In-Place Roadway Foundation Stabilization

Richard P. Murray

A new stabilization technique that uses a two-dimensional pile system and relieving platform is described. This technique has been used to preserve a short section of NY-23A about 16 km (10 miles) west of Catskill, New York. The two-lane roadway, built on the side of a steep mountain valley, suffered major damage in a 1935 hurricane. Reconstruction at that time used a series of stone-filled timber cribs and stone walls to support the roadway embankment. Subsequent weathering has caused deterioration of the exposed timber crib faces and resultant loss of stone filling from within the cribs. This loss of support has caused movement of the pavement. One location in particular, where the roadway was supported by a series of four timber cribs, was considered critical because a failure would be rapid and would involve the entire roadway. In early 1977, the stone wall in this area had moved so much that it threatened to topple. This wall was replaced by precast concrete wall units, which solved the immediate problem but not the deep-seated stability problem. Permanent stabilization alternatives included relocation into the hillside, a structure at grade, support of the downhill slope, and in-place treatment. Design constraints included maintaining one-way traffic during construction and minimizing environmental damage. In-place stabilization incorporating the root-pile concept was selected. Plans and specifications were prepared on this basis. This paper describes the project site, design features, analysis of the contractor's proposal, construction details, and postconstruction observations. Recommendations for the use of this in-place stabilization method on future contracts are made.

Each year, many highways in the United States are damaged by landslides. It is estimated that more than $100 million is spent annually to repair landslide damage. Studies have shown that up to 95 percent of all landslides are caused by water (1). The project described in this paper, however, did not involve a water-related slide: In this case, deterioration of timber cribs had removed support from upper retaining structures and the roadway. The movement of slides due to such causes is slow, and frequent patching of the pavement can keep the surface safe for travel. However, as movement progresses, these slides reach a point where complete failure is imminent and major stabilization techniques must be used. The choice of stabilization methods is often limited by design constraints such as lack of additional right-of-way, the need to minimize environmental damage, and the need to maintain traffic flow during construction. These constraints often require new and innovative stabilization methods. This paper describes the case history of the design and construction of an embankment-stabilization project having all of these constraints.

SITE DESCRIPTION

The project was located on NY-23A, about 160 km (100 miles) north of New York City and 16 km (10 miles) west of Catskill (see Figure 1). This road connects the Hudson Valley, a major north-south travel route, with the northern Catskill Mountains through a steep, narrow valley. The road is 6.4 m (21 ft) wide and rises approximately 365 m (1200 ft) in 5.6 km (3.5 miles). The Catskill Mountains are a year-round tourist and resort area that provides recreation activities for the urban New York area. Summer recreation, fall foliage, and winter skiing make the roadway heavily traveled with cars, buses, and commercial trucks throughout the year.

A cross section of the landslide area shows the need for stabilization (see Figure 2). The embankment slope combining the stone wall and the series of timber cribs has an overall inclination of about 75°. Borings taken adjacent to the dry stone wall showed a layered cobble and boulder fill over very compact glacial till that extends to bedrock. Borings taken on the uphill side of the road did not encounter the cobble and boulder fill.

The timber cribs were installed in 1935 after hurricane rains had destroyed many areas of the road (see Figures 3 and 4). Over the past 40 years, the timber facing deteriorated and either disappeared (Figure 5) or was seriously weakened. This resulted in a loss of support for the upper retaining structures and the roadway. In the main failure area (see Figure 6), the retaining structure settled and tipped, which created a depression of the pavement that required periodic maintenance. In the area just west of the main failure area (see Figure 7), deterioration and movement of the lower timber cribs removed support for the oversteepened soil slope, which led to subsidence of the shoulder and guide rail.

Slope indicator data showed movements at 3 m (10 ft) and 5.8 m (19 ft) below the roadway surface. The movements at 3 m were related to displacement of the dry stone wall. In 1977, a portion of this wall appeared ready to collapse and was replaced. The continuing movements at 5.8 m were approximately at the interface between the cobble fill and the glacial till (thus reinforcing the failure-mechanism theory that the deterioration of the timber crib facing was the cause of the loss of support for the facilities above). Failure at this level would remove at least half of the roadway and thus close the road.

CORRECTIVE DESIGN

Because the project is located within the Catskill State Park, strict controls to minimize environmental and esthetic damages had to be observed. Traffic conditions dictated that at least one-way traffic had to be maintained at all times. Possible design alternatives of relocation into the hillside, an at-grade structure supported by drilled-in caissons, and stabilization of the downhill slope with a rock fill were studied and rejected for not meeting the New York State Department of Transportation (NYSDOT) criteria. Reconstruction of the existing walls in the same location was impossible due to the restricted access to the cribs [15 m (49 ft) or more below the roadway] and the necessity to remove the roadway that this process would entail. Treating the face with shotcrete was determined to be impractical and not permanent. Thus, a solution was required that would provide a permanent stable platform for the roadway without the support of the timber crib walls; i.e., in-place stabilization was needed.

Several in-place stabilization methods were investigated. A structure at grade was considered in detail and rejected because of lack of accessibility of the site for large equipment, costs, and the necessity to close the road. A line of drilled-in caissons forming a wall was rejected because of the limited working area for large equipment, questionable cost estimates, and uncertainty as to whether or not sufficient bridging to support the roadway would develop between the caissons.

The last method considered was one involving drilling a three-dimensional array of small-diameter root piles to create an in-place gravity-stabilized retaining wall by knitting together the in situ soil, boulder, and rock masses. This patented process was selected because it fulfilled all of the design criteria.

Because the root-pile method was a patented process, legal complications arose regarding awarding a sole-source procurement contract of this magnitude. New York State law requires that all large contracts be awarded on a competitive-bid process and, therefore, it was necessary to develop some form of alternative-bid contract. Designs from other foundation specialty companies were solicited and found to be unsatisfactory. Alternative bids of root piles versus complete relocation into the hillside were considered and rejected. The NYSDOT legal department stated that the specifications must allow other contractors an opportunity to submit equivalent designs. NYSDOT was reluctant to be put in the position of having to evaluate alternative designs, but the roadway failure was accelerating and the project was essential, and so an "or equal" provision was included.

The plans and specifications were developed around the concept that the contractor would design and construct an earth-pile retaining wall based on a system of

Figure 1. Project location.

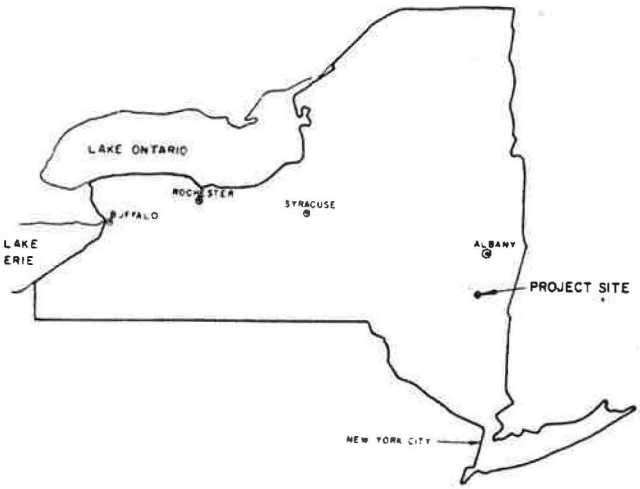


Figure 2. Cross section of critical section showing need for stabilization.

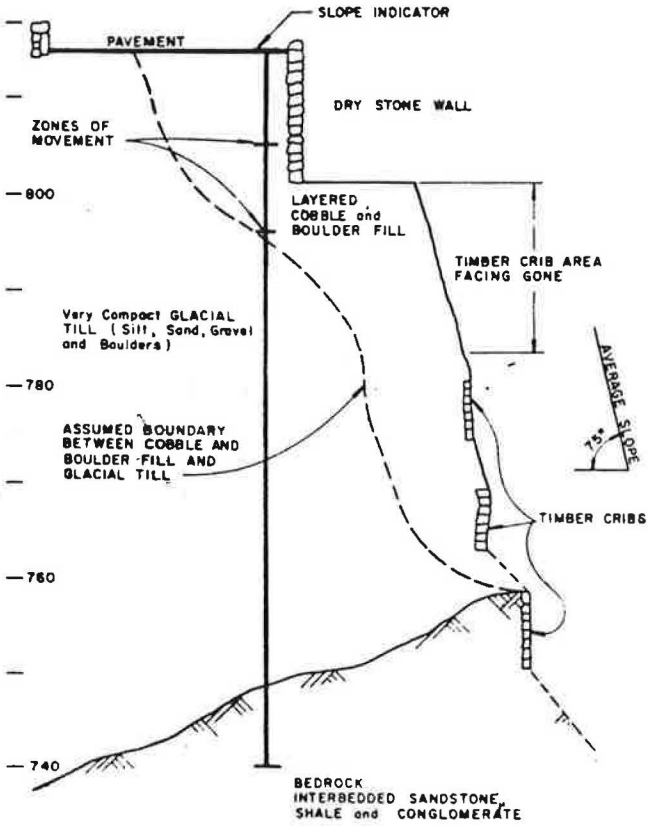


Figure 3. Conditions after 1935 hurricane.

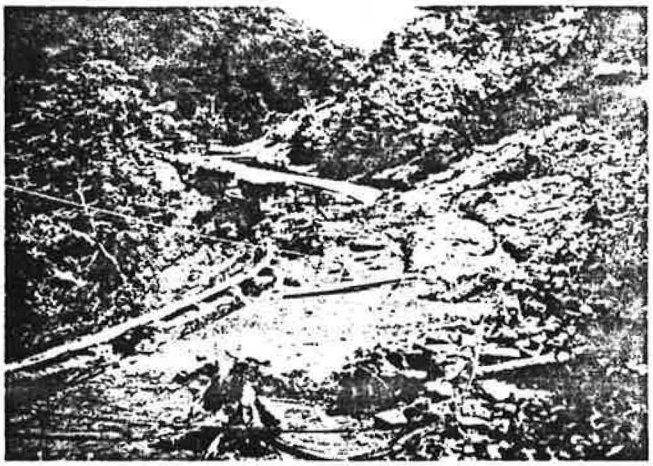


Figure 4. Construction of stone-filled timber cribs used to repair 1935 hurricane damage.

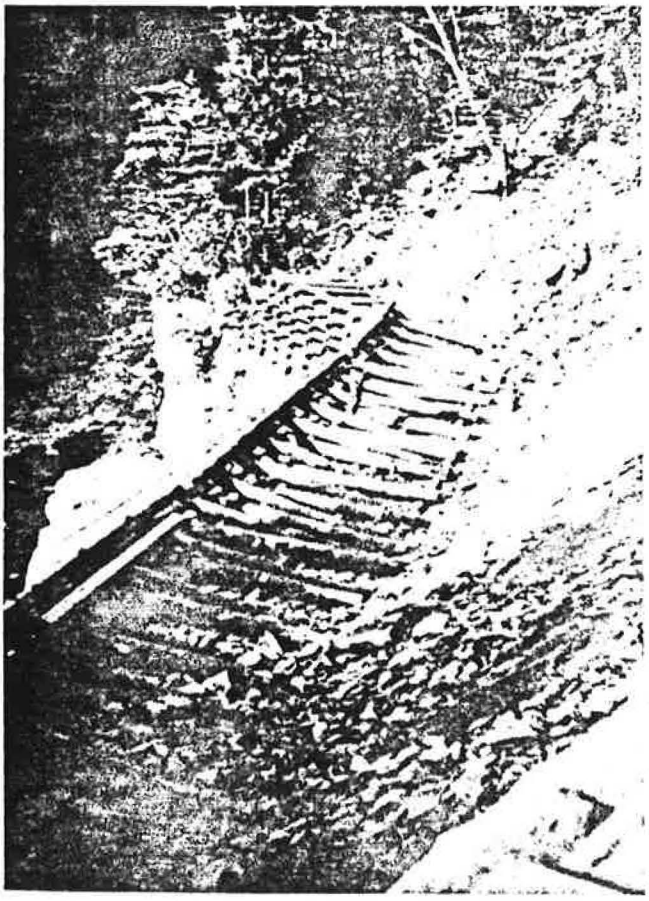


Figure 5. Current condition of timber cribs.

Figure 6. Main failure area.

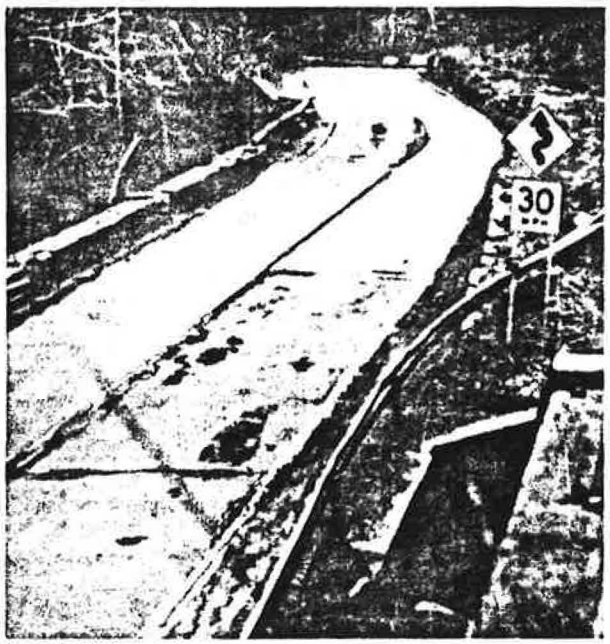
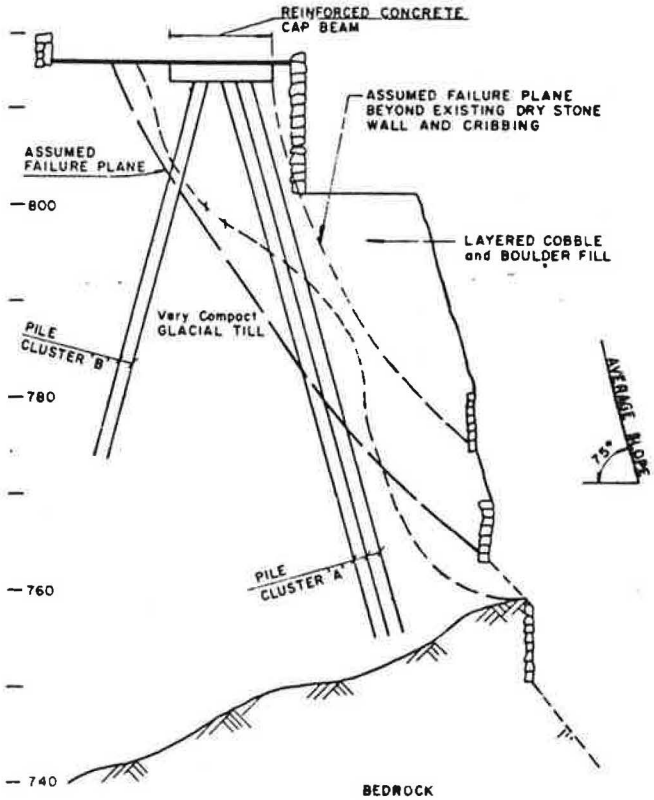


Figure 7. Area west of main failure area.

Figure 8. Typical pile layout at critical section.



cast-in-place reinforced concrete piles similar to those used in the root-pile method and topped with a reinforced concrete cap beam or an approved equal. The specifications required that the piles have a minimum diameter of 10 cm (4 in) and that the retaining wall contain an average of at least two 10-cm piles per linear foot (0.3 m) of wall measured at the ground surface. The payment items included excavation, piles, additional fluid mortar, concrete, and steel. Due to the unique location, topography, access problems, and storage space at the project site, the contract documents included as much information as possible to assist the contractor in the design, as well as a requirement that the site be visited before bid submission. Every effort was made to provide the contractor with enough information to design and construct the retaining structure.

The design that was approved for use involved installation of a two-dimensional system of four or five rows of reinforced concrete piles battered 15° in both directions from the vertical and supporting a reinforced concrete cap beam. Figure 8 shows a typical pile layout and section at the critical section. The length of the piles was varied based on the cross sections, and more piles were installed in the more critical areas. The pile and cap beam structure was analyzed as a rigid frame with sidesway. The legs (pile clusters A and B) were assumed to be fixed at the failure plane (because of their embedment below this point) and to act as beams and thus provide the capability to resist moments. These are valid assumptions because the loose, open rock allows the fluid mortar to fill the voids and the reinforcing steel knits the whole system together. All lateral resistance above the failure plane in front of pile cluster A was disregarded. Any material remaining in this position adds to the stability of the system. The soil above the failure plane, consisting mainly of layered cobbles and boulders, was assumed to have a unit weight of 1842

kg/m³ (115 lb/ft³), an angle of internal friction of 30°, and cohesion equal to 0.

The steps in the design analysis are shown in Figures 9-13. [The example analyzed in these figures does not represent the most critical section (that shown in Figure 8) but, rather, is based on preliminary data. The conditions at the critical section were discovered only during construction and were analyzed by the NYSDOT Soil Mechanics Bureau by using an approximate, somewhat conservative method.]

The forces exerted on the piles by the earth above the failure plane were resolved (see Figures 9 and 10) into axial and transverse loads on the two pile clusters. The transverse loads acting on the piles were distributed increasing linearly with depth. A rigid-frame analysis

Figure 9. Design analysis: resolution of forces from wedge 1.

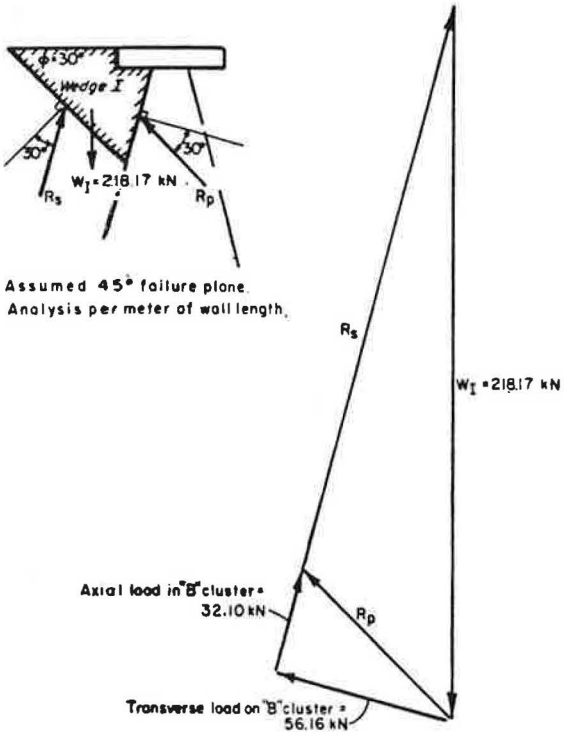


Figure 10. Design analysis: resolution of forces from wedge 2.

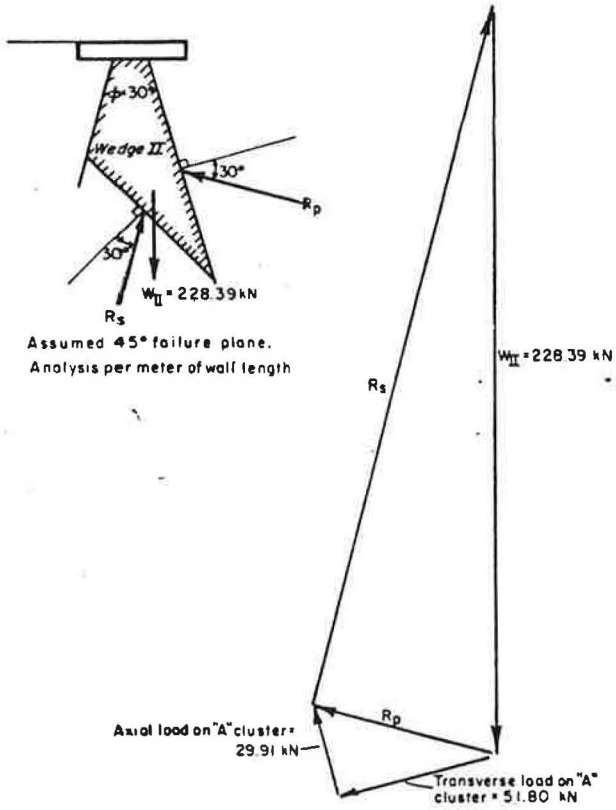


Figure 11. Design analysis: forces acting on rigid frame.

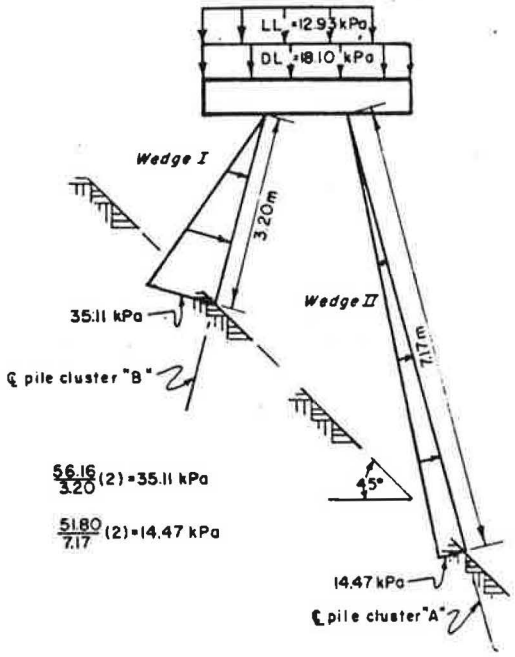


Figure 12. Design analysis: moments and shears determined by rigid-frame analysis with sidesway.

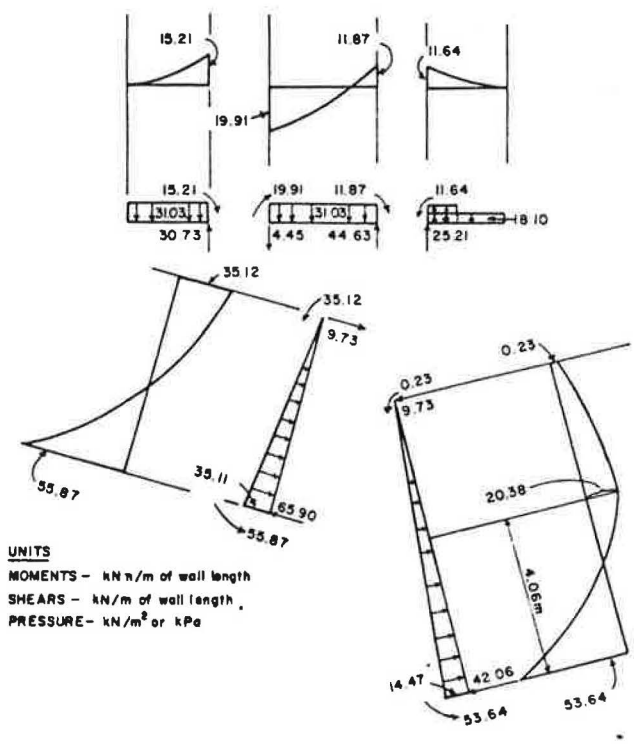


Figure 13. Design analysis: calculation of axial load on (a) pile cluster A and (b) pile cluster B.

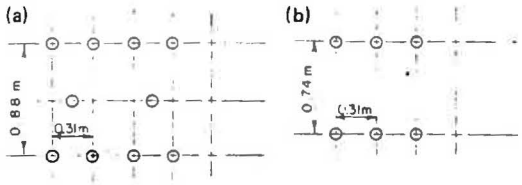
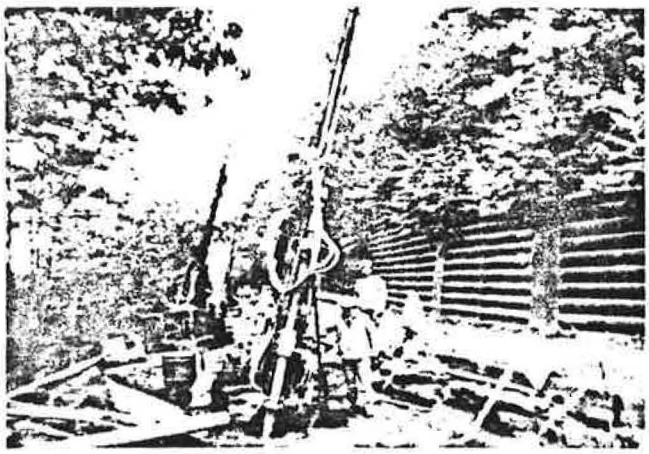


Figure 14. Construction: series of drills at work site.



with sidesway was performed for the geometry and loading shown in Figure 11. The resulting moments and shears are shown in Figure 12. The axial loads from the bending moments were added to those obtained in the steps shown in Figures 9 and 10. The allowable pile loads were calculated as shown below (1 kN/m = 0.222 lbf/ft and 1 kN = 224 lbf):

For pile cluster A, which has the geometry shown in Figure 13a,

Axial force per meter = (25.21 + 44.63)cos 15° + 29.91 = 97.37 kN/m,
Force from bending moment = 53.64/0.88 = 60.95 kN/m, and
Maximum axial load per pile = 158.32/8.74 = 18.11 kN (which is less than the design capacity of 111.25 kN).

For pile cluster B, which has the geometry shown in Figure 13b,

Axial force per meter = (30.73 - 4.45)cos 15° + 32.10 = 57.48 kN,
Force from bending moment = 55.87/0.74 = 75.50 kN/m,
Total axial load = 57.48 ± 75.50 = 132.98 kN or -18.02 kN, and
Maximum axial load per pile = 132.98/6.56 = 20.27 kN (which is less than the design capacity of 111.25 kN).

The contractor's computations were checked by the Soil Mechanics Bureau by using an approximate and conservative method. The ability of the pile clusters to sustain the computed bending moments was also investigated. It was found that, to enable each pile cluster to act as a composite beam, the shear strength of the rock fill between the piles had to be increased by permitting penetration of fluid mortar from the piles. After the design computations had been reviewed, the design was approved.

CONSTRUCTION

Construction began in August 1977. The contractor elected to excavate below the cap beam and pour an unreinforced concrete mud mat or working platform. This mat enabled him to lay out the piles on a smooth surface and to work in all kinds of weather. The pile layout was checked against the contractor's approved drawing and drilling began as shown in Figure 14. In a small number of holes, steel casings were used but, in most, there were no casings. The fluid mortar was hand poured into the holes, and 3.2-cm-diameter (no. 10) Dywidag threaded rods (reinforcing bars) were inserted. The rods had been cut into 3-m lengths, due to the necessity of installing them by hand. The pile lengths were determined by the length of the reinforcing steel placed and did not exceed the design length. The contractor was ordered to continue work until the piles and cap beam were completed to ensure the overall stability of the area.

Before the piles were installed, slope indicators were installed at various locations to monitor the stability during and after construction. Analysis of the movement data indicated that, as the piles were installed, the movements shifted to a lower depth in the compact glacial till due to the load transfer in the piles. Records indicate that up to 13 cm (5 in) of movement of the top of the existing dry stone wall occurred during construction operations. The movements slowed considerably as the front rows of piles were completed but remained in a constant state of flux from shallow to deep and vice versa due to readjustments of the shearing stresses from the soil to the piles. The piles transferred the loads deeper. Since completion of the cap beam on December 15, 1977, only minor movements have been recorded.

PERFORMANCE

The project was opened to traffic in 1978. It has been through two winters and springs, i.e., the most critical times of the year, with no significant adverse effects. The minor settlements of the pavement adjacent to the cap beam are believed to be due to discontinuity in the pavement cross section—the cap beam and the unexcavated pavement provide a rigid base, whereas the area between was filled with subbase material. The subbase material consolidated under traffic loading, which resulted in formation of a depression. There are also two areas on the downhill side of the work where the soil and rock have moved away from the cap beam. These movements were expected and were taken into account in the design shown in Figure 8. As this material moves away from the earth-pile retaining wall, the design concept of not relying on it for stability will be tested.

This method of in-place stabilization has proved to be effective and meets all of the design criteria. The contractor was able to work within one lane and maintain traffic on the other lane. Some dust was generated by the drilling operations; however, the effects on the environment were negligible. The cost of the earth-pile retaining wall and related items (no paving items) was about $10 000/linear m ($3000/linear ft) of stabilization. Much of this cost was due to the design constraints, the project location, and the area terrain.

RECOMMENDATIONS

New York State anticipates only limited future application of this stabilization method, principally due to its high cost. There may be other failure areas where con-

straints will dictate the use of this method. In these cases, the state will design the treatment rather than making use of the design-construct concept. Bidding competition by the many contractors who are not staffed to develop a design-construct contract should result in lower bid prices.

REFERENCE

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Embankment Stabilization by Use of Horizontal Drains

Stephen E. Lamb

An embankment 24 m (about 80 ft) high that traverses a narrow valley on I-81 about 56 km (35 miles) south of Syracuse, New York, began to fail several years after construction. In the spring of 1973, the pavement dropped several centimeters to bring the cumulative patch to 46 cm (about 18 in). Complete failure of the embankment was anticipated for the spring of 1974 because past movements had been largest during the spring and recent monitoring had indicated an increase in the rate of movement. A design for a stabilization project to counteract the effects of the hydrostatic pressure that was believed to be causing the failure was prepared that consisted of a berm and shear key at the toe of the embankment. Implementation of this design, however, could not be completed before spring. Therefore, it was decided to stabilize the embankment by decreasing the excess hydrostatic pressure by installing a system of horizontal drains, a project that could be completed before spring and at a cost saving of $1 000 000. The project was completed in early spring of 1974, and the embankment has been stable since then. This paper describes the design, construction, and postconstruction evaluation of the project. In addition, observations and comments are made that should be of assistance in evaluating this method of stabilization for future projects.

The problem discussed in this paper occurred in an embankment 24 m (approximately 80 ft) high that crosses a steep-sided valley 122 m (approximately 400 ft) wide. The embankment is part of I-81 in central New York State; in April 1973, about eight years after its construction, a severe crack was observed in the southbound roadway (see Figure 1). The pavement had dropped several centimeters and, by mid-July, approximately 46 cm (18 in) of asphalt paving had been placed to maintain the highway profile. A field review to determine the extent of the distress showed that stabilization would be beyond the scope of maintenance personnel.

DESIGN INVESTIGATION AND ANALYSIS

The investigation of the area consisted of a survey, a drilling program, and establishment of horizontal-movement-control stakes. The drilling program consisted of 21 borings. In 17 of the borings, observation wells were established to monitor groundwater fluctuations and to locate the depth of movement in the foundation soils (1). A plan of the site is shown in Figure 2.

Inspection of a 91-cm (36-in) diameter, corrugated metal pipe culvert in the center of the unstable area showed separations at several joints and a few separations between the joints. Also, the underdrains placed at the original ground surface during original construction were flowing.

The cross section shown in Figure 3 indicates the soil profile and observed static water table. The observation wells indicated a variable artesian pressure below the silt and clay layer, and measurements in these wells indicated significant lateral movement in the lower part of the silt and clay strata.

An analysis of the data obtained by the end of July 1973 suggested that a progressive wedge-type shear failure was occurring through the silt and clay. The most significant factor contributing to the failure was that, since construction, the static water table had risen 21 m (approximately 70 ft) in the embankment.

Analyses indicated that a counterweight berm would not be adequate to provide stability. One apparent solution was to form a key against sliding by a close-order sequence of excavation and backfilling and then construct a berm above the key. Analysis of this indicated that the slope would be stable but that the method would require an excavation for the key that extended approximately 9 m (30 ft) below the existing ground surface and cost about $1 million, because of limited access for equipment. Next, an at-grade structure spanning the failure area was considered, but the cost estimate for this was also about $1 million. Therefore, the decision was made to prepare plans to stabilize the area by using a shear key-berm solution, because a structure would require long-term maintenance. The design for this could have been completed by April 1974; however, the construction would not have been completed until several months later.

Because a major failure during the critical spring period was a significant possibility, a review was made of another method for solving this problem. In the western United States, groundwater is sometimes removed from slopes by installing horizontal drains. Such drains cannot be considered a permanent solution because of the complexity of underground water movements, but their cost is only approximately 4 percent (i.e., $40 000) of the cost of the more-permanent treatments. Consequently, a recommendation was made to attempt to achieve stabilization by installing horizontal drains.

A contract was negotiated, and work began in March 1974. Basically, the contract included a general description of the work to be performed and a 30-day completion date with a liquidated damage clause.

Because the schedule of the project did not provide sufficient time to complete the borings until just before beginning the contractual work, the description of the work to be performed was written before all the subsurface information could be obtained. However, the description was sufficiently general to allow making modifications after reviewing all the information.

The location and spacing of the drains was determined