ly be erected more quickly than the RE system can.

8. Where quality backfill would be a major cost item, the mesh (bar-mat) reinforcement could offer a significant economic advantage.

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Design and Construction of Fabric-Reinforced Retaining Walls by New York State

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The experience of New York State in the design and construction of two fabric-reinforced retaining walls is described. Crushed-stone fill is reinforced by horizontal layers of fabric placed at intervals dependent on the height of the wall, the strength of the fabric, and the internal friction angle of the fill. Design and construction procedures are detailed; the emphasis is on practical construction techniques. The design and construction are based on methods described by the U.S. Forest Service. The construction technique, although not commonplace, can be quickly mastered without special equipment or labor requirements. Instrumentation installed during construction to monitor vertical and horizontal movements indicates satisfactory performance 18 months after completion. The cost of this type of construction at this site compared favorably with alternative designs. Suggestions for cost reductions are offered for future installations, which may include embankment repair, similar to this project, or temporary works, such as construction detours.

New York State Department of Transportation (NYSDOT) has designed two retaining walls in which geotextile fabric is used as a reinforcing material. The walls, completed in August 1980, repair shallow failures in a side-hill embankment of NY-22 in Columbia County. The failures were observed in early 1976, when the easterly shoulder settled several inches in two areas 125 ft apart. The areas, designated A and B, extend for 110 ft and 150 ft, respectively.

A monitoring program was established in the fall of 1976 to measure any vertical or horizontal movements of the pavement, shoulder, and embankment slope. A subsurface investigation program was initiated in 1977; it consisted of a number of cased drill holes, one of which was converted into a long-term observation hole. The subsurface profile (Figure 1), determined from visual identification of soil samples and analysis of boring logs, shows 5-10 ft of loose clayey silt, sandy with gravel, overlying similar compact material; ledge rock is encountered at depths varying from 15 to 25 ft.

Movements up to 0.2 ft horizontally and 1.2 ft vertically, detected from September 1976 to April 1978, and distortion of observation hole casing at depths of 4-5 ft (see Figure 1) indicated a shallow failure in the loose material rather than a sliding failure along the rock surface. Subsurface water, due to side-hill seepage at area B and a broken box culvert at area A, was the primary cause for failure of the 1 vertical on 1.5 horizontal embankment slopes. This assumption was supported by the existence of 125 ft of unaffected 1 on 1.5 slope between the two failure areas and the natural flattening of slopes in the wet areas to 1 on 2.5.

The extent of the failures would have required remedial treatment beyond the capability of maintenance forces. Consequently, NYSDOT considered a number of design solutions. The criteria for an acceptable treatment included positive stabilization of the failure areas with low future maintenance, additional shoulder width, and safe traffic control during construction. Traffic restrictions by construction equipment and/or cost considerations eliminated a pile-and-lagging wall and extensive slope treatments. The earth-fabric wall concept was selected as the lowest-cost solution that met the criteria.

DESIGN

The design is based on methods described in a U.S. Forest Service publication (1, Chapter 5). The wall is designed as lifts of alternating fabric reinforcement and stone fill, as shown in Figures 2 and 3. The fabric within the theoretical failure zone cannot mobilize tensile strength to resist internal failure and is therefore discounted when the reinforcing length required is calculated. The fabric length embedded behind the theoretical failure plane is the fabric that reinforces the fill. The lift thickness is formed by the fabric, which overlaps the face of the wall and retains the fill material.

Site conditions controlled the length and the height of each wall necessary to stabilize the failure areas. The cross-section dimensions were determined to satisfy the internal and external stability of the wall. The minimum dimensions for internal stability were calculated with the strength parameters of the fabric and fill material by using appropriate factors of safety. These dimensions were increased to adequately resist lateral pressures acting on the wall, computed according to Rankine theory and the Boussinesq equation.

The reinforcing selected was Bidim C-34, a nonwoven, needle-punched, continuous-filament polyester fabric with high strength and permeability. The design tensile strength of 75 lb/in width of material represents approximately one-third the grab-test value (ASTM D-1117-69) reported by the manufacturer, a ratio that agrees with the Oregon State University ring-test results for other Bidim weights as report-

Figure 1. Typical subsurface profile at failure location.

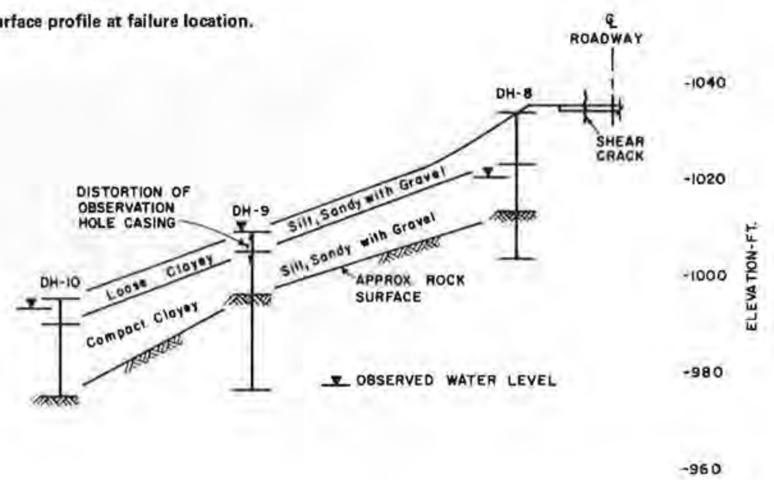


Figure 2. Typical lift detail.

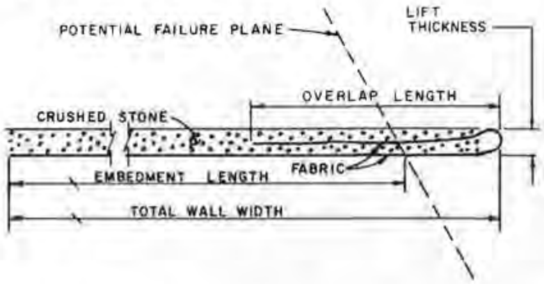
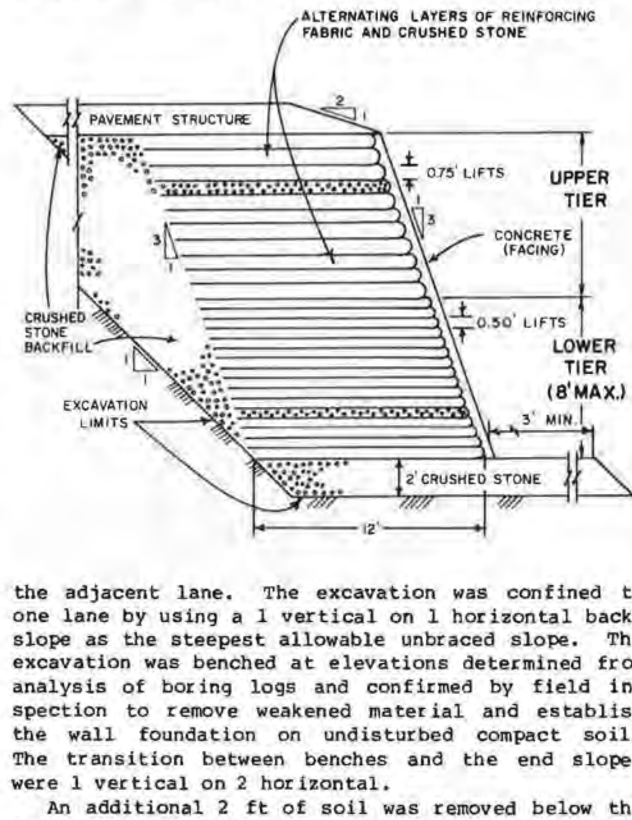


Figure 3. Typical wall section.



ed in the U.S. Forest Service publication (1). Crushed stone was selected as the fill material because of its high permeability and high angle of internal friction. The stone also develops high friction with the fabric, which is necessary to develop fabric tension.

The minimum dimensions calculated to satisfy internal stability did not satisfy the external requirements to resist sliding. This was primarily because of the low friction factor assumed between the fabric and the native material. The factor of safety against sliding was increased by widening the wall to gain mass and by placing a foundation of crushed stone 2 ft thick beneath the first lift to increase the friction factor. This crushed-stone layer also provided positive drainage for the wall and backfill.

The final cross-section dimensions, shown in Figure 3, were obtained by adjusting the calculated values to use the full manufactured width of fabric rolls, 17.4 ft. The wall was designed in two tiers, each with a different maximum lift thickness. The two-tier design was selected for practicality in construction, since the theoretical lift thickness may vary continuously with depth and pressure below the top of the wall. The overlap dimension was allowed to vary with the lift thickness to keep the wall width constant.

CONSTRUCTION

The site was prepared for wall construction by using normal clearing procedures. However, in the grubbing operation, care was taken not to disrupt the root systems of two large white pine trees adjacent to the limits of failure area A.

The excavation was progressed to the limits shown in Figure 3 while one-way traffic was maintained on the adjacent lane. The excavation was confined to one lane by using a 1 vertical on 1 horizontal backslope as the steepest allowable unbraced slope. The excavation was benched at elevations determined from analysis of boring logs and confirmed by field inspection to remove weakened material and establish the wall foundation on undisturbed compact soil. The transition between benches and the end slopes were 1 vertical on 2 horizontal.

An additional 2 ft of soil was removed below the base elevation and replaced with a crushed-stone foundation layer, as mentioned earlier. The excavation was continued at this elevation to the natural slope to provide a continuous drainage path from within and behind the wall. Wherever the distance from the toe of the wall to the existing slope exceeded 10 ft, French drains were installed, 20 ft on center, to provide positive drainage. This reduced the volume of excavation and stone backfill required. In area B, because of side-hill seepage, a separation layer of filter fabric was placed on the backslope of the excavation to prevent contamination of the stone backfill. Area A did not require this treatment because there would be no seepage after the culvert had been repaired under this contract.

This project incorporated recommendations from the U.S. Forest Service (1) to improve the temporary form. With the exception of placing the crushed-stone foundation and drainage layer, the sequence for wall construction was also performed as recommended (1). The construction sequence consisted of the following steps, which were repeated in order until the wall reached full height:

1. The temporary form system was placed to line and grade (Figure 4);
2. The fabric was positioned and the excess was draped outside the form (Figure 5);
3. Crushed stone was placed to approximately one-half lift thickness and reached full thickness at the face (Figure 6);
4. The excess fabric was folded back to overlap the fill, and the lift was completed to full thickness, burying the overlap (Figure 7); and
5. The fill was compacted, and the temporary forms were removed (Figure 8).

The fabric was placed horizontally, with the long dimension parallel to the centerline. Crushed stone was placed by using one of several methods, depending on the work area available. First, the contractor tried end-dumping the stone and placing it by hand. This method, which proved to be too laborious and time-consuming, was quickly abandoned in favor

Figure 4. Placing temporary forms.

Figure 5. Positioning fabric.

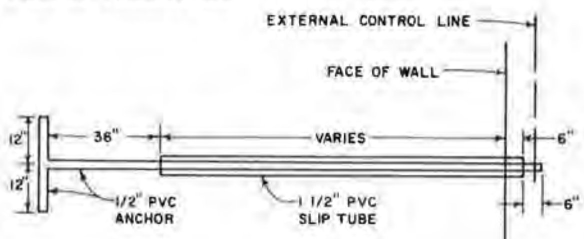
Figure 6. Forming berm for overlap.

Figure 7. Burying overlap length.

Figure 8. Compacting completed lift.

Figure 9. Concrete finish on face of wall.

Figure 10. Typical slip tube.



of dumping stone from a front-end loader and backblading it to the approximate thickness. Hand labor was necessary only to form the berm and overlap at the face. When the construction area increased sufficiently, the front-end loader supplied stone and a small bulldozer was used for grading.

Each lift was compacted by using a "walk-behind" (hand-guided) double-drum vibratory compactor with successive passes made parallel to the centerline that progressed from the backslope to the temporary form. When the work area increased, a small ride-on vibratory roller was used. Thorough compaction was desired at the wall face to minimize postconstruction settlements and horizontal movements in this area. However, the specified minimum of four passes was reduced to two passes when observations showed that the compaction effort was penetrating several underlying lifts. After the form had been removed, the vertical face gradually became curved as overlying lifts were compacted. Even when thoroughly compacted, the fabric at the face did not appear highly stressed and could actually be pinched by ordinary finger pressure.

The fabric was chosen mainly for its strength and permeability, although some resistance to ultraviolet deterioration was also desired. The maximum allowable exposure time for the fabric was specified as two weeks. However, for permanent protection, low maintenance, and protection from vandals, the wall face was covered with a mesh-reinforced pneumatically projected concrete. The reinforcing mesh, specified to be manufactured from No. 12 wires spaced 2 in in each direction, was supported by using No. 3 rebars inserted between layers to a depth of 3 ft. The bars were placed between layers during construction to form a grid measuring 3x3 ft. The mesh was included to minimize possible settlement cracks in the rigid concrete surface of the flexible wall (Figure 9).

INSTRUMENTATION

The walls were instrumented to investigate vertical and horizontal movement. Each wall was instrumented at the section of maximum height, which generally corresponded to the area of maximum past distress. Slope indicators and settlement devices were installed to monitor the behavior of the foundation soil. Slip tubes (Figure 10) were placed within the wall to detect lateral movements of the stone fill perpendicular to the wall face. The fabric was not instrumented. Minimal construction delays (typically only 0.5 h) were required to install a set of slip tubes or settlement devices that were prefabricated and installed by NYSDOT personnel. Installing the slope indicator casing, also done by NYSDOT, required several days of drilling. However, the contractor was able to schedule work without any conflicts.

Two settlement devices were installed under each wall 6 ft and 10 ft from the toe. The change in fluid level at the readout measures the cumulative foundation settlement. These devices measure the change to 0.01 ft, which is sufficiently accurate to detect significant movements.

The settlements measured during construction were less than 0.25 in. At wall B, the settlement measured one year after construction was 1.32 in at both devices. At wall A, settlements measured eight months after construction were 0.5 in and 0.25 in at the devices 6 ft and 10 ft from the toe of the wall, respectively. (These were the last measurements before the wall-A devices were vandalized.) These settlements have caused no noticeable effect on the roadway structure or the rigid concrete face.

The slope indicator casing was installed just outside the toe of the wall. The magnitude and direction of foundation movement with respect to depth can be measured as a change from the original slope of the casing. Foundation movements can thus be isolated, and any subsequent lateral movements detected by the slip tubes will indicate movement within the wall. Measurements made one year after construction indicate no significant foundation movement at either wall, which verifies the earlier assumption of a shallow failure.

A slip tube (Figure 10) is free to slide on an anchor rod that extends 3 ft beyond the tube. Internal horizontal movements of the fill are indicated by the change in distance between the end of the slip tube and a scribe mark on the inner rod. The scribe mark is referenced to an external control line. Thus, the relative movement and the absolute movement of the tube can be determined.

The tubes were installed in sets of four at various elevations to investigate the pattern of lateral movement. The tubes were buried 3, 6, 9, and 12 ft into the wall; each anchor rod is extended 3 additional ft beyond its respective tube. With anchors equal in length to the longer of the adjacent tubes, the wall is divided into several observation zones. Movements were compared within sets of tubes and with corresponding-length tubes at different elevations. Many tubes showed no movement, whereas others have not exceeded 0.25 in as late as one year after construction. Possibly, any short-term movements occurred during construction, before the control points had been set, or only small movements have occurred. Generally, the shorter tubes have shown the most movement, although continued monitoring may reveal a more definite pattern.

Figure 11. Stepping lifts to follow grade.

Figure 12. Bending wall to meeting existing slope.

LESSONS FROM FIELD EXPERIENCE

The contract plans originally specified crushed stone with a 2.5-in maximum size. A substitution was allowed at the contractor's expense to reduce the maximum size to 1.5 in. The smaller stone also provides a sufficient friction angle for this design but is easier to place by hand. This change, combined with improved handling methods developed during this project, increased the contractor's daily production. For example, exclusive of face treatment, the first wall, which has a surface area of 1630 ft², required two weeks for construction. The second wall, which has a surface area of 2100 ft², was also constructed in two weeks.

The fabric layers were placed horizontally without splices. Longitudinal splices were eliminated by designing the total fabric dimensions to equal the 17.4-ft roll width available commercially. Transverse splices were permitted if necessary by specifying a minimum overlap of 2 ft, or a 6-in minimum overlap, field-sewn double-stitched by using nylon or polypropylene thread. On this project, the contractor elected to place all lifts as a single piece of fabric. Consequently, we have no comments on the success or use of splices beyond the suggestion above regarding transverse splicing and a concurrence with others (1) that splices parallel to the wall face should be expressly prohibited.

Moisture was observed at the wall face before the fabric was covered with concrete. This was possibly due to surface water collecting in areas where irregular pockets formed in the fabric or perhaps where the fabric became clogged with rock dust. In future installations, placing the fabric horizontally in the longitudinal dimension should be considered, but the fabric should slope toward the rear of the wall for positive drainage, especially if the face treatment will be impervious.

Compaction vibrated some stone fill out of the open ends of lower lifts. This loss of fill was prevented by folding the fabric to form a bed corner (Figure 11). Figure 11 also shows how the upper lifts were stepped to follow the grade of the road. At the south end of wall A, each lift was folded to bend the wall back to meet the flatter existing slope (Figure 12). The north end of each wall intercepted the natural slope and was buried with light stone fill after the face treatment had been applied.

The rebar grid set for the reinforcing mesh required the addition of a top and a bottom row. These bars were installed after construction and had a minimal effect on the fabric. Future installations should specify a top and a bottom row of rebars. The 3-ft-square grid should be modified, perhaps by staggering alternate rows of rebars. Construction joints in the concrete face seem desirable to reduce cracking. Vertical construction joints, 50 ft on center, were included in the second wall.

The volume of concrete used on the facing exceeded the estimate by 40 percent, even with the thickness reduced from 3 to 2.5 in. This increase occurred because the ribbed surface of the wall and the wall batter formed a shelf between each lift.

COST

The cost, including items for excavation, fabric, stone fill, and concrete facing, was $42/ft² of wall face. This exceeds the 1978 estimate by $10/ft². This increase reflects the haul distance for the stone (35 miles) as well as the increased volume of concrete facing. The percentage breakdown by item was 10 percent for excavation, 30 percent for fabric, 40 percent for stone, and 20 percent for concrete.

CONCLUSIONS

Fabric-reinforced retaining walls can be built quickly and easily without unusual equipment or labor requirements. The wall can be shaped to the site by such methods as benching, bending, and stepping the lifts. The primary considerations in construction are compaction of the stone fill and drainage accommodations. The temporary forms necessary to confine the face area during compaction can be fabricated from common materials.

The instruments used were adequate to detect postconstruction movements. However, the slip tubes could not be adequately monitored during construction. Future installations should attempt to account for this condition or perhaps restrict observations to the external face of the wall. The arrangement with the contractor for installing instrumentation was satisfactory, since the instruments were prefabricated to minimize delay.

The cost of this project compared favorably with that of other alternatives at this site. Cost reductions are possible by using a cheaper fill material. However, positive drainage must be provided by using a very permeable fill or by constructing a permeable zone behind and beneath the wall. Also, a cheaper face treatment will reduce costs significantly.

ACKNOWLEDGMENT

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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.

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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