liquid limit	70-100	50-75	58-105
Plasticity index	40-77	25-45	18-46
Shrinkage limit	14-16	11-14	10—20
Clay Mineralogy by X-ray			
Montmorillonite, as a percentage	75-85	40-50	65-75
Cation	Ca	Ca/Na	Ca/Na
Quartz, as a percentage	2-5	25-35	3-10
Tangent of residual friction, primary layer	0.16 to 0.30	0.18 0.26 0.31	0.10 0.11 0.16 0.20
Envelope curved	Yes	No	Slight
Tangent of friction secondary layer	—	—	0.70

TABLE 4.—*Continued*

Data (1)	Pipe organ slide (2)	Golden slide (3)	Thrall 3 slide (4)
Correction			
Stabilized by	Drainage wells (buttress proposed)	Drainage wells, toe wall, unloading	Buttress toe, unload head
Pumping rate, in gallons per day (cubic decimeters per day)	30,000 (114,000)	250 (950)	None
Horizontal drainage, in gallons per day (cubic decimeters per day)	400 (1,500)	200 (760)	—
Present safety factor	1.15	1.11	1.16
Tangent of friction	(See Fig. 2)	0.26	0.11 and 0.70

the slope indicator readings show that some creep is still occurring. The computed safety factor with the buttress and 5:1 cut slope is now between 1.16 and 1.20, the actual value depending on the real average elevation of the basalt contact (which is not horizontal).

Comparison of Shear Strength Assumptions.—Fig. 8 shows how the safety factor, with the buttress, varies with the erratic elevation of basalt. The most reasonable estimate for the average elevation of basalt is 2,176 ft (664 m).

Safety factors for one and two-layer sections are given in Table 3. The safety factor for Case 2 is assumed to be 1.07; when backfigured, this results in a tangent of residual friction of 0.23 with the buttress in-place but before the 5:1 slope was excavated. The value of 1.07 is midway in the range of safety factors computed for Case 1, and is probably close to the actual safety factor.

Triaxial compression tests were performed on undisturbed samples of the green claystone, and the results are shown in Fig. 9 together with residual shear envelopes. The lowest triaxial envelope was obtained from a sample which already contained a dipping shear plane. The tangent of friction for this sample was 0.36. Using this value in a stability analysis with the buttress in-place yields a safety factor of 1.66 (Case 3), compared to safety factors of 1.07 backfigured for Case 2 and 1.05 to 1.10 for Case 1. (If tangents of 0.36 and 0.70 were used in a two-layer section, the computed safety factor would be more than 2.0.)

It is difficult to obtain large displacements with triaxial equipment and, even though the one sample contained an existing plane of weakness, the results are much too high.

With the 5:1 slope and buttress the safety factor of 1.16-1.20 for the two-layer section (compared to 1.37 for the one-layer section) occurs because about 40 ft (12 m) of stronger basalt has been removed, but the average backfigured one-layer value cannot take this into account. If the safety factor actually were 1.37 at this time, it is doubtful that there would be any creep.

Another possibility is that the safety factor varies throughout the shear zone. This could occur with release of tectonic stress during excavation and a resultant concentration of high shearing stress at the toe of the cut. Several aspects of this subject are considered by Bishop (2) and Peck (7).

Summary and Conclusions

Table 4 summarizes the data collected from all three slides, including the size, depth, rate of movement, index properties, clay mineralogy, and the tangent of residual friction. The cohesive intercept is zero in all cases. The failure envelopes approximate a straight line for the Golden and Thrall 3 slides, while that for the Pipe Organ slide is curved.

Overconsolidated montmorillonitic clays and shales with similar properties cover vast portions of the American West. Dozens of other landslides have been observed in such formations, but have not been thoroughly investigated. From the degree of slope and presence or absence of ground water, it appears that the strength in most of these slides must be approaching the residual according to stability calculations using these approximate data.

The following conclusions are made:

1. Measured residual shear strengths give safety factors that are compatible with observed behavior of the three landslides.

2. A satisfactory method to determine residual shear strength is by testing slurried samples in a direct shear device. Several reversals of the shearing load are required before the residual value is reached.

3. In situ tests using a 1-sq ft shear device in fissured clay gave results that were 50% too high when compared with the strength backfigured from the slide. It is difficult to cause enough displacement to reach the residual level of strength.

Peak strengths from triaxial tests are also too high as compared to the strength backfigured from slides. Even though a sample already contains a failure plane, an ordinary triaxial device may not permit enough displacement for a sample to attain the residual strength.

4. Drainage is a simple and effective way to stabilize a landslide in clay or shale. If ground water is present drainage should be attempted even though the prospect of removing a great deal of water may seem remote.

5. Buttresses at the toe of slides are effective only if there is a competent foundation stratum into which they can be keyed.

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