on the energy per drop but also on the sequence of drop points and the number of drops at each point. Available data from Belgium (3), Sweden (4), France (5), Scotland (6), Israel (7), and Chicago (8), as well as these, suggest that, for dry granular soils, the degree of compaction (as measured by q_c) correlates best with the product of the energy per drop and the total energy applied per unit of surface area (Figure 5). It appears that there may be an upper bound to the densification that can be achieved, corresponding approximately to q_c = 150 kg/cm², but more data are needed to verify this result.

CONCLUSIONS

1. In granular soils, the depth to which densification is significant is controlled mainly by the energy per drop: Relationship 1a given above is recommended as a guide for preliminary trials. The presence of clay layers or seams will greatly attenuate the effective depth of compaction.

2. The upper meter of soil is usually left in a relatively loose state, and surface recompaction is required.

3. For dry granular soils, the degree of compaction achieved seems to correlate best with the product of the energy per drop and the total energy applied per unit surface area. It appears that there may be an upper bound to the compaction that can be attained and that this corresponds to $q_c \approx 150$ kg/cm² (N = 30-40).

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Construction of a Root-Pile Wall at Monessen, Pennsylvania

Umakant Dash and Pier Luigi Jovino

A case history of the design, analysis, construction, and performance evaluation of a root-pile wall is presented in this paper. The root-pile wall was contracted for construction by the Pennsylvania Department of Transportation to correct a landslide near Monessen. The structure consisted of four hundred and fifty-eight 12.5-cm (5-in) diameter cast-in-place concrete piles placed at different inclinations to both the vertical and the horizontal axes. The piles were connected at the top by a 76.2-cm (30-in) thick by 1.82-m (6-ft) wide cap beam constructed in two 30.48-m (100-ft) sections. The cap beam was constructed first, and the root piles were then installed by extending drill holes through the cap beam to bedrock at predetermined locations and inclinations, inserting a single no. 9 deformed reinforcing steel bar (grade 60) into each drill hole, and grouting the holes. Nine survey targets were marked at the top of the cap beam to measure both horizontal and vertical movements and seven slope inclinometers were installed at various points both upslope and downslope from the structure to measure horizontal movements of the structure and the surrounding soil. This paper describes the soil and groundwater conditions, soil test results, slope stability analyses, design of the root-pile wall, and the findings of the horizontal and vertical measurements of wall movement. The following summary, observations, and conclusions are made: (a) a root-pile structure provides a fast and economical alternative to many conventional structures; (b) before the installation of the root piles, the movements of the cap beam varied from less than 2.5 cm (1 in) at the north end to more than 45.7 cm (18 in) at the sound end—these movements were due to movements of unstable soil in the slide area; (c) after the installation of the root piles, there were significant movements [up to 5 cm (2 in)] in the cap beam as well as in the soil below it, which indicated that some movement of the root-pile structure was needed before resistance to earth pressure could be mobilized; (d) no significant soil movement through the root piles could be detected—the small-diameter piles and the soil between them appeared to work as a single composite structure; and (e) conventional design procedures for retaining walls appear to provide adequate overall design for root-pile walls (the geometry of the root-pile structure described in this paper is patented and may not be the optimum design for all situations).

During the construction of a four-lane highway along the Monongahela River, just north of I-70, a series of landslides occurred. One of these landslides, at the northern end of the project, involved the new highway construction, as well as two water lines and a city street above the slope about 76 m (250 ft) from the northbound lanes.

A root-pile wall was designed and constructed to correct the landslide along PA-306 in Monessen, Pennsylvania. Several alternatives (such as tieback, reinforced-earth, and concrete-gravity walls) were considered, but the root-pile method of correction was

Figure 1. Cross section at center of landslide area.

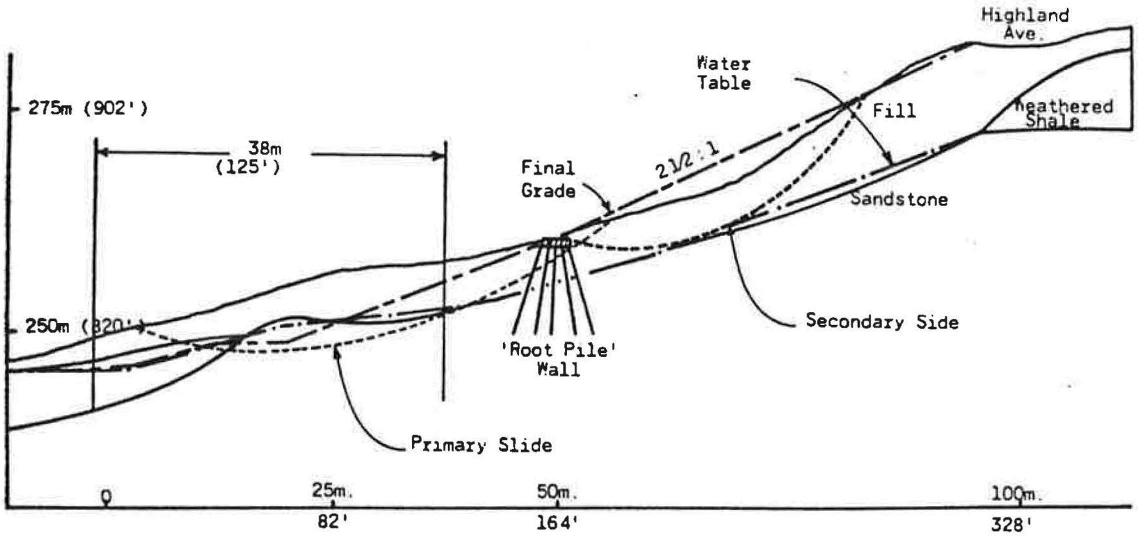


Figure 2. Aerial photograph of site.

selected because it would require the least amount of disturbance and the minimum time and have a cost comparable with that of the other systems. Another consideration in the selection decision was that this would allow evaluation of the procedure to determine its feasibility for future corrective works.

Root piles are small-diameter reinforced-concrete piles developed by the Fondedile Corporation specifically for strengthening soil or rock that is otherwise incapable of supporting its own load and/or an external load (1, 2). The method is efficient and economical and suitable for a variety of underpinning, restoration, and stabilization work.

SITE CONDITIONS

Stratigraphically, the slide area was confined to the upper portion of the Conemaugh formation of the Pennsylvania period. These strata vary from hard massive sandstone to red shales and have minor limestone interbeds. The overburden contains surface debris from mining operations, as well as foundations and other construction materials from demolished houses in the area.

A cross section at the center of the landslide area, including soil types and groundwater elevations, is shown in Figure 1. The section also shows the locations of the highway at the bottom, the root-pile wall near the middle, and the city street (Highland Avenue) near the top of the failed slope.

Figure 2 is an oblique aerial photograph taken soon after the failure and shows the general site conditions, the scarps, the acid mine-drainage channel, the location of a water pipe, and the location of the root-pile wall. The overburden soils (fill and colluvium) consisted of silty clays and clayey silts (AASHO A-6 and A-7) intermixed with rock fragments, cinders, and building materials.

The groundwater elevation varied from near the surface to about 3 m (10 ft) below the surface. Extremely wet conditions prevailed for most of the year, particularly around the acid mine-drainage channel.

SLOPE STABILITY

Slope stability analyses were performed by using a generalized soil profile and groundwater near the surface. The top and bottom scarps and the rock line were used as part of the assumed failure surface (Figure 1). Several slope-stability-analysis trials were made by using the Morgenstern-Price method and varying the effective angle of internal friction with each trial until a factor of safety nearly equal to 1.0 was obtained. The most-probable values of soil strength parameters obtained by using this method were $c = 4.79$ kPa (100 lbf/ft^2) and $\phi = 17°$. The maximum mass density (γ) [2146 kg/m^3 (134 lb/ft^3)] was obtained by using the Proctor compaction test.

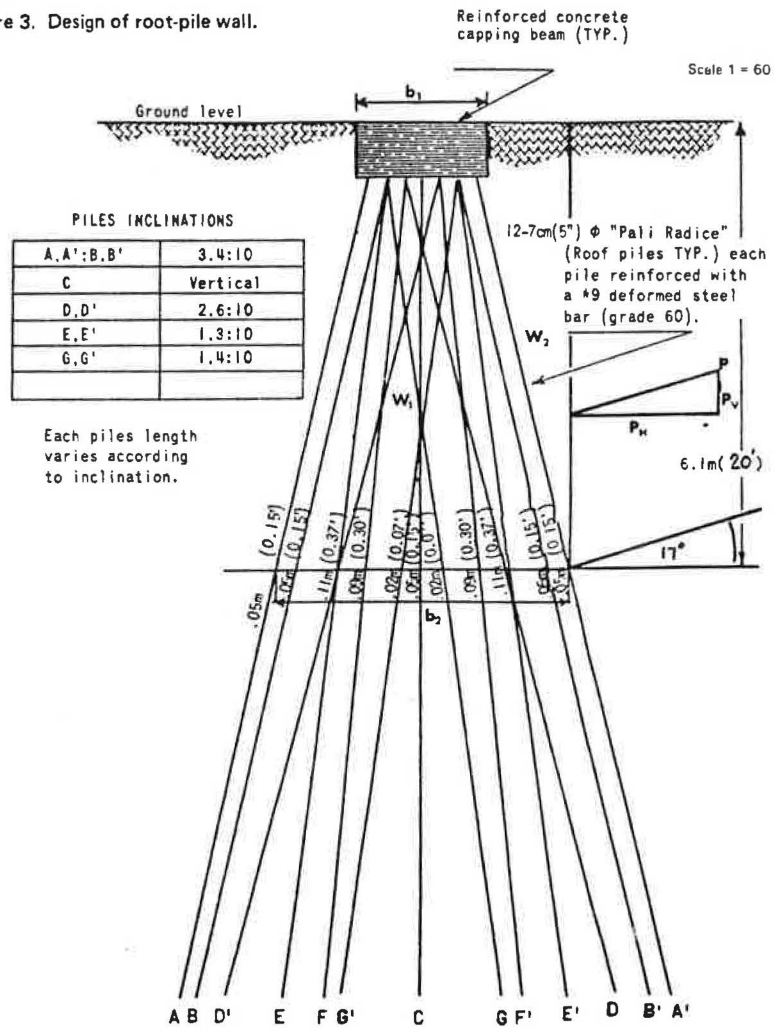
DESIGN

The design of a root-pile structure involves (a) selection of the location; (b) selection of the size; (c) selection of the pile arrangement—including spacing, inclination, length, and size of the individual piles; (d) checking the loads and stresses on the individual piles; and (e) checking probable movements of the structure. The method used for the pile arrangement at present (1979) is mostly derived from experience and is patented by the Fondedile Corporation (3, 4). The effective soil parameters used in the design of the root-pile wall were those cited above.

The resultant earth pressure (P) can be calculated by using Equation 1 and Figure 3.

$$P = (\tfrac{1}{2})\gamma h^2 K_a \tag{1}$$

Figure 3. Design of root-pile wall.



where h = height and Ka = coefficient of active earth force.

By assuming no effect of cohesion, $P = (1/2) \times 2146 \times 6.09^2 \times 0.757 = 295$ kN [66 400 lbf (66.4 kips)].
If the resultant direction is assumed to be at an angle of 17° to the horizontal, then $P_v = P \sin 17° = 86.4$ kN [19 400 lbf (19.4 kips)] and $P_H = P \cos 17° = 282.6$ kN [63 500 lbf (63.5 kips)].

The weight within the root-pile structure (W_1) is calculated by using Equation 2.

$$W_1 = \gamma h[(b_1 + b_2)/2] \quad (2)$$

where b_1 = width of cap beam and b_2 = width of root-pile structure at bedrock.

Thus, $W_1 = 2146 \times 6.09 \times [(1.82 + 3.96)/2] = 37\,778$ kg (83 112 lb) = 370.5 kN [83 300 lbf (83.3 kips)].

The weight of the soil wedge (W_2) is calculated by using Equation 3.

$$W_2 = \gamma h[(b_2 - b_1)/2] \quad (3)$$

Thus, $W_2 = 2146 \times 6.09 \times [(3.96 - 1.82)/2] = 13\,987$ kg (30 771 lb) = 137.2 kN [30 820 lbf (30.82 kips)].

The total vertical force (V) is given by Equation 4.

$$V = P_v + W_1 + W_2 \quad (4)$$

Thus, $V = 86.4 + 370.5 + 137.2 = 594$ kN [133 500 lbf (133.5 kips)].

The distance of the resultant (d) from "0" (Figure 3) is then $[(370.5 \times 1.98) + (137.2 \times 3.25) + (86.4 \times 3.96) - (282.6 \times 2.02)] \div (370.5 + 137.2 + 86.4) = 1.60$ m (5.24 ft)
and the eccentricity (e) is $(3.96/2) - 1.60 = 0.38$ m (1.25 ft).

At the base, where bedrock elevation is 6.09 m below the surface, the horizontal distances of the centers of the various root piles from the central (vertical) pile (i.e., pile C) are 0.27, 0.76, 1.16, 1.72, and 1.98 m (0.90, 2.50, 3.80, 5.65, and 6.50 ft). The corresponding numbers of piles per unit length of wall are obtained by considering a typical unit of root-pile wall (which repeats along the length of the entire structure) and dividing the total number of piles at the given distance from the center of the typical unit by the length of the typical unit (see Figure 4a) and are 0.23, 0.98, 1.21, 0.49, and 0.49/m (0.07, 0.30, 0.37, 0.15, and 0.15/ft), respectively.

The area moment of inertia (I) is $2[(0.23 \times 0.27^2) + (0.98 \times 0.76^2) + (1.21 \times 1.16^2) + (0.49 \times 1.72^2) + (0.49 \times 1.98^2)] = 11.16$ m^4 (26.8×10^6 in^4).
The total area of the root piles (A) is $2(0.23 + 0.98 + 1.21 + 0.49 + 0.49) = 6.80$ m^2 (73.2 ft^2).

Figure 4. Cap beam: (a) pile locations and (b) reinforcement details.

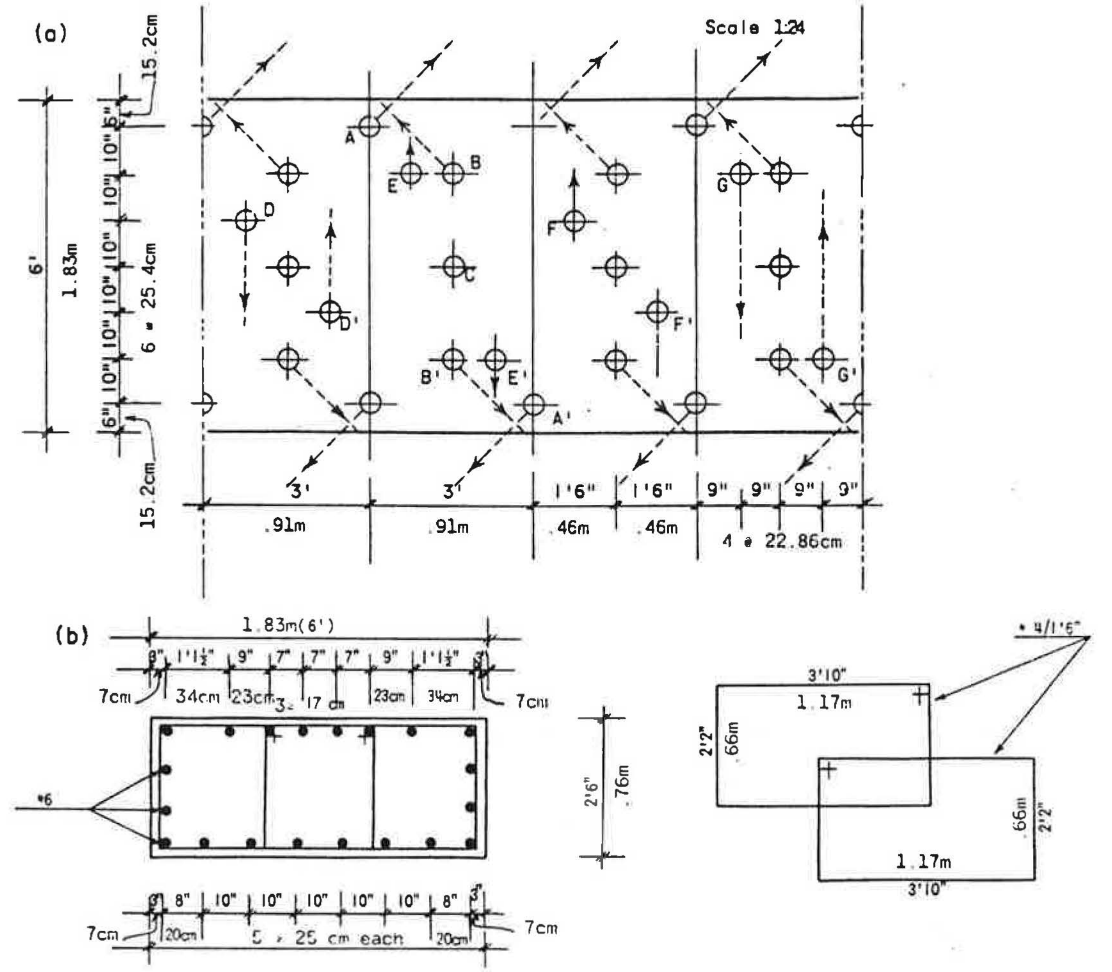


Figure 5. Form work for cap beam.

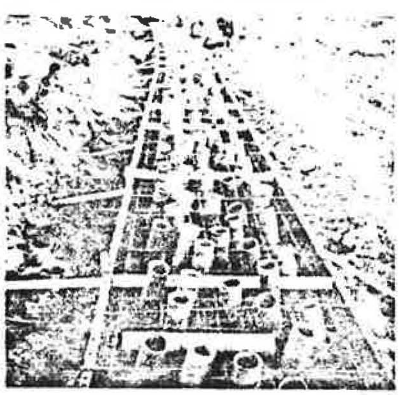


Figure 6. Drilling of holes through cap beam and preparation for grouting.

The pile loads are P_{max} = (594.0/6.80) + (594.0 × 0.38)/ 11.16 = 107.6 kN [24 180 lbf (24.18 kips)] and P_{min} = (594.0/6.80) - (594.0 × 0.38)/11.16 = 67.12 kN [15 090 lbf (15.09 kips)] and thus are within the allowable limits for the piles used.

For a 12.7-cm (5-in) diameter pile reinforced with a no. 9 bar, the allowable shear in concrete is 690 kPa × 126.64 cm^2 = 8.74 kN [1960 lbf (1.96 kips)], the allowable shear in steel is 110 316 kPa × 5.09 cm^2 = 56.22 kN [12 630 lbf (12.63 kips)], and the total allowable shear is 8.74 + 56.22 = 64.96 kN [14 600 lbf (14.60 kips)].
As the average number of piles over the length is 6.82/m (2.08/ft), the average shear resistance is 443 kN/m [13 500 lbf/ft (13.5 kips/m)].
Therefore, the factor of safety against shear is shear resistance from structure ÷ total horizontal force on structure = 443/282.6 = 1.56.

CONSTRUCTION

The construction of the root-pile wall was begun in December 1978. The cap beam was constructed in two 30.5-m (100-ft) sections (see Figure 4b). Figure 5 shows the form work for the cap beam. After the completion of the cap beam, construction was suspended during January and February 1979. There were movements of up to 46 cm (18 in) in the cap beam during this period. The holes for the vertical piles along the centerline of the cap beam were drilled first, and then the inclined holes were drilled. Most of the vertical holes were grouted before inclined holes were drilled. Figures 6 and 7 show the drilling operation, and Figures 8 and 9 show the mixing and grouting operations. The construction of the root pile was completed in April 1979.

Immediately after the holes were drilled through the cap beam, they were cleaned by using air pressure and a no. 9 reinforcing steel rod was placed in the drill hole. The grout was then poured into the hole until it was completely filled. No external pressure was applied to the grout during the grouting operation.

The grout mix consisted of 1 bag of cement, 22.7 L (6 gal) of water, and 0.071 m^3 (2.5 ft^3) of sand.

During the excavation for the northbound lanes downslope from the wall, the slope between the wall and the northbound lanes failed. This failure occurred during the second week of April 1979. Figures 10-13 show the

Figure 7. Drilling operation.

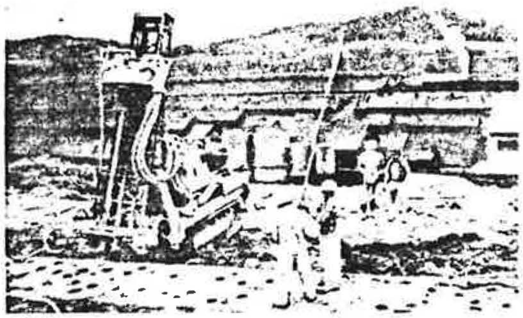


Figure 8. Mixing of grout.

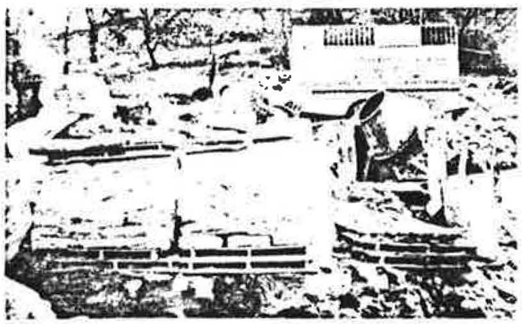


Figure 9. Grouting under gravitational pressure.

Figure 10. Soil movement during excavation downslope from root-pile wall.

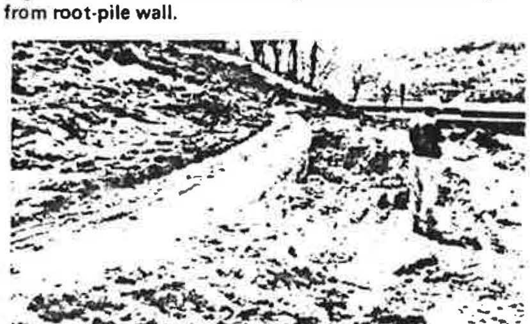


Figure 11. Additional movement that broke slope inclinometer pipes below wall.

Figure 12. Broken root piles.

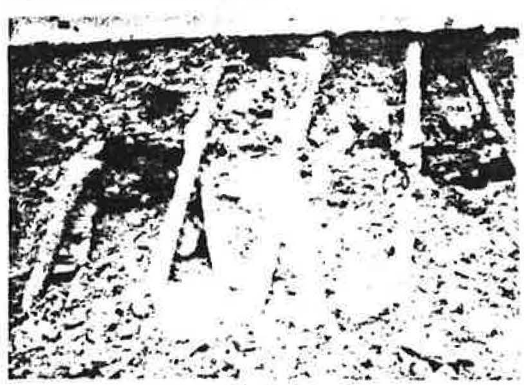


Figure 13. Testing of piles for soundness.

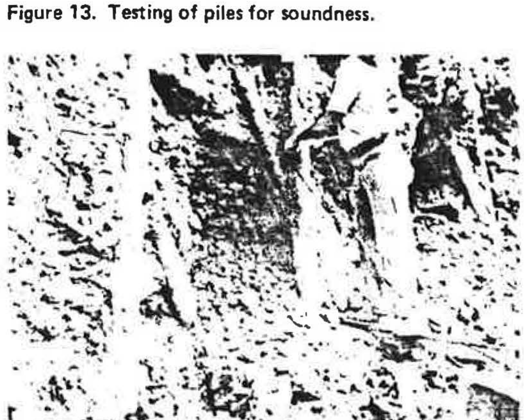


Figure 14. Slide conditions near large water pipe and removal of failed soil.

Figure 15. Root-pile wall after removal of downslope failed soil.

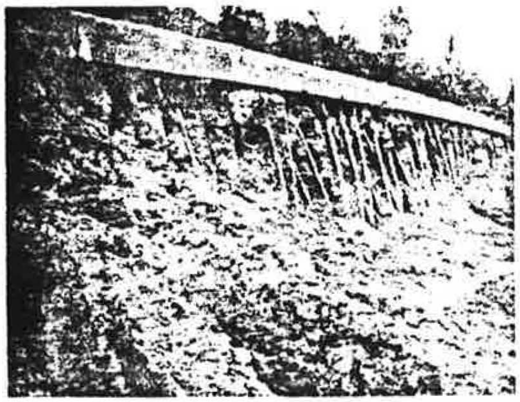


Figure 16. Completion of downslope soil removal.

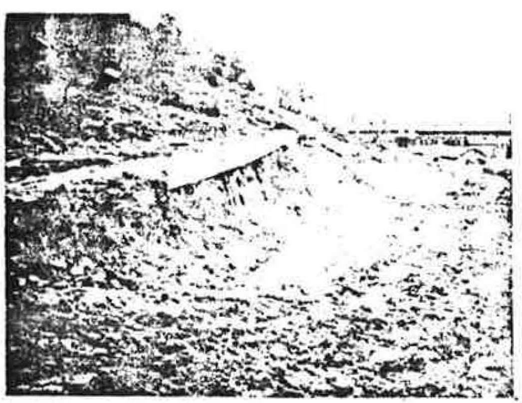
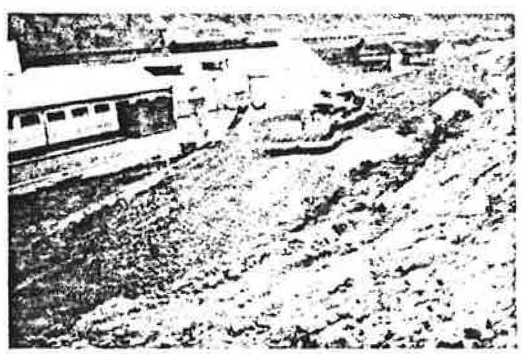


Figure 17. General view before reconstruction.

Figure 18. Installation of drainage ditch.

Figure 19. Beginning of reconstruction.



failed slope, as well as the condition of the root piles after the slope failure.

It was then decided to remove the entire failed slope in front of the root-pile wall and reconstruct the slope at a gradient of 2.5 horizontal to 1 vertical, using a well-compacted fill and a 1-m (3-ft) thick layer of granular material against the root-pile wall.

The drainage ditches were dug at right angles to the wall to drain a significant amount of the water that had ponded at the bottom of the exposed part of the wall and to serve as a permanent drainage system. Figures 14-17 show the general conditions after removal of the soil within the failed slope. Figure 18 shows the drainage ditch filled with stone.

The reconstruction work, particularly the compaction near the root-pile wall, had to be done with special care so as not to damage the piles. Figures 19-22 show the conditions during reconstruction in front of the wall.

The reconstruction work was completed in July 1979.

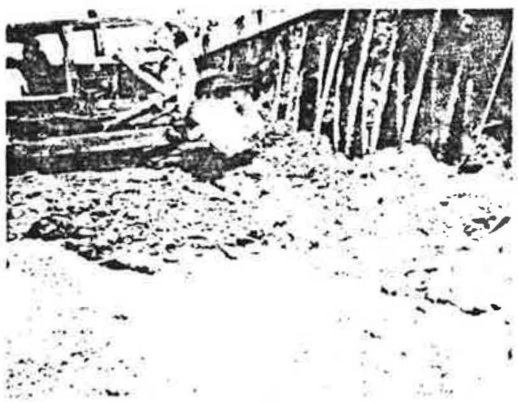
PERFORMANCE

The horizontal and vertical movements of the cap beam were monitored by taking survey readings at nine different points. These readings indicated that, before the installation of the root piles, the movements at the south end were about 46 cm (18 in) and those at the north end were less than 2.5 cm (1 in). The cap-beam movements ceased, however, after the installation of the root piles.

A total of eight slope inclinometers were installed—four on the downslope side and four on the upslope side

Figure 20. Placement of granular material against root-pile wall.

Figure 21. Placement of fill next to root piles.



of the cap beam. The slope inclinometers on the down-slope side (nos. 2, 4, 6, and 8) were sheared off during the slope failure of April 1979. The horizontal movements recorded from slope inclinometers nos. 1, 3, 5, and 7 are shown in Figure 23.

SUMMARY

The root-pile wall at Monessen provided a positive solution to the landslide problem. The method was rapid, requiring about eight weeks of actual construction time, although the total elapsed time was about four months, due to bad weather and other circumstances. The construction required practically no removal of existing soils or structures. The drilling and grouting could be done even in wet site conditions.

The original design of 7.6 m (25 ft) for the average length of root pile had to be changed to 8.8 m (29 ft) because the depth to sound bedrock was greater than had been anticipated. This delayed the completion of the

Figure 22. Reconstructed fill in front of root-pile wall.

Figure 23. Relationship between depth and deflection: (a) slope indicator 1, (b) slope indicator 3, (c) slope indicator 5, and (d) slope indicator 7.

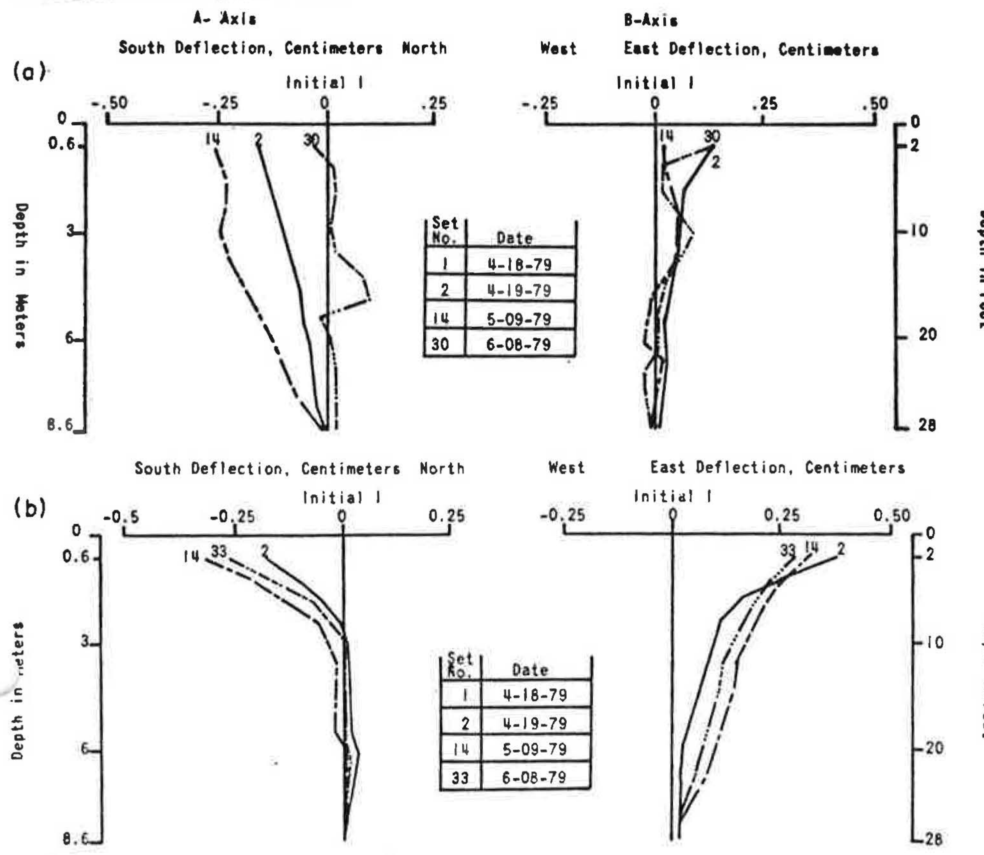
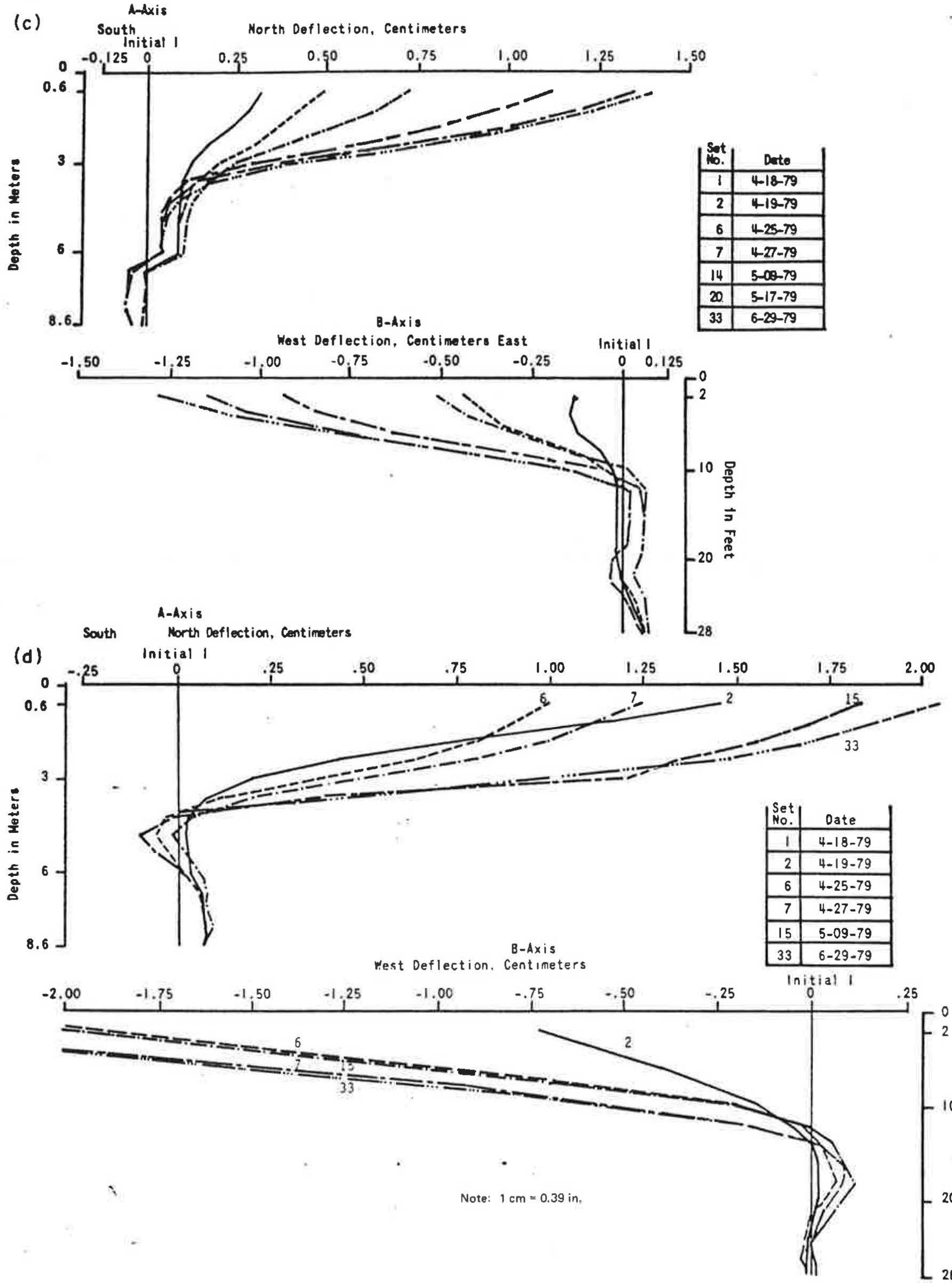


Figure 23. Continued.



construction and increased the total cost but had little or no effect on the design. There was a time delay of about 12 weeks between the casting of the cap beam and the installation of the first series of root piles. The movements of the cap beam could have been avoided by installing the root piles soon after the cap beam was constructed. The internal design of the root-pile structures is not well understood and is primarily based on experience. The arrangement of root piles (e.g., the number, size, spacing, and inclination) is based on empirical methods. The root-pile arrangement is patented and no rational method is generally available. Therefore, the actual design factor of safety cannot easily be determined.

The external stability of the root-pile wall is analyzed by using classical methods for gravity-wall analysis. This is simplistic because it does not consider soil-structure interaction aspects but appears, however, to be conservative.

The performance of the structure at Monessen provided a valuable test case for the adequacy of the current design practice because the structure supported the slope above it despite the unexpected slope failure below.

CONCLUSIONS AND RECOMMENDATIONS

Based on the experience with and the available structure-movement data from this project, the following conclusions are made:

1. The root-pile structure provides a fast and economical alternative to many conventional structures.
2. Before the installation of the root piles, the movements of the cap beam varied from less than 2.5 cm at the north end to more than 46 cm at the south end. These movements were due to movements of unstable soil in the slide area.
3. After the installation of the root piles, there were significant movements [up to 5 cm (2 in)] in the cap beam as well as in the soil below it. This indicates that some movement of the root-pile structure was needed before resistance to earth pressure could be mobilized.
4. No significant soil movement through the root piles could be detected; i.e., the small-diameter piles and the soil between them appeared to work as a single composite structure.
5. The construction of the root-pile wall was rapid and caused little or no disturbance to the existing terrain.
6. Conventional design procedures for retaining walls appear to provide overall design for root-pile walls. The geometry of the root-pile structure described in this paper is patented and may not be the optimum design for all situations. Therefore, the design procedure for the geometry and size of the individual piles within the root-pile structure should be investigated further. A rational method, one that considers soil-structure interaction, should be developed for the design of root-pile structures and verified by using actual field measurements of prototype construction.
7. There should be more test cases of root-pile construction; the instrumentation should be adequate to measure loads and movements so that the design methods can be evaluated.

ACKNOWLEDGMENT

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Analysis of an Earth-Reinforcing System for Deep Excavation

S. Bang, C. K. Shen, and K. M. Romstad

A limit-analysis procedure for a reinforced lateral earth support system is described. The system is composed of a wire-mesh-reinforced shotcrete panel facing, an array of reinforced anchors grouted into the soil mass, and rows of reinforcing bars that form horizontal wales at each anchor level. Excavation starts from the ground level and, after each layer, reinforcement is applied immediately on the exposed surface and into the native soil. This system thus forms a temporary earth support that has the advantages of requiring no pile driving, not loosening or sloughing the soil, and providing an obstruction-free site for foundation work. It has been successfully used for large areas of excavation to depths of up to 18 m in various ground conditions. However, in the past, no rational and proven analytical design procedure was available, a problem that resulted in considerable reservation toward the use of the system among engineers and contractors. The two-dimensional plane-strain limit-analysis formulation includes consideration of design parameters such as soils type, depth of excavation, length of the reinforcing members, inclination, and spacing. The analysis procedure can be used to evaluate the overall stability of the system and to determine the proper size, spacings, and length of the reinforcement for a given site condition.

In recent years, underground construction has been widely used as a logical part of the solution to many urban and city problems. Sewer and water conduits and other utility lines are usually installed underground in large cities, and vehicular tunnels and underground stations can decrease both intracity and intercity traffic congestion and thus improve both air quality and traffic safety. Even more important is that increased underground building construction is a desirable alternative that saves energy. To meet the challenge of increasing