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Design of Cut Slopes in Overconsolidated Clays

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The design of clay slopes for long-term stability is discussed in relation to (a) mode of failure, (b) soil test methods, (c) method of analysis, (d) selection of parameters, and (e) time dependency of stability. Failures of slopes in overconsolidated clays can be evaluated based on drained soil parameters with little groundwater drawdown below the cut face. The time dependency of slope failure is hypothesized to be a function of the Terzaghi hydrodynamic lag model. These failure criteria are evaluated for a case history of a cut slope in the western Allegheny Plateau region of New York State.

New York State cut-slope design procedures were previously published in the Highway Research Record series (1). It is the intent of this paper to discuss further the selection of design parameters for cut-slope design and to demonstrate, by a case history, the numerical methods used in evaluating a cut-slope failure. It is further intended to present a model to estimate the time to failure for unloading cut-slope soil conditions in the hope that other investigators will pursue the idea to develop a reliable estimation method.

DESIGN PROCESS

Seepage Force

Long-term cut-slope stability has been shown to be directly dependent on seepage force and, therefore, the ultimate groundwater level within the soils in the cut. As shown in Figure 1, the water table immediately after excavation is usually observed at the surface of the new cut slope. The free-water surface will usually then drop slowly to a stable zone at a variable depth below the new cut surface. This drawdown occurs rapidly for sand slopes but is very slow for clay systems. Although a number of investigators have attempted to mathematically model the rate and shape of the groundwater drawdown curves from cuts, none of these mathematical models has proved useful in correctly predicting the time or rate of drawdown of preconsolidated clays or clayey tills because the drawdown is a function of recharge, which has not been adequately modeled.

A number of open-well piezometers were installed in cut slopes before and after excavation to improve predictions for clayey tills and layered silt and clay systems. The records have only been kept for eight years, but no measurable drawdown was observed (2-5 ft maximum). It is therefore generally assumed that the seepage forces that tend to cause failure are based on a free-water surface no more than about 2 ft below the surface of the cut for most clay or clayey till slopes in New York.

Figure 1. Seepage conditions in cuts.

Shear Strength

The stability of the clay cut slope also depends directly on the shear strength of the clay. The shear strength of a clay, however, is not a constant. The in-place shear strength of an overconsolidated clay is a direct function of the prior load (P_p) if the clay is subjected to additional loading (fills) or short-term unloading conditions (temporary cuts). However, if the clay is subjected to long-term unloading conditions (permanent cuts), the strength of the clay no longer depends on the prior loading.

Figure 2 shows data from the case history. The strength of the clay under undrained (short-term) and drained (long-term) conditions is modeled from laboratory tests by use of the Mohr envelope. At the time the cut is made, the average shearing strength (at an average depth of failure plane 50 ft below the original ground surface) is 1850 psf (see Figure 3). After an indefinite time, the strength (at an average depth of failure plane 20 ft below the cut surface) is 610 psf, about one-third of the original strength.

This loss in shearing strength is generally attributed to the reduction in negative pore pressure after the excavation is made. This loss in strength

Figure 2. Shear strength for case history.

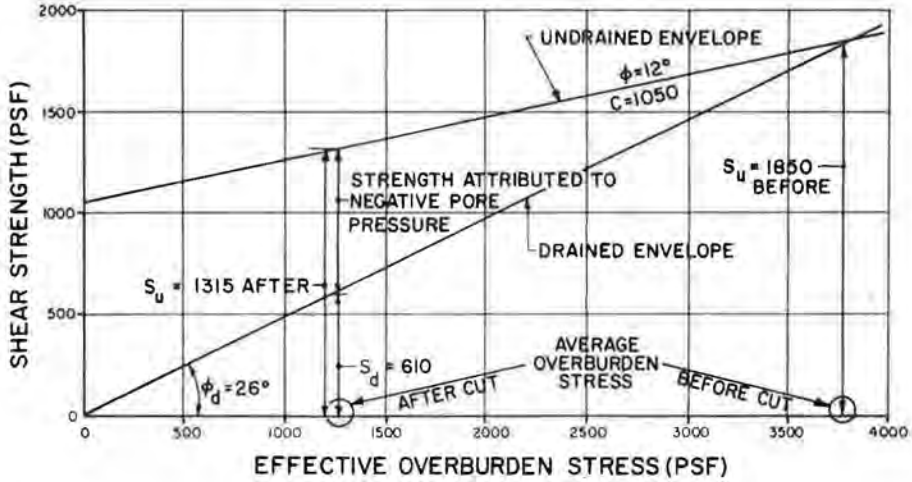
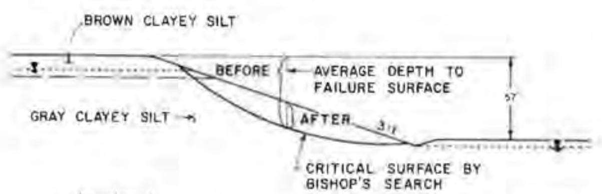


Figure 3. Cross section of slope failure.



has been observed to be a time-dependent function and appears to be related to the rate of dissipation of negative pore pressure.

Factor of Safety

The overall factor of safety of a slope can be evaluated separately for the time-dependent (a) decrease in seepage force (lowered groundwater) and (b) decrease in strength. Figure 4 schematically shows the two time-dependent functions and the corresponding factor of safety versus time function.

Since the water table does not draw down appreciably in most clay cuts in New York, the time dependency of the strength loss then usually controls the time to failure of a cut slope in clay soils.

Time-Dependent Strength Loss

Observations indicate that a large number of slopes more than 30 ft high have failed between 5 and 10 years after construction. Most of these failures occurred in clay or clayey till soils that had medium plasticity [with a plasticity index (PI) between 10 and 15] and a coefficient of consolidation (C_v) of approximately 0.1 ft²/day. Field observations indicate that the average depth of the failure surface through most of the clay systems was between 10 and 20 ft and the average depth was about 15 ft.

If it is assumed that the drained strength is reached at 90 percent consolidation, a simple calculation, using Terzaghi's theory of hydrodynamic lag (4), indicates that time for release of stress could be expressed as follows:

$$t = [(h)^2 \times T_{90}]/(C_v \times 365) = [(15)^2 \times 0.848]/[(0.1) \times 365] \quad (1)$$

where

h = drainage length = 15 ft,
T_{90} = time factor for 90 percent consolidation = 0.848, and
C_v = 0.1 ft²/day.

This simple calculation indicates that the time for stress release to occur, assuming the simple hydrodynamic lag model, would be in the order of 5.2 years. This seems to agree with the observed failures from the field.

An alternative to the simple hydrodynamic lag model is discussed below. It is hypothesized that the excess pore pressure (it would be negative in a cut) would be zero at the surface of the cut slope and it would also be zero at a point below the slope described by a one vertical on one horizontal line down from the top of the cut slope. Using this model, the estimated time to failure would be approximately 5.8 years:

$$t = (h^2\ T_{II}/C_v) = (15)^2\ 0.933/(0.1 \times 365) \quad (2)$$

where h = average distance from the slope face to the depth of the maximum negative pore pressure = 15 ft and T_{II} = time factor for 90 percent consolidation for "Case II" pore pressure distribution (4) = 0.933. Both of these models indicate values within the broad range of the average conditions.

CASE HISTORY

Site Conditions

Figure 3 shows a section through the middle of a cut failure on a section of NY-60 between Cassadaga and Fredonia in the glaciated Alleghany Plateau region of New York State. The soil in this cut was predominantly a silty clay with occasional silt layers and occasional stones. The soil has been overconsolidated, and the water table was near the surface. Construction of the road created two nearly identical cut slopes on opposite sides of the road.

History

The cut was made in 1958 at a slope angle of approximately 1:3 (vertical to horizontal) for a total vertical height of approximately 60 ft. Nine years later (1967) a drop at the top of the slope was noted with cracks occurring near the top of the slope on the east side. An investigation was started in 1969, and samples were taken through the natural materials from the top of the cut. By mid-1970, a drop of more than 15 ft had occurred on both east and west slopes.

Soil Parameters

Some test results are shown in Figure 5. Shear-strength test results are summarized in Figure 2. The samples were driven samples so that the undrained strength would be lower than the true strength, but the disturbance should have little effect on the drained tests.

One consolidated drained test and two consolidated undrained tests with pore pressure measurements were made. (Recent consolidated undrained testing with pore pressure measurements indicates that a good-quality test will yield drained friction angles almost identical with those obtained from good-quality long-term drained tests.) The test values obtained on the soil from the site were compared with the statewide curve of drained friction angle (ϕ_d) versus PI (see Figure 6). By correlating the moisture content and PI data, an average soil PI of 9.5 was obtained. At a PI of 9.5 (Figure 6), an average ϕ_d of 26° was obtained, and this value was used in this analysis. Prior studies have shown a residual cohesion of 50-200 psf for unfailed slopes at the time of failure. An estimate of 50 psf is used here. These values of strength parameters were assumed to best represent the soils on this site.

Stability Analyis

An approximate analysis of undrained failure was done. With an average shear strength of 1850 psf obtained from consolidated undrained strength tests (Figure 2), the factor of safety was $\geq$1.6 by stability charts, and therefore the undrained conditions do not represent the field performance.

Drained parameters were used in a series of modified Bishop stability analyses run on the cut slope. The groundwater table was estimated to be approximately 10 ft (from an observation well) below the surface at the top and to average about 3 ft below the slope face. The most critical failure circle gave a factor of safety of 0.96 at a drained angle of 26° and a cohesion of 50 psf. The most critical surface is shown in Figure 3.

An "infinite slope" analysis (2) was checked for $\phi_d = 26°$ $(C = 0)$ for an h_w/H (height of water surface above the failure plane divided by depth to the failure plane) of 17 ft ÷ 20 ft = 0.85 (see Figure 7). The factor of safety for a 1:3 slope is 0.87 from Figure 7. If the slope is flattened to 1:4, the factor of safety from Figure 7 is raised to 1.16, which is acceptable.

Factor of Safety

It is common for the factor of safety to be in the

Figure 4. Change of factor of safety with time.

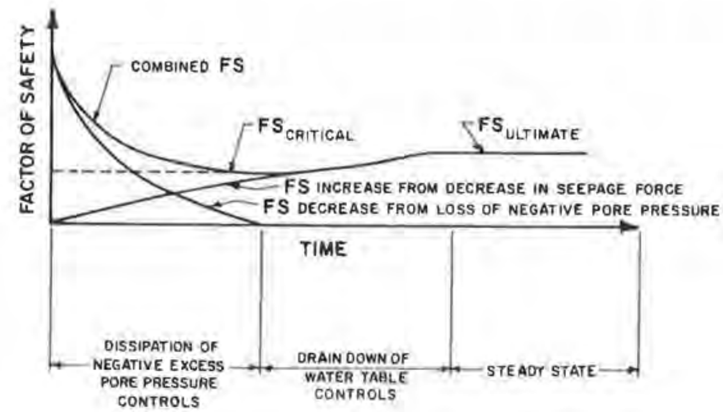


Figure 5. Subsoil conditions.

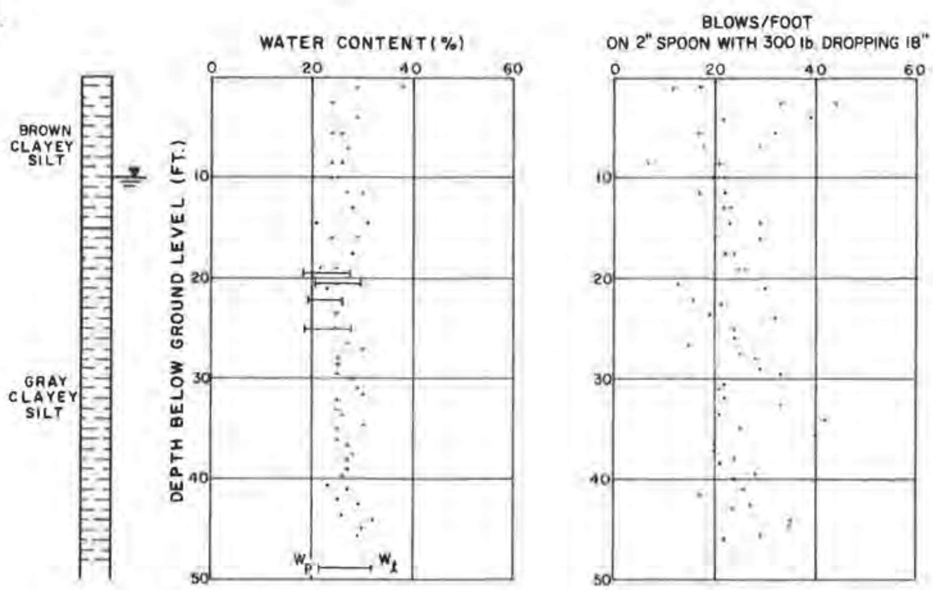
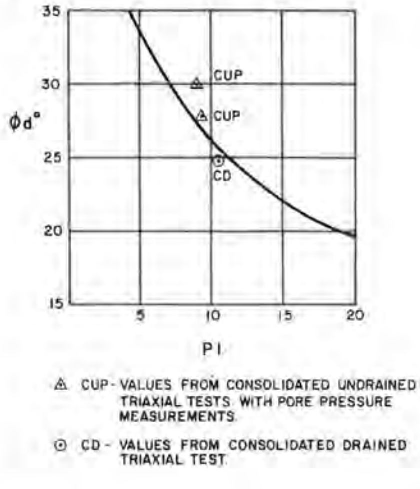


Figure 6. Drained friction angle versus PI for clay and till soils back-figured from failure.



range of 0.85-0.95 when back-figured from a cut-slope failure where movement in excess of 1 ft has occurred. This is assumed to be a result of a short-term higher water table, which produces a factor of safety appreciably below 1 and allows the system to overcome inertia and end restraint and to start movement. If the factor of safety was exactly 1, the soil would not start moving. After an initial movement of 1-3 ft occurs, the driving force of the soil is reduced slightly and the seepage force is also usually reduced as a result of the soil movement. Therefore, the landslide activity will cease and not recur until conditions worsen--for example, until another period of heavy rains, spring snow melt, or toe excavation.

Stabilization

The correction of the stabilization program was simple. The cut slope was trying to reach a stable equilibrium position of approximately a 1:4 inclination. With the help of the maintenance crews digging out the ditches and regrading the upper part of the slope, the slope was reworked to appoximately a 1:4 inclination with little further movement. This was predicted by additional stability analyses that used a 1:4 slope and the same drained parameters and high water conditions determined by modified Bishop analysis.

Estimate of Time to Failure

For this project, the C_v is 0.1 ft²/day and the depth to the failure plane is estimated to be an average of 20 ft below the slope surface (from the most critical surface). The time to failure was estimated to be 9.3 years by using the hydrodynamic lag model:

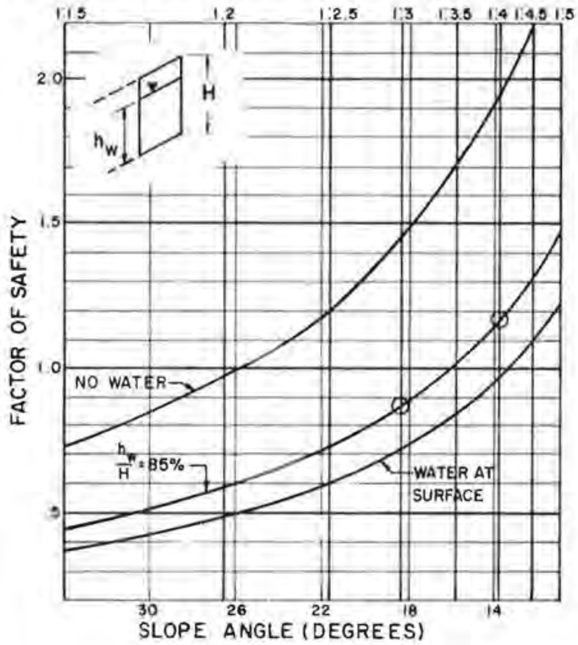
$$t_{90} = [T_{90} \times (h)^2]/(C_v \times 365) = 0.848\,(20)^2/0.1 \times 365 = 9.3 \tag{3}$$

This agrees very well with the approximate 9 years to first observed major movement.

PROPOSED GUIDELINES

For interpretive purposes, the use of the following guidelines is proposed:

Figure 7. Infinite slope analysis for ϕ_d = 26°.



1. If a clay slope with a factor of safety less than 1 for drained parameters has not failed within the time estimated by the simple hydrodynamic lag model, it will not fail.
2. If the performance of the slope is critical (if, for example, there is a hospital at the top of the slope), stabilization treatment is required if the factor of safety is less than 1.1 based on drained parameters and high water table.

CONCLUSIONS

Based on the research reported in this paper, the following conclusions can be drawn:

1. Long-term safety of overconsolidated clay slopes can be estimated by using the drained friction angle (plus a low residual cohesion) and seepage forces based on a high water table, determined by modified Bishop or a similar analysis.
2. Stress release in overconsolidated clays is a time-dependent function, and an estimate can be made from the assumption of the simple hydrodynamic lag model and a depth of failure plane based on the geometry of the most critical section.

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