Considerations for Stabilizing Drained Failures of Slopes

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Three drained slope failures bordering I-37 in San Antonio, Texas, occurred during the fall and early winter of 1986. One failure resulted from adverse natural seepage conditions, whereas the other two failures resulted from a broken water main and a disrupted surface drainage system. The latter two failures were stabilized by eliminating the water entering the slopes. Several procedures were considered for stabilization of the first failure. Use of horizontal drains was judged to be the most effective in raising the long-term factor of safety for the slope. The design of horizontal drainage systems is reviewed and sample calculations are provided to illustrate that many combinations of drain length and spacing can result in approximately equal gains in factor of safety with time and that longer drains that can achieve higher long-term factors of safety can be only marginally more expensive than systems with shorter drains. A second finding is that there is a limit on the benefit of increasing drain length to increase long-term factors of safety.

The Texas State Department of Highways and Public Transportation has experienced many slope stability failures in San Antonio over the last 20 yr. These failures have been repaired using several methods that have met with varying degrees of success. This paper discusses three recent failures of cut slopes, the factors causing them, and the methods considered to stabilize the slopes. In each case, the failure was the result of unfavorable seepage conditions, but the source of the seepage causing failure differed for each case. The latter part of this paper discusses the design of horizontal drainage systems for stabilizing slopes with adverse seepage conditions.

HISTORY

The slope stability failures discussed in this paper border I-37 in San Antonio, Texas (Figure 1). An 8,000-ft section of I-37 from south of Fair Avenue to south of Hot Wells Boulevard was placed in a depressed (i.e., a cut) section so crosstown streets could remain at grade. This section of I-37 was initially constructed without frontage roads. In 1986, plans were being prepared for the construction of frontage roads in this section. Because of the history of slope stability failures, the author began working with personnel of the San Antonio District in September 1986 to assess potential slope stability problems that might be associated with construction of the frontage roads.

The soil in this area is poor; it is a highly fissured, expansive clay with vertically oriented seams of coarser grained materials. The plasticity indices for the clay are in the 60–70 range. Problems with the expansiveness of the clay have plagued this highway since it was built. Expansion of the clay after the cut was opened affected ride quality to such a degree that leveling-up of the highway was required before it was first opened to traffic. The severity of the problem is evidenced by the damage many houses in the surrounding neighborhoods have suffered from the highly expansive clays. In addition to the vertical movements found under the pavement, significant horizontal swelling movements have been observed at overpass abutments and embankments. The swelling of soil at the abutments has been aggravated whenever the joint between the abutment and the concrete riprap below the overpass has opened. This opening allows any water entering the expansion joint at the abutment to enter the soil below the abutment.

In 1986, rainfall in San Antonio was greater than average. The long-term yearly average rainfall for San Antonio is about 27.5 in. per year. The rainfall in 1986 was approximately 44 in., with most of the rain occurring in the fall and winter months. Three slope failures occurred during the fall of 1986. The first failure occurred just north of Southcross Boulevard during September and October. The depth of the cut in this area is about 38 ft. The progress of this failure was slow. The second and third failures occurred on the same night in December 1986 after a period of intense rainfall; the former, north of New Braunfels Avenue where the cut is about 45 ft deep; and the latter, just south of Southcross Boulevard where the cut is about 35 ft deep. Investigation revealed that although each failure occurred because of unfavorable seepage conditions, the origins of the unfavorable conditions differed.

Several slope stability failures had taken place in cut slopes in this section before the three failures discussed in this paper, and more failures occurred in the spring. The earlier failures were usually shallow sloughs, no more than a few feet deep, and were repaired by stabilizing the clay

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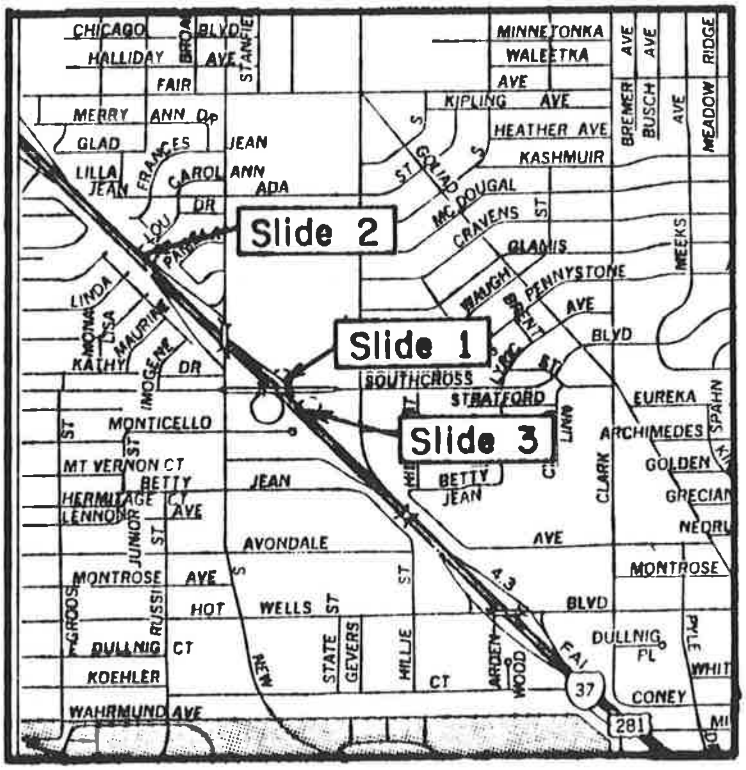

FIGURE 1 Location map.

with lime and constructing slide suppresser walls at one-third and two-thirds of the slope height. The lower slide suppresser wall was constructed of 24-in.-diameter drilled shafts with a spacing of 5 ft, with a 4-ft-wide panel supported by the shafts. The first slide suppresser wall failed structurally at the joint at the base of the panel. The shafts were repaired by reinforcing them with H-pile sections, and a second slide suppresser wall was constructed higher on the slope. The second slide suppresser wall was constructed of H-pile sections with surplus guardrail spanning between H-piles. The repair to this section has performed well and has required no further maintenance.

OBSERVATIONS

Observations were started in September 1986. At first, observations were concentrated on the area repaired with slide suppresser walls located about 100 yd north of the location of the first slope failure. Three slope indicator tubes and three groundwater observation wells were installed on a line parallel to the crest of the slope. Early photographs show that the surface of a shopping mall parking lot near the crest of the slope about 100 yd south of the repaired section had cracked, but that no large vertical movements at the top of the slide had taken place. The large crack in the concrete riprap had occurred several years previously and was believed to be symptomatic of erosion of the ground surface beneath the riprap and not of a slope failure. A program to take slope indicator measurements every 3 to 4 weeks at the three locations in the area to the north was started in October.

By middle to late fall, it had became apparent that the first slide (slide 1) was in the area south of the section being monitored. It is interesting to note that the progression of this failure was slow, occurring over several months. In early December, a 30-ft slope indicator casing was installed at the downhill edge of the entrance ramp, roughly in the middle of the slide mass. Slope indicator measurements were taken with time, and the measurements are summarized in Figure 2. The slope indicator measurements found a slip surface at a depth of 20.5 ft.

The direction of natural seepage in the area of the failures is from east to west. During dry summer months, the seepage can be detected by the the color of the grass: green on the eastern side of the highway cut and brown on the western side. The first failure is adjacent to a shopping mall parking lot. The asphalt parking surface forms a relatively impermeable barrier against evaporation and transpiration of groundwater in this area. The depth to the water table was measured to be steady at about 5.8 ft below the surface of the parking lot near the slope failure. The natural structure of the soil was revealed when the soil was excavated for lime treatment. The natural soil was found to contain two vertically oriented sets of joints: one set was brown due to iron stains, and the other was the grayish color of the parent material. The brown joints point off to the east toward an observation well that was

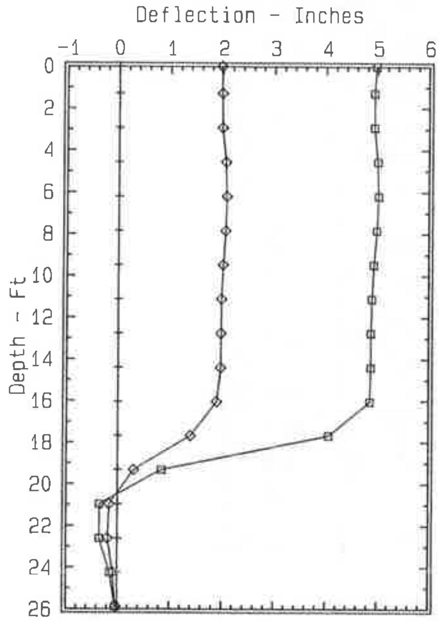

FIGURE 2 Slope indicator.

usually dry. The gray joints are oriented along a north-south alignment and line up with an observation well where the water levels are high. The orientation of the joints leads one to believe that the direction of natural seepage is not perpendicular to the alignment of the highway, but intersects the cut at an angle.

The second slope failure is adjacent to a residential area located north of slide 1 on the eastern side of the highway. Here, grass lawns and large trees consume some ground water. In September 1986, the houses along the alignment of the northbound frontage roads were destroyed or removed. When the second failure was inspected, a large flow of groundwater was observed to be saturating the slope. It was suspected that the groundwater source was a broken water main because of the steady, consistent flow. To confirm this suspicion, water samples were taken and tested for presence of chlorine. The water samples tested positive, and after a search was conducted, the broken main was found in the yard of one of the removed houses and repaired. The slope dried out quickly after the water main was repaired. The elapsed time between the removal of the house, when the water main was probably broken, and the second failure was about 4½ months.

The third failure occurred between Southcross Boulevard and a large structure that was constructed to drain the area above the cut. Inspections made after the failure found that construction of buildings adjacent to the cut had left a low depression outside of the right-of-way that could pond surface water and that it was possible that the newer buildings had not been connected to the sanitary sewer system. Inspection of the buildings also revealed architectural damage due to swelling soils. Later when the slide mass was being lime-stabilized, the soil was found to be saturated, suggesting that the combined effects of the ponded water and septic field had saturated the slope.

STABILITY ANALYSES

Analyses of slope stability are necessary for the evaluation and planning of any repair measures. The two primary reasons for performing stability analyses are to evaluate the conditions causing the first stability failure and to evaluate the effectiveness of the various options available for repair of the slope.

One can perform stability analyses using two different approaches. The preferred approach is to analyze the stability of the slide in question using a suitable technique, usually a computer program. This approach has the advantage that the specific slope geometry and seepage conditions can be considered. A second approach is to use stability charts for simple slopes. These charts are developed for slopes with specified seepage conditions. This second approach is less flexible than the first approach.

Both approaches require information on the shearing properties of the soil and the position of the water table. To evaluate an existing failure, the stability analysis is used to check the data for shearing properties that will be used in later analyses. Ideally, one should obtain a factor of safety of 1.0 and good agreement between the predicted and observed failure surfaces.

Stability analyses of slide 1 were made using UTEXAS2 (*1*). This program, an updated version of UTEXAS that has been modified to analyze reinforced slopes, uses Spencer's method (*2*) and can search for critical surfaces that are circular or noncircular. Spencer's method is similar to the method of Morgenstern and Price (*3*) in that all equations of equilibrium are satisfied. In Spencer's method, the problem is simplified by assuming that the angles of thrust between all slices are equal. The factor of safety calculated using this method is slightly higher and more accurate than the widely used simplified Bishop procedure (*4*). The reader is referred to Spencer (*2*) for further details.

Because the soil profile at slide 1 was relatively homogenous, the shearing properties were back-calculated for the observed failure surface by using the following procedure:

1. Values of effective stress shearing parameters (c' and ϕ') were assumed, and the factor of safety for the observed failure surface was evaluated. Holding cohesion constant, values of ϕ' were varied until the factor of safety equal to 1.0 was bracketed. The value of ϕ' corresponding to a factor of safety of 1.0 was calculated using linear interpolation. This procedure was repeated for several assumed values of cohesion. Values of ϕ' versus c' for the single surface analyses are plotted in Figure 3.

2. The procedure used in step 1 was repeated, using the search mode for critical failure surface of the program to find the combinations of c' and ϕ' for a factor of safety of 1.0. These combinations of ϕ' versus c' were plotted together with the values found in step 1. The back-

calculated shear strength parameters are the combination of c' and ϕ' where the two lines touch. Typically one will find that the c'–ϕ' line from the single surface analyses is straight and the c'–ϕ' line from the search analyses is curved and is tangent to the single surface line.

The above procedure finds the shearing strength parameters that allow a stability analysis to match the observed failure surface using the automatic search mode, given the assumed groundwater condition in the slope. The water table used in this procedure was based on field observations. The position of the water table at the top of the slope was fixed by long-term measurements at an observation well at the top of the slope. The position of the water table on the slope face was determined by the wet condition of the face and standing water found in post holes when the guardrail bordering the entrance ramp was removed.

The results of the analyses made for the two steps discussed above are shown in Figures 3 to 5. The water table was assumed to be that shown in Figure 6, where the line of seepage was near the slope profile. The comparison of the single-surface and search modes of analysis found $c' = 100$ psf and $\phi' = 18.8$ deg. The value for ϕ' is reasonable compared with published correlations of angle of friction with plasticity index.

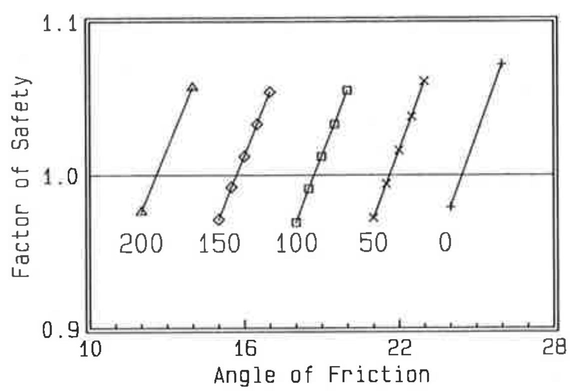

FIGURE 3 Single surface analyses.

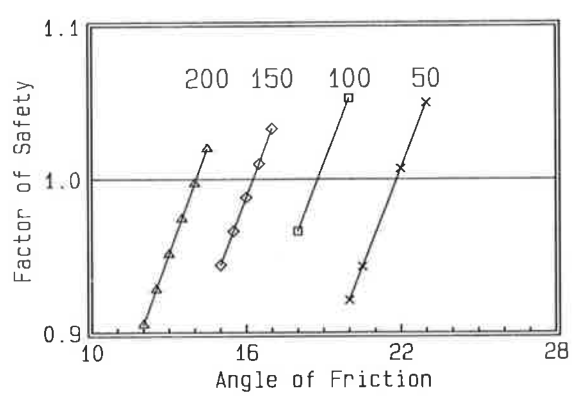

FIGURE 4 Search analyses.

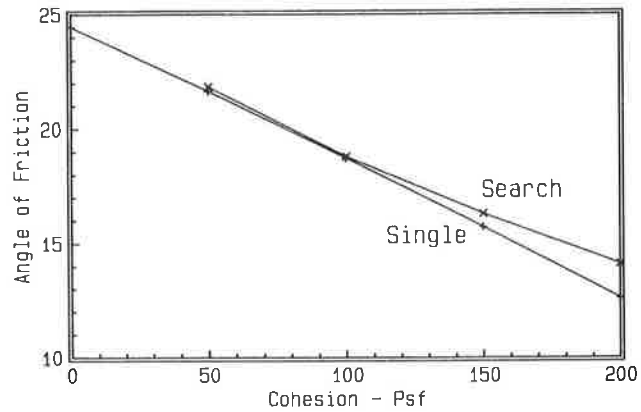

FIGURE 5 Back-calculated shearing parameters.

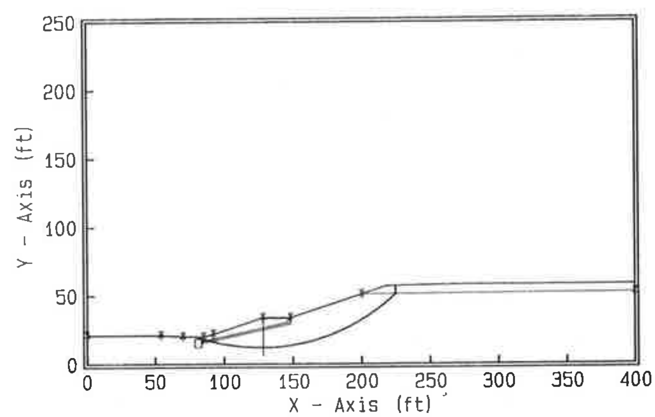

FIGURE 6 Existing conditions.

EVALUATION OF REPAIR MEASURES USING HORIZONTAL DRAINS

The obvious solution to stabilizing these slope failures is to lower the position of the groundwater in the slopes. For slide 2, this task is easily accomplished by repairing the broken water main and letting the slope drain naturally. For slide 3, the position of the water table was lowered by repairing the surface drainage facilities to eliminate ponding and by connecting the newer buildings above the failure to the sanitary sewer system. Controlling the natural seepage that caused the first failure will require the installation of subsurface drains.

A review of the relevant literature found that bored horizontal drains should work best for this application. Unfortunately, guidelines for design of horizontal drainage systems are not widely available, and the designer is faced with several questions concerning design:

- To what degree can horizontal drains raise the factor of safety against failure?

- What is the best scheme for installation of drains? Specifically, what combination of rows and lengths are most effective for draining the slope?
- How long will the drains take to become effective and raise the factor of safety to an acceptable level?

Because the conditions for each site are different, the best scheme is different for each site. However, some general comments can be made that are true for many sites. These questions are addressed in the following sections.

Change in Factor of Safety

Several approaches are available for estimation of the increase in factor of safety by the lowering of the water table by drains. Two simple methods are to use the stability charts developed by Hoek and Bray (*5*) or the charts developed by Bishop and Morgenstern (*6*). A third alternative is to make site-specific slope stability analyses using an appropriate computer program. These alternatives will be discussed in turn.

Hoek and Bray Charts

Hoek and Bray (*5*) have developed a set of five charts to estimate the factor of safety for simple slopes with assumed seepage profiles ranging from dry conditions to fully saturated conditions. One may enter Hoek and Bray's charts for the existing conditions and check the predicted factor of safety. Then one may refer to one of the charts for conditions of lowered groundwater to estimate the change in the factor of safety for lowered groundwater conditions. The user may have difficulty in using these charts for shallow slopes because the lines defining the angle of slope are not defined on some charts and the contours on the charts become quite dense in the region for shallow slopes.

Bishop and Morgenstern Charts

A second approach is to use the pore-pressure ratio, r_u. This factor is equal to the ratio of average pore pressure over vertical total stress along the shear surface. The pore-pressure ratio is defined as

$$r_u = \frac{u}{\gamma h}$$

where u is the average pore pressure, h is the depth of the point in the soil mass below the soil surface, and γ is the total unit weight of the soil. Bishop and Morgenstern (*6*) have developed design charts to estimate the factor of safety as a function of the pore-pressure ratio r_u, ϕ', and $c'/\gamma H$ for the case of a simple slope without a toe berm or bench. Bishop and Morgenstern have evaluated the factor of safety for various combinations of slope angle and shearing properties and fit regression lines through the results so that the factor of safety could be estimated using stability coefficients, m and n. The factor of safety is estimated from

$$F = m - nr_u \tag{1}$$

Assuming a factor of safety of 1.0 for a failed slope, one may estimate the value of r_u for the slope from

$$r_u = \frac{m - 1.0}{n} \tag{2}$$

The stability coefficients m and n were developed by Bishop and Morgenstern and presented in design charts as functions of ϕ', $c'/\gamma H$ (H = slope height), and bedrock depth factor D (depth to bedrock = DH). The value of r_u needed to raise the factor of safety to an acceptable level, F_d, is then calculated from

$$r_u = \frac{(m - F_d)}{n} \tag{3}$$

Now one can compare the two values of r_u and estimate how much the water table needs to be lowered to raise the factor of safety to the desired level. If the value of r_u is near zero or negative, it is likely that lowering of the groundwater alone will not be enough to raise the factor of safety to the desired level. In these cases, it will be necessary to flatten the slope in addition to lowering the ground water. In cases where r_u is positive, one may estimate the amount that the water table needs to be lowered from

$$\Delta z_w = \frac{r_u \gamma h}{\gamma_w} \tag{4}$$

where γ_w is the unit weight of water. Calculation of the distance that water should be lowered is useful in estimating how effective different locations for drains will be in raising the factor of safety.

Site-Specific Stability Analyses

In many cases, use of the stability charts is prevented because the unique conditions at a given site require that a site-specific stability analysis be made. The conditions at slide 1 will be used as an example of site-specific stability analyses. At this location, a freeway entrance ramp crossed the slope, forming a bench. The presence of the bench deviates from the slope geometry assumed by the developers of the stability charts. Stability analyses for several alternatives for repair were made so their relative effectiveness could be established. These analyses demonstrate the effectiveness of lowering the groundwater. The alternatives for repair of this slope follow:

- Installation of horizontal drains to lower the water table in the slope.
- Lowering of the ground water table by 10 ft. This case represents using a trench drain at the crest of the slope or several rows of short horizontal drains at several elevations on the slope.
- Reshaping the slope to put a bench near the top of the slope. This alternative was analyzed because it provided additional space to build frontage roads.

Installation of long horizontal drains from the toe of the slope was examined for four cases. The first case was to lower the ground water to an elevation 10 ft below the elevation of the main lanes of the highway as shown in Figure 7. This case is an ideal (and unrealistic) condition in which the water table is lowered below the critical failure surface and raises the factor of safety to the maximum level possible. The factor of safety for this condition is raised to approximately 1.76.

Three more realistic cases representing the effects of horizontal drains are those shown in Figures 8 through 10, which represent long-term conditions with horizontal drains installed at an elevation 4 ft above the toe of the slope. These cases differ in that the drain lengths are 50, 100, and 150 feet. In each of these cases, pore water pressures in the soil were assumed to be equal to zero above the water table. The assumption of zero pore pressures above the water table is conservative because any hydraulic gradient due to downward flowing seepage will increase the vertical effective stress, thereby increasing the soil's shear strength. For the case of the 50-ft drains, the factor of safety was raised to 1.28. For 100- and 150-ft drains, the factor of safety rose to 1.41 for both cases. Both cases have the same factor of safety because the critical surface does not extend into the zone covered by the additional length of the longer 150-ft drains.

In contrast, the second alternative, lowering the groundwater to 10 ft below the slope profile, could raise the factor of safety to only 1.26. This level is significantly lower than for the first alternative in which 100-foot drains were used.

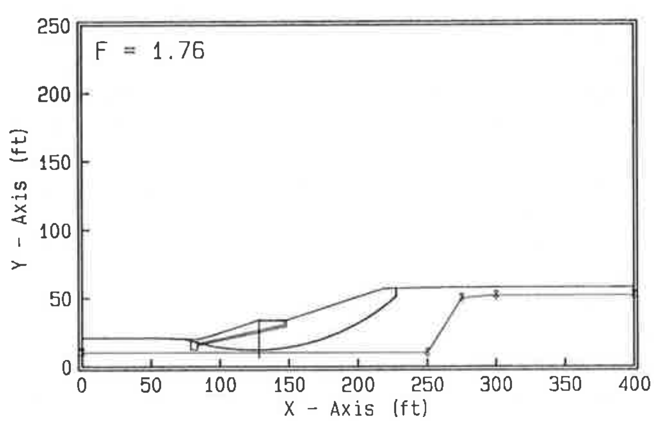

FIGURE 7 Maximum drainage.

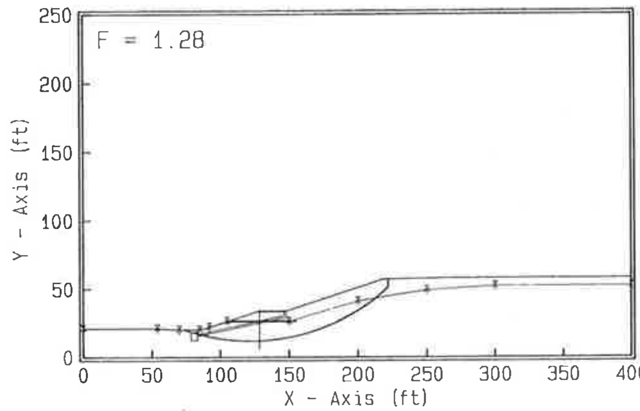

FIGURE 8 Fifty-ft drains.

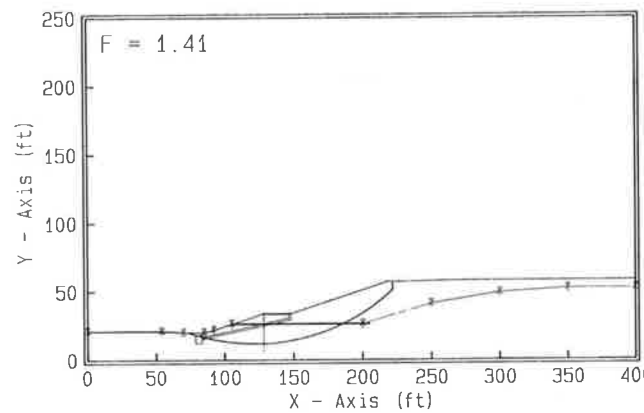

FIGURE 9 One-hundred-ft drains.

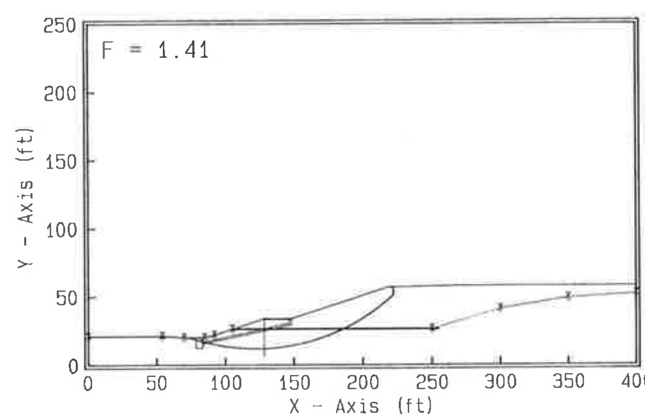

FIGURE 10 One-hundred-fifty-ft drains.

The critical surface and line of seepage assumed for this alternative is shown in Figure 11.

The third alternative, cutting the crest 10 ft, was found to be only marginally effective. This alternative could raise the factor of safety to only 1.04. Reshaping the slope is only marginally effective when the cut removes only a small section of the slide mass at the crest of the slope. The critical surface for this analysis is shown in Figure 12.

In summary, the most effective scheme for raising the long-term factor of safety is to install horizontal drains from near the toe of the slope. The general effectiveness of the horizontal drains is limited by the lower limit of elevation from which the drains can be drilled.

Installation Schemes for Drains

The second question regarding the use of horizontal drains to stabilize a slope is related to selecting the scheme for installing the drains that results in the best drain performance and lowest cost. Specifically the questions relate selection of the best location, length, and lateral spacing of drains and determination of the relative advantages and disadvantages of these factors related to performance.

Nonveiller (*7*) has investigated the efficiency and time response of horizontal drains using a three-dimensional finite difference seepage analysis coupled with a slope stability analysis that used the pore water pressures calculated in the seepage analysis to calculate effective confining stresses in the slope. Nonveiller considered three schemes of equal total drain lengths: in a single row from the toe of the slope, in two rows at the toe and midpoint of the slope, and on 10 levels up the slope. Nonveiller found that long drains from the toe are more efficient at lowering the pore pressures in the slope than multiple levels of drains and thus were able to raise the factor of safety the greatest amount. In addition, Nonveiller found that smaller horizontal drain spacings are more efficient in lowering average pore pressures and respond faster after installation because each drain covers a smaller volume of soil.

Kenney et al. (*8*) and Nonveiller (*7*) have studied the relative efficiency of spacing of drains and length of drains on raising the factor of safety. Both have found that closely spaced drains work better than widely spaced drains and that longer drains work better than shorter drains. Closely spaced drains are more effective becuase they can lower the average pore water pressure between adjacent drains to a lower value than more widely spaced drains can. Longer drains are more effective because they can lower the pore water pressures farther into the slope than shorter drains can. Obviously, there is a limit to the gain in efficiency from increasing drain length. For example, in the analyses presented in the previous section, both the 100- and 150-ft drains were found to raise the factor of safety to the same value. The increase in the factor of safety depends primarily on the lowering of the water table near the critical failure surface. Changes in the water table beyond the critical failure surface have no effect on the calculated factor of safety.

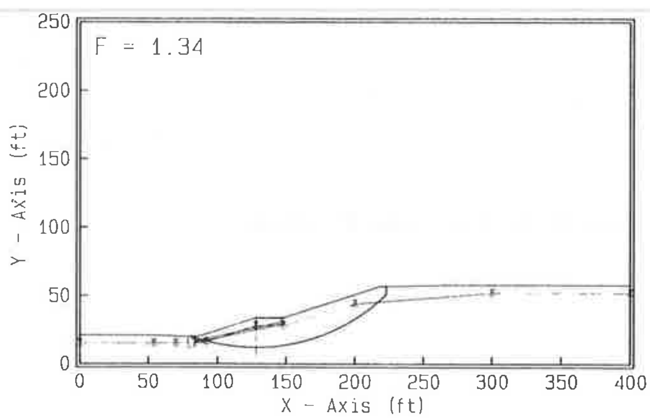

FIGURE 11 Ten-ft drainage.

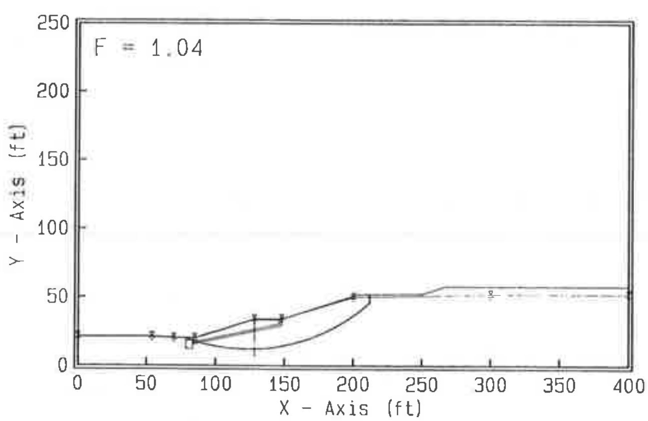

FIGURE 12 Cut-crest slope.

Time Response of Horizontal Drains

In any event, the selection of the spacing and length of drains should be based not only on the long-term factor of safety, but also on the effectiveness of drains in lowering the water table (and thereby raising the factory of safety) in a reasonable period of time. Nonveiller's study found that drainage systems with equal spacing ratios (drain spacings squared over drain length) responded similarly in obtaining a given factor of safety soon after installation. Nonveiller also found that larger differences were obtained at later times, with longer drains obtaining higher factors of safety. Thus, one can design several drainage systems that can obtain a given factor of safety in a fixed time period and that have a variety of different drain spacings and lengths. At this point, one needs to compare the relative cost of different systems of drain spacings and lengths that have approximately equal early time responses, but different long-term factors of safety, to determine whether the gains in long-term factors of safety are worth the additional expense. Ideally, this comparison should be made on a case-by-case basis. However, no readily available computer program is available that allows one to do this. Fortunately, Nonveiller's work allows one to make some sample calculations to approximate the costs involved.

Nonveiller presented data for the rise in factor of safety with time of horizontal drains installed with various lengths and spacings for the case of 2 horizontal : 1 vertical slopes. The time factor for a rise in the factor of safety with time is expressed by

$$\theta = \frac{tc_v L}{(H_i S)^2} \tag{5}$$

in which t is time, c_v is the coefficient of consolidation, L is the drain length, H_i is the height of the slope, and S is the drain spacing. The units of the time factor are 1/length. The value of θ for a rise in factor of safety from 1.0 to 1.2 is 0.0049/ft (0.016/m). Using drain lengths of 50, 100, and 150 ft, a time period of 180 days, a coefficient of consolidation of 0.093 ft^2/day, and a slope height of 40 ft, the following drain spacings, numbers of drains, total drain lengths, and adjusted drain lengths were calculated (Table 1).

These calculations were based on the conditions existing at slide 1 in San Antonio and on an assumption of a slide width of 200 ft. The spacing for each drainage system was planned so that a factor of safety of 1.2 could be achieved in 180 days. The number of drains is equal to a slide width of 200 ft divided by the drain spacing. The total drain length is simply the length of the individual drains multiplied by the number of drains, and the adjusted drain length is equal to the total drain length plus an arbitrary additional cost of 50 ft of drain per drain that was added to reflect the cost involved in moving and aligning the drilling rig, surveying drain locations, and providing facilities to handle outflow from the drains. The 50-feet-of-drain-per-drain charge is intended for illustration only and may not apply to all situations.

The purpose of these calculations is to illustrate that increases in the long-term factor of safety can be achieved for little additional expense over the cost of systems that have similar short-term effectiveness. A secondary purpose is to illustrate that there is an upper limit to the benefits of increasing drain length. For this example, there is no reason to use 150-ft drains because the long-term factor of safety is no higher than that achievable using 100-ft drains.

SUMMARY AND CONCLUSIONS

Drained failures of cut slopes often occur when unfavorable seepage conditions are present. The three slides discussed in this paper were all in the same soil formation and all failed due to adverse seepage conditions, but the source of ground water was different for each slide. Natural seepage caused one failure, a broken water main caused the second failure, and ponded water on the crest and a possibly unconnected sewer line caused the third.

Repair methods differed for each slide. Long-term stabilization of the first slide required control of the natural seepage in the slope. Installation of a system of horizontal drains was judged to be the most appropriate method for control of seepage. Repair of the other slides was accomplished by eliminating the source of water saturating the slopes.

Drained shearing parameters were back-calculated from the geometry of the failure and stability analyses with critical circle searches. Later analyses found that long horizontal drains installed at the toe of the slope were more effective in raising the factor of safety than either cutting the crest of the slope or using shorter horizontal drains or trench drains.

A review of the literature on horizontal drains revealed that long drains installed at the toe of the slope are more effective in raising the long-term factor of safety than are shorter drains installed at higher elevations. The time response of drains was found to increase with decreasing spacing of drains and increasing length of drains. Examples of calculations for drain spacings needed to raise the factor of safety to 1.2 in 180 days were presented for the conditions representing slide 1. These calculations illustrate that many combinations of drain spacing and length can achieve similar short-term time responses, and that the costs of systems that can achieve higher long-term factors of safety can be installed with minor increases in total cost. A secondary purpose of the illustration is to show that there is a limit to the benefits gained by increasing individual drain length.

TABLE 1 SPACING AND COST FACTORS FOR EXAMPLE HORIZONTAL DRAIN SYSTEM

	Individual Drain Length (ft)		
	50	100	150
Drain spacing (ft)	10.4	14.6	17.9
Number of drains/200 ft	19	14	11
Total drain length (ft)[a]	950	1400	1650
Total cost (drain-ft)[b]	1900	2100	2200
Long-term factor of safety	1.28	1.41	1.41

[a] Assuming a stabilized width of 200 ft.
[b] Assuming an additional cost of 50 drain-ft per drain.

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