LANDSLIDE INSTRUMENTATION FOR THE MINNEAPOLIS FREEWAY

Stanley D. Wilson, Shannon and Wilson, Inc., Seattle

In 1967 a landslide developed along a section of freeway under construction in Minneapolis. The following procedure was undertaken to ensure the stabilization of the landslide and completion of the project: Instruments were installed to detect the depth and rate of movement, and exploration was undertaken to determine the properties of the material in the failure zone; a temporary buttress was placed to control the movements while corrective treatment was being designed; additional instruments were installed to monitor movements during construction of the permanent treatment; and, after completion, all instruments were maintained and additional instruments were installed to monitor post-construction movements, if any. The corrective treatment consisted of a series of slit-trench concrete buttresses anchored into limestone below the failure plane. Details of exploration, instrumentation, testing, and design are included in the paper.

•FIELD instrumentation has become a working tool of the civil engineer faced with the problem of designing corrective treatment to control a landslide. The case history of the landslide described in this paper is a classic example of the essential role of instrumentation in all stages of investigation, design, construction, and post-construction monitoring.

No two landslides are identical in all respects. Likewise, no two installations of field instrumentation need be identical. Nevertheless, there are certain things that one needs to know about each landslide, e.g., the depth of sliding, the rate of movement, the distribution of the movements, the pore water pressure (or lack of it), and the properties of the material in the movement zone.

In 1967 a landslide developed along a section of freeway under construction in Minneapolis. The lines and grades were already established by new overpasses at each end, the materials appeared to be competent, and the design slopes were conservative. It was essential that adjacent property owners be protected and the freeway project be completed on a timely basis. Therefore, the following orderly sequence of events was initiated to bring about the stabilization of the landslide and completion of the project:

1. Instruments were installed to detect the depth and rate of movement, and explorations were undertaken to determine the properties of the materials in the failure zone.
2. A temporary buttress was placed to control the movements while corrective treatment was being designed.
3. Additional instruments were installed to monitor movements during construction of the permanent treatment.
4. After completion, all instruments were maintained, and additional instruments were installed to monitor post-construction movements, if any.

BACKGROUND AND SITE INFORMATION

Location and Description

The landslide occurred along a 1,100-ft (330-m) section of Interstate Highway 94 on the east side of Minneapolis (Fig. 1) bounded on the northwest by a railroad bridge and on the southwest by the Franklin Avenue Bridge.

At this site Interstate 94 was being constructed in a cut, and all of the previously existing streets and the railway crossed the highway on overpasses at close to their original grades. In the vicinity of the site, the original ground slopes to the west on an average grade generally flatter than 6 horizontal to 1 vertical. This gentle westward slope extends for approximately another ½ mile (0.8 km) to the east side of the Mississippi River. The area located immediately uphill from the right-of-way line is developed for single-family dwellings and a city park.

There were no significant stability problems in the general area prior to the start of construction, although some groundwater problems were encountered during the excavation of footings for bridges and retaining walls.

Construction Activities

To depress Interstate 94, it was necessary to make cuts more than 50 ft (15 m) deep. Most of the structural work was completed in the area prior to the start of roadway excavation. The railroad bridge adjacent to Prospect Park was essentially complete by June 19, 1967, and was founded on excavated shafts bearing on limestone. The Franklin Avenue Bridge and the other existing retaining walls in the area are founded on driven steel H piles.

The roadway excavation was started on June 19, 1967. By August 1 the eastbound roadway was complete and excavation was started on the westbound roadway, beginning at the Franklin Avenue Bridge. However, on August 11 all construction and excavation in the area were stopped because of the upheaval and cracking noticed on and below the cut slope in the vicinity of station 167 (Fig. 2). For approximately one month, all construction in the area was shut down while it was decided what course of action was to be taken.

During August the movements continued with increasing development of offsets and scarps upslope. The decision was made by the Minnesota Highway Department to engage a consulting firm with experience in both soil mechanics and instrumention, and the firm of Shannon and Wilson, Inc., was selected. The first site inspection was made on August 30, and a detailed site investigation, including installation of field instruments, was begun on September 2.

Within a few weeks the depth of the movements was identified and, although a minor amount of excavation was resumed in mid-September, the previously excavated subcut was subsequently backfilled with compacted granular base material. During late October, some slope flattening was accomplished below Prospect Park and to the southeast as far as station 170. This was done in an attempt to reduce the amount of earth movement that was occurring in this area. During the early part of November a perforated drain and storm sewer was installed along the toe of the slope to provide drainage for that area prior to the winter freeze.

By early November it was determined that enough information had been obtained to proceed with detailed engineering studies. Also, it was apparent that the earth movement was continuing even with the previously accomplished slope flattening. Therefore, in mid-November a sand berm was constructed about 5 ft thick and 40 ft wide (2 by 12 m) extending along the toe of the slope for most of the distance between the railroad bridge and the Franklin Avenue Bridge. Subsequent field observations showed conclusively that all of the earth movement in the area was essentially stopped by the berm by December 1, 1967.

General Geology

The Twin Cities of Minneapolis and St. Paul lie in a shallow basin intersected by the Mississippi River, which has cut a substantial valley into the underlying sedimentary rocks. In general, this is an area of gently rolling glacial topography dotted with numerous lakes. This glacial mantle consists primarily of tills and outwash materials. The sedimentary rocks that underlie the recent glacial deposits are of the Ordovician Period of the Paleozoic Era.

During this investigation, two of the sedimentary formations were encountered. The upper of these is the Decorah formation. In general, this is a blocky, blue-green to blue-gray shale with occasional limestone beds irregularly distributed throughout. In Minnesota, this formation varies in thickness from 20 to 80 ft (6 to 25 m). This formation is underlain by the Platteville formation, which varies from a light-gray to gray limestone to a calcitic dolomite. For all practical purposes, the rocks within the project are are flat lying. In addition, the limestones are of very good quality, with little evidence of jointing, faulting, or groundwater solution.

FIELD INVESTIGATION AND SUBSURFACE INSTRUMENTATION

The highway department had accomplished numerous borings in the area around Prospect Park prior to construction, including auger borings without samples and rotary borings where "undisturbed" samples were obtained using a Denison sampler. However, sampling was particularly difficult because of the presence of limestone stringers in the Decorah shale. Following the initial site inspection on August 30, an expanded field investigation was recommended that included additional borings, instrumentation, and other field explorations. Initially, six borings with Slope Indicator observation wells were recommended. Subsequently, the state drilled four additional borings in which Slope Indicators were installed and five additional holes with piezometers. Later, as a result of the field measurements, the investigation was expanded to include large-scale torsion shear tests and a program of test pits.

Borings

A well-planned program of exploratory borings provides a reasonably efficient and economic means of studying the subsurface conditions at a particular location. Specifically for this stability study, 15 borings were made; the depth of boring depended on its location and surface elevation. In addition, borings were located to serve the dual purpose of providing subsurface soil information as well as a hole for the installation of field instrumentation. Two sections in the vicinity of the slide were instrumented rather extensively.

An attempt was made to obtain good quality, undisturbed soil samples by using a Pitcher sampler. With this tool the sample is recovered in a 3-in. (76-mm) diameter thin-walled tube that is pushed behind the cutting edge. It was found that this sampler was able to cut through the thin limestone lenses with relatively little difficulty. In general, good sample recovery was obtained in the Decorah shale. However, it was not possible to obtain good samples of the material sandwiched between thin lenses of limestone. After recovery the tube samples were sealed with wax to preserve their natural moisture content and were then shipped to the laboratory for classification and testing.

Subsurface Instrumentation

The type of field instrumentation selected depends on the problem. In this instance it was obvious that movements of some type were taking place below the elevation of the subgrade, but it was not known whether these movements were at the shale-limestone interface or at some intermediate level, whether they were along a horizontal bed or a circular arc, whether they were uniformly distributed or progressive, and whether they were induced by high pore pressures or by a weak interbed.

An inclinometer was chosen because it is the best instrument to detect the depth, rate, and distribution of horizontal movements below the surface. Because of its proven record of performance on similar projects, and because of the immediate availability of both the sensing instrument and the casing, Slope Indicators were selected. This instrument is lowered down a grooved aluminum casing that controls the orientation of the instrument in a predetermined direction. Inclination readings are taken at frequent intervals of depth and are subsequently converted to displacement. Consecutive readings at predetermined depth intervals and at periodic time intervals are used to determine the depth and rate of ground movement. The 3-in. (76-mm) diameter Slope Indicator casing was installed by lowering it into a 6-in. (152-mm) boring, which was kept open with the use of drilling mud. The open space between the walls of the boring and the outside of the aluminum casing was subsequently filled with a weak cement grout. Initially readings were taken daily but later the reading interval was spread out to approximately three per week. Over the winter and after the remedial construction was accomplished, the reading intervals were further stretched out to about one per month.

Several of the Slope Indicators detected movement within a few days, and within a month it was apparent that slope failure was occurring as a result of stretching along a near horizontal plane at elevation 798 ± 1.0 ft. Figure 3 shows the geologic profile and the detailed movements as recorded by one of the Slope Indicators, and Figure 4 shows the distribution of these movements along a cross section, as well as the location of the failure plane.

Between September 1967 and January 1968, movements were as shown in Figure 4. The data indicated that the slide was of a progressive nature and that further upslope deterioration might be expected. Since the movements were not slowing down but on the contrary were increasing, a temporary buttress of sand and gravel was placed at the toe of the slope to stop the movements while remedial measures were studied. Figure 5 shows how the sand berm placement caused the movements to stop almost immediately at three of the Slope Indicators.

To evaluate the groundwater conditions on the site, seven Casagrande-type piezometers were installed. The two initial piezometers were installed at a relatively high elevation in the glacial drift material and in the upper zone of the underlying shale. The other five piezometers were installed so that the tips were set in the failure zone material at about elevation 798, and periodic observations were made to detect any change in water level with time. The data obtained during two months of observation showed that there were several perched water tables but none with high uplift pressures.

Water levels also were observed in the ten Slope Indicator casings. However, this was not always a reliable means of groundwater observation because grout may tend to seal water either in or out of the aluminum well casing.

The data from the Slope Indicator wells demonstrated conclusively that slope failure was occurring along a nearly horizontal plane only 10 to 15 ft (3.3 to 4.5 m) below the bottom of subcut elevation, and, since the borings had failed to detect any unusual material at this depth, it was decided that a test pit program would be useful in observing the materials along the failure surface.

Eight test pits were dug, varying in depth from 10 to 18 ft (3.3 to 5.5 m). During test pit inspection, it was possible to observe, sample, and test the soil within the failure zone. This was found to be a thin seam, averaging less than 1 in. (2.5 cm) in thickness, of a sodium bentonite. This material had been removed by the wash water during sampling and therefore had not been detected previously. Pocket penetrometer tests were conducted in most of the test pits to determine the in-place shear strength. In addition, Torvane [a 1-in. (2.5-cm) diameter hand-operated vane shear test apparatus designed for in situ testing of soft to medium stiff clays] and in-place torsion shear tests were conducted in one of the test pits.

The first test was conducted on a saturated shale at elevation 800. The second test was run on a moist shale at elevation 804. An attempt was made to run a third test in the soft, bentonitic clays at elevation 798, but because of adverse water conditions and difficulty in preparing the working surface it was not possible to complete this test.

LABORATORY TESTING

The need for an extensive laboratory program was precluded because of the mechanisms of sliding and the character of the soils and rocks involved. Hence, the testing program as such was developed primarily to establish the basic index and engineering properties of the representative samples of the materials encountered. In this respect, water content, Atterberg limits, grain-size gradations, and strength properties were obtained on selected samples.

Laboratory strength characteristics were measured in triaxial compression tests that were performed under conditions closely resembling the in situ condition. Unconfined compression tests were also performed on representative samples of cohesive soils. In addition, several small-scale torsional shear tests were performed in the

Figure 1. Vicinity map.

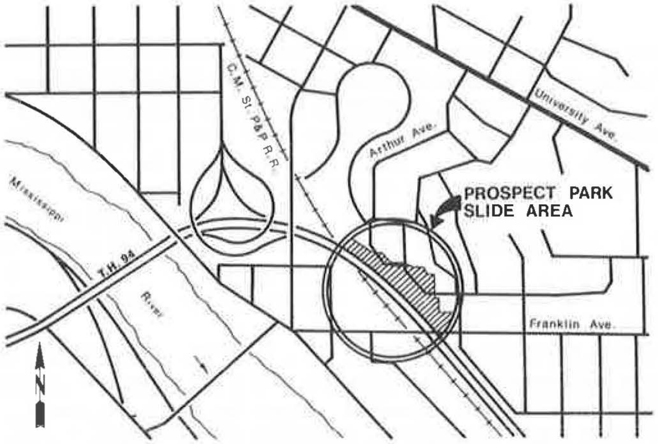
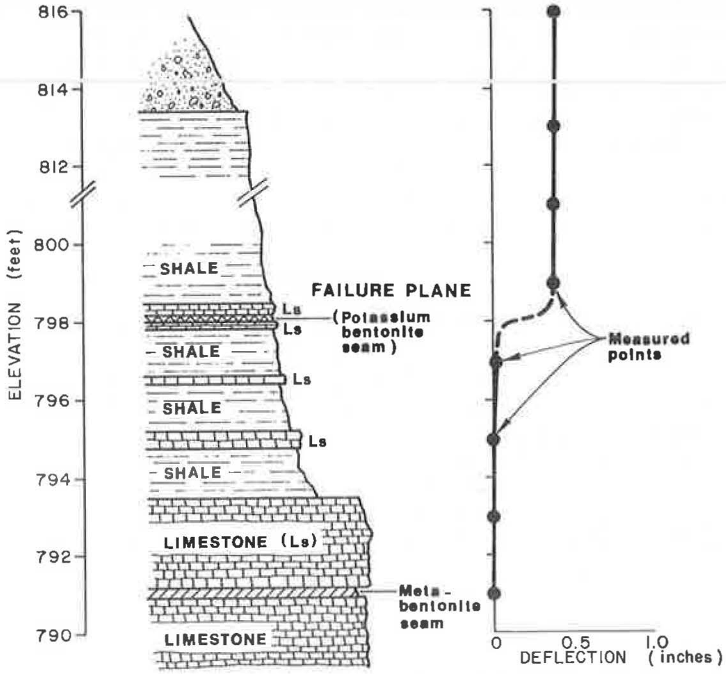


Figure 2. At Slope Indicator no. 4 looking toward railroad bridge and park.

Figure 3. Geologic profile and Slope Indicator observations at failure plane.



laboratory on undisturbed chunk samples obtained from the failure zone. The laboratory tests were designed to obtain an understanding of the shear strength properties of this clay, since knowledge of these properties was important to the design of the recommended remedial measures for this project.

All samples of the Decorah shale and the bentonitic clay were visually inspected and classified in the laboratory.

Atterberg limits were determined on representative samples, and the results showed most of the cohesive soils from the project area to be classified as clays of high plasticity (CH). The bentonitic clay from the failure plane had a liquid limit of 60 and fell 10 points above the A-line on the Casagrande plasticity chart.

In order to determine the shear strength properties of the bentonitic clay found at elevation 798, three small-scale laboratory torsion shear tests were conducted. The laboratory tests were performed in a torsion ring $3\frac{1}{2}$ in. (9 cm) in diameter, having a knurled or roughened face. Each specimen was cast in plaster of paris, fixed in a consolidation apparatus, and trimmed to expose a level test surface. A normal load was applied using dead weights on a loading frame. A rotational shearing force was then applied parallel to the failure surface through a system of cables, pulleys, and dead weights. The relationship between the normal stress and the shear strength was then determined using a procedure similar to that for the field shear test.

The results of the laboratory torsion shear tests are summarized in Figure 6. The results of numerous pocket penetrometer tests and Torvane tests conducted in the various test pits are also shown in Figure 6, as well as the average of results of six unconfined compression tests.

Four unconsolidated-undrained triaxial compression tests (R) were conducted on representative samples of the Decorah shale, and representative core samples from the Platteville formation were also tested. Visual inspection indicated that this latter material was a relatively sound rock with few joints or solution cavities. The results are not included here.

X-ray diffraction tests were conducted at the University of Minnesota on representative samples of the Decorah shale and clay. These tests are used to determine the clay mineralogy of various types of materials, and the results indicated that the shale located above the failure surface is composed almost entirely of illite. In contrast, the soft, gray-brown clays found at elevation 798 were found to contain approximately 50 percent illite and 50 percent mixed layers of montmorillonite clays. These soft clays appear to be similar to the potassium bentonites found in areas to the south of the Twin Cities. In any event, the clay in the failure zone is significantly different from that found either above or directly below the failure surface.

SUMMARY OF SUBSURFACE CONDITIONS

Generally, it was found that the subsurface conditions were quite uniform over the project area, although within various strata there were some complex or disturbed conditions. The subsurface soils are described in general terms in the following paragraphs.

Glacial Drift

Prior to excavation the entire area was mantled with a variable thickness of glacial drift. This material varies considerably in thickness and in consistency over the project area. Within the drift there is a considerable amount of brown, tan, or reddish-brown, silty, gravelly sand, which is believed to be glacial till. However, there are zones or lenses of relatively clean sand, gravel, and cobbles, which may represent outwash material associated with either the advance or retreat of the glacial ice. At various elevations within this project the drift is quite clayey, which might suggest that there were interglacial periods of lacustrine deposition.

Shale

Underlying the glacial drift is the Decorah formation. This formation generally

Figure 4. Typical section showing failure plane and movement distribution.

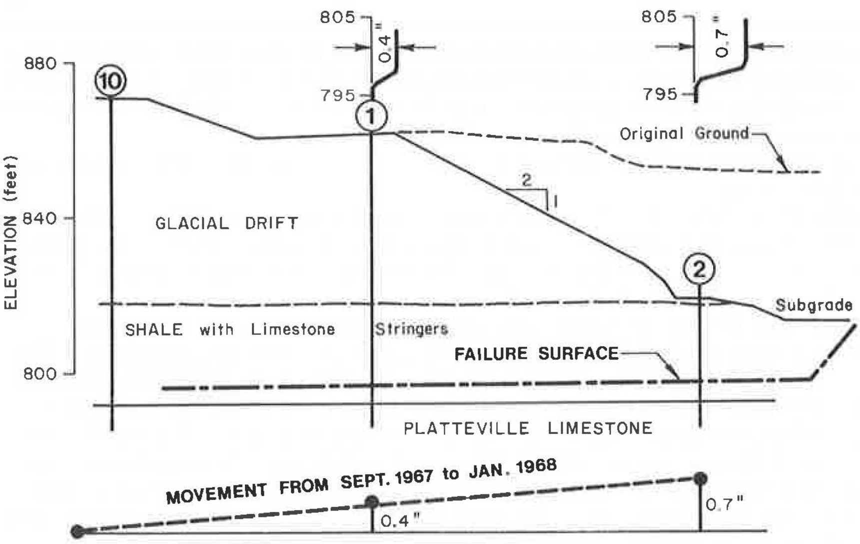


Figure 5. Slope Indicator dial changes with time.

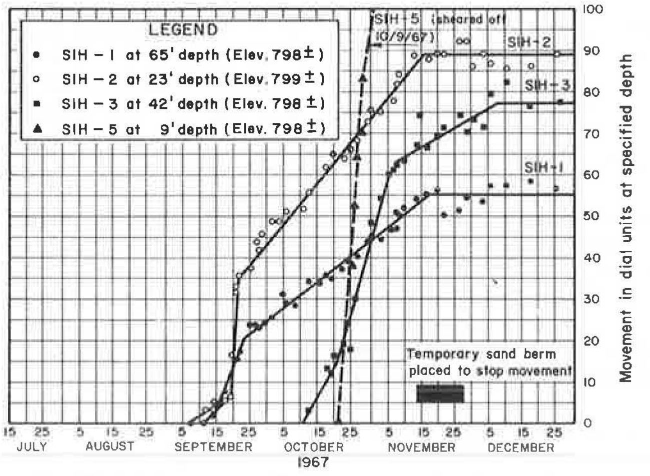
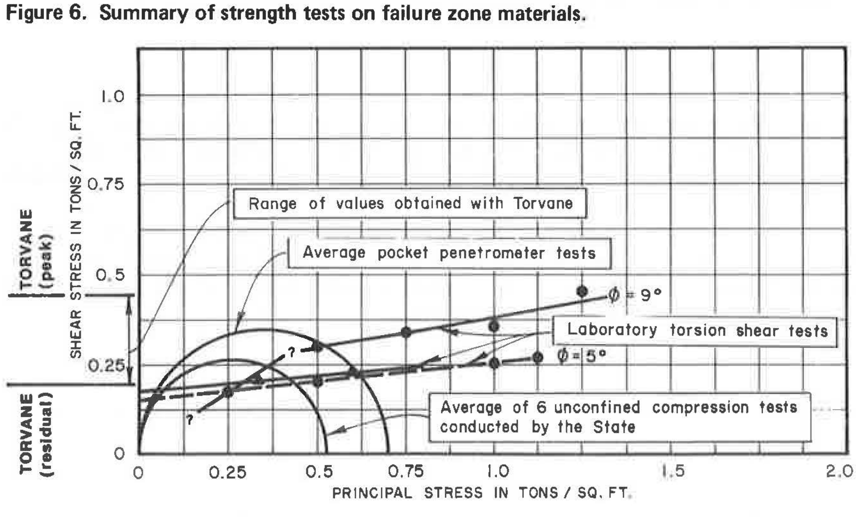


Figure 6. Summary of strength tests on failure zone materials.



consists of a gray-green, blocky, clay shale with thin interbedded (1 to 4-in., 2.5 to 10.0-cm) lime stone stringers. Locally, the formation appears to be quite disturbed. Within the project area, this formation is essentially horizontally bedded. The top elevation varies considerably because of glacial or pre-glacial erosion; however, the lower contact is found at elevation 793.

During the test pit program, a thin horizontal layer of soft, gray-brown, bentonitic clay was discovered in the Decorah shale at elevation 798. The clay was sandwiched between two of the thin limestone stringers. This material probably was not discovered earlier because of the difficulty in obtaining good undisturbed samples in borings under these conditions. As previously stated, the clay mineralogy of this material was found to be significantly different from that of the surrounding shales. It is believed that this bentonitic clay was originally deposited as a volcanic ash in a shallow sea.

Limestone

Underlying the Decorah shale is the Platteville formation. This consists of about 30 ft (9 m) of horizontally bedded gray limestones and dolomites. A thin bentonitic seam was also found in the Platteville formation. This was located approximately 2.5 ft (75 cm) below the top of the Platteville formation and was about 3 in. (7.5 cm) thick. Since the field instrumentation indicated that no movement was occurring in this zone or in the Platteville as a whole, this "metabentonite" zone was not extensively explored or tested.

Below the Platteville formation, sedimentary rocks extend to great depths, but these have no practical significance with regard to this investigation.

To some degree, groundwater was observed in most of the materials explored. Groundwater affecting the area of this study comes from two primary sources: surface runoff and deep aquifers from unknown sources. These water conditions undoubtedly influenced the stability of the cut slope, but relatively little preconstruction groundwater data were available. However, water levels in a few of the older borings and in an adjacent excavation indicate that the water table in the vicinity of the roadway centerline was considerably higher than the bottom elevation of the subcut. Therefore, the water trapped beneath the impermeable shale may have developed artesian pressures with relation to the excavated ground surface.

Primary emphasis in this investigation was directed toward an evaluation of the field, rather than the laboratory, properties of the soils and rocks encountered. An indiscriminate application of the laboratory strength properties in evaluating the stability problems might have been misleading. Because of the character of the materials that control the slope behavior and the mode of failure involved, the laboratory properties have served primarily as a means for relating quantitatively the soils existing throughout the site with those soils on which strength tests were performed. In addition, they have provided a basis for comparing the behavior of the Minneapolis materials with experience gained from other areas and with experience reported in the literature. Table 1 gives a summary of geology and engineering properties as used in subsequent failure analyses.

EVALUATION AND CORRECTIVE DESIGN

Slope stability computations were conducted on three representative cross sections where the limits of sliding were clearly defined by Slope Indicator data and by field observation of the heaving and cracking. In addition, the geology was more completely mapped at these locations.

Since the movement developed along a horizontal plane surface, the sliding wedge type of analysis was used. Hydrostatic pressures were assumed, based on available piezometric data. The location of the failure plane was determined from geologic observations and from the Slope Indicator data.

The parameters used for analyses, the general method of analysis, and the summary of results are shown in Figure 7. No seismic loadings were used in performing the stability analysis for this project.

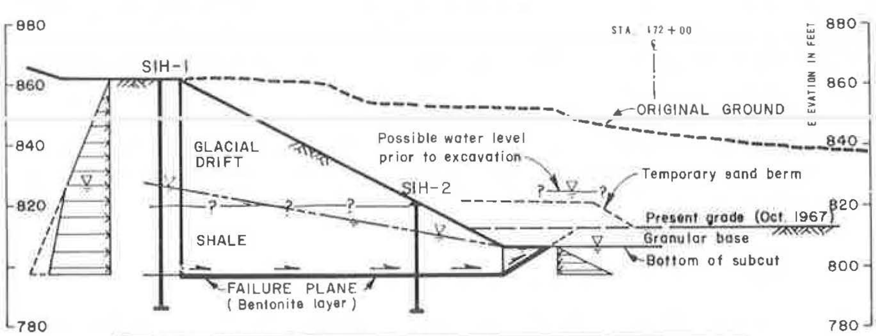
It was determined from testing that the strength of the bentonitic clay along the

Table 1. Description of materials encountered.

Material and Description	Properties	Testing Methods[a]
Recent deposits: Primarily glacial drift mantle (till and outwash materials); generally, brown, tan, or reddish-brown silty sand with varying quantities or lenses of gravel and cobbles	Unit weight (moist) = 130 pcf (2.0 kg/m^3) Shear strength ϕ = 35 deg c = 0	M, A
Decorah formation: Primarily a blocky, gray-green, clay shale with occasional thin limestone stringers	Unit weight (moist) = 130 pcf (2.0 kg/m^3) Shear strength ϕ = 9.5 deg (saturated) = 21.5 deg (moist) c = 0.22 tsf (0.22 kg/cm^2) (moist or saturated)	R, F, P, U
Failure zone: Within the Decorah at the elevation of the failure surface there is a 1- to 4-in. (2.5- to 10-cm) thick layer of soft, gray-brown clay (potassium bentonite)	Shear strength ϕ = 6 deg c = 0.2 tsf (0.2 kg/cm^2) residual = 0.4 tsf (0.4 kg/cm^2) peak	U, T, L, P
Platteville formation: Varies from a hard, gray to light gray limestone to a dolomitic limestone to a dolomite; the limestone has a relatively insignificant number of solution cavities.	Unit weight = 165 pcf (2.6 kg/m^3) Unconfined strength = 9,000 psi (600 kg/cm^2) min Modulus E = 5,000,000 psi (2500 kg/cm^2) min	S
Metabentonite layer: 0.25 ft (10 cm) thick, located 2.3 ft (0.6 m) below the top of the Platteville formation	Not tested during this investigation	

[a] Methods of determining average engineering properties:
M = Average of values supplied by state agency
R = Triaxial compression tests
U = Unconfined compression tests
T = Torvane
F = Field torsion shear
L = Laboratory torsion shear
A = Assumed values
S = Schmidt impact hammer
P = Pocket penetrometer

Figure 7. Stability analysis.



STATION	FACTORS OF SAFETY — AFTER ROADWAY EXCAVATION Aug. 1967: PEAK STRENGTHS AT REST PRESSURES	FACTORS OF SAFETY — AFTER ROADWAY EXCAVATION Aug. 1967: RESIDUAL STRENGTHS ACTIVE PRESSURES	WITH PROPOSED SLIT-TRENCH BUTTRESSES AND CANTILEVER RET. WALL
166+70	0.84	0.91	>1.50
170+00 EXCAVATION TO SUBCUT LEVEL	0.85	0.87	—
170+00 WITH SUBCUT BACKFILLED	0.94	0.99	1.49
170+00 WITH TEMPORARY SAND BERM	—	>1.15	—
172+60	1.05	1.00	>1.50

failure surface was reduced as movement progressed. Therefore, a peak strength value of $c = 0.4$ tsf ($0.4\ kg/cm^2$) was assigned to the clay for the condition immediately prior to the start of movement, but once movement remolded the clay, the strength dropped to the residual value of $c = 0.2$ tsf ($0.2\ kg/cm^2$).

With a horizontal failure surface, the only available driving forces are those produced by the hydrostatic forces and the lateral earth pressure. Prior to roadway excavation, lateral earth pressures existed but were balanced so that no movement could occur. Excavation unbalanced these to the point where the lateral earth pressures, along with other factors, caused strain along the failure surface. Prior to excavation, the lateral earth pressure was in the at-rest condition. To cause failure, it was determined that the coefficient of lateral earth pressure at rest (K_o) would have to have been equal to 0.43 in the glacial drift and 0.76 in the Decorah shale. Once movement started, the lateral earth pressure dropped to a minimum value equal to the coefficient of active earth pressure (K_a). This value was determined to be $K_a = 0.26$ in the glacial drift and $K_a = 0.50$ in the Decorah shale. The results of these studies indicate that the reduction in lateral earth pressure from the at-rest condition to the active condition has a greater effect than the reduction in shear strength in the clay from the peak value to the residual value. Therefore, the slide did not accelerate with time.

The highway department desired that any structural scheme for hillside stability provide a factor of safety of approximately 1.5. By working backwards through the stability computations, it was determined that there would have to be an additional resisting force of 70 kips per lineal foot to provide the desired factor of safety. Because of this conservative factor of safety, it was recommended that the remedial structures be designed for ultimate stresses with no superimposed factor of safety.

In addition, it was determined that a retaining structure should extend at least 10 ft (3 m) above the final roadway elevation. The weight of the backfill behind this wall would prevent the possibility of a slope failure, which would come over the top of a lower structure. Further, it was considered desirable that all schemes be keyed or tied into the Platteville limestone at or below the elevation of the "metabentonite" seam at elevation 791.

Based on these design criteria, six different remedial schemes were studied, as summarized in the following paragraphs.

Slope Flattening

Although slope flattening is occasionally feasible, there were disadvantages:

1. Slope flattening is not a positive means of stopping movement, and a creep-type movement could continue for an indefinite period of time.
2. This scheme would require the purchase of private and public land and homes in a well-established neighborhood.
3. Even if landscaped, a large flat slope probably would not esthetically fit in with the design of adjacent sections of the highway.

Cylinder Pile Retaining Wall

Another scheme provides for stabilizing the cut slope by means of drilled, cast-in-place, reinforced-concrete cylinder piles. In essence, a cylinder pile is a cantilever member that is embedded deep below the failure surface and designed to resist the design loads. These are usually constructed as a continuous wall.

There are several significant advantages in the use of cylinder piles:

1. They provide a positive means of stopping the movement.
2. They may be constructed with a minimum of excavation, prior to the removal of the temporary sand berm.
3. Even under wet conditions, the drilling and casting of cylinder piles is a relatively simple construction procedure.

However, the estimated cost of this scheme was found to be excessive.

Anchored Cylinder Piles

A modified cylinder pile wall was considered that would reduce the quantity of drilling and the quantity of reinforcing steel. Batter tension or compression piles would be used to tie back the top of the drilled-in cylinder piles. However, the cost of preboring and driving the piles offset the savings in reduction of drilling and reinforcing steel.

Braced Soldier Piles

In a scheme that utilizes methods commonly used for making braced cuts for large trenches and for building foundations, the first step would be to drive a continuous row of soldier piles to the top of the Platteville elevation. These soldier piles probably would be steel H piles. Then, in short segments, a continuous wale would be constructed in front of the soldier piles. Finally, at regular intervals, reinforced concrete struts would be cast in narrow trenches excavated under the future roadway. With this system, the design loads would first be transmitted into the soldier piles, which in turn would distribute them into either the continuous wale or into the shale formation below the failure surface. That portion of the load carried by the wale would then be transmitted to the struts. In turn the struts would act essentially as long, horizontal friction piles. Any load not transferred out of the struts by friction would be carried by end bearing of the strut on the opposite side of the roadway. A 10-ft (3-m) cantilever wall, required to prevent slope failure over the top of the soldier piles, would be founded on the soldier piles plus any other piles required for stability.

As in the case of the other alternate schemes, this would be a positive means of stabilizing the hillside; it has the advantage of eliminating the expensive rock drilling required for cylinder piles. The primary disadvantage would be the need to excavate parallel to the toe of the slope during construction of the required wale. If not done properly, this excavation would trigger additional movement which might damage the soldier piles. In addition, there was a possibility that pile-driving vibrations could cause damage to existing structures or renew slope movement.

Anchored Slab

The anchored slab scheme essentially replaces a portion of the vertical load previously removed by roadway excavation with a new vertical load developed by tensioning prestressed vertical anchors. The Slope Indicator observations proved that a relatively small sand berm stopped the movement. A prestressed, anchored slab would provide the equivalent of a 15-ft (5-m) high berm over the entire roadway. To construct the anchored slab, it would be necessary first to remove small sections of the existing temporary berm prior to the casting of the slab. Anchor holes would then be drilled and steel anchors grouted into place. The final step before paving would be to stress the anchors to provide a 1.0 tsf (1 kg/cm^2) area load on the slab. The 10-ft (3-m) high cantilever wall along the toe of the slope would be pile-supported, as in most of the other schemes.

Although considered feasible, there are no significant advantages to this scheme as compared to the others. The main disadvantage would be the need to excavate areas of the temporary berm prior to the casting of the slab. In addition, anchors would have to be protected against the possibility of long-term groundwater corrosion.

Slit-Trench Buttresses

The remedial scheme finally recommended and adopted utilized reinforced concrete buttresses cast in narrow slit trenches excavated normal to the roadway centerline (Fig. 8). The design loadings on the buttresses are also shown in Figure 8. These buttresses were 3 ft (1 m) thick and spaced 12 ft (4 m) center to center and were designed as gravity sections. Each buttress was founded on the top of the Platteville limestone but was keyed into the Platteville with two 3-ft (1-m) diameter concrete filled calyx holes, heavily reinforced for shear. The horizontal load was transferred to the upper layer of the Platteville limestone, which was assumed to act as a start, and

Figure 8. Typical section of recommended remedial slit-trench buttresses.

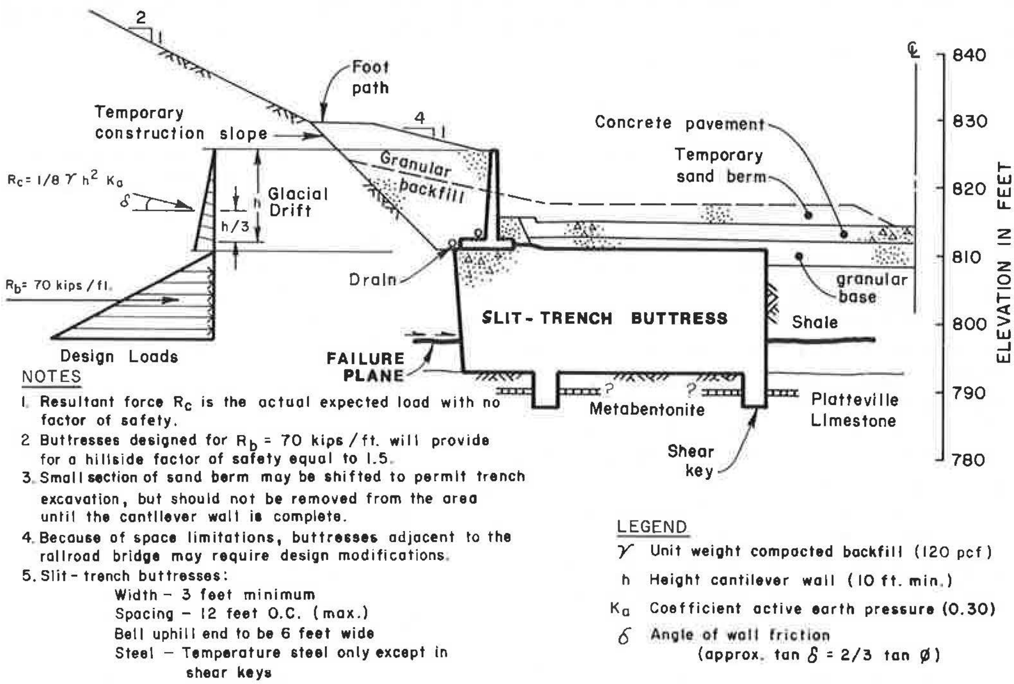


Figure 9. During construction, May 4, 1968.

the vertical load was transferred into the limestone below the metabentonite seam.

The retaining wall constructed above the slit-trench buttress was designed as a conventional cantilever retaining wall except that the footing was designed to span the 9-ft (3-m) distance between adjacent buttresses.

CONSTRUCTION

An essential feature of the recommended remedial design was that construction not further endanger the stability of the existing hillside. Therefore, all phases of construction were closely inspected and Slope Indicators and piezometers located uphill from the work area were read at least every other day during construction.

It was anticipated that at least three buttress trenches could be under construction at any one time, without significantly reducing the stability of the area, so long as there was at least 50 ft between adjacent excavations. Figure 9 is a view of construction in progress, showing the many activities going on at any one time. After a few initial problems, construction proceeded in an orderly, systematic fashion.

The temporary sand berm was left in place during buttress and retaining wall construction. However, to facilitate the construction operation, local portions of it could be temporarily moved. It was specified that, whenever possible, buttress construction was to be accomplished as one continuous operation. This meant that, once excavation had started on a particular buttress, it was not to stop until the final concrete was poured. During the actual construction, the importance of excavating the slit trench and pouring the concrete as soon as possible was confirmed. Several of the trench excavations were left open for 24 hours or longer. As a result, the trench sides caved in during the concrete pour, resulting in delays. This caving occurred even with bracing.

The Slope Indicator observations reflected the construction procedures. As areas of the berm were removed and the trenches excavated, especially where more of the berm was removed than for one buttress, the Slope Indicators showed the resumption of movement. As the buttresses were completed, the movement ceased. No movement has been detected since the buttresses have been completed. A total of 89 buttresses were constructed.

Although the construction took place during a very wet summer, groundwater was not a major problem. Sump pumps were able to dewater the keyways and the bottom of the excavation without any problems.

With completion of a series of adjacent buttresses, wall construction followed closely behind. Thus, the Franklin Avenue end of the wall was completed before the buttresses toward the railroad end were complete. This procedure enabled the contractor to complete and open the job to traffic on schedule on December 9, 1968.

ACKNOWLEDGMENTS

The detailed design of the remedial measures described herein was accomplished by the Minnesota Department of Highways Bridge Section. The staff of F. C. Fredrickson, Materials Engineer, was particularly helpful in handling details of the exploration program and in installing and reading the field instrumentation. Ray Adolfson maintained continuous liaison.

This investigation was under the direct supervision of the author, with the able assistance of E. D. Schwantes.