Compacted Clay Embankment Failures

ZENON G. KYFOR AND RAYMOND L. GEMME

Embankment instability that occurred along several completed sections of a major interstate highway located in the western part of New York State is described. The affected embankments ranged in height from 4.6 m (15 ft) to 9.15 m (30 ft) and were constructed of silty clay over a silty clay foundation. Field investigations revealed that movements were taking place within the embankments and were not a result of the foundation soil. These movements were consistent with an undrained mode of failure. The embankment material was obtained from borrow excavations made outside the project limits and classified as a medium to highly plastic clay. The moisture contents of the borrow ranged from 25 to 55 percent and the plasticity index ranged from 15 to 32 percent. Drill holes progressed through the affected fills indicated that the embankment moisture contents at the time of distress exceeded the optimum moisture content (20 percent) by 5 to 10 percent based on a standard Proctor compactive effort. The undrained shear strengths at these moisture contents were not sufficient to provide for internal stability of the embankments. The results of the field and laboratory investigations are presented. They consist of inclinometer results of embankment monitoring, compaction curves for material used, and moisture contents and densities of fills obtained during construction and after movement began. A relationship between moisture content and undrained shearing strength is also shown. Stabilization consisted of flattening all embankments greater than 4.6 m (15 ft) from a 1 (vertical) on 2 (horizontal) to a 1 (vertical) on 3.5 (horizontal) side slope.

The current New York State Department of Transportation specifications require embankment material be compacted to at least 90 percent of the maximum Standard Proctor Density. The contractor determines the type and size of compaction equipment, selects lift thickness, and exerts proper control over the moisture content of the material and other details necessary to obtain satisfactory results.

Many hundreds of miles of embankments have been constructed with few serious problems, indicating for the most part that the earthwork specifications currently in use are effective. This can be attributed primarily to the following factors:

- Many of the embankments constructed in New York State are built out of granular materials.
- Contractors will often opt to waste highly plastic excavation materials and substitute granular or low-plasticity materials for embankment construction whenever economically feasible.
- The majority of the embankments constructed do not exceed 6.1 m (20 ft) in height. Although embankments exceed this height at many bridge approaches, the material used at the abutment locations is mostly a select granular material.

Notable problems have all been related to embankment construction using plastic soils. The problems have ranged from the "during construction" type, such as difficulty in handling and placing material, rutting, and weaving, to internal instability of the embankment soon after construction is completed. The internal stability of embankments is the subject of this paper.

The failure of compacted clay embankments containing low-plasticity [plasticity index (PI) = 8 to 13] soils in New York State has previously been reported (*1*). These failures were attributed to the condition in which the moisture contents of the fill material exceeded the optimum moisture content for standard compaction.

The present paper deals with the instability of highway embankments that occurred in the fall of 1982 on a major Interstate project. The embankments were constructed of higher-plasticity soils (PI = 15 to 35). The type of embankment failures that occurred on this project were short-term failures in which the undrained soil strength parameters would govern. The results of the investigation and recommendations for remedial treatment of the problem areas and for use of similar embankment materials on future projects are presented.

BACKGROUND

Project Description

The project is located in Erie County just northeast of Buffalo (Figure 1). A closeup of the location of the problem areas is shown in Figure 2.

Site Geology

Geomorphologically the project area is a broad, flat, glacial lake bed dissected by several streams. The soils consist mainly of lacustrine bottom deposits, with alluvial deposits found on the flood plains of the streams.

General Foundation Conditions

The general foundation conditions in the project area consist of up to 1.5 m (5 ft) of sandy silt, underlain by 1.5 m (5 ft) of stiff red/brown clay over 3.1 m (10 ft) of soft red/brown clay. The soft clay is underlain by compact glacial till extending to rock. Figure 3 diagrams the embankment geometry showing embankment distress. Figure 4 shows the stress history and undrained shear strengths of the foundation soils.

Embankment Material

The embankment material was obtained from borrow excavations made outside of the project limits and is classified as medium to highly plastic clay (Unified Soil Classification System: CL to CH).

New York State Department of Transportation, Soil Mechanics Bureau, Bldg. 7, State Campus, 1220 Washington Avenue, Albany, N.Y. 12232.

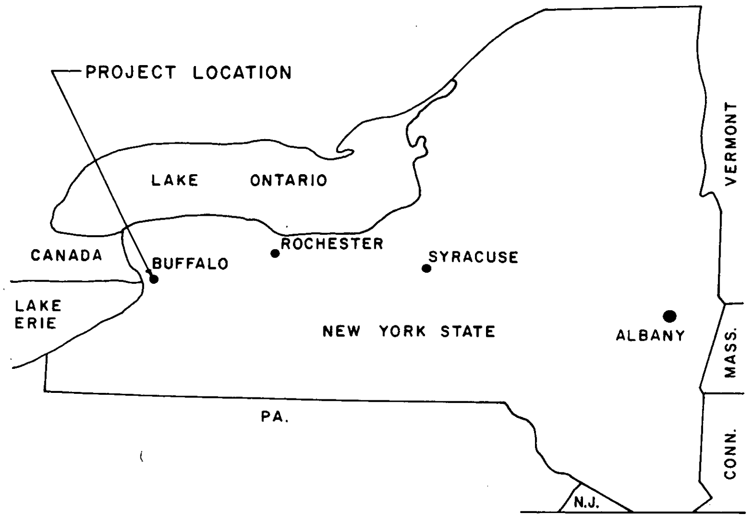


FIGURE 1 Project location.

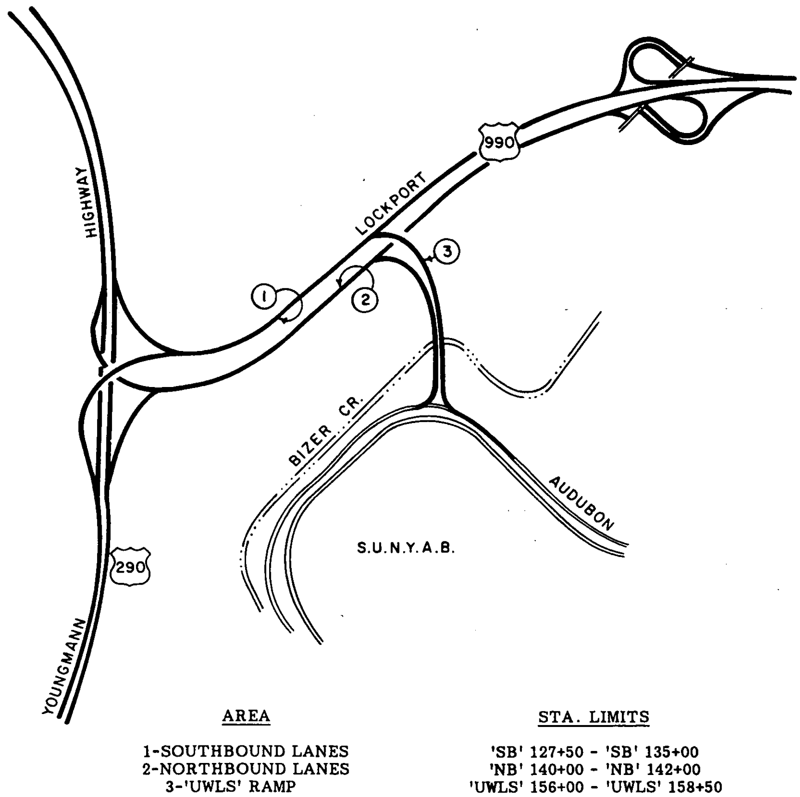


FIGURE 2 Project plan and location of problem areas.

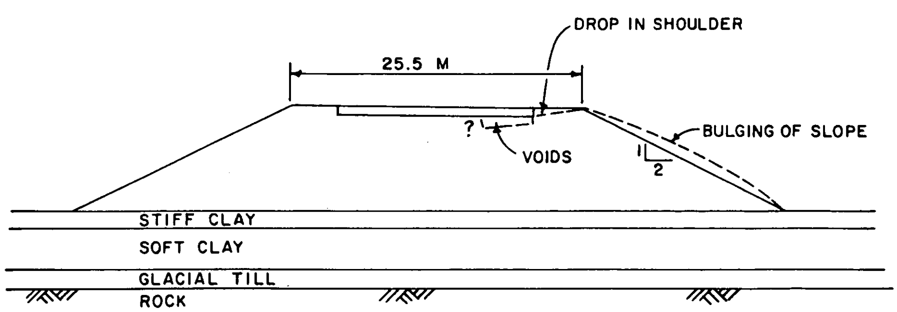


FIGURE 3 Embankment geometry showing embankment distress.

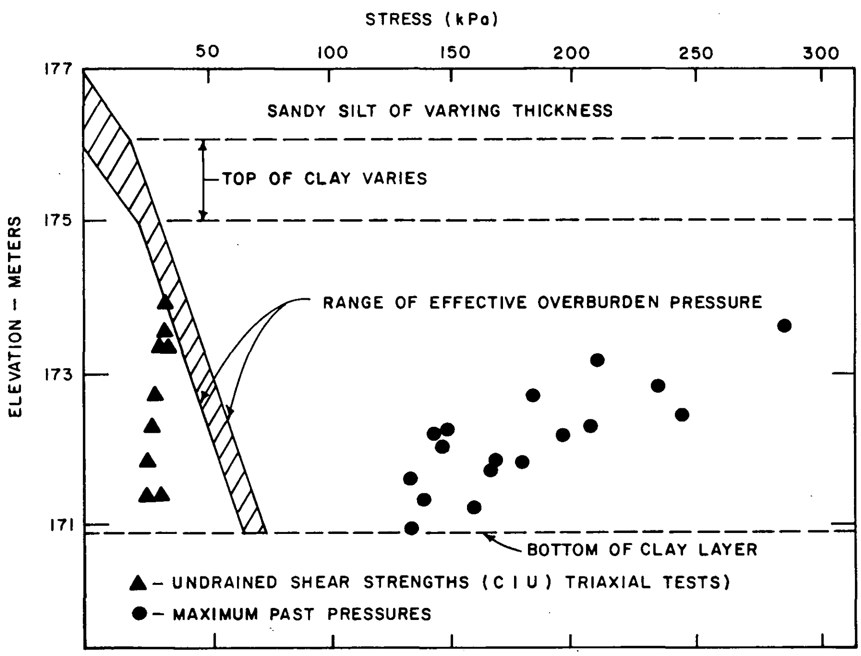


FIGURE 4 Stress history and undrained shear strengths of foundation soils.

The moisture content of the borrow ranged from 25 to 55 percent and the plasticity index ranged from 15 to 32 percent. The optimum moisture content at the standard Proctor compactive effort was determined to be ±20 percent (Figure 5). As indicated, the natural moisture content of the material from these excavations was wetter than optimum.

Earthwork Construction Method

Where necessary a working platform of granular material was established above the original ground surface up to 1 m (3 ft) in depth. The wet silty clay excavated from the borrow sources was placed in approximately 0.23-m (9-in.) lifts and then aerated with discs and tractors for 2 to 3 days, depending on weather conditions. The material was then compacted using a sheepsfoot roller until the roller effectively walked out of the lift with an approximate 76.2-mm (3-in.) rebound behind the roller. Compaction tests were performed for the lifts to be approved.

INVESTIGATION OF FAILURES

Description of Problem Areas

Several months after project embankments were completed and pavements were in place, three embankment areas (Figure 2) began experiencing unexplained movements (Figure 3). Eventually all embankments greater than 4.6 m (15 ft) in height would experience similar movements. The southbound (Area 1) and northbound (Area 2) lanes of the project mainline were paved with concrete, whereas the third, designated the UWLS ramp (Area 3), was paved with asphalt. The fill heights of the southbound lanes, northbound lanes, and UWLS ramp were 7.6 m (25 ft), 6.1 m (20 ft), and 9.2 m (30 ft), respectively. During the design phase of this project it was determined that the foundation clay was not capable of supporting the UWLS embankment for its full height (*2*). Stabilizing berms were selected as the most cost-effective way of preventing embankment foundation failure. The berms were 2.4 m (8 ft) high and 22.9 m (75 ft) wide and placed on both sides of the embankment.

On one side of the southbound lanes it was noticed that a drop of less than 25 mm (1 in.) had occurred in the asphalt shoulder relative to the concrete pavement. No bulging of the side slopes was noticeable. It could not be determined visually whether this movement was due to the foundation soils or to the materials used in constructing the embankments. By coincidence, an instrumentation program for monitoring embankment foundation performance, initiated during the project design phase, was in place at this particular location. The instrumentation consisted of piezometers, settlement gauges, and inclinometers. The information obtained from this instrumentation indicated that the foundation soils had completely consolidated and gained strength under the weight of the embankment.

The northbound lanes had experienced movements similar to those on the southbound lanes; however, they were more pronounced. The asphalt shoulder had dropped vertically 51 mm (2 in.) to 76 mm (3 in.). A separation between the shoulder and concrete pavement was measured to be 25 mm (1 in.).

Of the three embankments, the movements that had occurred on the UWLS ramp were the most visually dramatic. In October of 1982, a crack 30.5 m (100 ft) long was noted along the centerline of the ramp (Figure 6) and a small bulge existed along the bottom portion of the embankment slope above the berm. The initial crack was approximately 25 mm (1 in.) wide and 77 mm (3 in.) deep. By February 1983, the crack was 61.0 m (200 ft) long and the asphalt pavement had dropped vertically 0.8 m (2.5 ft) (Figure 7). By this time, a large bulge was apparent in the slope.

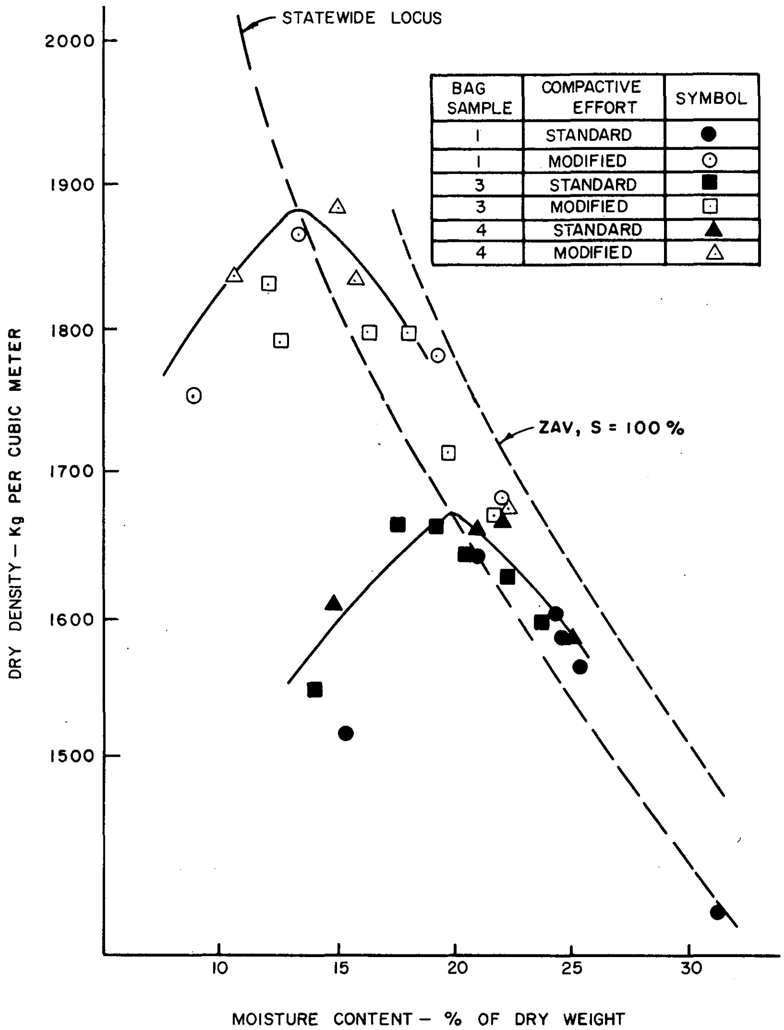


FIGURE 5 Dry density versus moisture content for bag samples.

Investigation Program

The investigation program of the distressed areas consisted of the following:

- Laboratory classification, consolidation, and triaxial compression tests performed on undisturbed Shelby tube samples of the foundation soils.
- Laboratory classification, density, moisture content, and triaxial compression (unconsolidated-undrained) tests performed on brass liner samples of the embankment materials taken after the failures. (It was difficult to press Shelby tubes through the embankment materials, therefore a split barrel-driven sampler having a thin-wall, 76.2-mm (3-in.) diameter brass liner was used to obtain representative samples. After sampling, the liner was sealed in a manner similar to a Shelby tube: capped with plastic caps, coated over with wax, and sent directly to the laboratory.)
- Comparison studies of standard and modified laboratory compaction with unconsolidated-undrained triaxial compression tests performed on bag samples of the silty clay embankment materials taken after the failures.
- The results of field density tests performed during construction of the embankments before the failures.
- The results of several inclinometers installed to measure horizontal movements of the embankments and foundation soils both before and after the failures.
- Optical surveys to detect both vertical and horizontal movements of embankment pavements and slopes after the failures.
- Visual observations made during several field inspections after the failures.

RESULTS OF INVESTIGATION—VERIFICATION OF CAUSE OF MOVEMENTS

Inclinometer Borings and Optical Surveys

Inclinometers were installed at the problem sites to determine the location of the zones of movement. At the mainline locations (northbound and southbound lanes) the inclinometers were progressed through the fills into the foundation soils and at the toe of slope. The inclinometer at the UWLS ramp was progressed at the

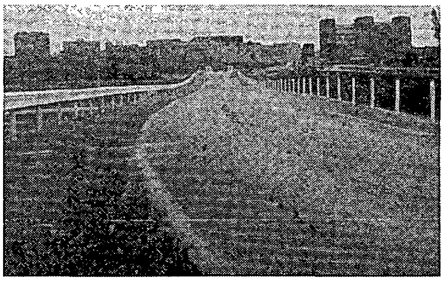


FIGURE 6 Crack developing on UWLS ramp, October 1983.

FIGURE 7 Drops of 2 to 3 ft on one side of UWLS ramp, February 1983.

toe of the ramp slope, through the berm and into the foundation soil. Figure 8 shows the results of the inclinometer monitoring. The data indicate that the movements were all occurring within the embankments. The inclinometer progressed through the berm at the toe of the UWLS ramp slope indicated no movement.

A series of optical survey points were established on the pavement and along the side slope at the UWLS ramp site. The edge of asphalt pavement dropped at a relatively uniform rate (102 mm per month) since the start of the optical survey in mid-October. During this period, the slope moved out horizontally 0.3 m (1 ft). No significant vertical movements were detected on hubs located on top of the berm.

Strength of Fill Material

Laboratory Testing

Bag samples of the clay material used to construct the embankments were obtained to perform laboratory compaction and strength tests. The samples were compacted under both standard and modified compactive efforts, and the results are shown in Figure 5. At the standard compactive effort, the optimum moisture content was 20 percent and the maximum dry density was 1 681 kg/m^3 (105 pcf). An optimum moisture content of 13 percent and a maximum dry density of 1 874 kg/m^3 (117 pcf) was obtained for this material compacted at the modified effort.

On completion of the compaction tests, each sample was taken out of the compaction mold and full-diameter unconsolidated-undrained triaxial compression tests were performed at 100 percent saturation. A confining pressure of 172.5 kPa (25 psi) was applied to each sample. A relationship between moisture content and undrained shear strength was developed and is shown in Figure 9.

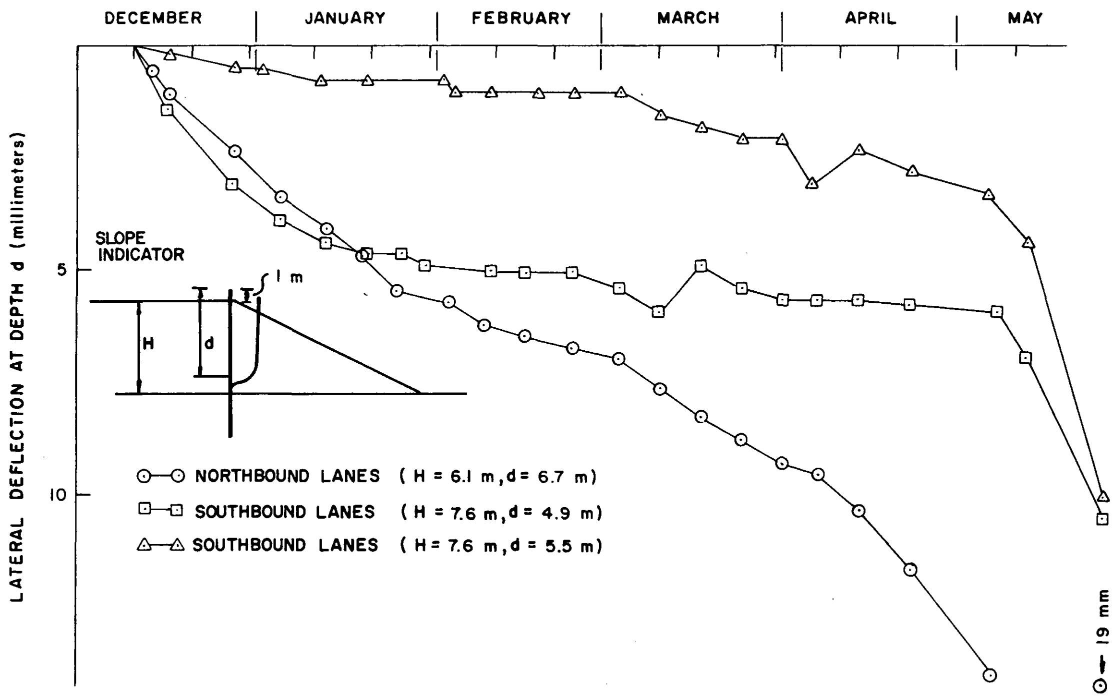


FIGURE 8 Slope indicator monitoring results within fill.

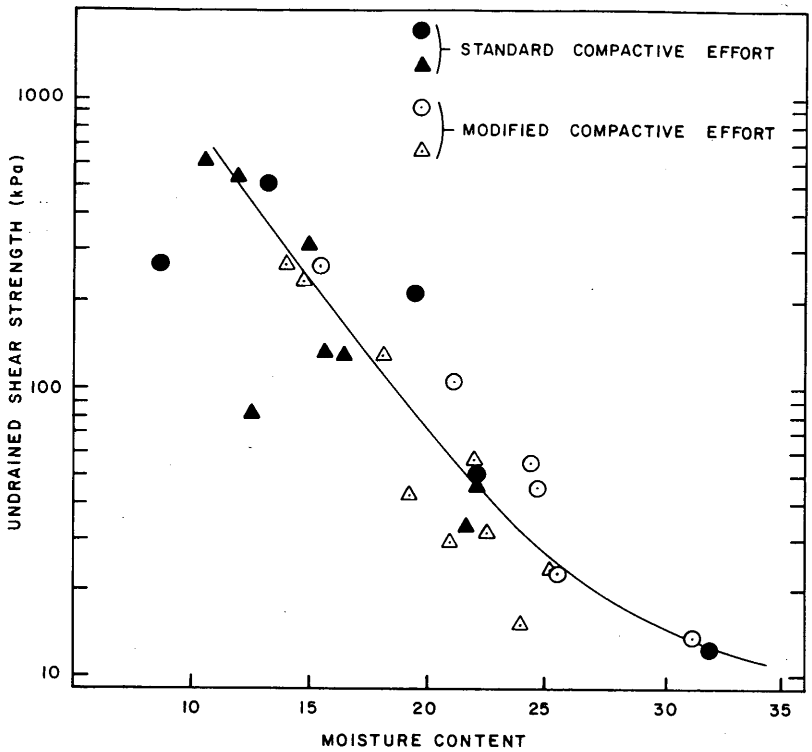


FIGURE 9 Undrained shear strength versus moisture content for bag samples.

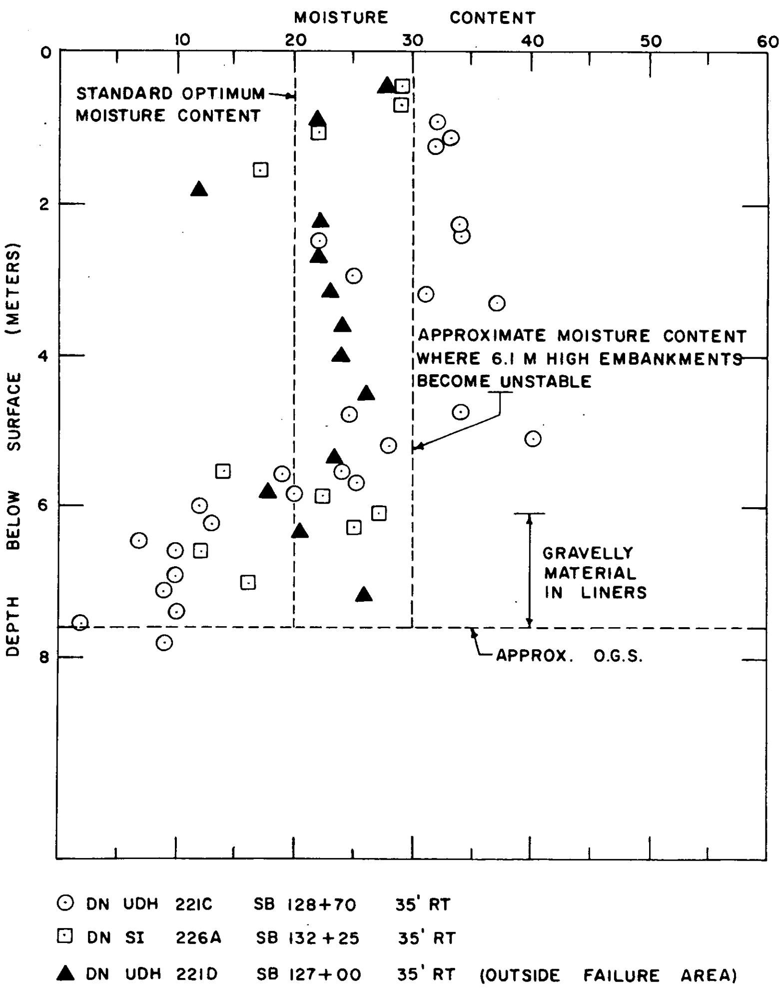


FIGURE 10 Depth versus moisture content for southbound lanes.

As can be seen, strengths as low as 14.4 kPa (300 psf) were obtained at a ±30 percent moisture content. It made no significant difference whether the samples were compacted at the standard or the modified effort.

Density, moisture content, and strength determinations were made on the brass liner samples of the embankment material obtained from borings progressed through the problem embankments. Moisture content profiles are shown in Figures 10–12. Density versus moisture content relationships are shown in Figure 13. As can be seen, the densities of the embankment samples were determined to be above the minimum required 90 percent standard Proctor density and the moisture contents were considerably wetter than optimum. The relationship between moisture content and undrained shear strength is shown in Figure 14. As indicated, the shear strengths obtained from tests performed on the brass liner samples of the embankments were generally higher than those for the bag samples. An explanation for this is that most of the material in the embankments had a chance to gain strength after placement. The sampling procedure may also have had an effect on the strengths obtained. The bag samples, on the other hand, were tested soon after they were compacted and thus were not allowed to gain strength with time (i.e., no thixotropic strength gain). The bag sample shear strengths are more typical of the shear strengths of embankments experiencing movement. This is true because along the active plane of movement, within the embankment, the soil has not had a chance to gain strength because of its constant movement and remolding action. This view is in agreement with other investigators (*3–6*) who have performed research on the strength characteristics of compacted clay embankments. Therefore, shear strengths obtained from the bag samples were used in analyzing the stability of the embankments.

Field Testing

The results of recorded field density tests in which moisture contents were obtained during construction are shown in Figure 13. The densities obtained were all above the minimum required 90 percent standard density, and the majority of the moisture contents were at or below the optimum moisture content of 20 percent as determined in the laboratory. The moisture contents determined during construction of the embankments are considerably lower than the moisture contents determined after construction. Some possible reasons for this are

- Field density test time and location (i.e., selective testing);
- Infiltration of water into previously accepted embankment lifts;
- Material placed wetter than optimum; and
- Sampling disturbance.

Strength of Fill Used in Stability Analyses

The engineering behavior of compacted cohesive soils is affected by moisture content, degree of saturation, method of compaction,

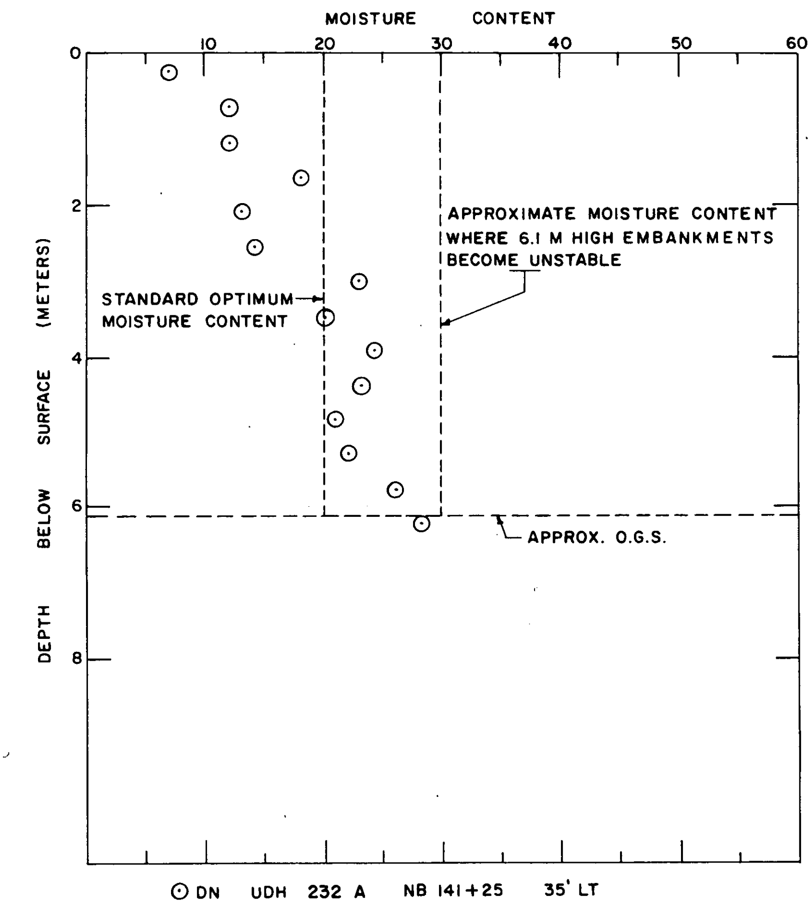


FIGURE 11 **Depth versus moisture content for northbound lanes.**

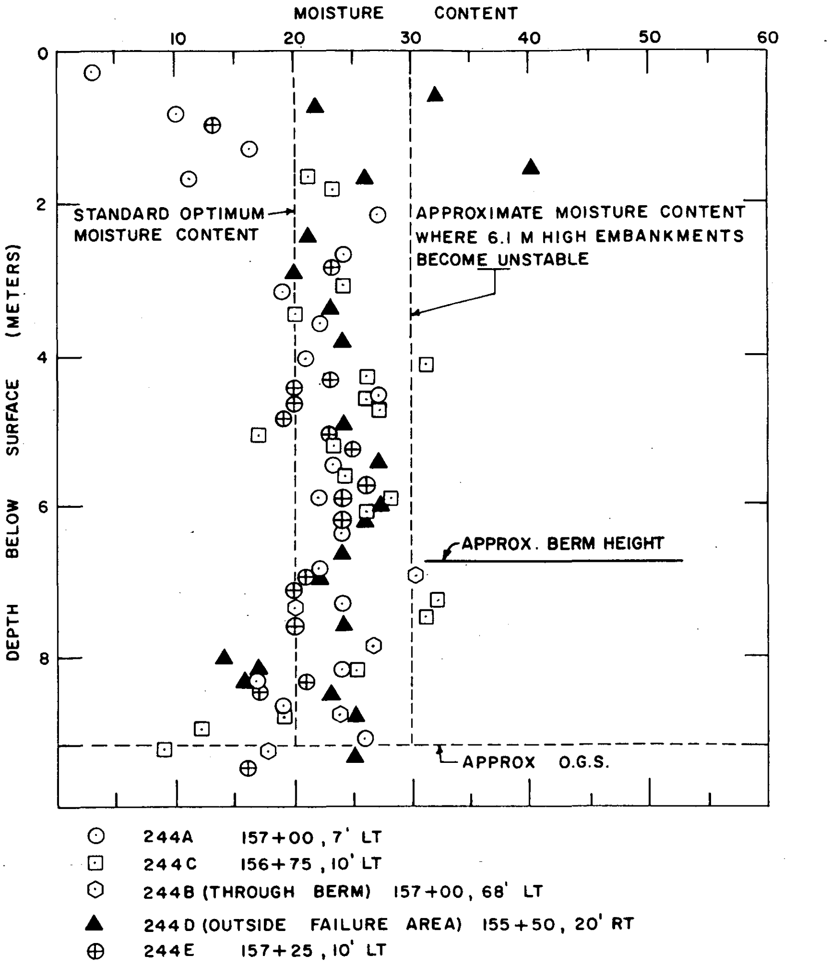


FIGURE 12 Depth versus moisture content for UWLS ramp.

and compactive effort. Of all these factors, moisture content appears to play the most important role in the strength of the soil, as indicated from the plot of moisture content versus shear strength shown in Figure 9.

In addition, a distinction must be made between long-term stability and short-term stability, as this affects the selection of soil strength parameters used in the analyses of these embankment failures. Long-term failure starts to take place a relatively long period of time (perhaps years) after completion of embankment construction. Failure usually takes place close to the surface of the slope [less than 2.4 m (8 ft.)] deep as a result of loss of shear strength as the soil swells and takes on water to reach a point of equilibrium with respect to the elements. Shear cracks usually first show up in the slope or outer portions of the shoulder. Long-term drained soil strength parameters are used in the stability analyses of embankments failing in this manner.

In the short-term case, failure starts to take place deep within the embankment (verified by inclinometers) and within a short time (days or months) after completion of embankment construction. Failure is associated with a deep outward squeezing of soil from within the lower portions of the embankments due to the overlying loads from the upper portions of the embankments. Because the movements are initially deep within an embankment, the movement cracks first show up in the shoulder or pavement. Because the movements exhibited in the embankments were consistent with what would be expected for the short-term case, undrained soil strength parameters were used in all stability analyses.

Figures 10–12 show the moisture content profiles found to exist in borings progressed through the roadway embankments. These figures indicate that moisture contents approached or exceeded 30 percent at the time of failure in these embankments. Using these moisture content profiles, undrained shear strengths were selected for analysis from the relationship of moisture content versus shear strength obtained from Figure 9. From this relationship a shear strength of 14.4 kPa (300 psf) was used for analysis in the lower portions of the compacted embankments.

Stability Analyses

Slope stability analyses were performed to equate safe fill heights to shear strength using the simplified Bishop analysis. A fill height of 6.1 m (20 ft) was used for the problem embankments for the following reasons:

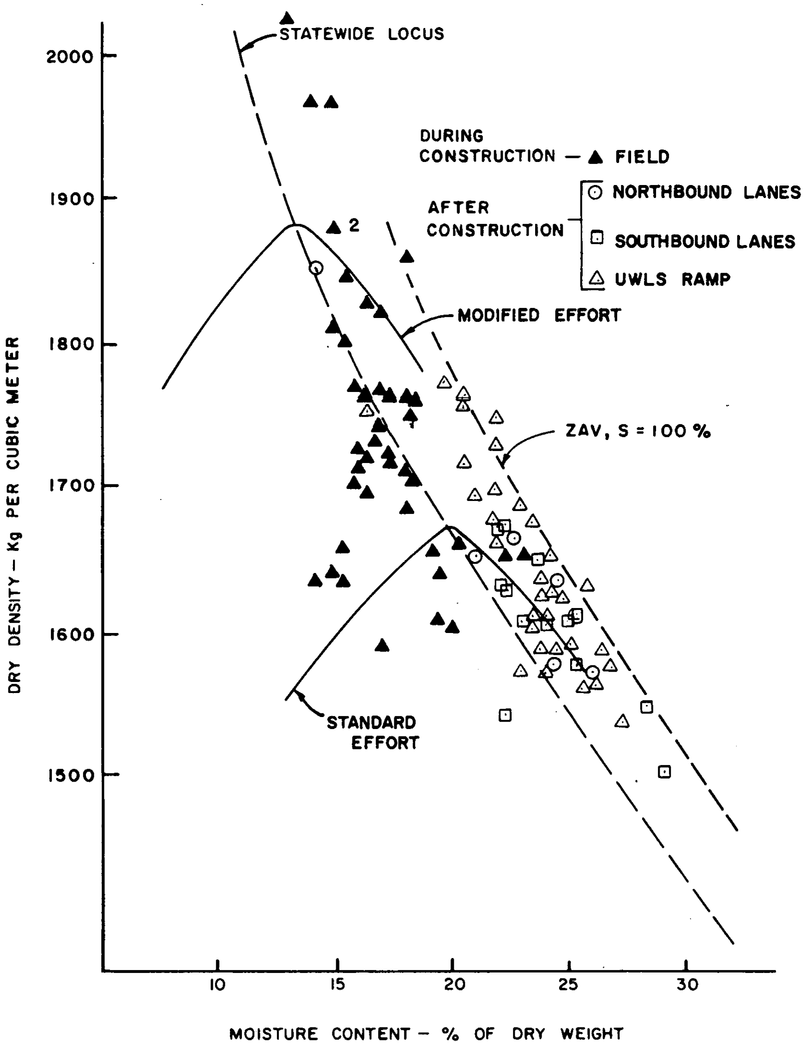


FIGURE 13 Dry density versus moisture content for field and laboratory testing.

- The bottom 1.5 m (5 ft) of the 7.6-m (25-ft) high southbound embankment contained granular material.
- The fill height was actually 6.1 m (20 ft) for the northbound embankment, and the maximum moisture content occurred at the bottom.
- The fill height above the berm was 6.1 m (20 ft) for the UWLS ramp.

When incorporated with the results of laboratory testing, the results of these analyses show that embankments constructed of this material possessing an undrained shear strength of 14.4 kPa (300+ psf) can be built safely [factor of safety (F.S.) = 1.25] to a height of approximately 4.6 m (15 ft) and that the effective fill heights of 6.1 m (20 ft) in these areas run a high risk of failure, as actually happened. Figure 15 shows the results of a stability analysis performed on the UWLS ramp. An undrained shear strength of 26.3 kPa (550 psf) would have been required in order to ensure short-term stability of the embankment. Based on the relationship developed in Figure 9, the undrained shear strength required for stability (F.S. = 1.25) corresponds to a moisture content of 25 percent. The moisture content found to exist in the lower portion of the fill (above the berm) approached 29 percent, corresponding to an undrained shear strength of 19.2 kPa (400+ psf).

REMEDIAL TREATMENTS

Recommendations were made to stabilize all the project embankments that were greater than 4.6 m (15 ft) high. Stabilization consisted of flattening the embankment side slopes because material was readily available and right-of-way was not a problem. Based on an average fill height of 6.1 m (20 ft), a required F.S. of 1.25, and an undrained shear strength of 14.4 kPa (300 psf), the slopes were flattened to 1 (vertical) on 3.5 (horizontal).

CONCLUSIONS

The results of the investigations into the embankment failures that occurred on this project revealed that the moisture contents in the fills exceeded the optimum moisture content at the standard effort

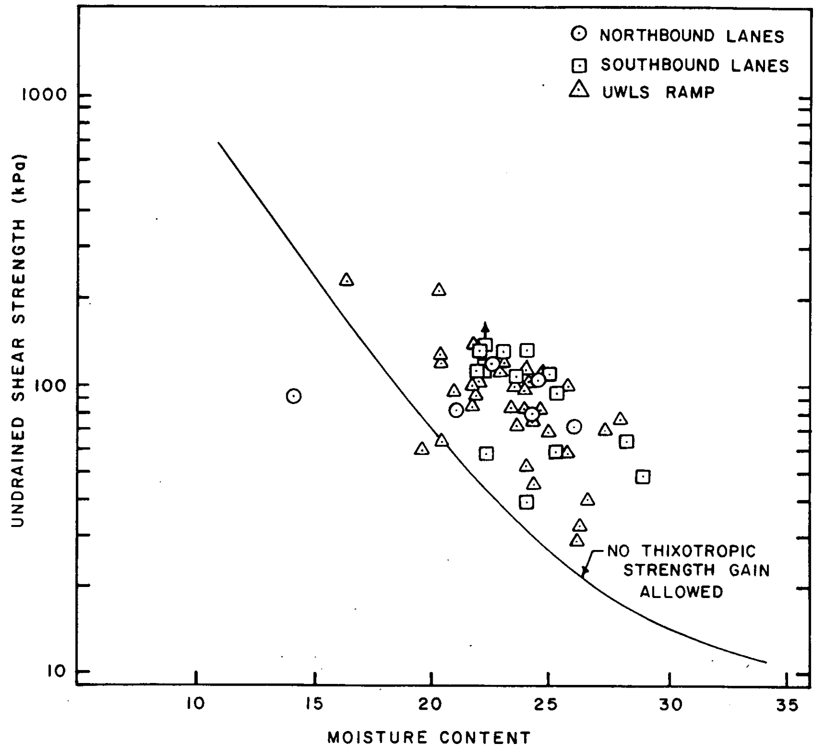
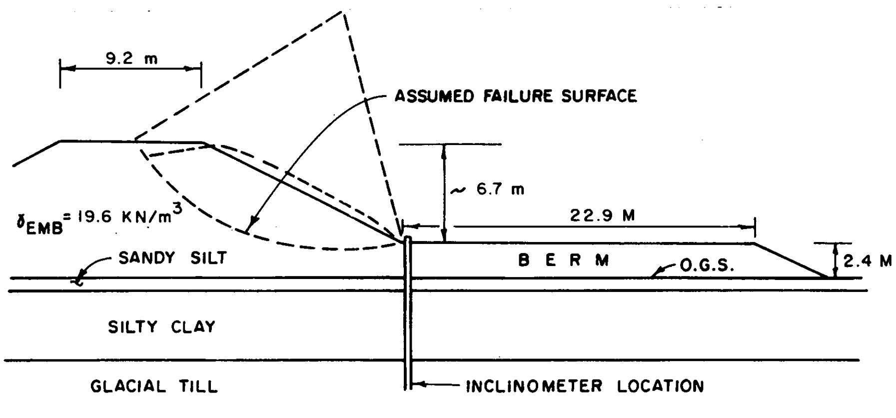


FIGURE 14 Undrained shear strength versus moisture content for brass liner samples.



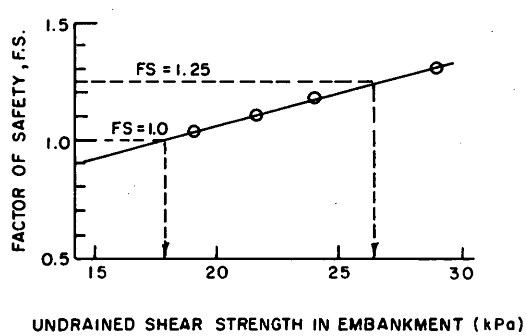


FIGURE 15 Stability analyses for UWLS ramp.

by as much as 10 percent. These moisture contents corresponded to undrained shear strengths deemed inadequate in ensuring stable embankments greater than 4.6 m (15 ft) high.

Moisture control would appear to be the answer if it were not for the fact that construction records showed that for the most part the fill met the density and moisture content requirements for the particular soil used. The difference between the construction records and post-failure results could not be adequately explained.

Since these high-plasticity clays are not used all that often in embankments of any great height, perhaps it would be better to assume that moisture contents in completed fills will exceed those desired by a certain percent and then design accordingly.

EMBANKMENT DESIGN IN NEW YORK STATE

The New York State Soil Mechanics Bureau seldom designs, in the strict sense, highway embankments. It has been the position of the bureau that strict adherence to the standard specification requirements will produce internally stable embankments constructed with side slopes of 1 vertical on 2 horizontal. For the most part this approach has worked quite well. Experiences in recent years have caused the bureau to modify its operating procedure concerning the use of low- to high-plasticity soils as embankment construction material. Embankments that are to be constructed 4.6 m (15 ft) or higher and have a strong probability of being constructed out of plastic soils are now more closely evaluated and, if deemed necessary, designed to prevent problems during construction. Provided right-of-way is not a problem, flattening the embankment side slopes is a common approach. Other methods might be the inclusion of geosynthetic reinforcement, stabilization of embankment soil with chemicals, or zoned construction (placement of granular materials in selected areas of the embankment).

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