ACKNOWLEDGMENT

I would like to acknowledge all the personnel of the Soil Mechanics Bureau who were involved in this project, especially B.E. Butler, V.C. McGuffey, A.R. Schnore, R.S. Grana, and E.A. Cardinal.

The efforts of M. Duval, engineer-in-charge for NYSDOT, and S. Cornell, supervisor for Schultz Construction, Inc., were uniquely responsible for the quality of the structure and the innovative solutions to field problems detailed here.

REFERENCE

1. J. Steward, R. Williamson, and J. Mohney. Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-TS-78-205, 1978.

*Publication of this paper sponsored by Soil Mechanics Section.*

# Stabilization of Wedge Failure in Rock Slope

DUNCAN C. WYLLIE

**A wedge failure in a rock slope 21 m (70 ft) high was stabilized by unloading the top third of the failure, constructing a concrete wall to reinforce the toe, and installing tensioned rock anchors. Because the potential failure was located above a major railroad line, it was necessary to carry out this work with minimal interruption of traffic. This required careful blasting to ensure that the track below the slope was not damaged by falling rock and the use of equipment that would not block the track. The use of decision analysis to review the alternative stabilization methods and select the optimum method is discussed. The probabilities of slope failure, which were used as input in decision analysis, were obtained from a Monte Carlo analysis. The purpose of the Monte Carlo analysis was to quantify the uncertainties in the rock strength and structural geology parameters that were used in the design.**

Stabilization of rock slopes above major transportation routes requires a high degree of reliability because failures that delay traffic can be extremely costly. For the same reason, the method of stabilization must allow the work to be carried out with the minimum disruption to traffic as well as be cost effective.

In this paper, the stabilization of a wedge failure of a rock slope above a heavily used railway is described. The unstable wedge was discovered during a routine preventive-maintenance program to scale loose rock from the face of rock slopes and to widen and deepen ditches. This program had been set up as a result of a survey along the entire mountainous section of the railway to identify and classify potentially hazardous slopes (1). At this location a tension crack had opened behind the crest, and excavation of accumulated debris at the toe of the slope revealed that the rock was heavily fractured and had moved as much as 300 mm (6 in). The unstable rock mass was defined by two intersecting joint planes that formed a wedge that could be analyzed by standard limit-equilibrium techniques (2).

The extent of the movement made it necessary that the wedge be stabilized as soon as possible. This work was made more urgent by the approaching winter, which would halt construction. These time restrictions made it impossible to carry out an extensive investigation program, so design work was carried out by using available data and previous experience in similar geologic conditions. In order to quantify the uncertainties in the input data, probability analysis was used in addition to the limit-equilibrium method as an aid in evaluating different stabilization options.

## CUT SLOPE IN MASSIVE GRANITE

The wedge failure had developed in a rock cut 21 m (70 ft) high that had a face angle of 75°. The toe of the cut was within 6 m (20 ft) of the railway, so even a minor slope failure could reach the track. The rock type was a very competent, massive granite that was sufficiently strong not to be fractured by the stresses imposed by a slope of this height. However, the rock contained several sets of joints that were planar and had continuous lengths that often exceeded 3 m (10 ft) and were sometimes as long as 30 m (100 ft). These conditions meant that the volumes of unstable rock formed by these joints could be substantial.

Figure 1 shows the two joints that formed the base of the wedge and one of the near vertical joints that formed the tension cracks behind the crest. A stereographic projection of the two inclined joint sets and the slope face is shown in Figure 2. This shows that the line of intersection of joint sets A and B dips toward the track and is undercut by the face, so there is a potential for sliding to occur.

There were two contributory causes of instability. Excessively heavy blasting in the original excavation had fractured the rock at the toe of the slope and reduced the forces that resisted failure. It was also likely that water pressures in the slope could be substantial following heavy rainfall or periods of sudden snow melt. However, the slope would drain quickly because of the continuous open fractures in the rock and because the slope had been cut in a "nose" of rock that was free-draining on three sides.

## BACK ANALYSIS OF EXISTING FAILURE

Although the orientation of the planes that formed the wedge could be determined with some confidence, there was no direct means of measuring the strength of the joint surfaces. There was insufficient time to carry out core drilling to obtain samples for laboratory testing, so back analysis was used to calculate the strength. The joints contained no in

Figure 1. Sketch of wedge failure.

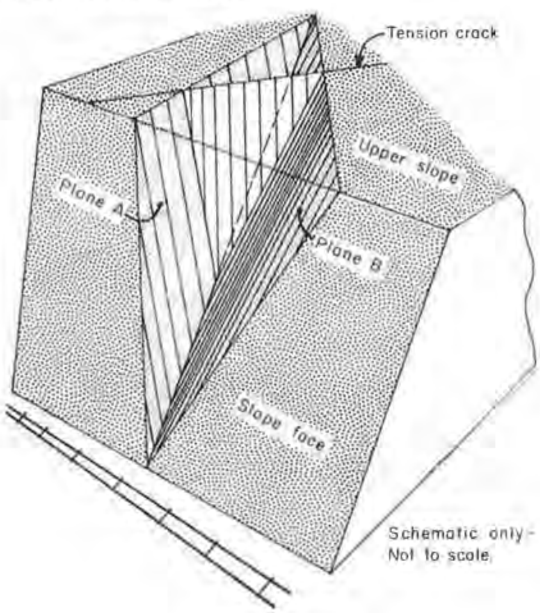
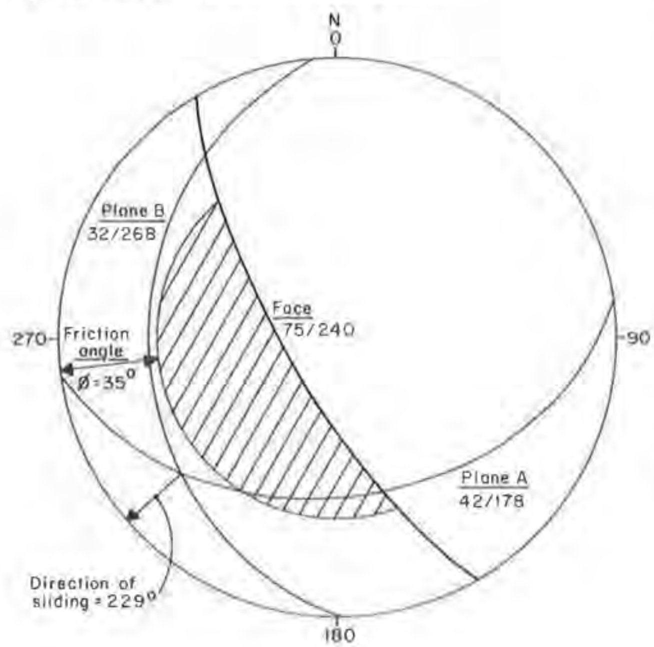



Figure 2. Stereographic projection of wedge geometry.




filling and were planar, so it was assumed that they had no cohesive strength and that the friction angle had a small roughness component.

The friction angle was determined by using a standard limit-equilibrium analysis for a wedge failure and varying the friction angle until the factor of safety equaled unity. It was assumed that the height of the water table was about one-half of the slope height. The limiting friction angle was found to be 35°, which is shown as a friction circle on the stereographic projection (Figure 2). This shows that, because the point of intersection of planes A and B lies outside the friction circle, the dry slope will be stable.

## ALTERNATIVE STABILIZATION MEASURES

The objective of the stabilization program was to increase the factor of safety to 1.5. The alternatives considered were drainage, unloading, and bolting. The merits of these alternatives are discussed below.

### Drainage

A fully drained slope would have had a factor of safety of 1.6. However, it was considered that the long-term reliability of horizontal drains to lower the water pressures was uncertain because if they became partly blocked, say with ice, high water pressures could build up in times of rapid snow melt or heavy precipitation. In addition, drainage through existing continuous open fractures was likely to occur naturally and drains would not improve the overall permeability significantly.

### Bolting

In order to raise the factor of safety of the existing slope to 1.5 by installing bolts, a total bolting load of about 22 MN (5 million lb) would have to be applied. A bolting force of this magnitude would require the installation of a number of high-capacity, multistrand anchors. The holes for these anchors would have to be about 125 mm (5 in) in diameter. Drilling holes of this diameter in a near-vertical face while maintaining traffic would have been a difficult and expensive undertaking. Furthermore, the support system would rely on a few high-capacity anchors, failure of one of which would have produced a significant decrease in the support force.

### Unloading and Bolting

Removal of the upper one-third of the slide and the installation of bolts with a total working load of about 5.5 MN (1.2 million lb) would increase the factor of safety to 1.5. This support force could be produced by using bolts with a working load of 450 kN (100 000 lb). The advantages of this system were that these bolts could be installed in holes drilled with hand-held equipment and that failure of a few bolts could not significantly decrease stability of the slope. However, the disadvantage was that the unloading operation would be slow. Only small volumes of rock could be moved in each blast in order to prevent damage to the track and to allow time for cleanup of broken rock between trains. It should be noted that the unloading operation alone does not increase the factor of safety in this condition where friction is the only factor contributing to the force that resists sliding. The effect of unloading is to decrease the required bolting forces to achieve the required factor of safety.

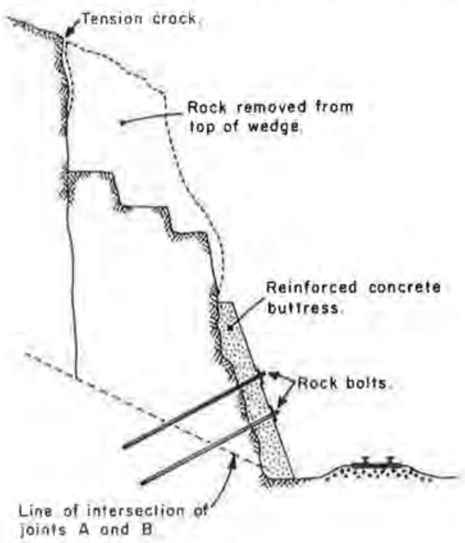
## STABILIZATION PROCEDURE

The following is a description of the unloading and bolting operation used to stabilize the slope (Figure 3).

Unloading the top 9 m (30 ft) of the slope was carried out by a four-person crew who used hand-held pneumatic equipment to drill blast holes. The blasting was carefully controlled to ensure that there would be no further damage to the rock, and smooth-wall blasting was used on all final faces. This technique consisted of drilling holes parallel to the final face on a 750-mm (30-in) spacing that were lightly loaded by using wooden spacers between half sticks of explosive. This produced an even distribution of explosive load equivalent to about 0.34 $kg/m^2$ (0.07 $lb/ft^2$) of face. The holes were detonated on a single delay to ensure that the rock broke on the line defined by the holes. Before each blast the track was protected from falling rock with a 1.2-m (4-ft) layer of gravel. In this manner about 1150 $m^3$ (1500 $yd^3$) of rock were removed.

The bolting operation was carried out as follows. The rock at the toe of the slope was so highly fractured that the installation of bolts, which produce highly concentrated forces, would have been insufficient reinforcement. Therefore, a reinforced concrete wall 9 m (30 ft) high was poured at the toe to act as a massive bearing plate for the rock bolts and distribute the stabilizing load into the slope.

Figure 3. Section through failure showing stabilization procedures.




Plastic pipes, at the required orientation of the bolts, were cast into the concrete to facilitate later drilling of the bolt holes.

All the rock bolts were 6-m (20-ft) long continuous threadbar that had grout anchors. The anchorage length of 2.4 m (8 ft) was determined by using the assumption that the rock/grout working-bond strength was 1.2 MPa (170 psi) (3). The bolts were tensioned by using a hydraulic jack before the bars were fully grouted to lock in the tension and provide corrosion protection. Five of the bolts were installed through the wall and the remainder in the sounder rock above.

## COMPARISON OF ALTERNATIVES BY DECISION ANALYSIS

The following is a description of how decision analysis (4-6) can be used to determine whether a stabilization program was economically justified and, if so, which has the optimum program.

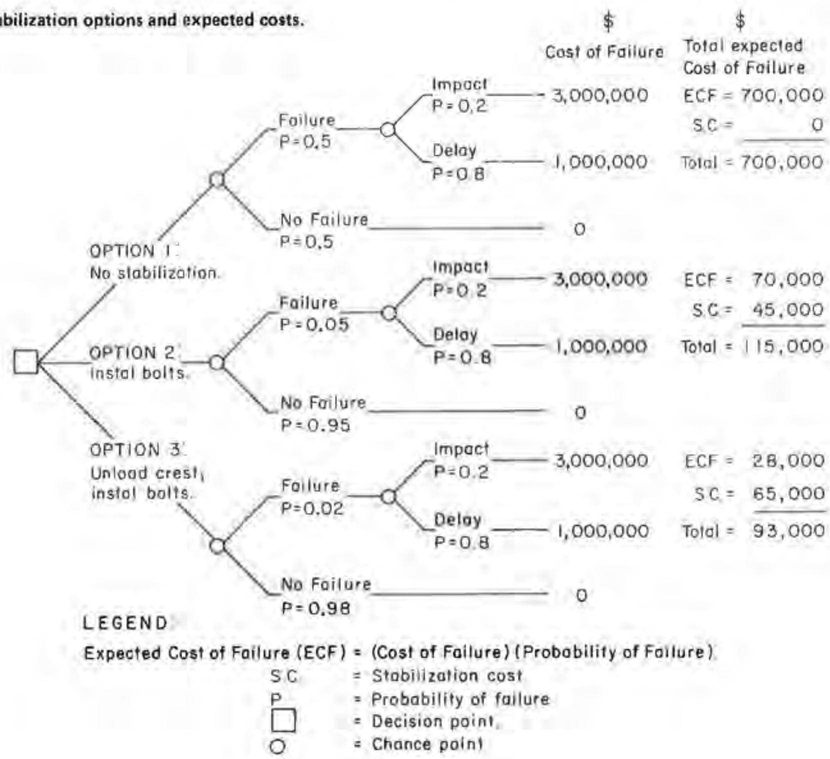
Decision analysis involves examining the probabilities and associated costs of the various possible events that can occur for each alternative course of action. These probability and cost figures are used to calculate the expected cost for each alternative so the alternative with the lowest expected cost can be determined. The value of decision analysis is that the designer must quantify his or her uncertainty in the design and the owner must evaluate the consequences of failure. There is then a rational basis for comparing alternative courses of action.

The first step in the analysis is to draw a decision tree that shows the decisions that can be made and the events that can arise from these decisions. Figure 4 shows the three options:

1. No stabilization work,
2. Installation of bolts, or
3. Unloading and installation of bolts.

Whichever option is selected, the same events can

Figure 4. Decision tree that shows stabilization options and expected costs.



occur. Namely, a failure will or will not occur, and if failure does occur, there is the possibility either that it will cause a delay or that a train will hit the fallen rock. The difference among the three options is the probability with which the events take place.

## Probabilities of Failure

The probabilities are estimated in two ways. The probabilities of slope failure are calculated by using Monte Carlo analysis (7,8), whereas the probability of a delay or impact is determined by examination of previous failure records.

Monte Carlo analysis involves giving a range of values to all those parameters in an analysis that cannot be determined precisely. A random number is then generated that is used to select a value for each of the variable parameters. These values are input to the stability analysis to calculate the factor of safety of the slope. A new random number is then generated and the calculation procedure is repeated until several hundred factors of safety have been determined. The proportion of the number of times that the factor of safety is less than unity is the probability of failure; i.e., if 10 out of the 100 trials have factors of safety less than unity, the probability of failure is 10 percent. A probability of 50 percent is synonymous with a factor of safety of 1.0.

In the stability analysis of the wedge failure, triangular distributions were assumed for those parameters the precise value for which was in doubt. The triangular distribution was defined by a mean value, which was the most likely value, and by upper and lower bounds, which are the expected extreme values (Figure 5). Because limited field and laboratory work had been carried out, judgment based on previous experience in the area was used to select these distributions. It was assumed that the extreme values for the dip direction of the joints were ±15° of the mean value, whereas the extreme values for the dip and friction angle of the joints were ±5°. The possible variation in water pressure was between one-quarter and three-quarters of the slope height.

In the case of the support by the bolts, it was also assumed that it was possible that they could produce more support than the nominal working load but also that some of the bolts could fail. The slope geometry and water and rock densities were given point values in this analysis.

Figure 6 shows a typical result of the Monte Carlo analysis. The X's represent the results of each stability analysis. Out of 100 analyses, 8 had factors of safety of less than unity, so the probability of failure is 8 percent. The probabilities of failure calculated in this manner are shown on the decision tree. The probability of failure of option 3 (unloading and bolting) is lower than that of option 2 (bolting only) because there is no uncertainty in decreasing the slope height, whereas rock bolts can fail.

**Figure 5. Triangular distribution of dip angle of plane A.**

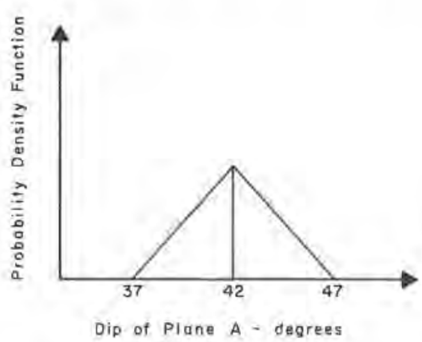



**Figure 6. Calculation of probability of failure by using Monte Carlo analysis.**

```
--------------------------------------------------
FREQUENCY DISTRIBUTION OF FACTOR OF SAFETY VALUES
--------------------------------------------------

                     FREQUENCY
                     0         10        20        30        40        50        60
CLASS   INTERVALI.........I.........I.........I.........I.........I.........I........
  1     .73     .79 .XX
  2     .79     .86 .XX
  3     .86     .93 .XX
  4     .93     .99 .XX
  5     .99    1.06 .XX
  6    1.06    1.12 .XXX
  7    1.12    1.19 .XXX
  8    1.19    1.25 .XXXX
  9    1.25    1.32 .XXXXXXXXX
 10    1.32    1.39 .XXXXXX
 11    1.39    1.45 .XXX
 12    1.45    1.52 .XXXXXXXXXX
 13    1.52    1.58 .XXXXXXXXXXXXX
 14    1.58    1.65 .XXXXXXXX
 15    1.65    1.71 .XXXXXXXXXXX
 16    1.71    1.78 .XXXXXXX
 17    1.78    1.85 .XXX
 18    1.85    1.91 .XXXX
 19    1.91    1.98 .XXXXX
 20    1.98    2.04 .X
                    I.........I.........I.........I.........I.........I.........I........
                    0         10        20        30        40        50        60

------------------
SAMPLE STATISTICS
------------------
MEAN= 1.481    S.D.=  .295    SKEWNESS= -.248    KURTOSIS=  2.75    SAMPLE SIZE= 100

-----------------------
PROBABILITY OF SLIDING
-----------------------
P.F.=  5.17 :  BASED ON SAMPLE OF KINEMATICALLY VALID CASES AND ASSUMING THAT IT IS
               NORMAL WITH MEAN = 1.481 AND STD. DEV. =  .295
P.F.=  8.00 :  BY COUNT WHERE KINEMATICALLY IMPOSSIBLE CASES ARE CONSIDERED TO BE SAFE
               ***NUMBER OF SUCH CASES WERE     0
```

### Expected Costs

The next step in the decision analysis is to determine the expected costs of failure (ECF) of each alternative. Expected costs are the product of the cost of that event and the probability of its occurring. The costs of failure, i.e., $1 million for a delay and $3 million for an impact, are estimates based on likely delay times, injuries, damage to track and equipment, cleanup of rock, and slope stabilization. Indirect costs such as legal fees and insurance premiums should also be included in the costs of failure. The expected cost of failure for each alternative is shown on the decision tree (Figure 4).

The final step in the analysis is to determine the likely cost of implementing each decision, i.e., the stabilization costs. The costs are estimated from previous construction projects and are added to the expected cost of failure. As shown on the decision tree, the most effective means of stabilization is to unload the crest and install rock bolts, even though this is more expensive than bolting only. This shows the sensitivity to expected costs of the probability of failure.

## CONCLUSIONS

In this paper, use of both limit-equilibrium and probability methods to design stabilization measures for a wedge failure in a rock slope is described. The advantages of using both methods are that limit-equilibrium analysis has been well proved in rock engineering practice, whereas probability analysis allows the designer to assess the effect of uncertainty in the input data on the design. The calculation of the probability of failure of the slope for different courses of action allows the relative merits of the alternatives to be evaluated. The use of decision analysis requires an assessment of the consequences of failure, which can involve the owner in the decisionmaking process.

## REFERENCES


1. C.O. Brawner and D.C. Wyllie. Rock Slope Stability on Railway Projects. American Railway Engineering Association, Vancouver, Canada, 1975.
2. E. Hoek and J. Bray. Rock Slope Engineering, 3rd ed. IMM/North American Publications Center, Brookfield, VT, 1981.
3. G.S. Littlejohn and D.A. Bruce. Rock Anchors--State of the Art. Ground Engineering, May and July 1975, May 1976.
4. D.C. Wyllie, N.R. McCammon, and W. Brumund. Planning Slope Stabilization Programs Using Decision Analysis. TRB, Transportation Research Record 749, 1980, pp. 34-39.
5. H. Raiffa. Decision Analysis. Addison-Wesley, Reading, MA, 1970, 309 pp.
6. S.O. Russell. Civil Engineering Risks and Hazards. The B.C. Professional Engineer, Jan. 1976.
7. Pit Slope Manual. Canada Centre for Mineral and Energy Technology, Ottawa, Ontario, Canmet Rept. 77-5, 1977, Chapter 5.
8. D.L. Pentz. Slope Stability Analysis Techniques Incorporating Uncertainty in Critical Parameters. Proc., Third International Symposium on Stability in Open Pit Mining, Vancouver, B.C., American Institute of Mining Engineers, New York, 1981.


*Publication of this paper sponsored by Committee on Soil and Rock Properties.*

# Repetitive-Load Behavior of Unsaturated Soils

E. SABRI MOTAN AND TUNCER B. EDIL

**The resilient and residual deformation behavior of a number of soil samples under unconfined repetitive loading was investigated in the unsaturated state as a function of matrix suction. Matrix suction values within the range of 50-1500 kPa were achieved by using pressure-plate extractors. Results obtained during the first phase of the study on low-plasticity sand and clay mixtures are extended to high-plasticity clay during the second phase after a series of repetitive-load tests on statically compacted samples of grundite at both dry- and wet-of-optimum compaction moisture content. The resilient modulus is shown to be related to the matrix suction in a rather unique form for all soils tested; the maximum is at 800 kPa suction and it decreases thereafter and subsequently increases again significantly at very high suction values (as in the air-dried or oven-dried state). This maximum resilient modulus obtained at 800-kPa suction increases with decreasing plasticity index. The position and the form of the curve of resilient modulus versus matrix suction may undergo changes if deviator stress and confining pressure are introduced as variables. It is also shown that the resilient behavior of cohesive soils does not follow the same pattern as the relationship of unconfined strength versus suction beyond a suction value of 800 kPa. Furthermore, the postrepetitive testing unconfined strength of dry- and wet-of-optimum compacted samples seems to be better correlated with the moisture content than with the soil suction value.**

The critical role of moisture in controlling the mechanical behavior of partly saturated cohesive soils through changes in the state of stress in soil and its modifying effects on the soil fabric have been well recognized. Strength and deformation characteristics of saturated soils can be related consistently to the stress state in the soil skeleton through the use of the effective-stress principle. However, in partly saturated soils, with decreasing degree of saturation, the evaluation of the effective stresses becomes less and less reliable due mainly to experimental difficulties. At low degrees of saturation, soil suction, which is the only measurable soil-water stress parameter, can be used analogous to the effective stress in saturated soils. The use of soil suction here as the critical soil-moisture parameter instead of the water content is also supported by the dependence of the consistency limits of cohesive soils of different origins on the stress conditions in soil water rather than on the amount of water.

Partly saturated soils usually do not create critical bearing-capacity problems. However, the magnitude of the recoverable (resilient) and irrecoverable (residual) deformations in base-course and subgrade materials caused by repetitively applied traffic loads is the basic concern in flexible pave-