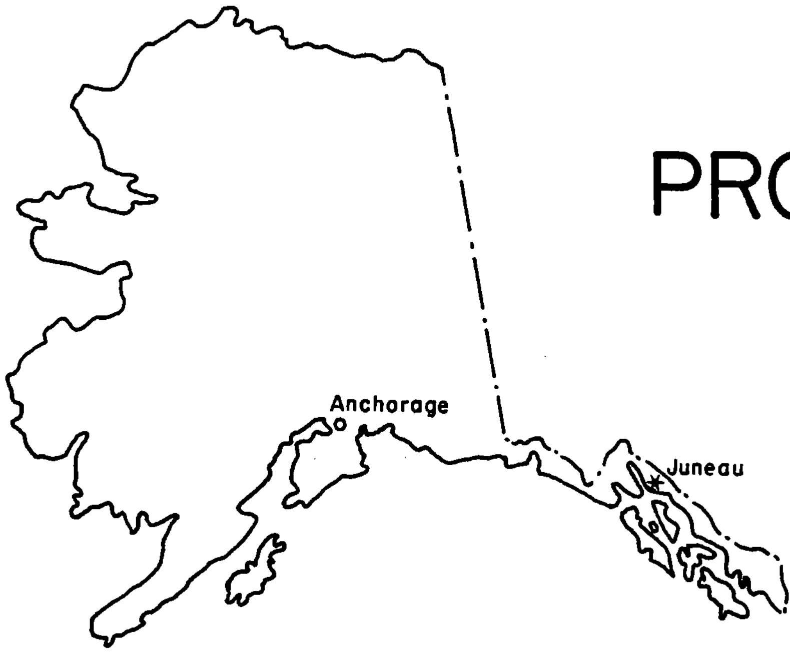
THIRTIETH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 8-10, 1979 PORTLAND, OREGON



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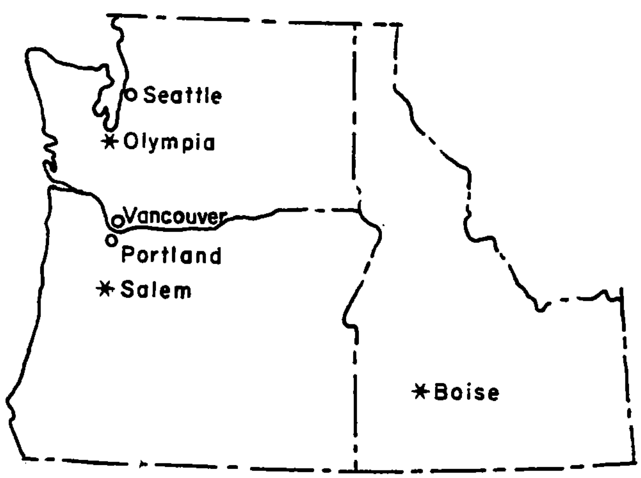

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Table of Contents

Titles and Speakers

LANDSLIDE TESTS REINFORCED EARTH WALL

By

Ronald G. Chassie (1)

ABSTRACT

In October, 1977, a landslide occurred on an Oregon Highway project in Southwestern Oregon, U.S.A. The landslide occurred during construction through an area where the roadway fill was retained by a 30 foot high Reinforced Earth (RE) retaining wall. The slide failure surface passed behind and below the RE wall. The slide affected the middle 300 feet of the 800 foot long wall. This paper describes (1) the cause of the landslide, (2) the wall movement caused by the slide, (3) the effects of the movement on the RE wall, (4) the remedial measures undertaken, and (5) the lessons learned.

(1) Regional Geotechnical Engineer, Federal Highway Administration, Region 10, Portland, Oregon

DESCRIPTION OF PROJECT

The landslide occurred on October 5, 1977, on a section of the Roseburg-Coos Bay highway (SR 42) that was being reconstructed approximately 30 miles west of Roseburg, Oregon. The project is located in Coast Range of southwestern Oregon. The natural terrain is very rugged with the roadway constructed in a sidehill cut-fill section near the base of a steep canyon. The north fork of the Coquille River flows along the base of the canyon parallel to the highway (photo 1).

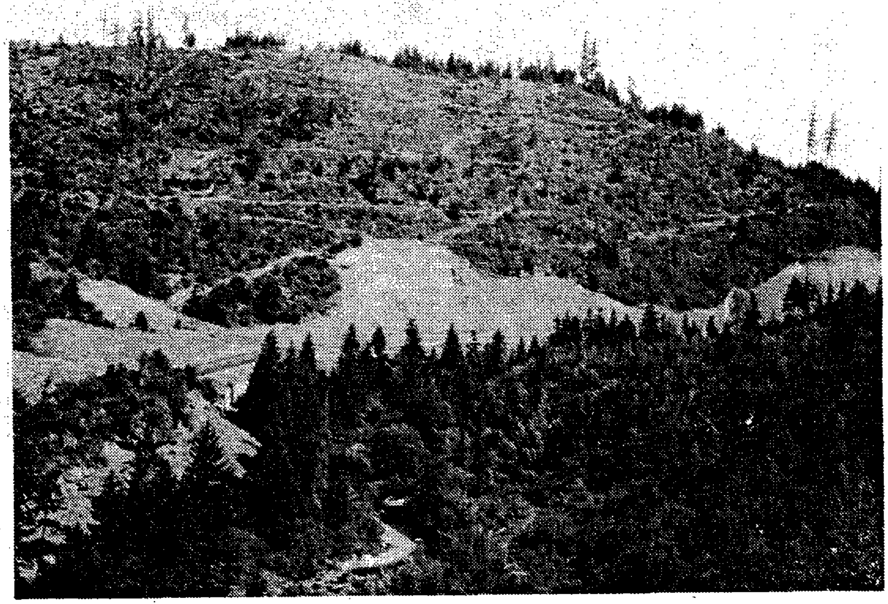

Photo 1 View of Terrain Through Project Area

GEOLOGY

The project area is underlain by sedimentary rocks composed primarily of sandstone and siltstone. Some massive sandstone outcrops are between 30 and 40 feet thick. The sandstone is well cemented and competent. The siltstone beds are up to 60 feet thick. When exposed in highway cuts or outcrops, the siltstone weathers rapidly. This characteristic is the principal cause of landslides and rock falls in the project area. The overburden soil is shallow colluvium consisting of a mixture of boulders, rock fragments, silt, clay, and sand.

ROADWAY DESIGN

The roadway design through the area where the landslide occurred has the roadway placed in a sidehill cut-fill approximately 105 feet above the river. A maximum 30 foot high Reinforced Earth (RE) wall was located between the roadway and the river. The top of the RE wall was approximately 70 feet above the river. The wall was to retain a 35 foot fill placed on a 1½ H to 1V slope from the top of the wall up to roadway grade. The purpose of the RE retaining wall was to prevent fill encroachment into the river. The wall was 800 feet long. The highway fill was within approximately 8 feet of grade when the landslide occurred. (figure 1)

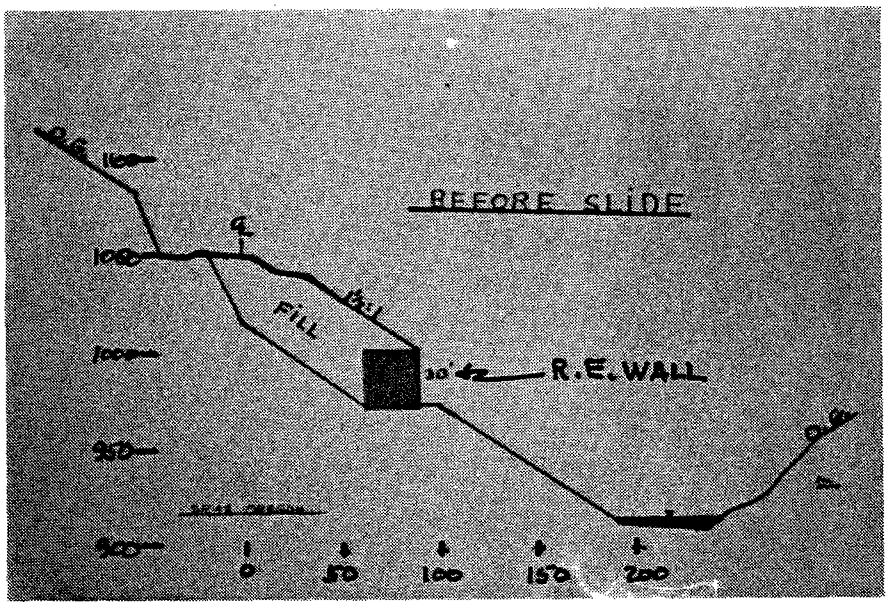

Figure 1 Cross-section Just Before Slide Occurred

LANDSLIDE

Unfortunately, a landslide occurred at the location of the RE wall. Approximately the middle 300 feet of the 800 foot long wall was involved in the slide movement. The slide failure surface passed behind and below the R.E. wall. The slide head scarp passed through the roadway shoulder and the slide toed out about 30 feet above river level.

RE WALL MOVEMENT

The RE wall underwent a significant displacement as a result of the slide movement. The top of the wall in the center of the slide was displaced <u>18 feet</u> horizontally, the bottom of the wall was displaced <u>23 feet</u> horizontally, and the wall dropped <u>12 feet</u> vertically (photo 2).

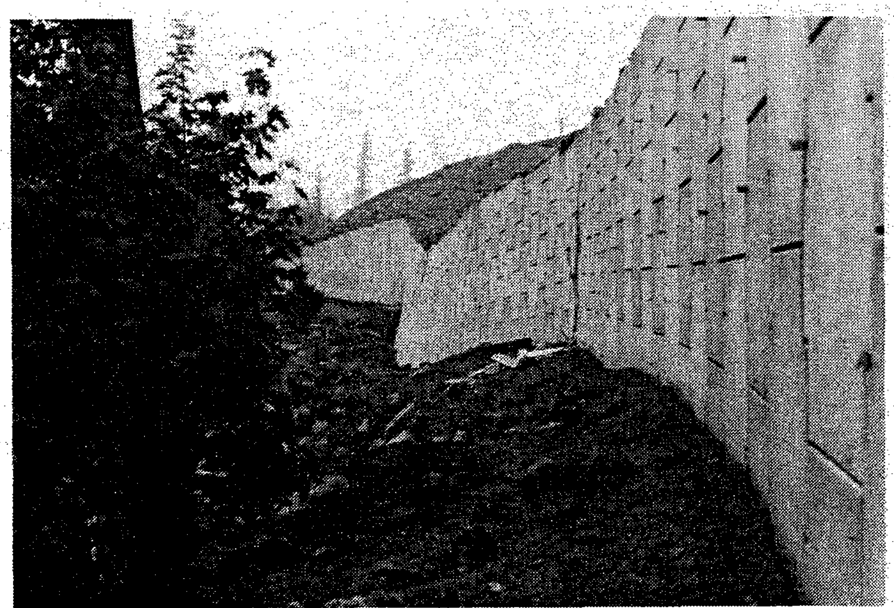

Photo 2 RE Wall Movement Caused by Landslide

In spite of these large movements, the RE wall <u>remained intact</u>. Some spalling and cracking of the wall concrete facing panels did occur near the top of the wall (photos 3, 4, 5, and 6)

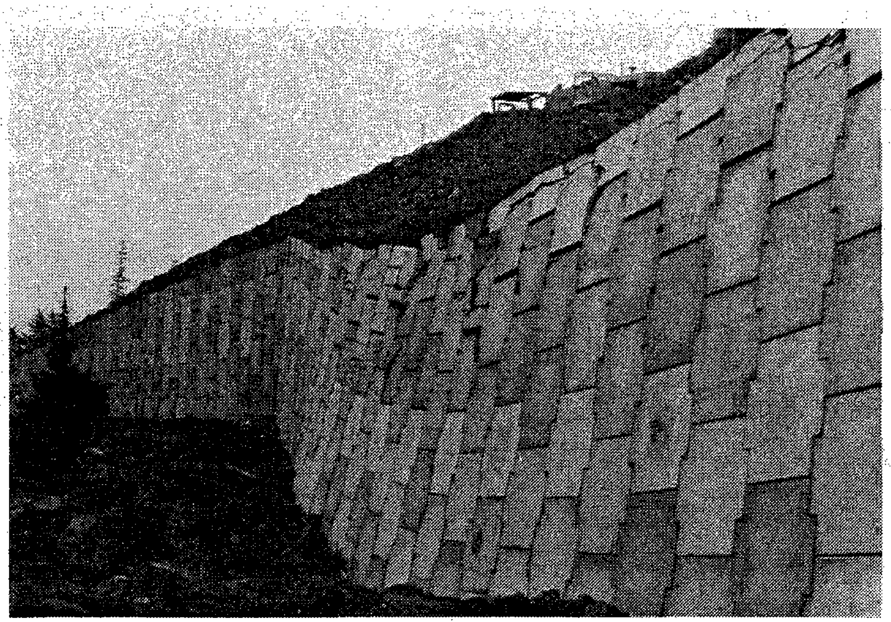

Photo 3 Cracked Facing Panels Near Top of Wall

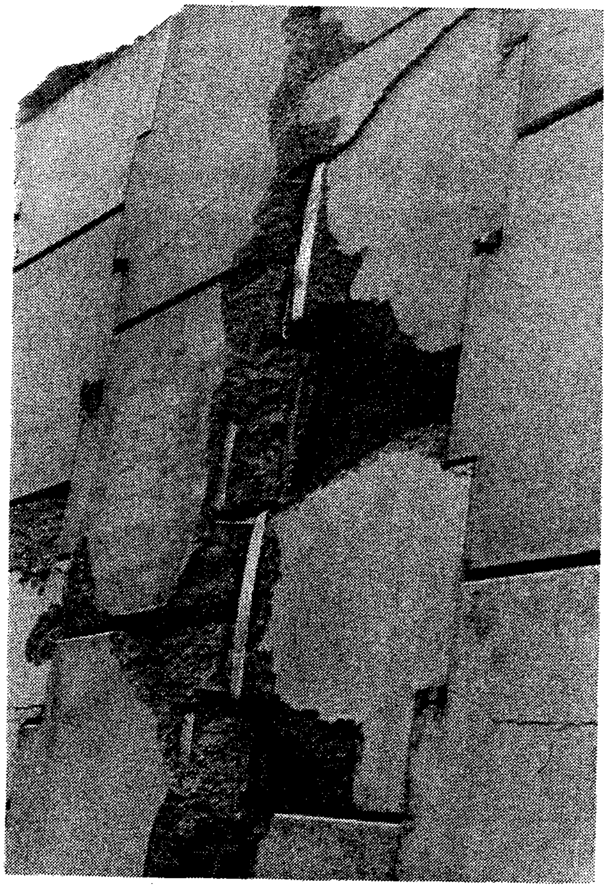

Photo 4 Spalled Facing Panels Near Top of Wall

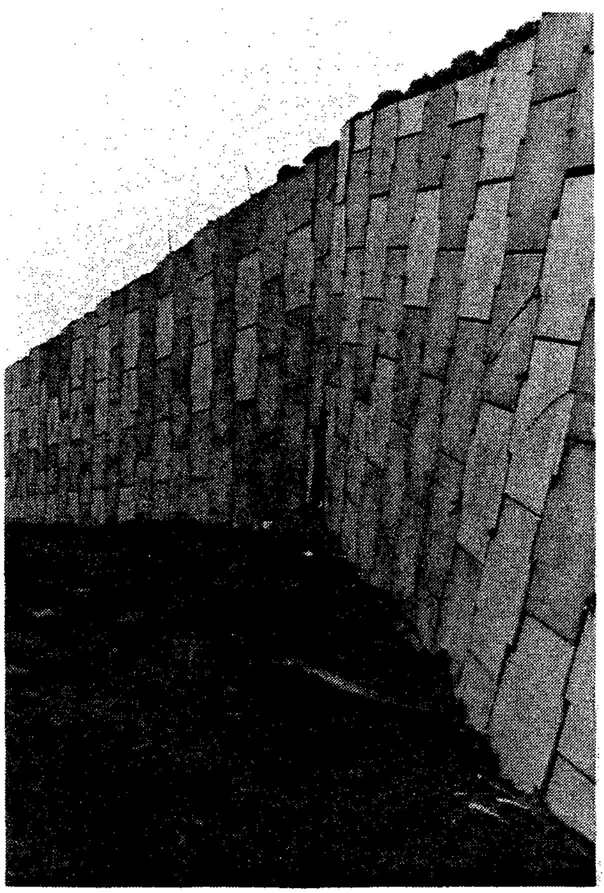

Photo 5 Transition Between Middle Portion of Wall That Moved With Slide and End Section That Did Not Move

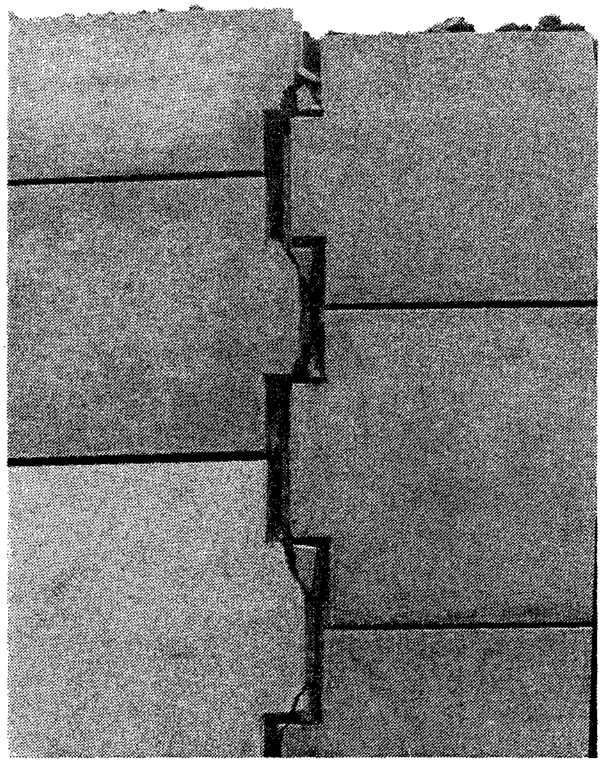

Photo 6 Joint Opening at Transition Between Portion of Wall That Moved and Portion That Did Not Move.

CAUSE OF LANDSLIDE

Immediately after the slide occurred, Oregon Highway Department forces installed six slope inclinometers through the slide area in order to locate the depth of the slide failure surface and to determine ground water levels. Slope inclinometers were installed in front of and behind the RE wall. To speed installation, the slope inclinometer holes were drilled with an air-trac drill and no samples were recovered. At the writer's recommendation, two more borings were made in the failure area using a rotary core drill rig and the holes were continuously sampled. The borings both revealed sandstone immediately underlying the RE wall. The sandstone graded into siltstone at about 20 feet below the base of the wall. Visual inspection of the core samples revealed a thin 2 inch thick layer of soft silty clay existing at the siltstone contact. The depth of the thin clay seam coincided with the depth of movement in the slope inclinometers. Therefore, the investigation revealed that the landslide failure occurred along this thin clay seam which dipped toward the river. (figure 2). Figure 2 also shows the amount of RE wall movement that occurred in the center of the slide area.

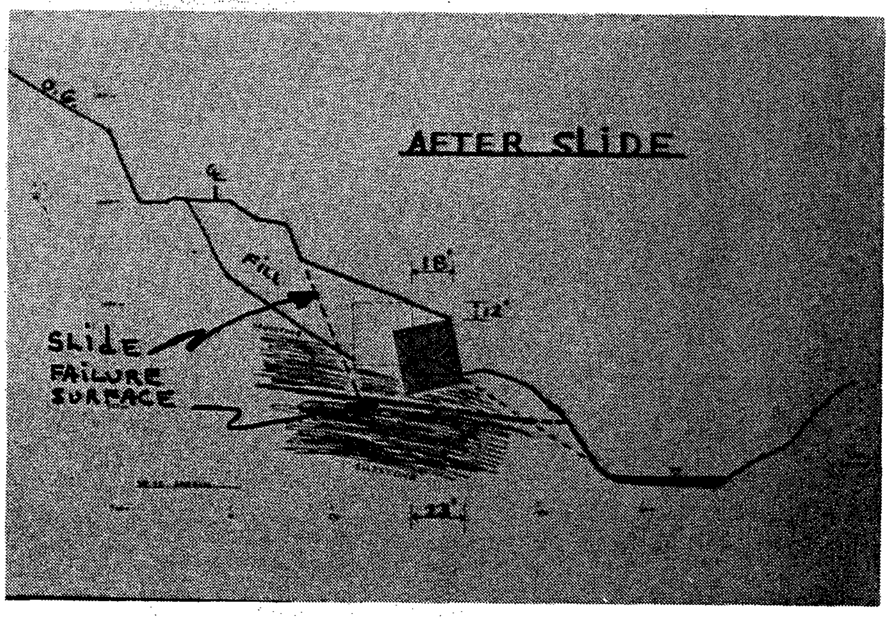

Figure 2 Location of Landslide Failure Surface
And RE Wall Movement

The thin clay seam provided a plane of weakness. The cause of the landslide was simply due to the clay not having sufficient strength to resist the forces imposed by the new highway fill placed above.

CORRECTION SCHEME

The slide continued to move after the initial failure. Several of the slope indicators installed in the failed area sheared off within a two-three week period. It was obvious that time was critical.

Based upon the nature of the slide and the fact that waste rock and soil excavation was available on the project, it was decided that a stabilizing buttress would provide the most positive and most economical solution to the slide problem.

Based upon the slide geometry and the slope inclinometer and boring data, the landslide was backanalyzed to determine the soil shear strength existing at failure. Using this backfigured soil strength, additional slope stability analyses were conducted to size the rock buttress. Based upon anticipated ground water levels, the buttress was sized to provide a factor of safety of 1.25 with the highway fill slope reconstructed.

Due to the environmental concerns relating to placing a rock buttress in the river, Oregon DOT obtained the necessary clearances from appropriate State and Federal agencies.

The correction scheme included bringing the buttress up to mid-height of the RE wall through the 300 foot section of wall moved by the landslide, and carrying the reconstructed highway fill slope down over the top of the RE wall to where the fill met the top of the buttress. In effect then, it was decided to "bury" the portion of the RE wall damaged by the slide movement. None of the wall was removed or replaced. A cross-section of the correction scheme is shown in figure 3.

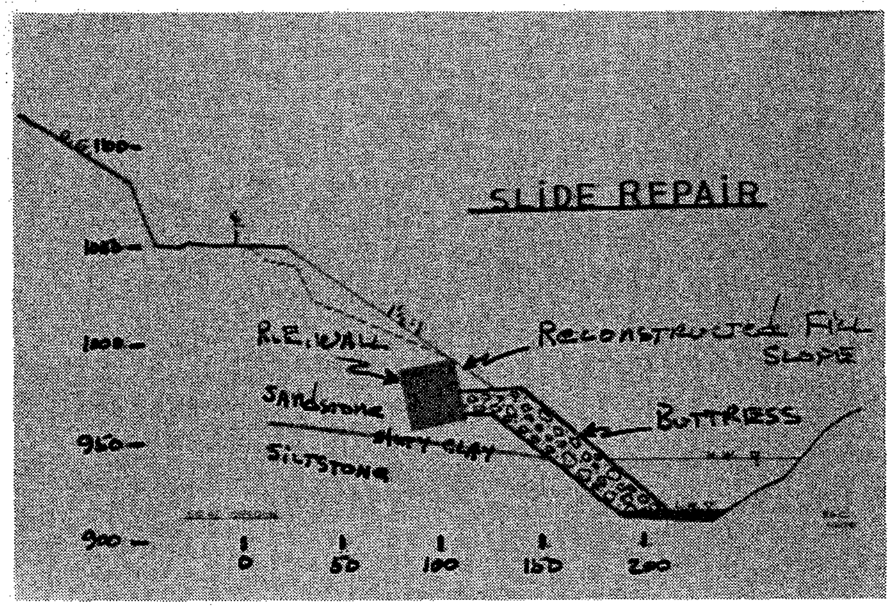

Figure 3 Cross Section of Buttress Correction

Buttressing of the middle 300 feet of wall that had moved as a result of the landslide was completed approximately 5 weeks after the slide occurred. Construction on the project then ceased due to heavy winter rains.

As noted previously, approximately the middle 300 feet of the 800 foot wall was involved in the slide movement. This left about 250 feet of wall on each side of the failure area that did not move. However, since the slope geometry and amount of fill retained was similar to that through the failed area, we were concerned with the long-term stability of the remaining wall. Additional borings were drilled near the base of the RE wall sections that had not moved. Borings on the east end revealed similar foundation conditions to those encountered in the failed area. Therefore, when project work began again in the Spring of 1978, the buttress was extended an additional 250 feet to the east end of the wall. This additional length of buttress was only brought up to the base of the RE wall.

Approximately 60,000 cubic yards of rock and soil were placed to form the total buttress. Cost of the buttress was approximately $120,000. The low cost was due to the fact that the buttress material was waste material utilized off the grading project, and the $120,000 was simply the cost for the extra handling and placement of the waste material.

The completed correction work is shown in photos 7, 8, and 9.

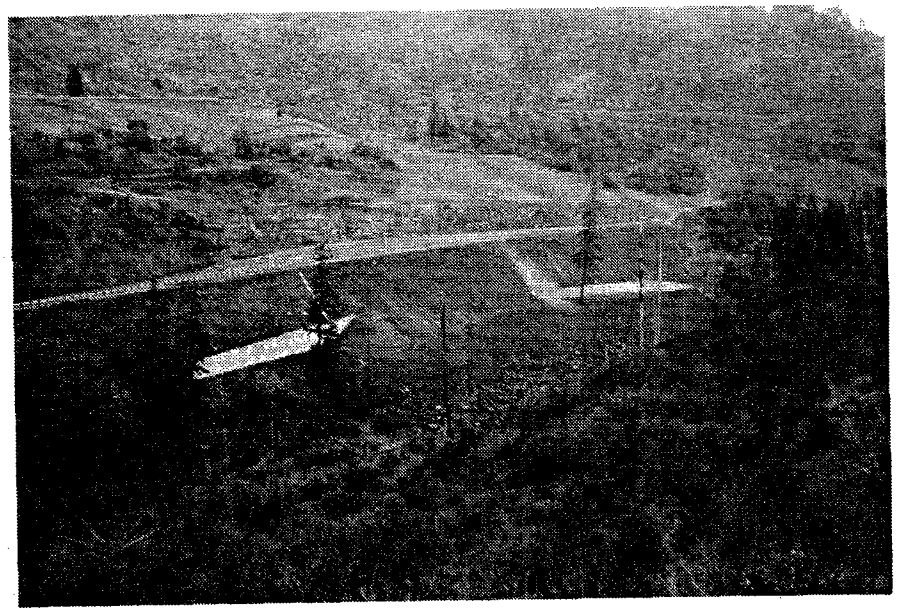

Photo 7 Completed Buttress Correction

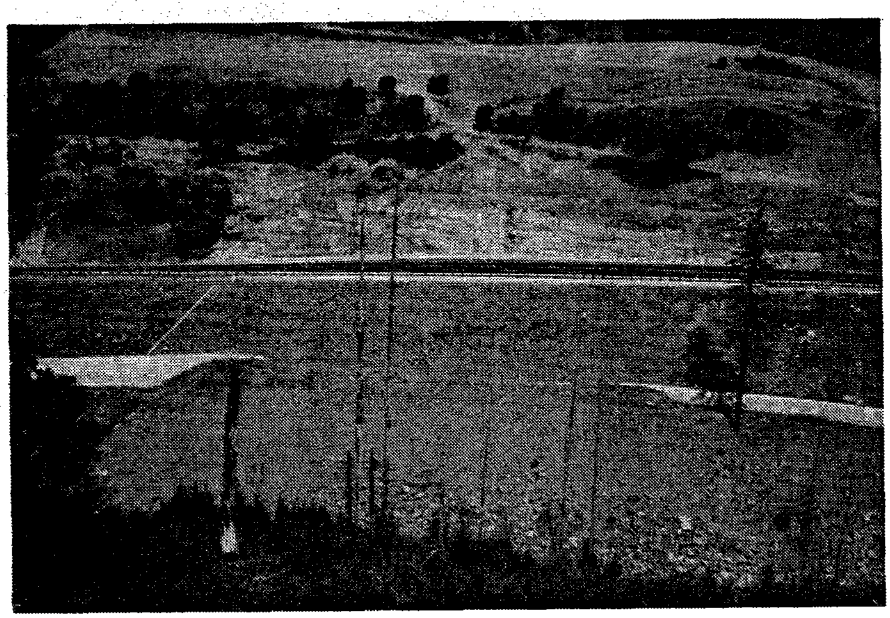

Photo 8 Front View of Completed Buttress Correction
Note Middle Portion of RE Wall Buried by Fill Slope

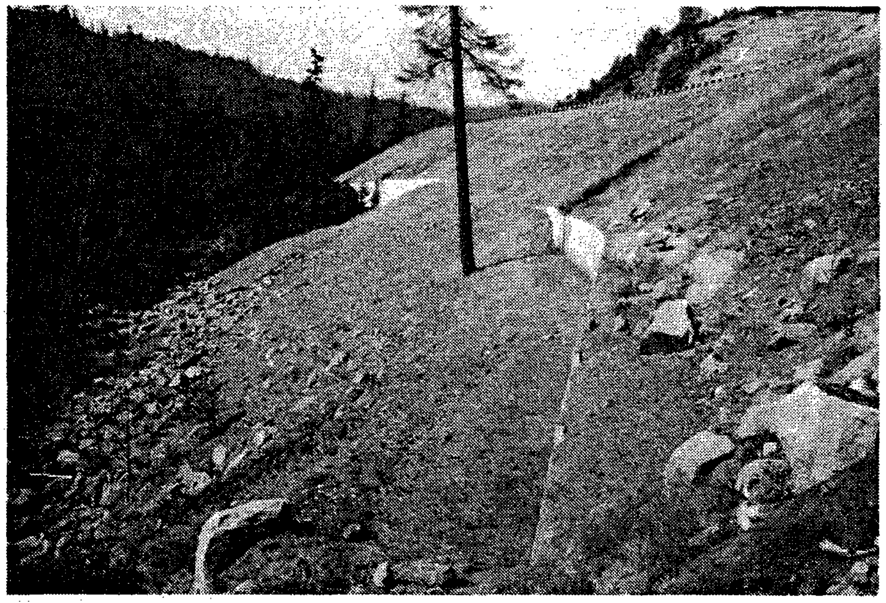

Photo 9 End View of Buttress Correction

LESSONS LEARNED

The landslide movement resulted in a full scale test of the Reinforced Earth wall. Obviously, such a full scale test was unplanned but it did reveal some important facts on the performance of a Reinforced Earth structure.

First, the amount of movement that the wall withstood, without rupturing, was amazing. As mentioned previously, the top of the wall was displaced 18 feet horizontally, the bottom of the wall 23 feet horizontally, and the front of the wall dropped 12 feet vertically. These movement amounts were measured at the midpoint of the 300 foot length of wall that moved with the slide. This means that the wall was able to withstand up to 23 feet of differential horizontal movement and 12 feet of vertical differential movement in a 150 foot length, without rupturing. One of the advantages of a Reinforced Earth structure is that they are flexible and can withstand some settlement. A rule of thumb is that vertical differential settlements of 1 percent of the length (such as 1 foot in 100 feet) are tolerable. For the subject wall, this would mean that 1.5 feet of vertical settlement through the 150 foot length would be considered tolerable. Twelve feet actually occurred. The 12 feet vertical differential movement was tolerated by the sturucture in that it did not fail, however the movement was intolerable for aesthetic reasons due to the wall bulge and some cracked and spalled facing panels. (photo 3)

Second, this full scale test showed that a Reinforced Earth structure does perform as a gravity structure, as theory says it will. The Reinforced Earth volume moved as an intact gravity mass. This was evident both by the final configuration of the moved wall and by visual inspection of the bolt connections where the reinforcing strips attach to the back of the facing panels (for those panels where the concrete facing had spalled and exposed the connection). The connections were not broken.

In closing, it should be emphasized that the subject problem was a landslide problem and not an RE wall problem. The RE wall just happened to be sitting on the landslide and went for an "unplanned ride." Unfortunate as it was, the landslide did provide a dramatic full scale test of a Reinforced Earth structure and demonstrated dramatically (1) the internal strength of a RE structure, (2) that an RE wall does in fact perform as a gravity structure, and (3) that an RE structure is capable of withstanding large settlements and horizontal movement without rupturing.

ACKNOWLEDGEMENTS

I wish to acknowledge the professional association of Messers Ed Johnson, Fred Yarbrough, Eliott Parker, Jim Gix, and Al Bátes of the Oregon Highway Department and Mr. Tom Lowe of the FHWA Oregon Division Office, with whom I had the pleasure of working on the analysis and solution of the subject landslide problem.

FABRICS IN THE HIGHWAY: THE STATE OF THE ART IN CIVIL ENGINEERING APPLICATIONS

BY

J. RAY MULLARKEY

CELANESE FIBERS MARKETING COMPANY

Editor's Note: Only the following synopsis of this paper was received for publication in the Proceedings.

CELANESE

THE USE OF FABRICS IN ROAD CONSTRUCTION

The problems associated with the construction of high performance, low cost roads over poor soils have challenged road builders and engineers for centuries. Today, these problems still exist; and new challenges abound, brought on by tighter construction and maintenance budgets, environmental regulations and ever increasing prices. However, a rapidly expanding technology now offers the engineer a new tool to achieve more cost effective solutions to a variety of road building problems. This new technology involves the use of engineered construction fabrics. Their use has been demonstrated to save significant amounts of money in properly designed road systems, drainage structures, sedimentation control devices and embankments. During the session, this new technology will be examined in order to answer several basic questions: What are construction fabrics, and what functions do they perform?, What are the benefits derived from the use of these materials?, What are the major fabric selection criteria?

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STRUCTURAL PERFORMANCE OF BURIED CORRUGATED POLYETHYLENE TUBING

by

Reynold K. Watkins

and

Ronald C. Reeve

Presented at:

Thirtieth Annual Highway

Geology Symposium

Portland, Oregon

8 Aug 1979

Sponsored by:

Federal Highway Administration

Office of Federal Highway Projects

Vancouver, Washington

SYNOPSIS

Corrugated Polyethylene tubing, in just one decade, has emerged as the principal material used in the drainage industry. This is due primarily to its light weight, which makes it easy to handle, and to its low cost. Beginning first in the agricultural and building markets as early as 1967, its use has now extended to highways and roads and other industrial buried drainage applications where it is subjected to various types of traffic loads. This paper provides information on structural performance and design criteria for buried corrugated polyethylene tubing.

It is shown that as a result of certain unique properties of corrugated polyethylene tubing, such as longitudinal flexibility and the rapid internal stress relaxation capability of the material, the design requirements for load supporting applications reduce to a single criterion - ring deflection. As with all flexible conduits, the deflection of corrugated polyethylene tubing can be controlled by envelope selection and placement. Thus, design requirements for load carrying capability involve the combined response of tubing and envelope in each particular loading situation.

Data are reported for full scale field tests where tubing deflection was measured for a range of envelope materials and installation conditions under AASHTO Standard H-20 Truck loading. Tubing deflection, in percent of nominal diameter, are reported for each of four installation cases as a function of cover expressed as a ratio of height of cover to nominal diameter, H/D. Recommendations are presented for both minimum and maximum cover. In general, tubing deflection under load was found to be equal to the vertical compression of the sidefill envelope material.

Where control is exercised over envelope material selection and placement, the structural performance of corrugated polyethylene tubing must be rated as excellent for a wide range of applications where both dead loads and live loads are involved. Such cases include highway berm and slope drainage, culverts, storm drains and other industrial drainage situations.

STRUCTURAL PERFORMANCE OF
BURIED CORRUGATED POLYETHYLENE TUBING

by

* Reynold K. Watkins 1/

** Ronald C. Reeve 2/

INTRODUCTION

Highway Geology is inextricably linked to drainage. In the short period of just 12 years, corrugated plastic tubing has emerged as the principal material used for drainage in agriculture and in the building industry. It is rapidly gaining widespread usage for drainage in Highway and other commercial type applications. The smaller diameters (4-6 inch) have been used extensively for highway berm and slope drainage, and the larger diameters (8, 10, 12, 15, and 18 inch) are being used in various water disposal applications, such as culverts for highways and city and county roads and for industrial site drainage. Corrugated polyethylene tubing is currently included in approved products listings for 27 states and approvals are in process in at least 12 more.

Its wide acceptance in these varied applications is due primarily to (1) resistance to deterioration, (2) light-weight and ease of installation, and (3) low cost.

Corrugated polyethylene tubing is resistant to both chemical and biological deterioration. Its low weight, from 1/4th to 1/25th that of the weight of steel or concrete, makes for easy installation with significantly reduced labor cost. With savings in labor and low tubing cost, corrugated polyethylene tubing is attractive for many drainage and culvert applications.

Its rapid increase in use has called for evaluation of its structural performance and development of design criteria. This report is a summary of findings on the structural performance of buried corrugated polyethylene tubing. It is reduced to simple design criteria for typical installations.

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SCOPE

Some considerations that are ordinarily paramount in buried tubing design do not apply directly to corrugated polyethylene tubing because of its unique properties:

1. Most cylindrical pipes are stiff longitudinally like a beam. So they must be designed as a beam to carry heavy soil loads. Corrugated tubing is flexible longitudinally. It conforms with the bedding in the trench. Therefore beam design is unnecessary.

2. The bearing strength of polyethylene is much less than the bearing strength of rocks. Clearly polyethylene tubing cannot resist the concentrated load of a large rock bearing against it. So a "select" soil, such as minus 1 inch gravel, or sand with high internal friction is specified as an envelope. When so designed, the bearing problem no longer exists.

3. Corrugated plastic tubing has a flexible circular cross section herein called the ring. The corrugations are of such a depth that the ring is stiff enough to hold its circular cross section during transportation and backfilling, the ring is flexible enough to deflect slightly and conform with the soil envelope. A rigid ring, such as clay or concrete pipe, cannot conform, and therefore must be designed to carry the vertical load (dead load plus live load) as the sidefill soil is compressed downward. (See figure 1). For rigid pipe, the sidefill support is very small or nonexistant, so the ring must be strong enough to resist the large bending moments caused by unbalanced loads; i.e.

$$\text{BENDING MOMENT} = R^2(P_v - P_H)/4$$

R = mean radius

P_v = vertical soil pressure

P_h = horizontal soil pressure

In contradistinction the flexible ring flattens down slightly. This is called ring deflection. The ring deflection relieves the ring of a large part of the vertical load and develops an almost equal sidefill support such that the loading becomes balanced. The bending moment is essentially zero. Thus a light flexible ring is adequate. In fact, the "select" soil envelope forms a crude masonry arch, which supports most of the load. See figure 2. Soil materials are an inexpensive building material.

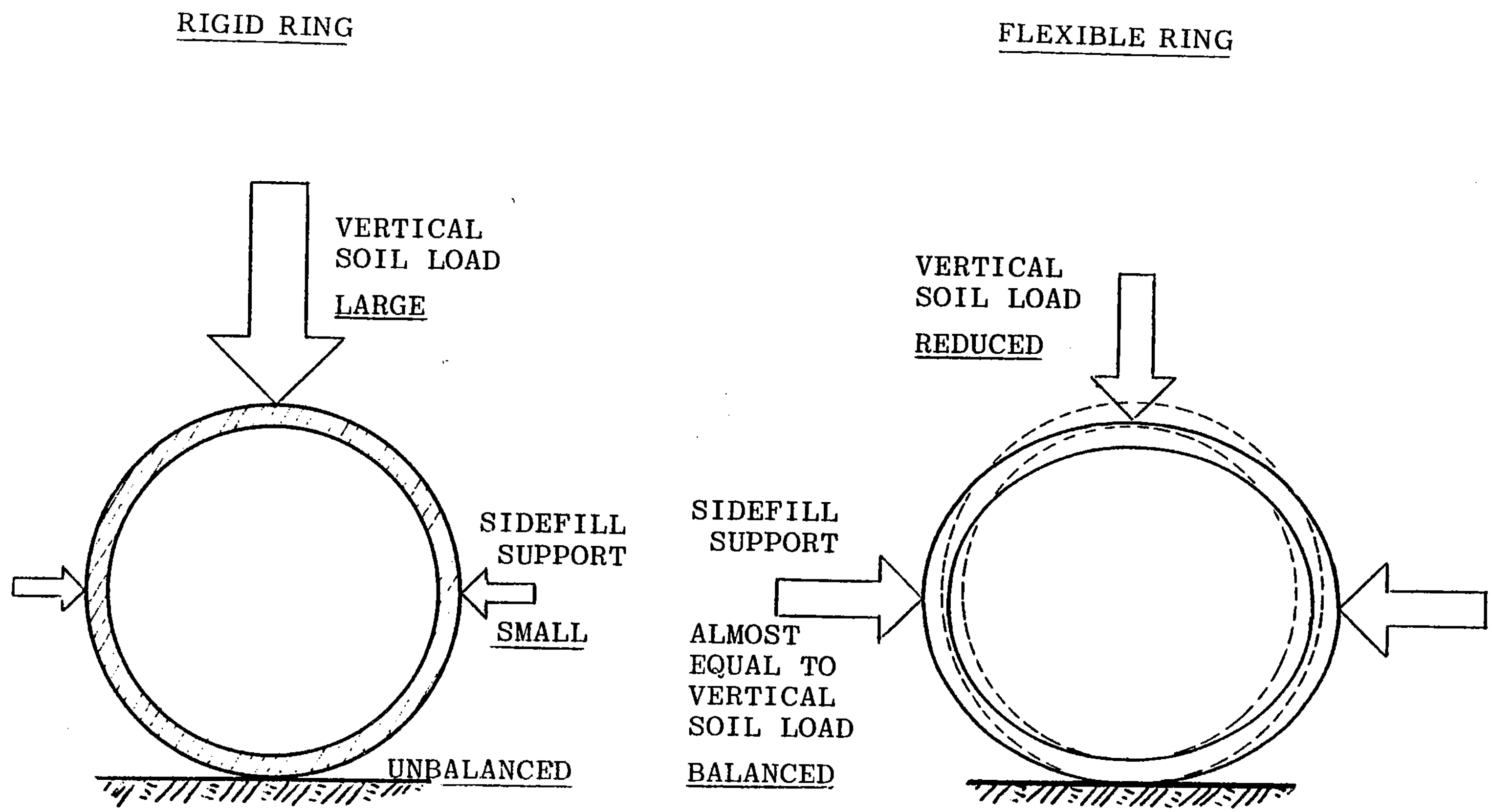


Figure 1. Because of its rigidity, the rigid ring must support a large vertical load. Little or no sidefill support is developed. The flexible ring conforms with the soil and so gives up a large part of the vertical load to arching action of the "select" soil envelope. Note that the flexible ring depends on sidefill forces which support the ring. Vertical and sidefill forces are balanced, so the flexible ring is structurally adequate.

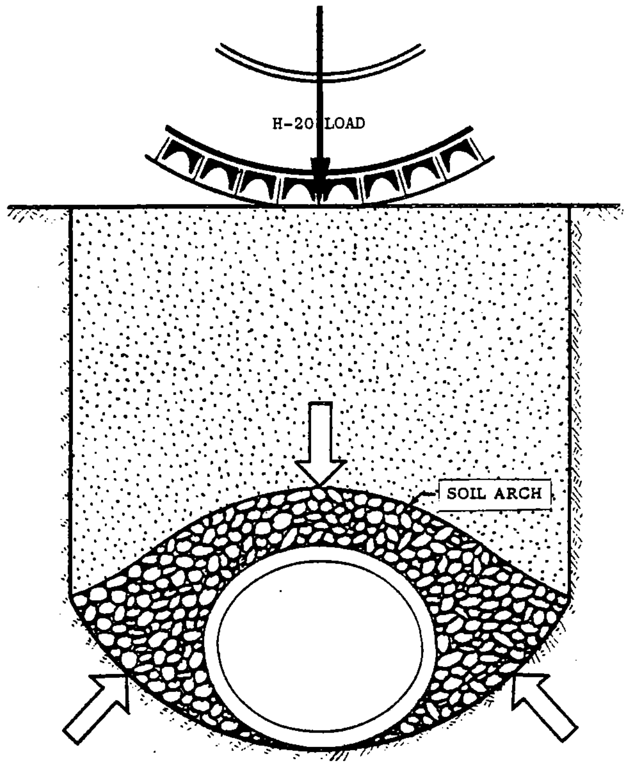


Figure 2. A"select" soil envelope placed over and around a flexible ring, becomes a soil arch, similar to a masonry arch, which supports much of the vertical load and so reduces the strength requirements of the tube ring.

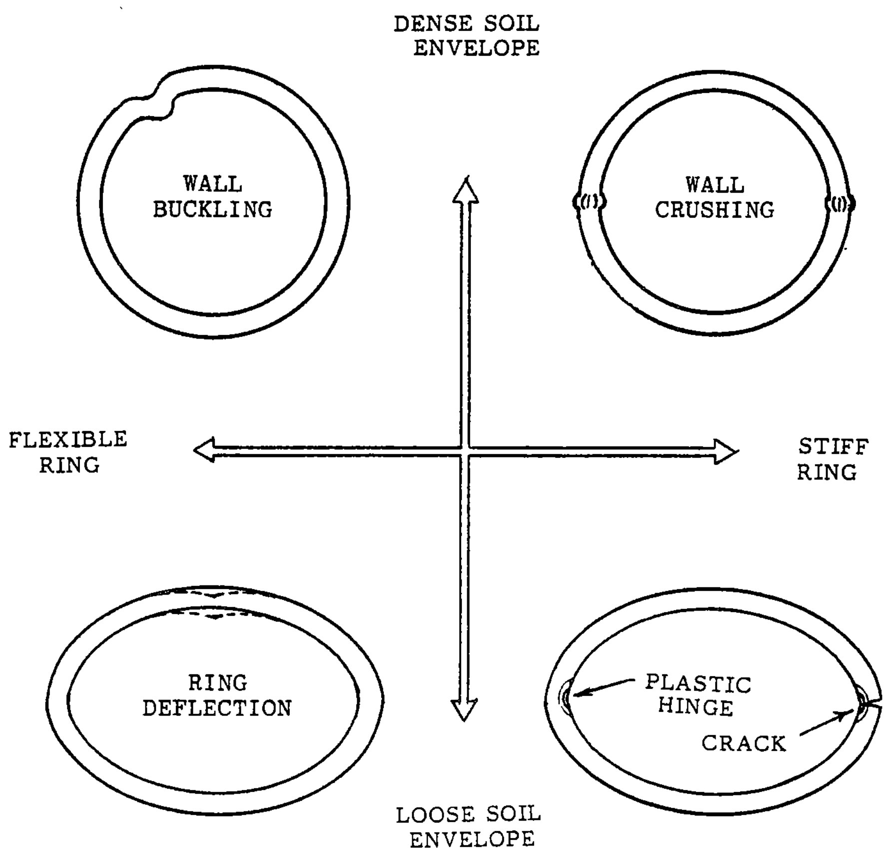


Figure 3. Classification diagram of performance limits (deformation limits) for flexible tubing.

DESIGN

For design we start with the universal design concept,

$$\text{PERFORMANCE} = \frac{\text{PERFORMANCE LIMIT}}{\text{SAFETY FACTOR}}$$

where the performance limit is reduced by a safety factor. The words performance limit are used rather than failure because failure connotes catastrophic collapse. Collapse usually does not occur - especially in the case of buried flexible tubing which may deform beyond design limits but still perform its function. For this reason, a safety factor of 2 is usually adequate.

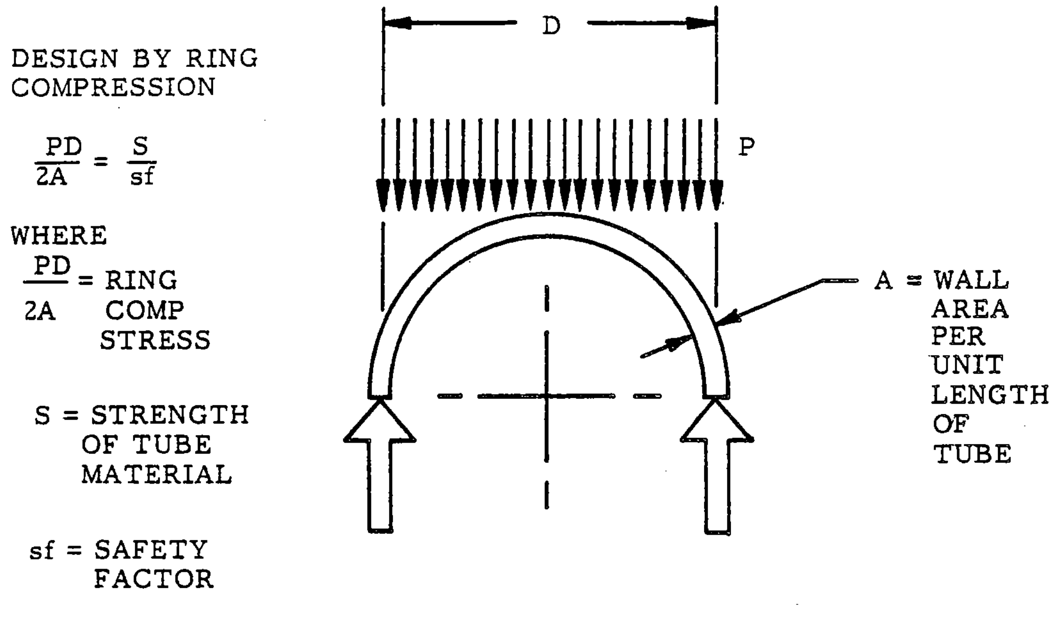
Performance limits of buried tubing are deformations. Figure 3 is a general classification of these deformations for which limits can be established. If the soil envelope is densely compacted, performance limits may be wall buckling (very flexible ring) or wall crushing (relatively stiff ring). If the soil envelope is loose, performance limits may be 1) excessive ring deflection (including a possible reversal of curvature on top due to wheel load), or 2) the formation of plastic hinges or cracks in the case of brittle material.

Design in dense soil is by ring compression, and design in loose soil is by ring deflection. Let us consider each separately.

Ring Compression

In general, the concept of ring compression follows the free-body-diagram of figure 4. The vertical load, PD 3/ must be resisted by compression in the ring (P = soil pressure, D = mean diameter). For the usual design of buried pipes, ring compression stress must be less than the strength of the material. The stress PD/2A is based on a vertical soil pressure P caused by a surface live load P_l and dead load of the soil P_d. See figure 5. However, for many plastics, including polyethylene, the materials have a unique property that eliminates the need for conventional design procedure. Stresses in polyethylene relax under constant deformation. Moreover, the rate of stress relaxation is faster than the decrease in strength with time. So the ring relaxes to a safe stress level and the envelope picks up the load in arching action over the tube. The structural performance of polyethylene in some respects is analagous to human performance. Polyethylene loses strength with time. Suppose you support a 50 kg barbell overhead. Initially you are OK, but the load feels heavier and heavier as the minutes drag on until you reach your limit and let go. For you it's an endurance limit. For polyethylene it is service life. Incidentally, your endurance is less if the temperature is very high; the same applies to plastics. Now suppose you support the 50 kg's overhead but somebody raises platforms up to the weights on either side until they just make contact. As you yield under the weight, the platform picks up a large part of the load. In time you relax, and you are able to hold your portion of the load without distress.

3/ The symbol D is used to refer to ring or tubing diameter. There are three diameters of importance: outside diameter, inside diameter and mean diameter. Although D is used separately in this paper to refer to each of the above, it will be made clear each time it is used which diameter is intended. The inside diameter is the standard nominal diameter used in the industry.



FREE-BODY-DIAGRAM

Figure 4. Ring compression stress is defined as the tangential stress, PD/2A in the wall of the tube due to the external soil pressure.

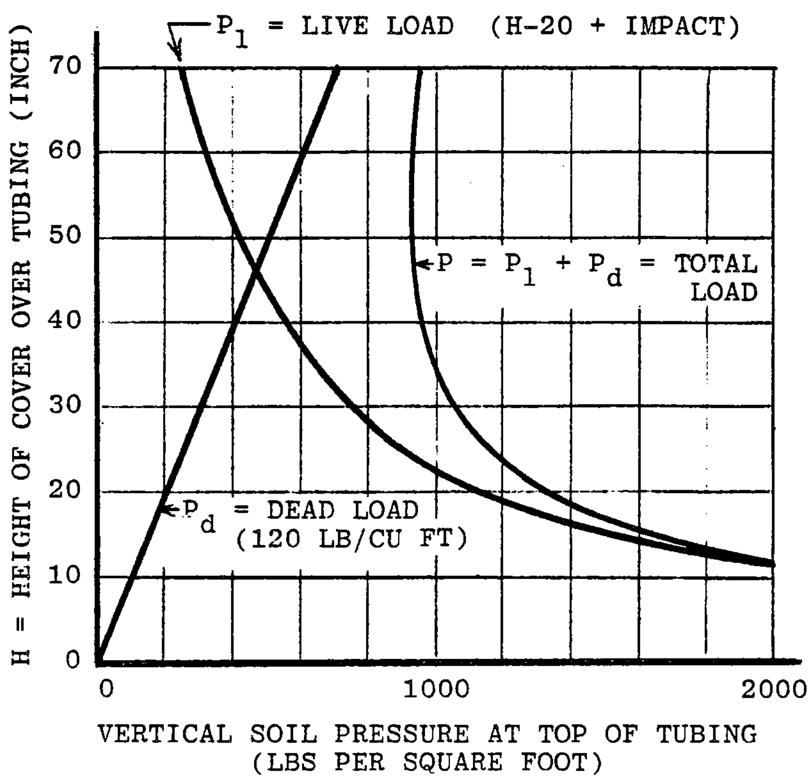


Figure 5. Typical chart of values for vertical soil pressure due to live load pressure P_l, and dead load pressure P_d. See also AISI Handbook of Steel Drainage and Highway Construction Products, 1971, page 86.

Thus, ring compression is not a critical design criterion, provided that an adequate envelope and sidefill has been placed to pick up the load as the polyethylene relaxes.

Up to this point, for the case of corrugated polyethylene tubing, all of the usual structural design criteria have been eliminated by specifying a "select" soil envelope. However, one design criterion remains, ring deflection. It cannot be eliminated.

Ring Deflection

We define ring deflection as $\Delta y/D$ caused by the vertical load PD (D = mean diameter). See figure 6. The performance limit for ring deflection of flexible drain tubing is set by many designers at about 5%, but under some circumstances, 10% may be acceptable.

For design, there are two cases for which ring deflection must be considered, maximum soil cover, and minimum soil cover.

For maximum soil cover, several equations are available for predicting ring deflection. Figure 7 is a convenient, conservative summary of these equations. It is a plot of the ring deflection term $(\Delta y/D)/\epsilon$ as a function of the stiffness ratio $E'/(F/\Delta y)$. The ring deflection term is the dimensionless ratio of ring deflection $\Delta y/D$ to vertical strain of the sidefill. The stiffness ratio is a dimensionless ratio of soil stiffness E' to pipe stiffness $F/\Delta y$.

Stiffness is simply the slope of a load-deflection diagram. For soil, the stiffness E' is the average slope of the stress-strain diagram from a laboratory compression test. It is similar to a modulus of elasticity E of other structural materials. For the ring, the stiffness $F/\Delta y$ is the slope of a plot of diametral load F per unit length of tubing applied to a ring of the tubing by parallel plates, as a function of the ring deflection.

Figure 7 is used as follows. The pipe stiffness $F/\Delta y$ for corrugated polyethylene tubing is usually about 40 psi, but may be up near 70 psi. Soil stiffness E' for loose sand envelope ranges up from roughly 700 psi. Thus, the least stiffness ratio for buried corrugated polyethylene tubing is about 700/70 = 10 which is off the chart to the right of figure 7. Therefore, for all practical purposes, $(\Delta y/D)/E = 1$. In other words, the ring deflection $\Delta y/D$ is roughly equal to the vertical soil strain ϵ.

The flexible ring is compressed downward nearly as much as the sidefill is compressed downward by the vertical load. With a select soil envelope, it would take a very high soil cover to compress the sidefill as much as 5%. The limits of maximum soil cover shown in figure 8 include a safety factor greater than 2.

For minimum soil cover, the equations available to predict ring deflection are complicated and imprecise. The best way to predict ring deflection is to run load tests and measure ring deflection. Such a series of tests was conducted in this study.

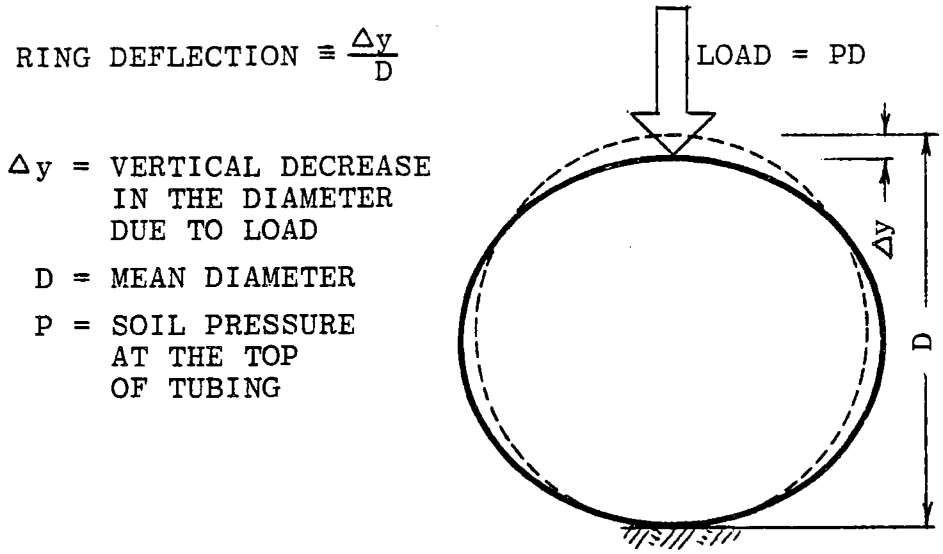


Figure 6. Ring deflection is defined as the vertical decrease in diameter divided by the mean diameter. Ring deflection is restricted to that deformation, basically eliptical, caused by increasing vertical soil pressure after the soil envelope has been placed up to the top of the tube. Ring deflection is <u>not</u> the deformation caused by carelessness in placing the soil envelope. During the placement of the soil envelope, the ring should be held in its circular cross section.

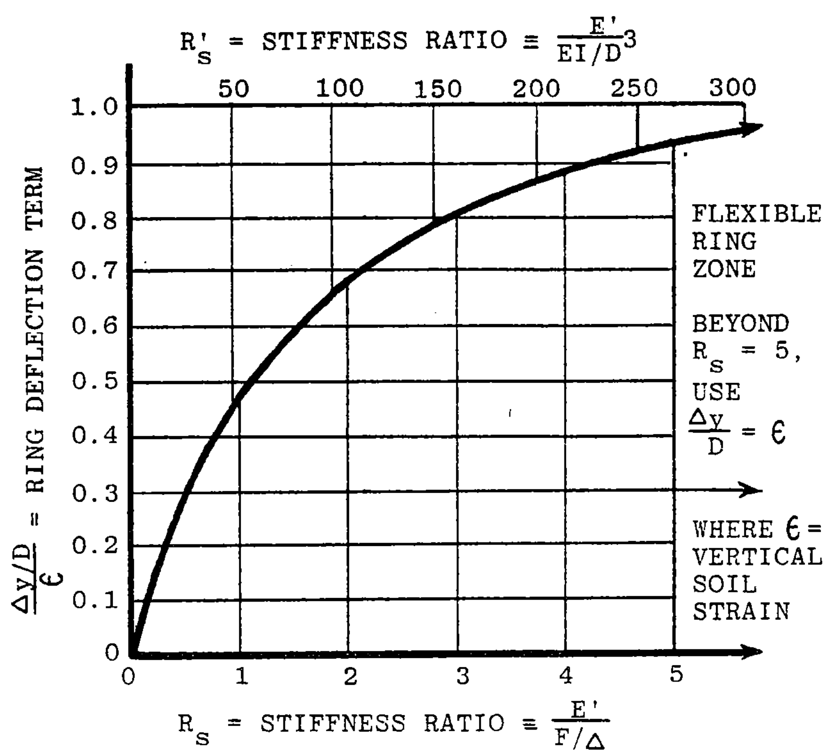


Figure 7. Summary of much data on the dimensionless ring deflection term for buried tubing as a function of the dimensionless stiffness ratio. It is notworthy that corrugated polyethylene tubing generally falls to the right of the graph in the flexible ring zone, where $\Delta y/D = \epsilon$.

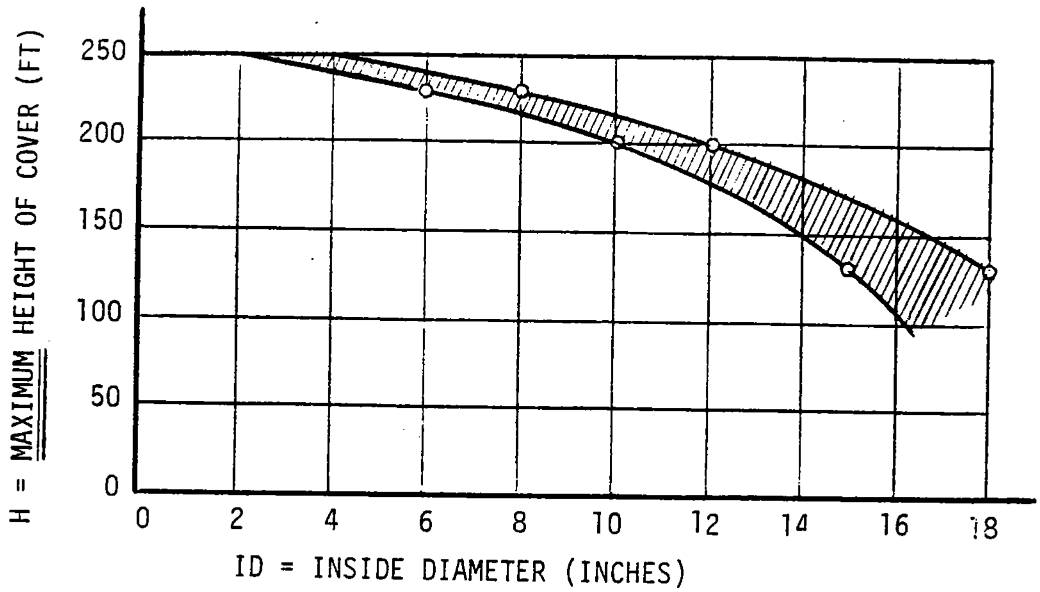


Figure 8. Conservative values of maximum height of soil cover over typical corrugated polyethylene tubing, installed with "select" soil envelope.

TEST PROCEDURE

The experiment comprised a series of full scale tests on nominal 12-inch diameter corrugated polyethylene tubing at Hamilton, Ohio, installed according to procedures already described. The objective was to determine the ring deflection $\Delta y/D$ (dimensionless) as a function of depth of cover term H/D (dimensionless) for four common installation conditions with the passage of an H-20 truck load (D= nominal inside diameter). Three of the four installation cases are shown diagrammatically in figure 9. The case 4 was for complete backfill using native soil carefully compacted which gave results similar to Case 3, and was, therefore, not included in Figure 9. Twelve-inch corrugated polyethelyene tubing was used, but by principles of similitude the results can be extended from the smallest available tubing, 3-inch, up to 24-inch tubing. The test procedures are further described by a series of photographs, (Figures 12 through 17) shown later in this report.

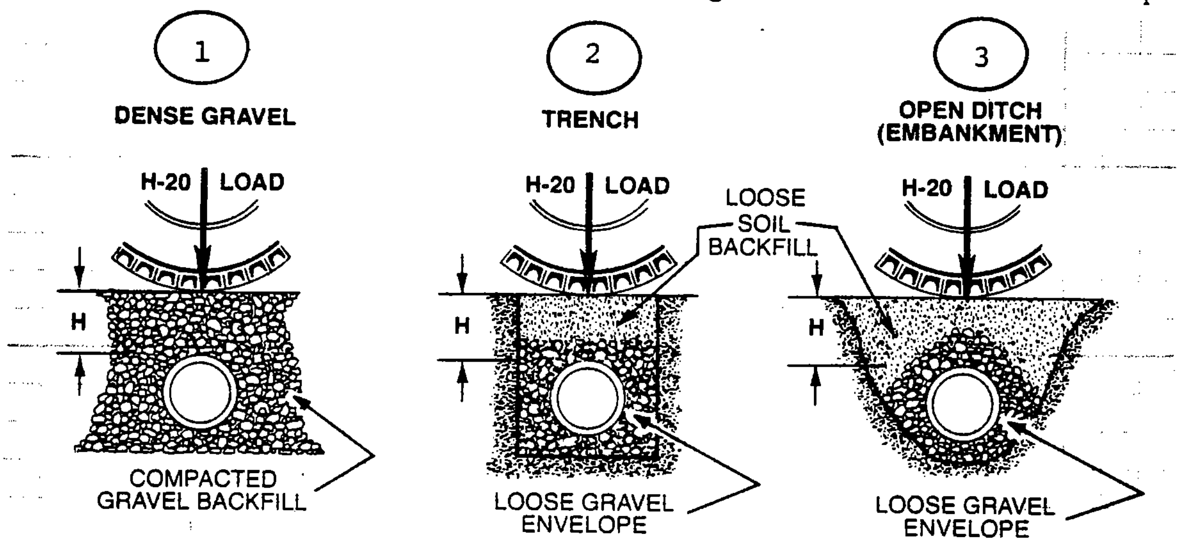


Figure 9. Three cases of installation for testing to find the ring deflection as a function of depth of cover H. Case 4 consisted of complete envelope and backfill with native soil. The response of case 4 in these tests was similar to case 3.

Above 24-inch, more testing is needed because installation procedures would be different. In these tests, the in-situ soil was good enough to support the tire loads without rutting. To be conservative the "select" soil envelope material was pit run gravel which contained a fairly high proportion of fines. The standard AAHSTO H-20 truck loading is shown in figure 10.

RESULTS

The field test results are shown in figure 11 for each of the three installation Cases. Here it is noteworthy that each curve approaches a vertical asymptote. This indicates that:

1. Ring deflection is sensitive to soil cover H/D; and

2. The absolute minimum soil cover is well defined at the asymptote. However, a safety factor is needed to keep ring deflection under same specified amount. For many purposes a deflection of 10 percent is acceptable. However, a deflection less than 5 percent may be required in some cases.

Recommended minimum cover

With a safety factor, the minimum cover recommended is shown in figure 11 at the right edge of each cross hatched zone. The numerical value is shown in each box for installation cases 1, 2, and 3. For Case 4, (native envelope and backfill) use Case 3 values. The recommended minimum cover for the 3 installation cases, as shown in figure 11, are reproduced in the following table:

RECOMMENDED MINIMUM SOIL COVER H/D*

INSTALLATION CASE		1	2	3 & 4
VALUE OF H/D	=	1.0	1.25	1.67
OR H/D	=	$\frac{5}{5}$	$\frac{5}{4}$	$\frac{5}{3}$
SAFETY FACTOR	=	2.0	2.8	4.0

* D is the inside diameter, which is the nominal diameter used by manufacturers.

The safety factor is at least 2.0 because the fraction of the surface pressure (wheel load) that reaches the tube as given by the well-known Boussinesq equation 4/ varies inversely as the square of the depth of cover H. In other words, for case 1, by increasing the depth from H/D = 0.69 at the asymptote, to H/D = 1, the pressure reaching the tube decreases to $(0.69/1.00)^2 = 0.5$. Clearly the safety factor is 2. The safety factor increases to 2.8 for case 2 and to 4 for cases 3 & 4. Higher safety factors for cases 2, 3 & 4 are recommended because of the greater risk that the "select" soil arch may not be carefully shaped and so could break down.

4/ Terzaghi and Peck, Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley, N.Y. p271.

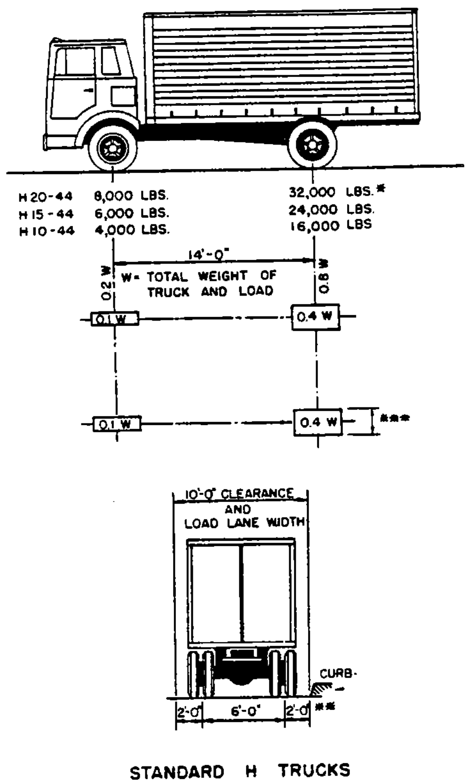


Figure 10. AASHTO standard "H" trucks. The H-20 standard truck loading was used in these studies.

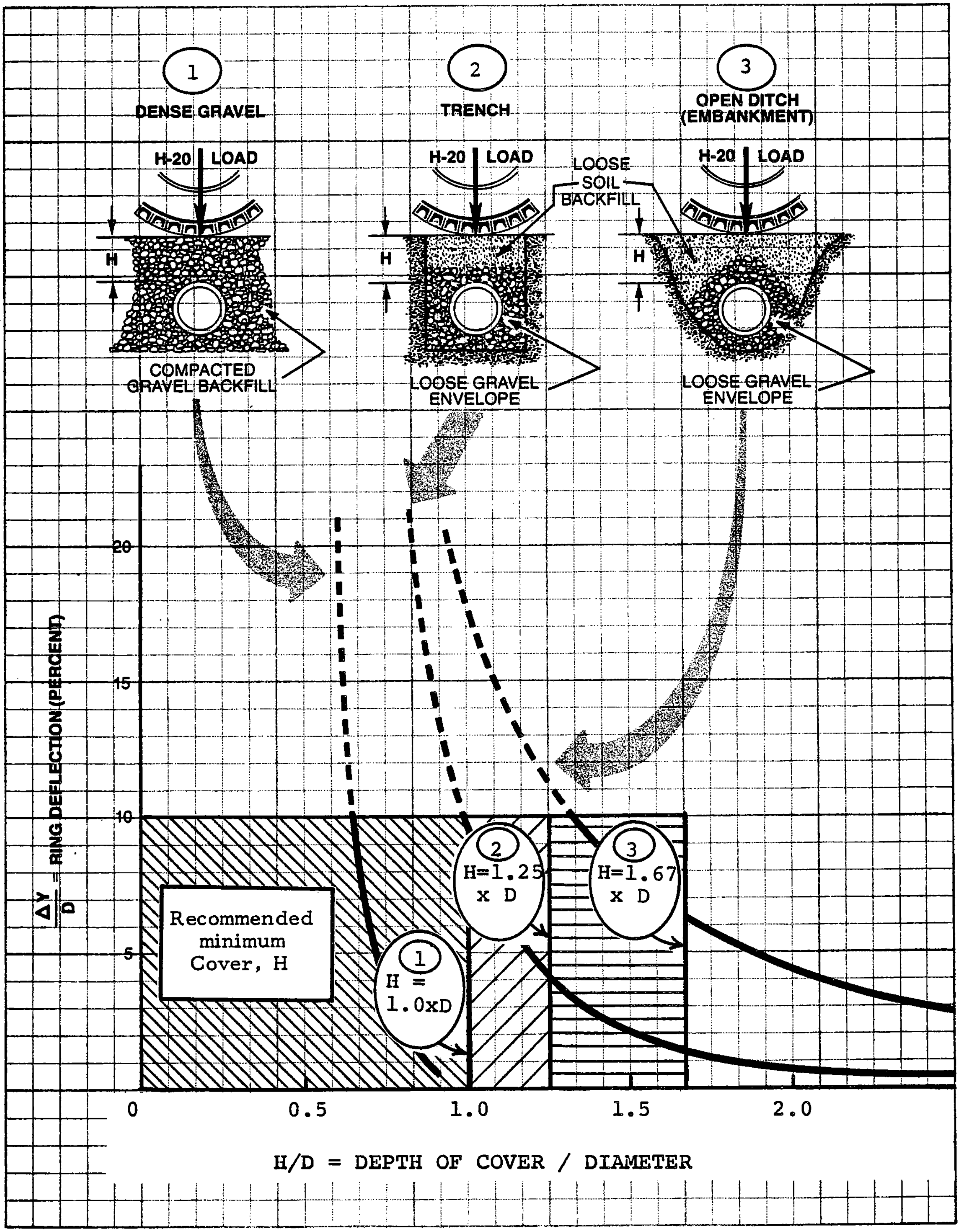


Figure 11. Results of testing shows ring deflection, $\Delta y/D$ as a function of depth of cover term, H/D, for AASHTO standard H-20 truck load. Recommended minimum cover is indicated for each installation case.

Figure 12. Corrugated polyethylene tubing is installed with a pit-run gravel envelope in a trench. This is a Case 2 installation.

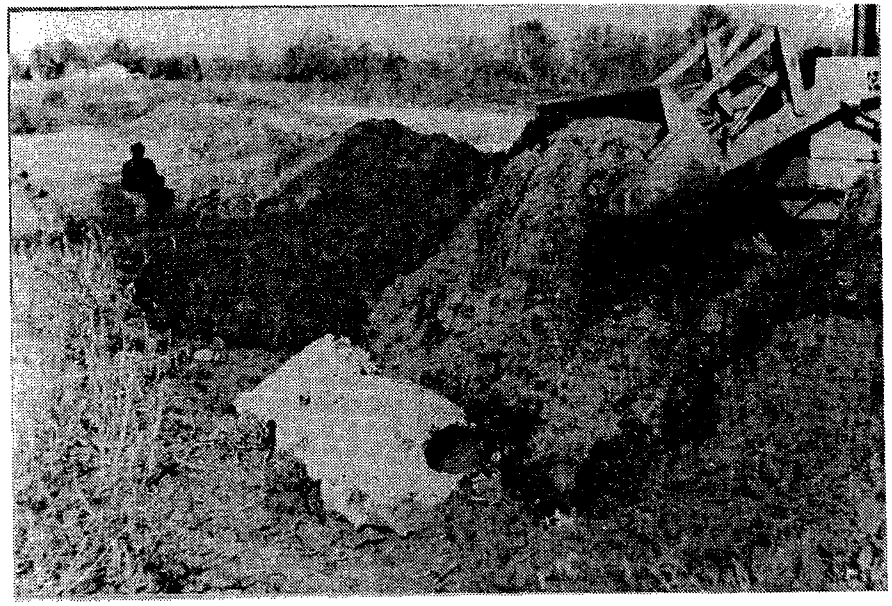


Figure 13. After the gravel envelope is placed, in a ditch in this case, native soil is used to complete the backfill. This is a Case 3 installation. The gravel envelope comprises four to six inches of bedding and four to six inches of "select" soil envelope (pit-run gravel, which is conservatively poor for a "select" soil envelope). The "select" soil is specified with a friction angle of no less than 30°, a compressibility of no less than 5% at the vertical soil pressure anticipated, and adequate qualities as a filter if the tubing is to serve as a slotted drain.

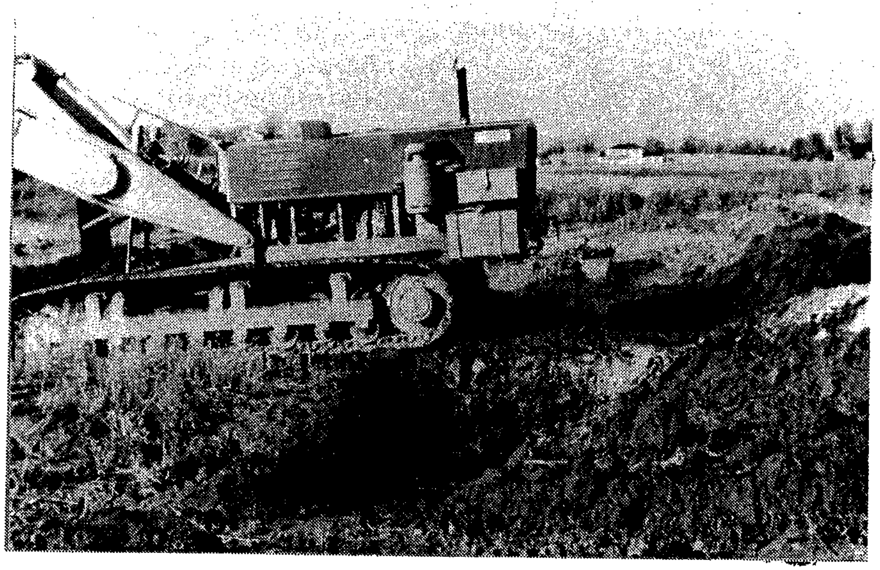


Figure 14. The backfill soil was compacted lightly by passing the trencher over the surface. The track pressure was approximately 3 psi This compares about the same as bearing pressure under an average person wearing ordinary shoes.

Figure 15. A completed installation after compaction.

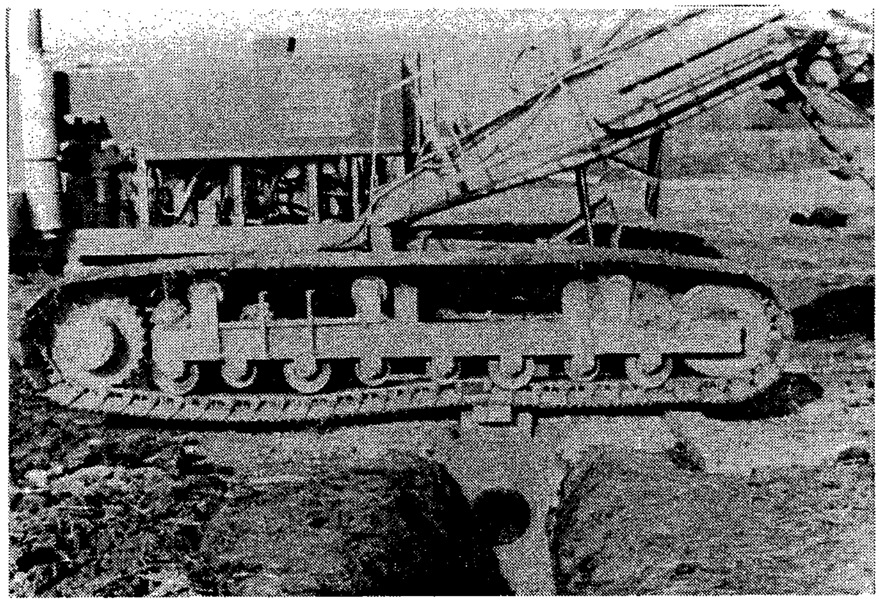


Figure 16. An H-20 truck load is applied by the 32000 pound trencher on blocks with an area equal to the contact areas of the H-20 dual tires.

Figure 17. Inside diameters before and after loading were measured by means of a special inside caliper.

CONCLUSIONS

From this study and the related testing program, the following can be concluded:

1. Corrugated polyethylene tubing is flexible. Longitudinally it conforms with the bedding and so eliminates the need for beam design. In ring deflection, it conforms with the soil envelope and so develops essentially a uniform external pressure which eliminates the need to design a moment resisting ring. Thus, the structural requirements of corrugated polyethylene tubing are considerably less than other more rigid pipe or tubing products.

2. Polyethylene has less stiffness and less strength than rock. It should not have to resist the bearing forces of large rocks. A select soil envelope with no rocks greater than one inch in diameter is recommended for buried corrugated polyethylene tubing.

3. Ring compression design also can be eliminated if a "select" soil envelope is specified. This is so, because at constant strain, the polyethylene relaxes without failure and the "select" envelope picks up the vertical load in arching action over the ring. The material in the envelope is "select" if it has a friction angle greater than 30°; a compressibility less than 5% at the anticipated vertical soil pressure; and in the case of a slotted drain, 5/ if it meets the conditions for an adequate filter. Any deviation from these requirements should be investigated.

   The question arises as to how much "select" soil envelope material is required. The answer is - "the envelope should extend outward far enough to fully contain any potential 'failure wedge' for an envelope material having the minimum allowable friction angle". The subtended failure wedge angle is two times the friction angle, which in this case is 2 x 30° = 60°. For the trench case, assuming a poor soil with essentially zero internal friction, that provides containment of the envelope material only, the envelope dimensions are 2D x 2D. For both the trench case and for the open-ditch, or full projection case, the minimum edge distance is D/2, as shown in figure 18. For this usage, D is the tubing outside diameter.

4. With a select soil envelope, if corrugated polyethylene tubing is designed within maximum and minimum depths of cover (see the design charts) its structural performance is excellent. If a Case 1 installation is specified, maximum and minimum cover, H, and bedding depth, B, are as recommended in table 1.

5/ SLOTTED UNDERDRAIN SYSTEMS, Implementation Package 76-9, U. S. Department of Transportation, Federal Highway Administration, Offices of Research and Development Implementation Division, June 1976

Table 1. Height of cover limits for ADS heavy duty corrugated polyethylene tubing installed with gravel or crushed stone envelope for installation Case 1 (dense gravel with high internal friction).

D Inside Diameter	Minimum Values for live load (H-20)		Maximum Cover H for Dead Load (Soil Cover) Plus Live Load (H-20)
	Cover Depth H	Bedding Depth B	
(inches)	(inches)	(inches)	(feet)
6	12	4	230
8	12	4	
10	12	5	199
12	12	6	
15	15	6	132
18	18	8	

It is recommended, on the basis of much experience in controlling field installations, that the least minimum cover should be 12 inches, even for the 6, 8, and 10 inch diameter tubing. From a structural point of view, corrugated polyethylene tubing can be used successfully for land drainage, for culvert, and for other services as buried tubing.

5. For practical purposes, the amount of deflection in the corrugated polyethylene under load, is equal to the strain in the sidefill material. In other words, the compression in the sidefill material governs the deflection of the tubing. Therefore, the selection and placement of the "select" soil envelope is the means by which the performance of the combined tubing-envelope structure is controlled. The more dense the envelope sidefill material, the less the compressibility and hence the smaller the tubing deflection.

The design criteria summarized above apply to buried corrugated polyethylene tubing up to 24 inch diameter. Some typical applications are shown in figures 19 and 20.

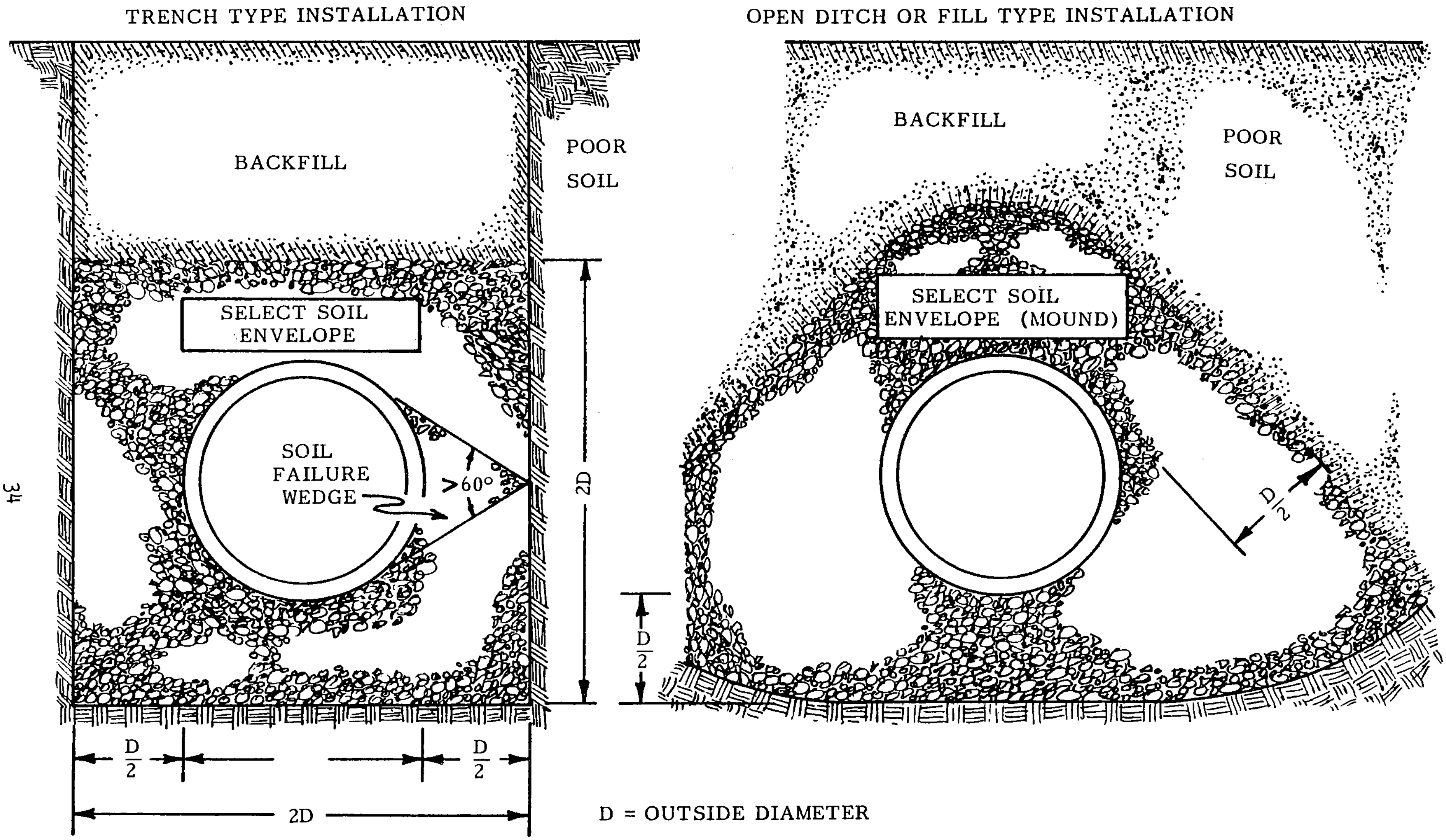


Figure 18. Shown here are minimum dimensions for "select" soil envelopes (gravel, crushed stone, or other high internal friction materials) that are required when corrugated polyethylene tubing is buried or backfilled with soils that have poor structure and low internal friction. Such soils provide containment of the envelope only in the form of an external pressure force. Note that if the minimum dimension, D/2 is maintained, the "failure wedge" is contained within the envelope boundary.

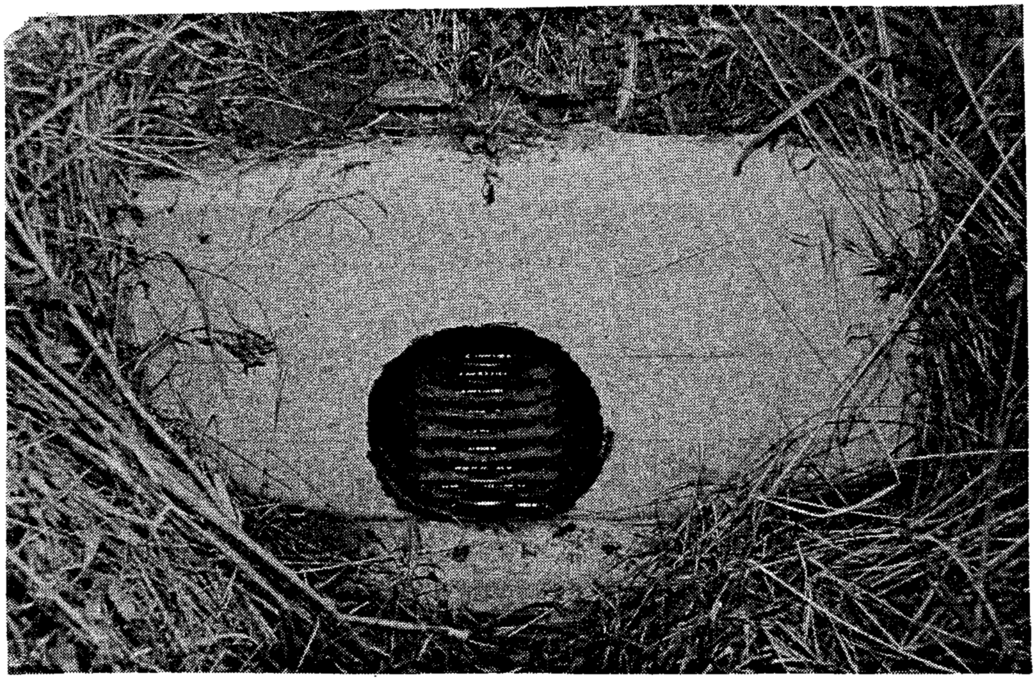


Figure 19. Twelve-inch corrugated polyethylene tubing used for surface water drainage along a county road in Madison County, Ohio. Drain, 1800 feet long, was finished off with a concrete headwall at both ends.

Figure 20. A driveway application for 12-inch corrugated polyethylene tubing. Eleven cubic yard concrete truck--total load 48,000 pounds, with 9,000 pounds per set of duals.

REVISION OF THE 1967 AASHTO MANUAL ON FOUNDATION INVESTIGATION

Allen W. Hatheway, PE PG[1]

The Transportation Research Board (TRB), through its National Cooperative Highway Research Program (NCHRP), has recently funded a 21-month contract with Haley & Aldrich, Inc., to compile a draft revision of the 1967 AASHTO Manual on Foundation Investigations.

The revision effort recognizes that significant advances in engineering geological, geotechnical and engineering geophysical techniques, among others, have come about in the years since compilation of the present manual, which is now out of print. In this same time period, transportation facility design and construction has shifted broadly to encompass not only highways but a wide variety of non-traditional structures. Such changes are bringing a notable increase in the familiar dimensions of highway structures; bridges will be longer and founded on less favorable materials; tunnels and underground structures will be utilized to avoid sociological impacts and to save transportation energy; slopes and embankments will be higher and longer; and new construction methods will change the more familiar foundation investigation requirements.

Advances that have paralleled these developments are numerous and the pace of development has meant that many of the techniques have been proven but are not yet widely disseminated. The revision intends to bring together the best of these developments as found in the published and unpublished reports of the Federal DOT activities, the various state transportation agencies, consultant experience and the appropriate literature.

In addition to revision of existing chapters, the outline will be expanded to include treatment of such topics as:

- engineering geophysics
- earthquake engineering
- geohydrology
- rock mechanics
- construction mapping
- instrumentation
- blasting and rock excavation
- erosion control
- contract documents
- remote sensing

In all cases, a major objective will be to identify key aspects of application of techniques, along with advantages, pitfalls, and background references. Contributions from persons involved in the engineering geology and geotechnical engineering of transportion structures are solicited. The final draft of the revised Manual is to be submitted by December 1980.

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File No. 4384
31 July 1979
A.W. Hatheway

SECOND DRAFT OUTLINE

REVISED AASHTO MANUAL
ON
SUBSURFACE INVESTIGATIONS
FOR TRANSPORTATION SYSTEMS

N.C.H.R.P. RESEARCH PROJECT 24-1

HALEY & ALDRICH, INC.
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS
238 MAIN STREET
CAMBRIDGE, MASSACHUSETTS

31 July 1979

SECOND DRAFT OUTLINE
REVISED AASHTO MANUAL ON SUBSURFACE INVESTIGATIONS FOR TRANSPORTATION SYSTEMS

1.0 INTRODUCTION

- 1.1 Background
- 1.2 Purpose of Manual
- 1.3 Existing Standards
- 1.4 Limitations
- 1.5 Contents

2.0 SUBSURFACE DATA REQUIREMENTS

- 2.1 General
- 2.2 Common Requirements
  - 2.2.1 Material properties
  - 2.2.2 Excavation parameters
  - 2.2.3 Support of Structural Loads
  - 2.2.4 Slope Stability
  - 2.2.5 Ground Water
  - 2.2.6 Seismic Evaluations
  - 2.2.7 Geologic Hazards
- 2.3 Retention Structures
- 2.4 Cuts and Embankments
- 2.5 Bridges and Viaducts
- 2.6 Turnpikes and Freeways
- 2.7 Railroads and Rapid Transit
- 2.8 Underground Structures
- 2.9 Pavements
- 2.10 Erosion Control Features

3.0 CONDUCT OF INVESTIGATIONS

- 3.1 General
- 3.2 Site Selection
- 3.3 Planning
  - 3.3.1 Concept Development
  - 3.3.2 Phasing
  - 3.3.3 Reconnaissance
- 3.4 Remote Sensing
- 3.5 Reconnaissance
  - 3.5.1 Purpose
  - 3.5.2 Levels of Effort

9.0 REPORT COMPILATION

- 9.1 General
  - 9.1.1 Types
    - 9.1.1.1 Factual Reports
    - 9.1.1.2 Interpretive Reports
  - 9.1.2 Use
    - 9.1.2.1 Design
    - 9.1.2.2 Construction
- 9.2 The Data Report
  - 9.2.1 Introduction
  - 9.2.2 Geologic Mapping
  - 9.2.3 Subsurface Exploration and Field Testing
  - 9.2.4 Laboratory Testing
- 9.3 The Interpretive Report
  - 9.3.1 Design considerations
  - 9.3.2 Construction considerations
  - 9.3.3 Special considerations
- 9.4 Presentation of Data
  - 9.4.1 Logs
  - 9.4.2 Profiles
  - 9.4.3 Maps
  - 9.4.4 Charts
  - 9.4.5 Tables
- 9.5 Contract/Legal Implications

APPENDICES*

*Includes simple guideline procedures or reference to AASHTO/ ASTM standards

- A. Drilling Equipment
  - A-1 Soil
  - A-2 Rock
- B. Sampling Methods
- C. Field Testing - Soil
- D. Field Testing - Rock
- E. Field Data Collection Forms
- F. Laboratory Testing
- G. Clay Mineral Determinations
- H. Laboratory Testing - Rock
- I. Engineering Properties of Rock
- J. Petrography

LIST OF ILLUSTRATIONS (Working Documents)

References

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION
MATERIALS OFFICE

A SUMMARY OF THE USE OF SAWDUST FOR HIGHWAY FILLS

by
N. C. Jackson
Pavement Design Engineer

Presented at the 30th Annual
Highway Geology Symposium
Portland, Oregon
August 8-10, 1979

R. V. LeClerc
Materials Engineer

Materials Office
Report No. 153

August, 1979

A Summary of the Use of Sawdust for Highway Fills

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Department of Transportation

Abstract

Prompted by a need to correct numerous small slides on existing highways, the Washington State Department of Transportation has, since 1972, used wood-waste products as lightweight embankment materials. With the lack of definitive information concerning the long-term performance of sawdust as an embankment material, its use has been limited to applications where there is either a substantial economic advantage or specific circumstances which limit normal construction techniques. Its use has also been confined to the coastal region which experiences both moderate temperatures and high rainfall. To date, ten projects involving the use of wood-waste products as a lightweight embankment material have been constructed. Presented is a summary of the projects and their resulting performance, along with the general engineering properties, determined from a combination of laboratory and in situ testing, used in design.

A SUMMARY OF THE USE OF SAWDUST FOR HIGHWAY FILLS

By

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ABSTRACT

Prompted by a need to correct numerous small slides on existing highways, the Washington State Department of Transportation has, since 1972, used wood-waste products as lightweight embankment materials. With the lack of definitive information concerning the long-term performance of sawdust as an embankment material, its use has been limited to applications where there is either a substantial economic advantage or specific circumstances which limit normal construction techniques. Its use has also been confined to the coastal region which experiences both moderate temperatures and high rainfall. To date, ten projects involving the use of wood-waste products as a lightweight embankment material have been constructed. Presented is a summary of the projects and their resulting performance, along with the general engineering design properties determined from a combination of laboratory and in situ testing.

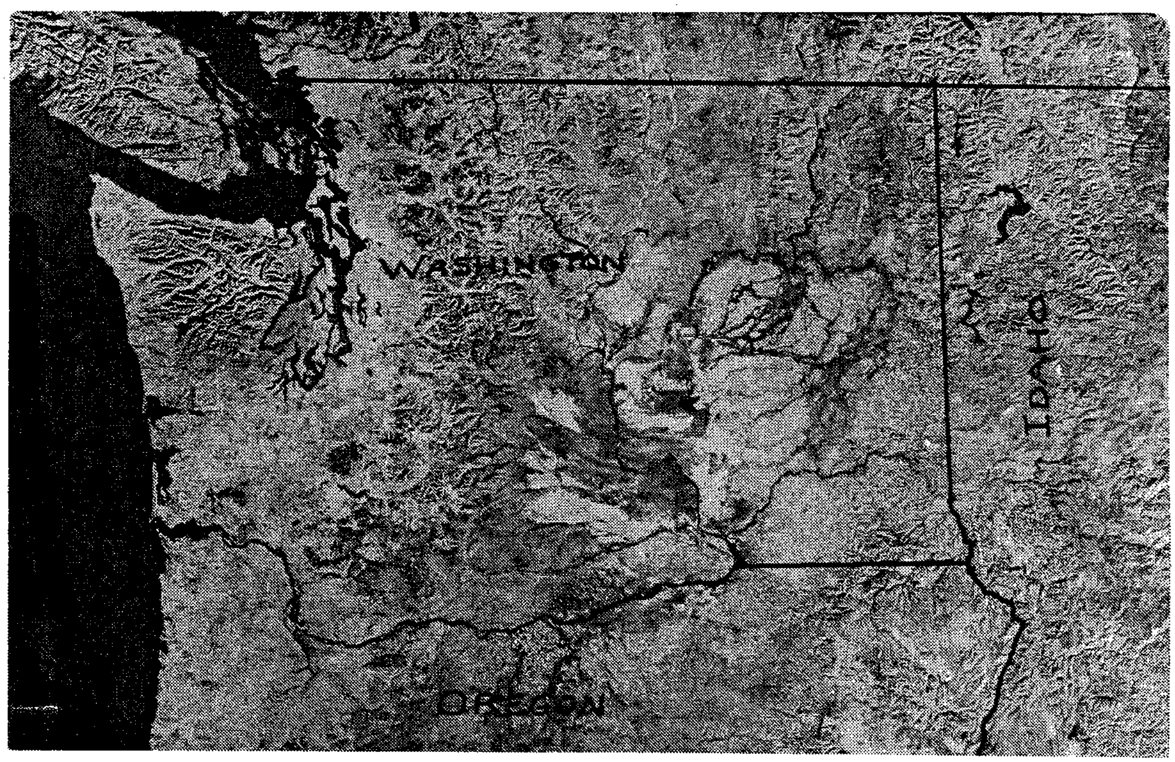


Figure 1 - Vicinity Map

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

INTRODUCTION

Both climate and geology combine along the southwest coast of Washington to produce a region which has numerous existing natural landslides, with a high potential for new landslides resulting from any change in land form. The geologic units that are found in this area range in age from Eocene to Recent, the oldest being the Crescent formation, a series of basaltic rocks, the youngest the recent alluvium and tidal mud commonly found in the low-lying river mouths. The great preponderance of landslides occur in only one of the various formations comprising this very thick series of rocks. Somewhere near the middle of the stratigraphic column is a formation known as the Astoria. It varies from claystone to conglomerate with an intraformational basalt.

The Astoria formation is most easily differentiated from the underlying Lincoln Creek formation by the abundance of mica and carbonaceous material, lack of tuffaceous material, and better bedding. It differs from the overlying Montesano formation in that the sandstone beds are usually fine-grained and silty, and the conglomerate poorly sorted. The siltstone units contain considerable amounts of clay minerals and are, therefore, much less competent in outcrop than the Montesano.

The inordinate number of landslides in this formation is probably due to the high percentage of both mica and clay minerals. Bedding may have an influence in some areas, but in the majority of cases the mineralogy of the material combined with high moisture to cause sliding. The heavy rainfall, which often exceeds 100 inches per year, in addition to producing high ground moisture, has produced deep and intensive weathering, resulting in greater instability. The early economy of this region was primarily timber-oriented. Early roadbuilders routinely used wood products as construction materials. Many of today's roads overlay original corduroy roads. These are roads that were built over logs or planks, usually placed crosswise to the direction of travel. Sawdust fills are also quite common in the region. Many of the logging towns located next to rivers or salt water were built on sawdust fills placed over mud or tide flats.

With this heritage, it is not surprising that there has been, for some time, many proponents who have recognized the potential of using wood-waste products to help solve our most perplexing stability problems in the region.

In March of 1972, a slide took out a section of SR 101 several miles south of the town of Cosmopolis, an all too common occurrence on this section of highway. The slide involved approximately three-fourths of the roadway for a length of 200 ft, dropping it 40 ft. This highway is a major commercial route for the area's timber industry. To reestablish the route, the local maintenance division used wood-waste products, for the first time, to reconstruct the failed section. The material was chosen because its light weight would reduce the driving force on the old slide mass, and because its strength is relatively unchanged by major changes in moisture content, thus allowing it to be used in very wet weather. In a region where many attempts at slide correction or stabilization have been only marginally successful, this "emergency work" seemed to do the job.

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

Within a year, sawdust was again used in an attempt to stabilize a slow-moving slide mass within a mile of the first project. This section had required constant effort to maintain adequate geometry and presented the constant threat of major movement, which might again close the highway. On this project, the top 10 feet of the roadway embankment in the area of the slide was removed and replaced with sawdust, and internal and diversionary drainage was installed. Both projects have been successful in stopping all slide movement, to date.

Details concerning these projects are covered in a Status Report published by the Federal Highway Administration in 1974, authored by David Nelson and Bill Allen. A film which shows the sawdust embankment construction was also made at the time, and is available through the Federal Highway Administration.

We have used wood-waste products on nine additional projects and are presently considering their use on another project. The wood-waste product used has been either sawdust, hog fuel, planer or bark chips, or a combination of one or more of these products. For simplicity's sake, I will not try to differentiate between these products, but will refer to them simply as sawdust.

On most of these projects, sawdust has been used in an attempt to reduce the driving force of the roadway embankment on existing slide masses, as typified by our first two projects. It has, on a few projects, been used as a lightweight fill material to construct embankments over very soft foundation soils, generally consisting of peat and/or organic silts and clays. Considering the number of installations where sawdust was used, a discussion of each is not warranted. Instead, the two specific uses of sawdust will be discussed using a couple of typical projects as examples.

LANDSLIDE RECONSTRUCTION OR STABILIZATION

Our greatest use of sawdust has been in the area of landslide control, identical in all respects to our first two projects. We have thus used sawdust to either reconstruct a section of highway lost as a result of a landslide, or replaced a portion of the roadway involved in a slow-moving slide, in an attempt to stabilize the slide.

As you might expect, the reconstruction of a missing section of the roadway prism, varying from a short section of shoulder to a portion of the roadway, accounts for the greatest number of installations, but the least quantity of sawdust. We have approximately a dozen such installations. These projects typically have been put together by the Project Engineer in the field, either as an individual project or as part of a larger pavement rehabilitation project. Each installation has been quite simple. Typically, the old slide mass is dressed up, internal and diversionary drainage is installed if possible, and the missing section of roadway is reconstructed using sawdust. This use has been simple and successful. Though some of the sections have experienced minor consolidation of the sawdust, there has been no renewed movement of the original slide.

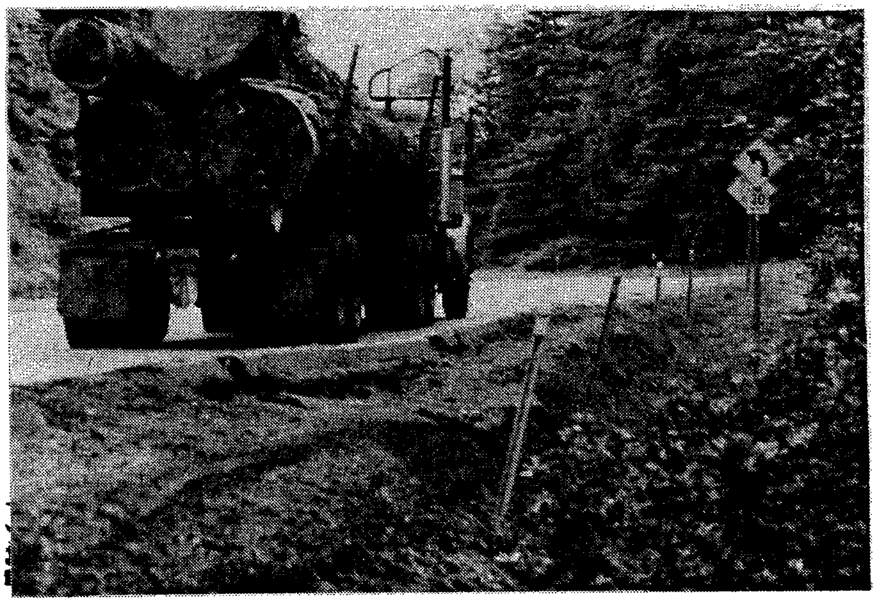


Figure 2 - Sawdust Fill - 6 years old

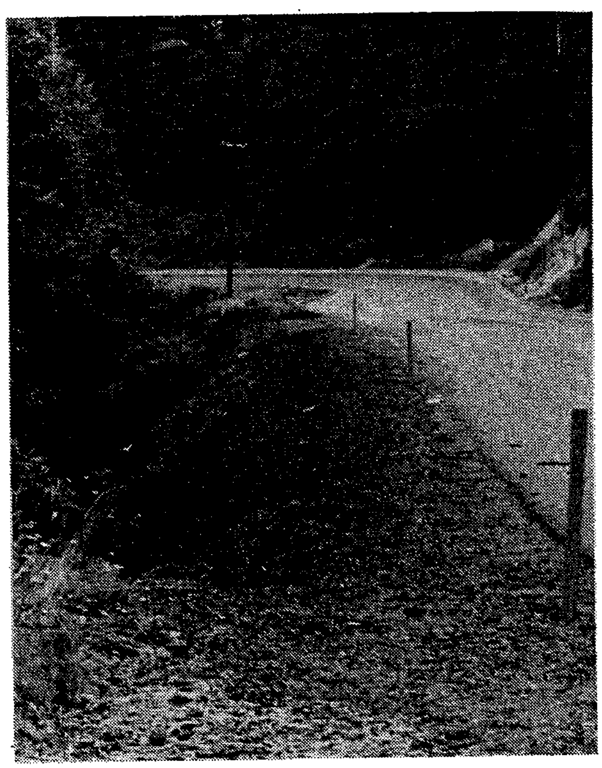


Figure 3 - Sawdust Fill - crack typical
on fills with steep side slope

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

On four separate projects, sawdust was used in an attempt to stabilize an active slow-moving landslide. In most of these sections, continued patching by Maintenance personnel to provide a passable grade had actually resulted in total pavement depths ranging from four to eight feet. These sections thus maintained have become quite narrow, with most unusual horizontal and vertical geometry. In addition, as the driving force is increased on the slide mass, one must recognize that the risk of total failure also increases.

Rock Crusher Slide repair on SR 101, between Raymond and Aberdeen, is typical of this type of project. This is one of many sections on this route where the roadway apparently crossed over an existing landslide. There have been several attempts at stabilizing the slide through various drainage strategies since at least some time in the 1940's.

In 1975, a soils investigation was begun to determine means of stabilizing the slide. Five test holes were drilled to depths of 70 ft in the slide block, with inclinometer casing installed in each. The test holes encountered 35 ft to 50 ft of soft to stiff clay-sands and silts overlying very stiff silts and clays layered with soft shales. Although there were several active slide planes, the major plane appeared to be in the deeper clays, silts, and soft shales. Our analysis indicated that we could gain a 20 to 30 percent increase in stability by lowering the water level in the slide mass 15 to 30 ft. This could then be improved an additional 10 percent by replacing the top 15 ft of embankment with sawdust, or 20 percent by replacing the top 25 ft with sawdust. As there was some question concerning the extent of the slide mass, we chose to use both drainage and sawdust in the correction. A buttress was considered but rejected because it was much more efficient and cheaper to reduce the driving force than try to counterbalance it.

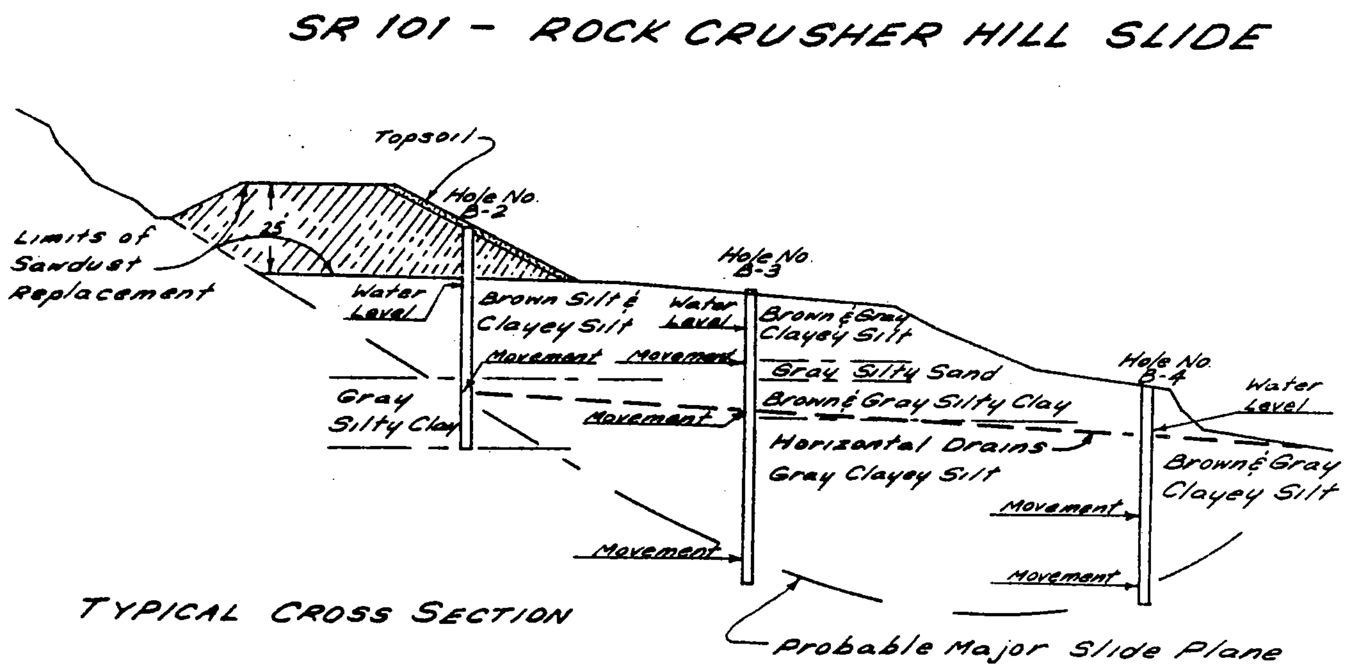


Figure 4 - Cross Section - Rock Crusher Hill Slide

The project was constructed in the summer and fall of 1978. There has since been no sign of renewed slide movement. It has, however, experienced some significant consolidation of the sawdust, resulting in the surfacing dropping uniformly six to eight inches within the limits of the sawdust replacement. It is interesting to note that the guardrail placed some time after surfacing shows only minor change in grade.

In general, we have experienced the greatest consolidation at the ends of the sawdust fill where the compactive effort is least, most of it occurring quite rapidly after completion of the surfacing. Some areas have required minor maintenance patching. As we have anticipated added long-term maintenance cost over more standard construction, this comes as no surprise.

To reduce the effect of post-construction consolidation of the sawdust, we recommend that the slide slope contact between sawdust and natural ground be as flat as possible. The amount of consolidation may be reduced by placing a small surcharge, if construction timing allows. If not, the surface of the roadway may be cambered in proportion to the depth of sawdust.

We have also experienced some slope distress on the outside edge of the sawdust embankment where the sides were constructed to 1:1 slopes. As the distress is minimal and not yet endangering the roadway, we have not taken any action. We do, however, recommend maximum side slopes of 1½:1 in sawdust.

Other than the few problems noted, all of these slide corrections have been successful. There are, at present, no signs of renewed movement in any of the old slide masses. Considering past successes in these endeavors in this region, we are quite satisfied with the results.

Figure 5 - Rock Crusher Hill Slide Correction

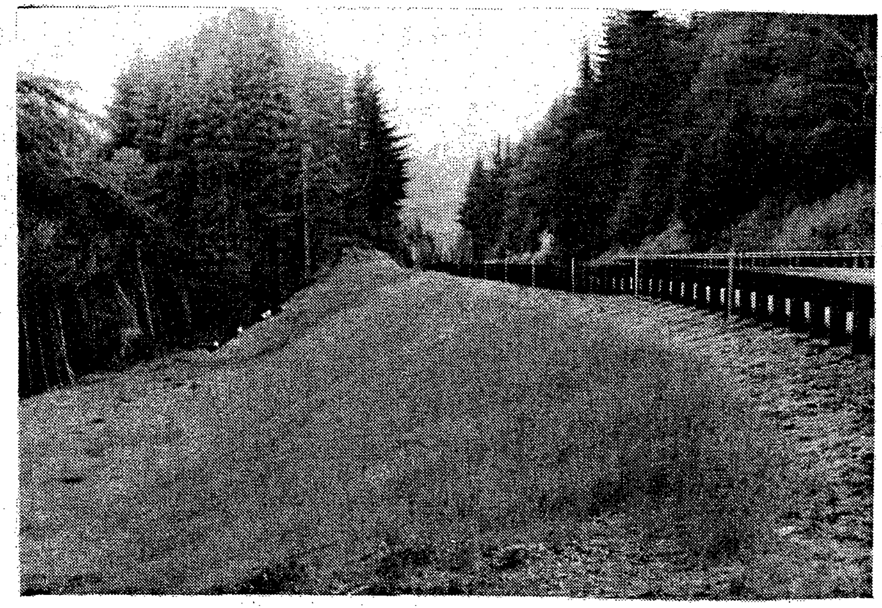


Figure 6 - Sawdust Fill - Rock Crusher Hill Slide

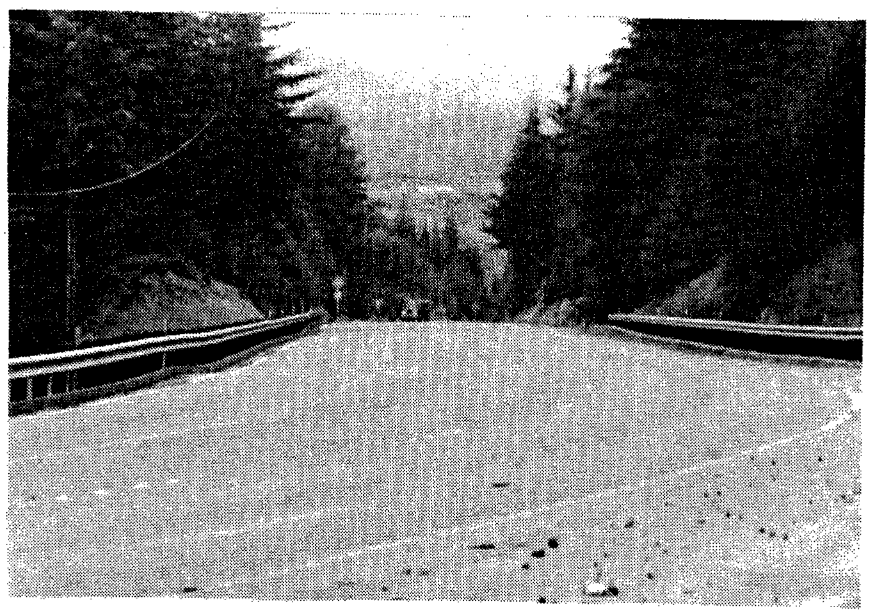


Figure 7 - Rock Crusher Hill Slide - Dip defines limits of sawdust replacement

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

LIGHTWEIGHT EMBANKMENTS OVER SOFT FOUNDATION SOILS

We have, on four separate projects, used sawdust to construct either a new embankment or to widen existing embankments over deep deposits of very soft peat and/or organic silts and clays. Typical of this type of project was a 100,000 CY sawdust embankment constructed as part of a grading and surfacing project on SR 101 between South Bend and Raymond. Here, the alignment crossed a 600-ft-long section of tidal flats before tying back into the existing alignment. Test holes indicated that the soils consisted of 20 to 70 ft of very soft peat, together with organic silts and clays overlying steeply dipping sandstone. Standard penetrometer blow counts of less than one were encountered to a depth of 25 to 30 ft.

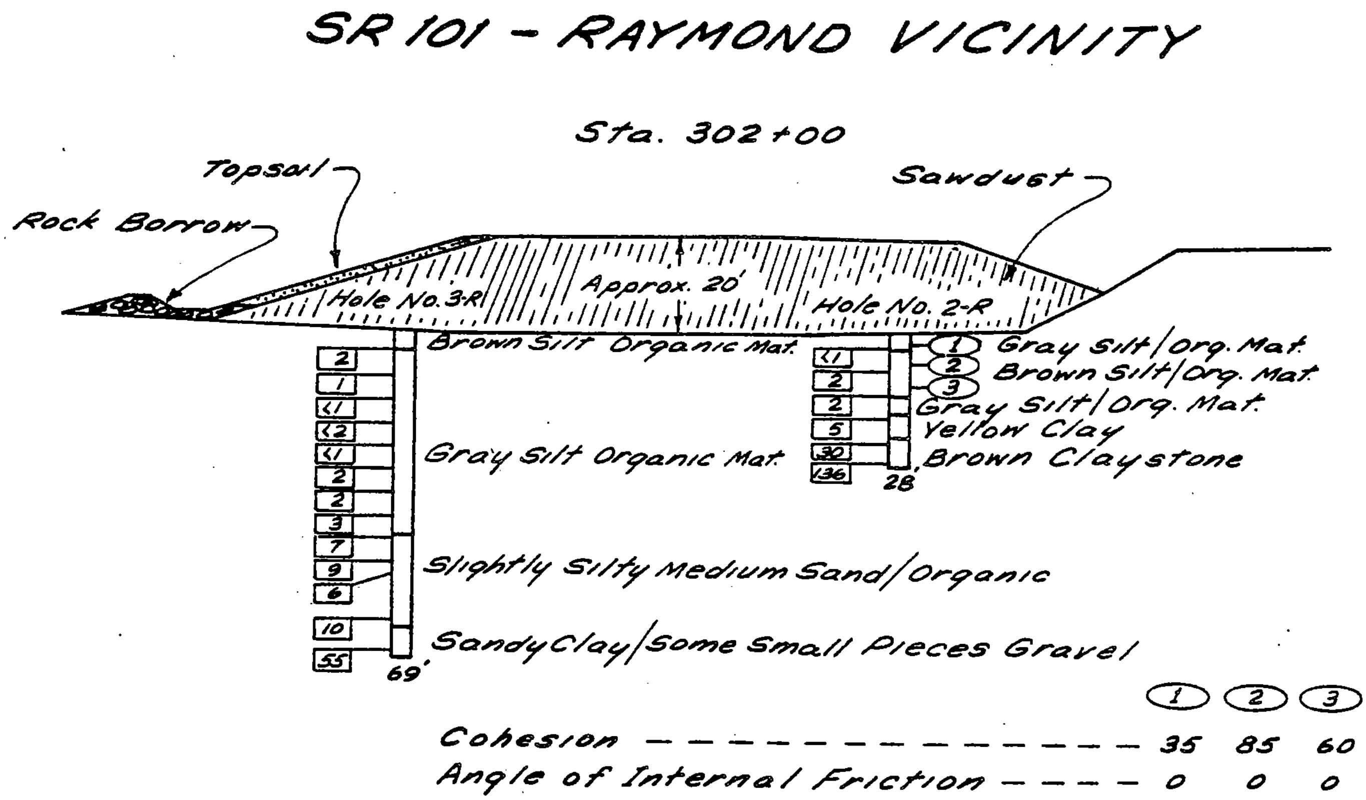


Figure 8 - Raymond Sawdust Fill

To construct a reasonably stable embankment, our original design called for removal of the very soft foundation soil to a depth of 20 to 25 ft, to be replaced with quarry rock. Even with the partial removal of the very soft foundation soils, our analysis indicated that we could anticipate total embankment subsidence of over 3 ft, occurring rather slowly. Thus, a surcharge was also called for.

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

With fairly recent success of the first two sawdust installations 20 miles north of this location, an alternate design using sawdust was considered. It was determined that a stable sawdust embankment could be constructed without removal of the underlying foundation soils. It was also determined that, with the markedly reduced settlements, a surcharge was not warranted. The original comparative estimate made in 1973 indicated approximately $250,000 savings by using sawdust. Considering the saving in money and time, this option was selected. By today's costs, the difference could easily be over $500,000.

The sawdust embankment was constructed during the summer of 1976. It was topped off with ballast and served as a detour through the winter, and was finally paved in the spring of 1977. The performance of the embankment has been about as expected. Considering the lengthy construction of the embankment, we experienced somewhat more consolidation of the sawdust than expected.

Maintenance patching was required in one area where a trench was cut in the embankment to place a 12-inch water line crossing after completing the embankment. It was not possible to backfill the sawdust to the same density as the surrounding embankment, thus resulting in additional consolidation in the area of the trench after construction. It is interesting to note that this did not show up under the weight of partial surfacing and traffic during the winter, as one might expect it should.

Most of our in situ testing was performed on this embankment. The various tests and subsequent results will be discussed later in the report.

The remaining projects involved the widening of an existing embankment over similar, very soft foundation soils. As with the first project, the difference in cost between using sawdust and other more standard construction techniques was significant on each project. All but one have been in service for several years. The one, though constructed last year, is not yet in service as it is part of a project being completed this year. A surcharge has been used on several projects to minimize post-construction subsidence and consolidation. This appears to have worked as expected, as we have not yet experienced any post-construction distortion of the roadway prism where it was used.

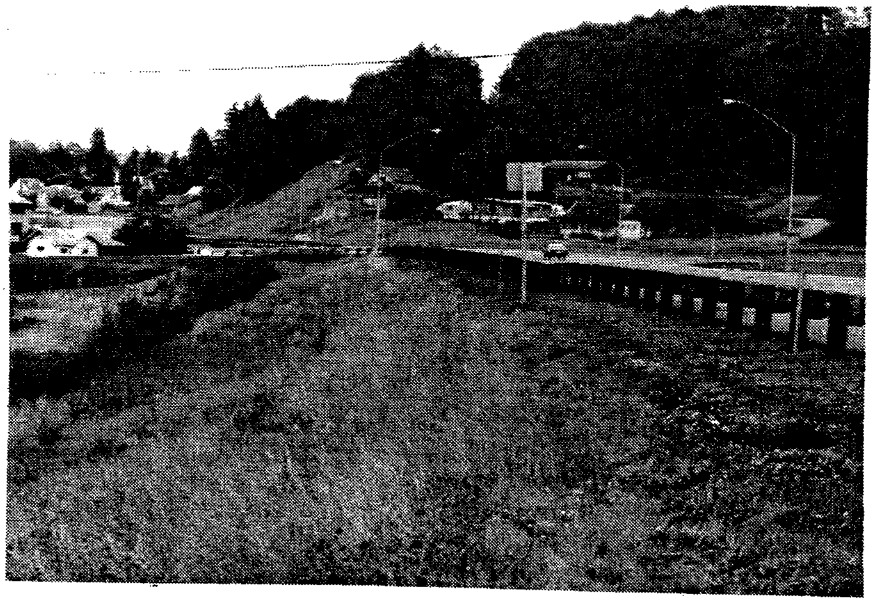


Figure 9 - Sawdust Fill - Raymond Vicinity

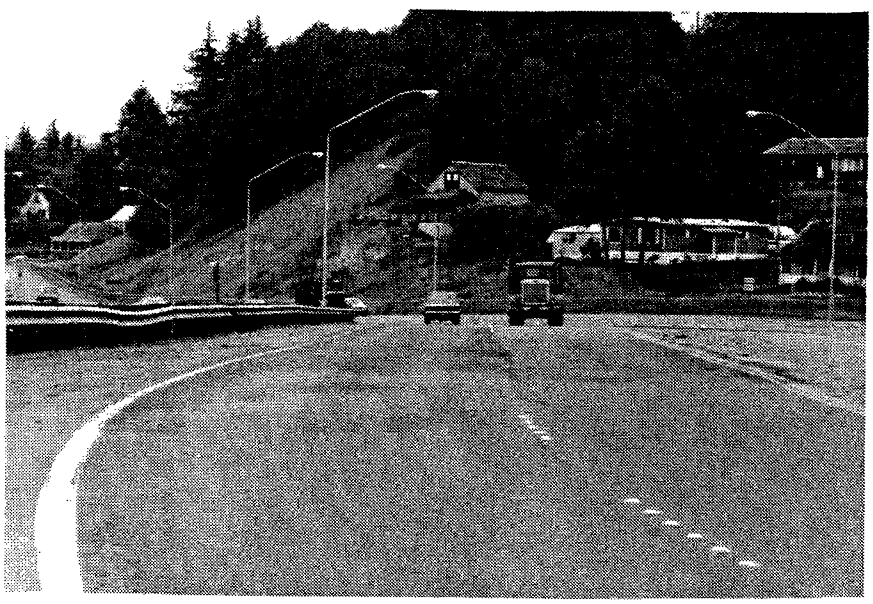


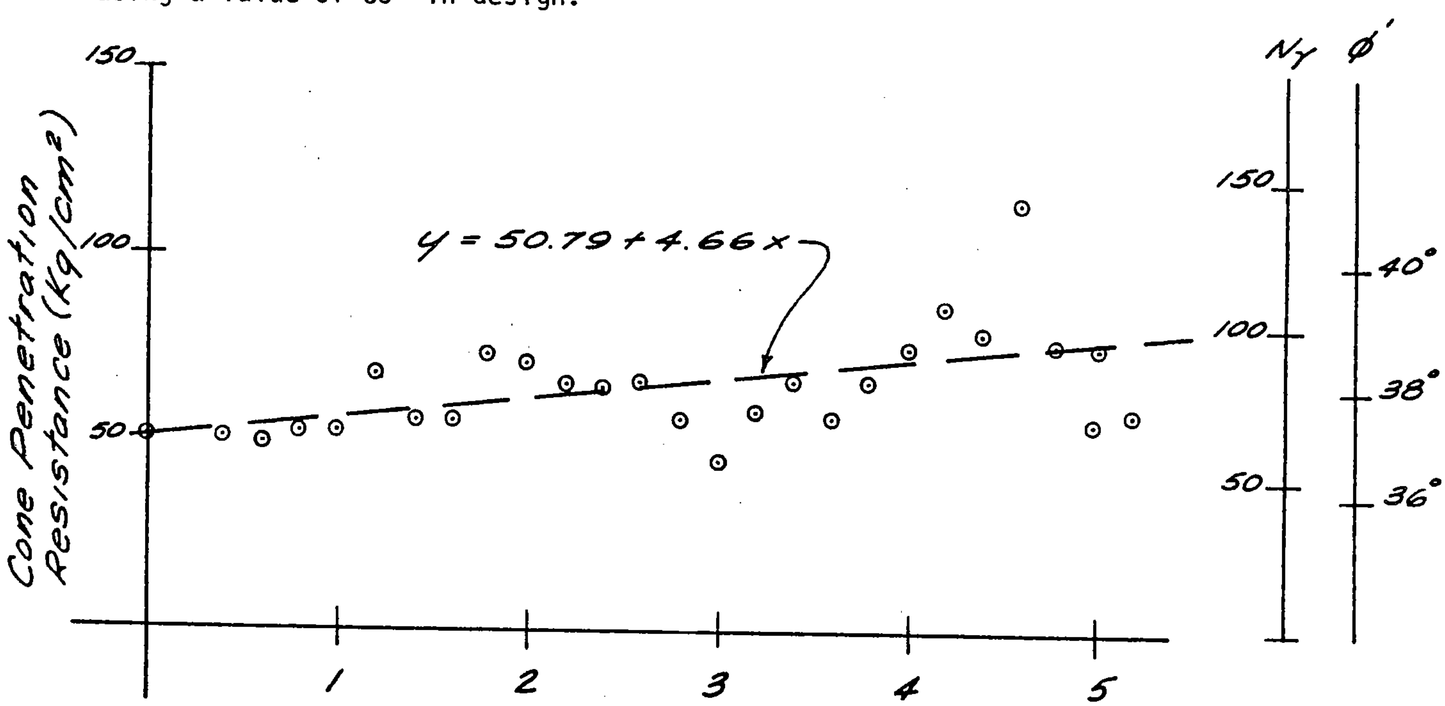
Figure 10- Dip in guardrail and patching defines
culvert crossing in sawdust fill

IN SITU TESTING

Various in situ tests were performed on the sawdust embankment constructed in Raymond. As you might expect, we found it most difficult to imitate field densities in the laboratory with repeatable results. Nelson and Allen[1/] determined that the angle of internal friction of the hog fuel that they tested was 31°, using a triaxial test. Lea and Brawner [2/] reported an angle of internal friction of 50° for sawdust, determined by direct shear tests. We were not as successful, as our laboratory tests indicated values of 6° to 25°. It was apparent that we would probably obtain better information if we were able to test the sawdust in place.

The large sawdust embankment constructed in Raymond provided us the best opportunity we have had to test sawdust in place. Most sawdust fills are constructed and covered quickly. This fill was available to us several months through the summer and most of the winter. A settlement plate, surface elevations, and the internal temperature of the sawdust were monitored for over a year after construction of the embankment. Individual tests were conducted on the sawdust consisting of in-place density, plate bearing, and Dutch Cone penetration resistance. Rebound deflection data was obtained at two different times, once after placing the untreated surfacing and again after placing the final lift of asphalt concrete pavement. To confirm the rather low subgrade modulus value we were finding, a facsimile seismograph was even run on the grade.

Our data indicate that we achieved maximum in-place wet densities of 59 to 60 pcf. This was greater than anticipated; we are now using 60 pcf in our design. The Dutch Cone Penetrometer resistance indicates a maximum angle of internal friction ranging from 30° to 39°, increasing with depth. We are now using a value of 35° in design.



Depth Below Subgrade (Meters)
Figure 11 - Dutch Cone Penetration Data

A Summary of the Use of Sawdust for Highway Fills
By N. C. Jackson

Data from the plate bearing tests, rebound deflection, and facsimile seismograph indicate that the surface of the sawdust is not nearly as strong as originally considered. Our analysis of the data from these tests indicates that the sawdust embankment has a subgrade modulus E of 1400 psi in the top 1 to 2 ft. This equates roughly to a CBR of 1. We now require a minimum of two feet of surfacing over sawdust--more if the traffic warrants.

The settlement data indicated that we did experience somewhat more settlement than predicted. As the unit weight was also higher, this was not surprising. Post-construction consolidation of the sawdust was in the range of 6 to 8 inches where the total depth was approximately 20 ft.

The embankment in Raymond was constructed primarily from sawdust stockpiled next to the site by an adjacent sawmill. This material had started to decay and was at a temperature of +85°F when placed in the embankment. The temperature of the embankment rose to over 100°F before ultimately declining to ambient. We have since restricted sawdust accepted on a project to either fresh sawdust which has not started to decay, as evidenced by ambient or lower temperatures, or to very old sawdust which has stopped decaying, as evidenced by temperatures equal to or lower than ambient.

DISCUSSION

My first recommendation to anyone considering the use of wood-waste products as an embankment material, as we have, is to approach it with a reasonable degree of caution. In spite of the number of installations just described, we have tried to limit the use of sawdust to specific applications where there is either a substantial economic advantage, or circumstances which limit normal construction techniques. Its use has also been limited to our coastal areas where there is both high rainfall (100 in. or greater) and moderate temperatures.

One must consider the risk of spontaneous heating and ignition of the sawdust. This does occur in loose sawdust piles, even in our coastal region. With the restricting of sawdust sources and the use of normal embankment construction techniques resulting in high densities, I think the risks are reasonable in our coastal region. In dryer, warm regions, or when used with minimal compaction control, the risks are probably quite great.

Associated with the construction of all sawdust fills is a potential water quality problem, due to an aqueous solution of wood extractives commonly known as leachate. It is evidenced by its dark color, oily sheen, foaming, and septic smell. It generally has a high metal ion content, high T.O.C. and B.O.D., low pH, and may be toxic in sufficient concentrations. It has been our experience that the oxygen demand and accompanying characteristics peak rather quickly and diminish rapidly with time. In general, the water draining from the sawdust is almost completely normal within six months to a year. It has also been our experience that the total quantity of flow is quite small. On projects where there has been a receiving stream in close proximity to the sawdust embankment, there was a very high dilution ratio between the leachate and the stream. Possibly the best treatment for leachate is by surface application to a highly forested area. We have not experienced any problem with leachate in these areas. Our major problems have occurred in urban areas. Again, the leachate is unsightly and has a foul odor.

For anyone considering the use of sawdust, I strongly urge you to read "A Study of Woodwaste Leachate"[3] by Schermer and Phipps, and "Consequences of Leaching from Pulp and Paper Mill Landfill Operations"[4] prepared by Econotech Services Limited of Canada.

Of final concern is the obvious lack of any definitive information concerning the life expectancy of a reasonably large sawdust embankment. Present popular predictions are for reasonable performance for at least 15 years. Inspection of much older sawdust piles in the area indicates that this may be quite conservative. I anticipate that we will experience progressive deterioration in the geometry of the roadway prism, increasing with time until some remedial reconstruction will be warranted. Thus, we will not be faced with total reconstruction of these projects in 15 years, but a need to perform some partial reconstruction at approximately that interval. This final question will only be answered with time.

REFERENCES

1. D. S. Nelson and W. L. Allen, Jr., "Sawdust as Lightweight Fill Material," Report No. FHWA-RD-74-502, May 1974 Status Report.

2. Norman D. Lea and C. O. Brawner, "Highway Design and Construction Over Peat Deposits in Lower British Columbia," N. D. Lea and Associates, Vancouver, B.C., and British Columbia Department of Highways, Victoria, B.C.

3. Eugene D. Schermer and James B. Phipps, "A Study of Woodwaste Leachate," Choker Research, Grays Harbor College, Aberdeen, Washington, 1976.

4. Econotech Services Limited, "Consequences of Leaching from Pulp and Paper Mill Landfill Operations," CPAR Project Report 363-2, September, 1977, Ottawa, Ontario.

ABSTRACT

Modeling Erosion and Sedimentation from Roadways

By

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Temporary access roadways or roads under construction are primary sources of sediment in urban and forest environments. Sediments derived from these roadways can adversely affect adjacent and downstream waterways. Alteration of aquatic and riparian ecosystems can result in loss of plant and animal life, and changes in channel morphology can create flood hazards. In addition, sediments and attached substances such as phosphorous or pesticides may cause water pollution.

In order to avoid these problems, methodologies are needed that realistically estimate erosion and sediment yield from roadways. A useful approach is through physical process simulation models. This type of model uses formulations of the controlling processes, such as overland water flow, that produce erosion and sedimentation. Two types of models have been developed at Colorado State University; a complex type that routes water and sediment in time and space using a finite difference technique and a simplified model that spatially averages runoff and sedimentation processes over a storm period.

This paper presents both complex and simplified physical process models that simulate water and sediment yields from roadways, Physical processes considered include rainfall, infiltration, overland flow, sediment transport in the flow, and sediment detachment from raindrop impact and surface runoff. Both models are designed to analyze complex roadway geometries such as cut slopes, road surfaces, fill slopes, ditches, and culverts. Controlling physical characteristics including slope gradient, soil type, length of surface flow, and gravel or vegetative surface treatments are also considered. Inclusion of all these factors aids the design engineer in selecting the roadway configuration that will minimize sediment yield. The complex or finite difference model can be used to study the response of the roadway to variable intensity and constant intensity rainstorms. However, the simplified model is presently formulated to only consider constant intensity design storms. The complex model can compute a time variant hydrograph at each point in the roadway configuration while the simplified model yields an average constant discharage for the entire uniform storm.

For short overland flow lengths and constant rainfall, conditions often encountered in road design, the approximation of a constant discharge is justified. Although the complex model is more versatile in processing rainfall inputs into outflow hydrographs, the simplified model is much easier to use for preliminary design purposes and constant rainfall conditions. Comparisons of the two models for selected design storms and three roadway configurations indicate the simplified model is valid for these applications. Additional attributes of both models along with applications for determining surface treatments, slope gradients, and overland flow lengths are also discussed in this paper.

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SYNOPSES

Roadways are often primary sources of sediment in natural and urban environments. Erosion and sedimentation from roadways during and after construction may increase the sediment load to streams thus affecting water quality, riparian and aquatic ecosystems, and the flood capacity of the channel. Techniques are needed to estimate sediment yield from roadways. A useful approach is through physical process simulation models that use formulations of the controlling natural phenomena, such as overland water flow, to describe erosion and sedimentation. This paper presents a complex finite difference model and a simplified time and space averaged model that simulate water and sediment yields from roadways. These models can be used to assess the relative sediment yields between different roadway configurations. Application of the models are presented and comparisons between model results are discussed.

INTRODUCTION

Unimproved or poorly maintained roads are recognized as primary sources of accelerated erosion and sedimentation. Significant quantities of sediment delivered to channels may adversely affect aquatic and riparian systems, reduce channel capacity, and increase flood hazards. A properly built and maintained roadway with adequate sediment control can effectively reduce sediment yields and decrease erosion impacts on downstream channels. The evaluation of alternative routes and designs of road cross sections, road gradients and surfaces, cut slopes, embankments, and spacing of cross drains requires a method to predict sediment yields.

Soil erosion from roadways is a complex process of detaching soil particles from the ground surface by raindrop splash and water flow. Factors affecting soil erosion and subsequent sediment yields are climate, topography, soil characteristics, and activities of man. Methods for estimating sediment yield must consider these factors.

Despite the complexity of the physical processes governing soil erosion, numerical modeling is a physically realistic way to estimate the time and space dependent erosion and sediment yield from roadways. Estimation of sediment volume should include the amount of sediment transported and its origin.

Numerous methods are available to estimate sediment yield from watershed areas, but few have been developed for roadways. Knowledge gained about watersheds can be applied to roads. In this paper water and sediment yield estimation models are presented. Based on previous studies of watershed erosion and sedimentation, these models use formulations of physical processes to simulate sediment yield from roadways.

FORMULATION OF MODELS

Erosion, water runoff, and sediment yield are hydraulic and hydrologic phenomena governed by complex physical processes. Finite difference mathematical models can be constructed to simulate these complex processes in time and space. Model simplification can reduce this complexity and if simplification leaves the basic physical processes intact, the model may not lose its accuracy.

Governing factors considered in the models include rainfall intensity, storm duration, surface water ponding time, infiltration rate, soil detachment rate, sediment size, ground cover conditions, cross drain spacing, ditch and culvert size, road radient, cut and fill slopes, sediment discharge, and water discharge.

The simplified method is composed of the same components found in the complex model.[1] The difference between the two methods is the complex model routes water and sediment in real time and space while the simplified method uses a time-space integration. In addition, the complex model can examine sediment availability from a storm at a discrete time step. However, the simplified method only approximates overall sediment availability during the storm and total sediment transport capacity for the whole runoff period. The simplified method neglects the protecting effect of the shallow ponded water layer and the armoring effect of the loose soil but it is easier to use and understand.

The complex road sediment model has been only partially validated using field data. Currently, the complex road sediment model and the simplified method can only be applied qualitatively. That is, the models can be used to assess the relative quantities of sediment from roadway configurations but not as a method for precise prediction of the amount of sediment produced. Acquisition of adequate measured field data of rainfall, runoff, and sediment yield will help make both complex and simplified models more accurate in predicting water and sediment yields.

Basic Components

Processes in the road sediment models are similar to those in the watershed surface erosion model.[2] The road sediment models also consider road surfaces and flow in ditches and culverts.

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[1]Simons, D. B., R. M. Li, and L. Y. Shiao, "Formulation of a Road Sediment Model," Colorado State University Report CER76-77DBS-RML-LYS50, prepared for USDA Forest Service, Rocky Mountain Forest and Range Experiment Station, Flagstaff, Arizona, March 1977.

[2]Simons, D. B., R. M. Li, and M. S. Stevens, "Development of Models for Predicting Water and Sediment Routing and Yield from Storms on small Watersheds," Colorado State University Report CER74-75DBS-RML-MAS24, prepared for USDA Forest service, Rocky Mountain Forest and Range Experiment Station, Flagstaff, Arizona, August 1975.

Basic components of the models are mathematical formulations of the physical processes that control erosion and sedimentation. For the planar roadway surfaces of cut slope, fill slope, and road bed, the basic formulations follow those presented elsewhere.[1,3] These processes include rainfall, infiltration, runoff, sediment detachment by raindrop splash and surface runoff, and sediment transport capacity.

The complex model uses variable intensity rainfall hyetographs as input. The simplified model is currently limited to a single constant intensity hyetograph. Although the complex model is more versatile in calculating natural rainfall input, the similified model is adequate for treating design storms or experimental rainulator rainfalls.

Infiltration is estimated by formulations of the Green-Ampt Equation. In the complex model, the unsteady rainfall requires a time variant infiltration scheme. Because the simplified model only considers steady rainfall, an analytical solution is utilized.

Water discharge characteristics on planar surfaces (road and cut and fill slopes) and in ditches are determined by using Darcy-Weisbach friction factors and the kinematic wave assumption. If required, Manning's equation can be used instead of the Darcy-Weisbach equation.

Water and sediment discharges through culverts are computed by Manning's equation and a sediment transport equation. The sediment transport capacity of the culvert is found from a generalized conveyance system equation.[4] The complex model routes rainfall excess through a road cross section using a finite difference scheme. In contrast, the simplified model treats the total runoff excess as a constant discharge hydrograph during the rainfall period. Sediment transport in the complex model is computed using a finite difference formulation of the sediment continuity equation in conjunction with the Meyer-Peter, Müller and Einstein's equations. Transport in the simplified model is computed using the constant discharge rate and is integrated over time to give the sediment yield. The complex model provides more information on water and sediment discharge because time and space variant dynamics are considered while the simplified model integrates the time and space variations of the system. However, for roadways with short overland flow lengths, this integration still provides realistic simulation results for water and sediment yields. In both models, error messages are printed if water and sediment inflows exceed ditch or culvert carrying capacities.

Sediment Supply and Capacity

Sediment yield from roadways is controlled by sediment supply and transport capacity. If supply is less than the transport capacity, supply controls sediment yield. If supply is greater than the transport capacity, the transport capacity controls the sediment yield. Transport capacity can exceed available supply for small sediment sizes such as silts and clays, but is less than supply for the larger sizes. Sediment supply is assumed to occur from raindrop splash detachment and overland flow detachment in both models.

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[4] Graf, W. H. and E. R. Acaroglu, "Sediment Transport in Conveyance Systems (Part 1)," Bulletin of the International Association of Scientific Hydrology, June 1968.

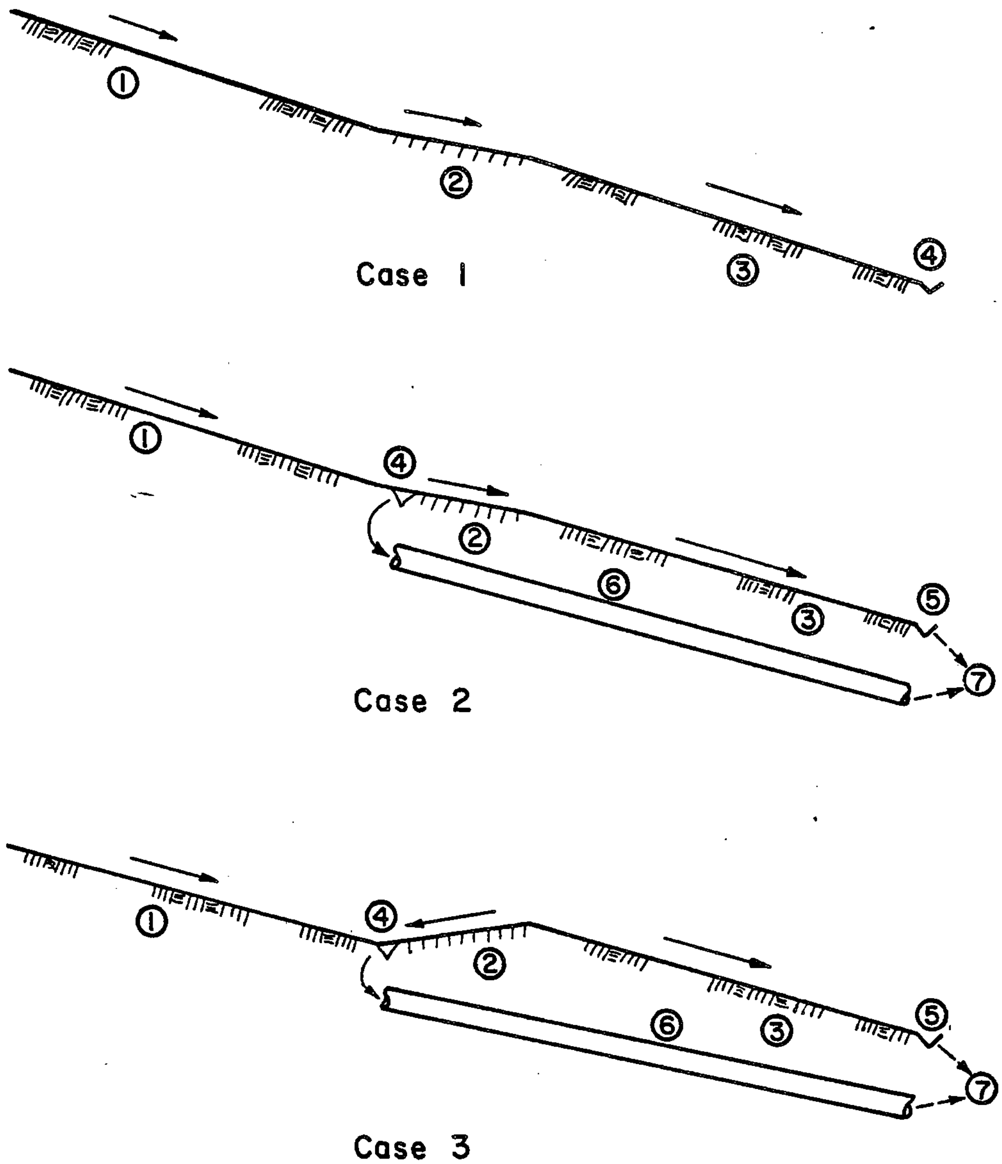

Figure 1. Roadway cross sections.

Linkage of Roadway Components

Roadway components of cut and fill slopes, road surfaces, ditches, and culverts can be linked to represent surfaces that contribute to water and sediment yields (Figure 1). Water yields are additive from all components but sediment yields are dependent on the supply and transport capacity.

Comparison of Complex and Simplified Models

There is no good set of road sediment yield data presently available for an entire road prism. Simplified and finite difference solutions for data from overland flow plots was compared with excellent results.[5] Reese[6] compared a version of the simplified model with the complex model to the same realistic hypothetical data base and the three roadway cases shown in Figure 2. Sediment yields differ since the simplified model does not accurately represent the complex hydrograph that may be developed from a series of roadway components. Simplification of peak and duration of the hydrograph alters sediment transport thus causing differences in yields between the two methodologies.

Loss of precision through simplification is insignificant compared to the relative accuracy of the results and complexity of using the more sophisticated finite-difference scheme. The finite-difference scheme is the best approach if variable intensity storm events are to be analyzed or information about water and sediment discharge rates are required at discrete space and time points.

APPLICATION OF MODELS

The models can be used to assess relative sediment yields from different roadway configurations when subjected to a variety of conditions. For example, given the same rainfall input and 100 percent gravel cover on the roadway and 30 percent cover on the cut and fill slopes, the yields from the three cases shown in Figure 2 are quite different (Table 1).

Such information is useful in comparing sediment impact on downstream channels and the cost of developing a road prism to control erosion and sediment yield.

Either model can be used to estimate sediment yields occurring from slight changes in roadway geometry. For example, the simplified model can be used to study the variation of sediment yield with changes in inclination on the road surface and cut and fill slopes. Table 2 lists four design alternatives with changes in slope for Case 1.

A 30 percent cover was chosen for the cut and fill slopes and no cover for the road surface. The simplified model was applied for a 30 minute duration rainfall with the results as shown in Figure 3.

This example shows that additional reduction in slopes can significantly alter sediment yields.

These limited examples illustrate the flexibility the road sediment models can provide the roadway designer when analyzing various alternative plans. Although not shown in these examples, sediment yields are divided into the various size classes to allow a better estimation for the types that may be entering the waterways.

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[5]Simons, D. B., R. M. Li, and T. J. Ward, "Simple Procedural Method for Estimating On-Site Soil Erosion," Colorado State University Report CER76-77-DBS-RML-TJW38, prepared for USDA Forest Service, Rocky Mountain Forest and Range Experiment Station, Flagstaff, Arizona, February 1977.

[6]Reese, A. J., "Simplified Small Watershed Sediment and Water Yield Modeling," M.S. Thesis, Department of Civil Engineering, Colorado State University, Fort Collins, Fall 1977.

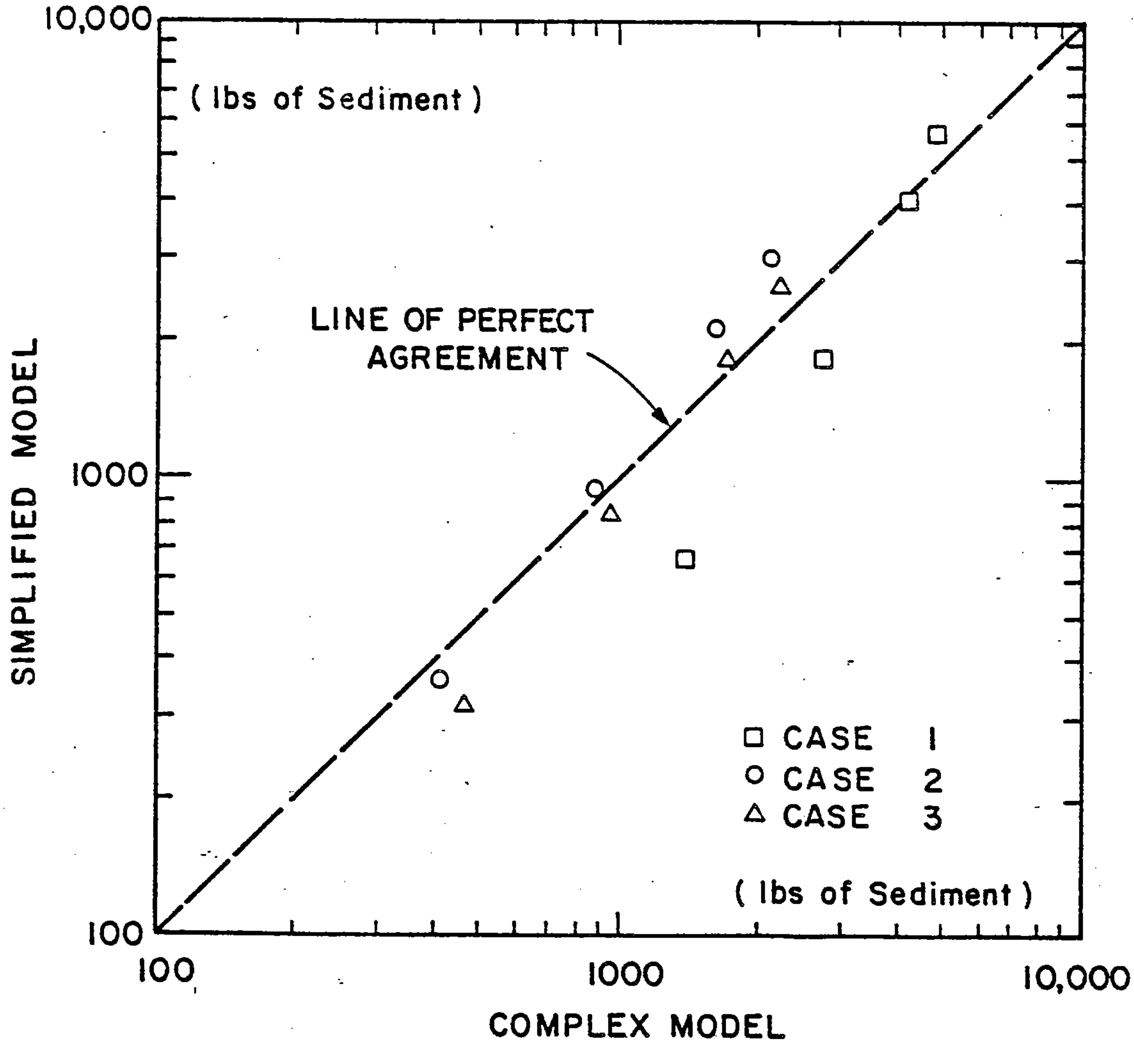

Figure 2. Comparison of sediment yields for simplified and complex models.

Table 1. Sediment yields from three alternative roadway prisms.

Case	Sediment Yield, lbs	Fraction of Case 1
1	1520	1.0
2	982	.65
3	990	.65

Table 2. Alternate roadway prism surface inclinations.

	Slope Inclinations			
Surface	Alternative I	Alternative II	Alternative III	Alternative IV
Cutslope	0.10	0.20	0.33	0.50
Road surface	0.025	0.05	0.10	0.075
Fill slope	0.10	0.20	0.33	0.50

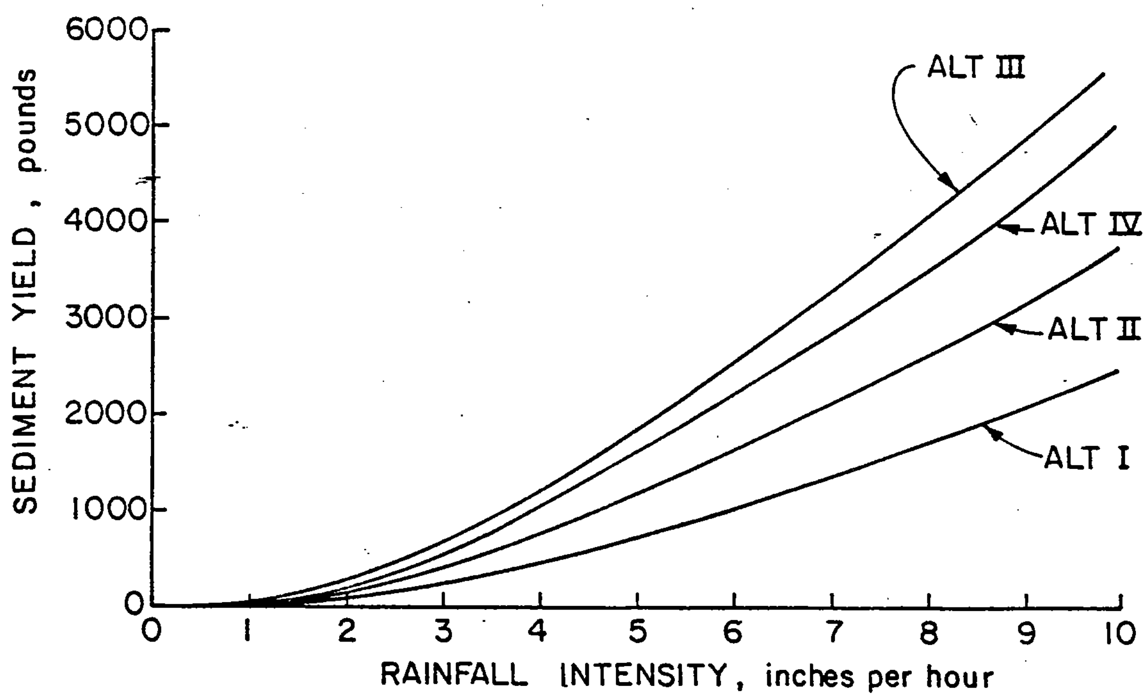

Figure 3. Sediment yields for different rainfall intensities with alternative roadway geometries of changing inclinations.

CONCLUSIONS

Estimation of sediment yields from alternative roadway design can be aided by mathematical models that describe the governing physical processes. Two levels of models are available depending on the complexity of the situation and accuracy of the required results. For many design cases, the simplified model provides as accurate results as the complex finite difference model. These models can provide the roadway designer with relative comparisons of various roadway designs and erosion control practices.

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DECISION AND RISK ANALYSIS
AS A PRACTICAL TOOL FOR
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

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Introduction

This paper demonstrates the use of decision and risk analysis in geotechnical engineering. It is not the intent to present a mathematical treatise on decision and risk analysis as excellent references are available (Keeney and Raiffa, 1977; Keeney and Raiffa, 1975; Schlaifer, 1969; Benjamin and Cornell, 1970; Vanmarcke, 1979). Rather the intent of this paper is to present a non-mathematical explanation of the general decision and risk analysis concepts emphasizing that it can be a useful tool to the practitioner. Through several examples the utility of these concepts on highway projects is demonstrated.

Put in simple terms, decision and risk analysis quantifies what the geotechnical engineer and geologist must do anyway. . . take incomplete and imperfect information and reach a decision. By formalizing the decision process, we are being more honest, we are forced to examine the information and our preserences in detail, and we develop a bettwe framework for communication. Through a clear understanding of the concepts and proper implementation of the procedures, our work is made easier, not harder, our recommendations are more honest and less subjective and the ultimate decision maker (owner, government agency and/or public) has a better understanding of the uncertainties and implications of a decision.

General Concepts

Fundamentally, geotechnical engineers and engineering geologists are asked to forcast the future. We are a little like the weatherman; except everyone expects us to be right all the time while no one expects the weatherman to be so clever. Now assuming we had a crystal ball, we would probably do the following:

1. Identify the various design alternatives and possible construction procedures.
2. Look into our crystal ball and see the consequence of each alternative.
3. Relate the alternatives and consequences to the decision maker (owner, government agency, public).
4. Choose the best design and construction procedure. Either the decision maker would make this decision based on our forcast and his preferences; or alternatively we would incorporate his preferences into our perception of the situation and we would make the choice.

There would be no risks in our decisions since we would have no uncertainty. Unfortunately we do not have crystal balls and we can not see into the future. Thus we must rely on the imperfect tools of our trade (subsurface investigation methods, expert observations, analytical evaluations and past experience) to make uncertain predictions leading to decisions which contain risks. The decision and risk analysis technique allows us to perform the five basic decision steps defined above; in lieu of step 2, (crystal ball) we make various predictions and express these predictions with levels of confidence or probabilities of occuring.

Several other general concepts are necessary to understand the general uncertainty and risk analysis method. These include:

Risk

Fundamentally, any decision involves the risk of being wrong. Normally we only think of risks which result in failures or problems developing. However, decisions which result in an overly conservative and therefore costly design should also be considered a risk.

Examples of these risks are:

Unconservative Design: Engineer A underestimates the potential settlement of a major building and designs the structure on shallow spread footings. After construction, $25,000 is spent repairing cracks, replaning doors and underpinning portions of the structure. The owner estimates an additional $100,000 liability due to the loss of value from the potential for additional settlement problems and the harm to his reputation. Thus the engineer's poor estimate apparently cost the owner $125,000.

Overly-conservative Design: Engineer B overestimates the potential settlement of another building and designs the structure on a pile foundation with a structural floor slab. The piles and structural slab adds $400,000 to the cost of the building. In reality, the building could have been constructed on normal spread footings with a slab-on-grade. The engineer's inaccurate estimate cost the owner $400,000.

Let us assume that both engineers did a reasonable technical job, the differences being one of philosophy and fear of being sued/desire to provide cost effective recommendations. Since Engineer A's error resulted in an obvious $125,000 additional cost, Engineer A is sued and his reputation badly damaged. However Engineer B whose bad prediction cost the owner $400,000, is not sued and in fact his reputation improves with another "successful" design. In reality, Engineer A's design may have been the most cost effective since the actual cost of his recommendation is the difference between the repair cost and a more conservative initial design (such as piles, large footings, surcharge, etc.).

It is important to realize that these risks can be minimized but not eliminated. However, it is not a simple matter of minimizing the risks since risk reduction is costly. Thus the fundamental design problem is "what is the optimum trade-off of cost, benefit, and risk?" Since nothing is known for sure, the optimum design is the design which has the highest probability of providing the best result.

Risk and Uncertainty

Risk and uncertainty are in fact one and the same. If we had no uncertainty (knew everything and could perfectly predict the future), there would be no risk since we would know exactly what would happen. However since there is always uncertainty, there is always risk. Thus to quantify our uncertainties is to understand the risks.

We use probability to express our uncertainty or level of confidence. As hypothetical example, let us assume that you have to reach a decision based on the current population of Buffalo, New York and you are not allowed to resort to any outside information. Figure 1 is an uncertainty curve the author developed expressing his knowledge and confidence level about the population of Buffalo. This curve was readily developed by asking what is my best estimate? What is the smallest population that I am sure Buffalo exceeds? What is the largest? What is the probability the population falls between 300,000 and 400,000?, etc. The uncertainty curve is by its very nature subjective, it is a level of confidence.

Engineers and engineering geologists, being applied scientists, find the concept of subjective uncertainty curves offensive. In fact, the subjective uncertainty curve is probably the main stumbling block any of us have in accepting the decision and risk analysis methodology as being valid. Many of us are unwilling to admit that our predictions and decisions always include subjective preferences; we would rather say that our evaluation was based in part on our past experience and engineering judgment. However call it what you want, geotechnical engineering is both an art and a science. We must rely heavily on subjective opinions whether it is our own or the combined opinions of others developed over the years and incorporated into current practices.

The uncertainty curve need not be based on a simplistic "guesstimate" as depicted in the population example explained above. It can be based on a wealth of data, past experience and numerous complex geotechnical analytical and numerical evaluations. However the ultimate curve will always contain subjective elements.

Uncertainty and Factor of Safety

Conventional engineering evaluations utilize the concept of "best estimates" of parameters. These are based on the available data and subjective experience of the engineer. However, there is always an uncertainty in this "best estimate" since there is a certain probability that the actual value may be more or less than the "best estimate". To compensate for these uncertainties, engineering analyses normally utilize the concept of "factor of safety". However, such an approach is incapable of quantifying the uncertainties, comparing the relative significance of uncertainties in different parameters, or relating the changes in uncertainties assóciated with improving the data base. Also using "best estimates" is only telling part of the truth and does not express the unceraínty in our knowlege. In fact, one can make a good case for the point of view that the only way to speak the truth is to speak in terms of uncertainty since the only thing we can "truly" talk about is our state of confidence.

There is an inherent relationship between the probability of problems developing (failure), factor of safety (level of conservatism), and level of uncertainty. Figure 2 presents these relationships. Figure 2a shows that a factor of safety implies a specific risk associated with the level of information available. Figure 2b indicates that an acceptable level of risk can be obtained through various combinations of factors of safety and decreased uncertainties. By expressing these factors, their inter-relationships and costs, we can determine the most economical design.

Uncertainty and Frequency Curves

Confusion arrives over the differences between uncertainty curves and frequency curves. A frequency curve is a method for representing data. Thus if 1000 highway embankment settlement readings were taken, a frequency curve, could be used to present the data. On the other hand, if we had to predict the settlement of a new embankment at a particular location, we would express our prediction as an uncertainty curve. Obviously the frequency curve of actual observed data would be very useful in developing our uncertainty curve. In fact, if both embankment conditions were identical, the unceraínty curve may essentially become the frequency curve.

Decision Theory

The essence of decision theory is contained in Figure 3. At each point of decision there are various options and each option has various possible outcomes. Each outcome has a set of consequences or impacts which can be represented as a linear quantity called impact vectors. To evaluate the decision, it is necessary to combine these impact vectors into a single value representing the preference of the option. The process which combines the impact vectors is known in decision theory as the utility function.

Comprehensive Decision and Risk Analysis

An entire highway project can be designed and constructed using a decision and risk methodology. The basic methodology would consist of a decision theory model in which all the data and results are expressed with uncertainty distributions. The model would be used to evaluate the overall system, determine the dominate factors, and develop the optimum design. The two major problems would be developing the uncertainty distributions and combining unlike impacts (such as cost and asthetics). However, these problems are not weaknesses in the method but inherent difficulties in the design process. To complete a design, these problems must be solved; they can be solved implicitly (as is normally done) or explicitly as required by the methodology.

Figure 4 shows a model framework which might be constructed to evaluate a major highway project. The framework includes:

1. Project Characteristics: This task involves the computation of a data base of significant site characteristic factors including the design factors. The results would include summary maps and other appropriate site characteristic descriptions. This effort represents the majority of the technical input in the methodology and is, in essence, identical to the work required in the more conventional design studies.

2. Impact Vectors: These might include construction costs, maintenance costs, costs associated with problems developing, noise impact, air quality impact, safety, etc. In the model, each impact vector would be represented as a probability distribution of consequence versus probability of occurring. Within each major category, the consequences would be described with the same measurement such as cost, auto accident, etc. The probability distributions would incorporate both the parametric and analytical uncertainties. Each set of detailed impact vectors would be summed to qualitatively represent the primary impact of the site on each major impact category. The results, which would be the summation of a set of probability distributions, would also be a probability distribution.

3. Rating (Utility Function): Each primary impact probability distribution is then summed to yield an overall rating of the design. This requires a relative weighting of impact categories in order to combine costs (dollars), safety (auto accidents), etc. Establishing this weighting or utility function requires subjective judgment and can be the source of considerable discomfort and disagreement. However, it cannot be avoided. In fact, we seek with our formalism not to avoid these confrontations, but to make them explicit, quantitative, and as productive as possible. In some cases, it may be more publicly acceptable that the common denominator the utility function uses to compare impacts need not be dollars or lives but could be "utiles" (decision theory utility function units). The resulting utility function or rating would also be expressed as a probability distribution.

The general procedure would be to set up the model, identify factors and preferences, perform sensitivity analysis to identify key factors and uncertainties, and initiate the decision processes. The procedure would be an iterative process with each successive iteration eliminating alternatives, obtaining more data and reducing uncertainties, and focusing in on the final decisions.

The method has several advantages including:

1. Forces all factors and preferences to be identified and put into perspective.
2. Results in a full disclosure of information since we have to define the project's preferences and our uncertainties.
3. The framework is flexible allowing changes in our information base and changes in preferences to be readily incorporated.
4. Since it puts all factors and consequences into persepctive, it minimizes the possibility of unimportant factors being blown out of proportion; and conversly, the chance of an important factor being ignored.
5. It provides structure and continuity to the study. In fact the basic framework does not change as the work progresses from preliminary through final design; only the level of uncertainty changes.
6. By identifying the key factors and dominant uncertainties, it provides us with guidance in determing where to concentrate our study efforts.
7. It provides a framework for communication; implementation of these procedures invariably results in much better communication and understanding of the problems, factors and solutions. When presented to the public, it can facilitate public acceptance.

In essence, the general procedures outlined above are incorporated into any large design project. The merit of the methodology is that it requires these procedures to be done systematically, comprehensively, and explicitly. It will result in a more timely and cost effective design study. It enables many complex issues to be systematically examined such as:

1. Which design/construction risks are worth taking?
2. At what level do costs associated with aesthetics become excessive? When should compromises be made between the ideal aesthetic design and a more cost effective design.
3. How can criteria associated with costs, safety, noise, air quality and aesthetics be best combined to develop an optimum design?
4. How can risks be best incorporated into design decisions? These might include delays in obtaining government/public approval, construction risks, maintenance risks, safety risks, etc.
5. How can design decisions best be presented to the public? What is a workable format for discussing issues?

Applying Methodology to Geotechnical Decisions

The previous section briefly discusses how the decision and risk methodology might be used to design and construct a major highway project. Even if these concepts are not incorporated into the overall project study, the concepts can be very effectively used to express geotechnical evaluation and to reach cost effective geotechnical decisions.

The basic procedure would include:

1. Initial examination of geotechnical alternative and the factors that affect the associated uncertainties, risks and costs.
2. Formulation of a site investigation program which optimizes the resulting decrease in uncertainty at a level of expenditure appropriate for the associated project risks and costs.
3. Geotechnical analysis and evaluation presented in terms of uncertainty and risk (probability of problems developing) and consequence (both known costs and probability of problems developing times the associated costs).

The results of the work would be used to reach decisions about geotechnical design and construction.

In practice, rigorous implementation of these procedures is probably not normally warranted. However, several different levels of implementation may be effective depending on the type of project and importance of the geotechnical decisions.

On many projects and on most phases of virtually all projects, many geotechnical design and construction decisions are relatively straight forward. In these cases there are no significant design and construction alternatives to be examined. Thus an appropriate application of the decision/risk methodology would be to informally perform step 1 (examination of factors and alternatives) and incorporate the general philosophy of steps 2 (optimizing investigation program) and 3 (analysis and evaluation) into our general program. The geotechnical engineers and engineering geologists would make the geotechnical related design decisions.

Frequently one or two key geotechnical decisions on a project have a significant impact on the cost and risks of a project. In those cases, it is the author's opinion that the geotechnical consultant should not make the decision. Rather he should provide the decision maker (owner, government agency, public) with the necessary information and expert opinions to reach the best decision possible based on the available data. For the consultant to play the role of a decision maker is inappropriate for several reasons including:

1. The decision should be based on a firm understanding of acceptable risks; overall project costs; overall impact of problems developing including maintenance costs, adverse public reaction, reputation; and various other factors. These are factors which only the owner can assess.
2. The consultant should not be required to make risk decisions involving major project economics for which he receives no benefit. To do so is unfair to both the consultant and owner; often leading to overly conservative (and expensive) solutions. It is in a sense a conflict of interest for the consultant to make the decisions.

3. It is inappropriate for the consultant to make risk-cost preference decisions for the owner. Such decisions are subjective and the consultant would be presumptuous to assume he knows what the owner wants.

The following sections present examples of possible applications of the risk-decision analysis methodology on geotechnical projects.

Developing Geotechnical Exploration Programs

In developing a geotechnical exploration program many factors must be considered such as complexity of subsurface conditions, impact of geotechnical decisions on project costs and risks, cost of exploration program, and other factors. Normally these factors are weighed implicitly in a rather arbitrary manner. As shown on Figure 2, there is an inherent relationship between risk or probability of problems developing (failure), factor of safety (i.e. our level of conservatism), and level of uncertainty. Ideally there exists an optimum combination of these factors. By expressing these factors, their inter-relationship, and costs, we can determine the optimum combination which would result in the most economical design. Unfortunately, the extent of the geotechnical program is all too often based on general practices on similar projects and budget constraints rather than the actual cost effectiveness of the program.

A systematic procedure for determining the optimum exploration program would consist of the following:

1. Identify Required Geotechnical Decisions: This might include evaluation of slope stability, design of lateral earth support systems, geotechnical criteria for foundations of ramps, undercrossing structures, tunnels, and bridges, evaluation of groundwater control, and identification of construction problems and requirements.
2. Determine Significance of Geotechnical Decisions and Identify Key Decisions: Based on virtually any level of information, a preliminary geotechnical evaluation can be performed expressing the recommendations as an uncertainty range. Thus based on only published geologic information, past experience and site reconnaissance it would be possible to determine probable ranges in geotechnical criteria. The design exploration program would then be intended to confirm the foundation conditions and reduce the uncertainty ranges allowing for a more economical design. Based on the initial evaluations, the potential cost savings between design recommendation based on the best probable conditions and the worst probable conditions can be computed. These cost estimates place the geotechnical problems in perspective, indicate which decisions are most important, and provides guidance to the appropriate level of the exploration program.

3. Identify the Required Geotechnical Information: This might include lithology, location and fluctuation of groundwater, areas of marginal slope stability, locations and properties of water bearing zones, strength and deformation characteristics of each lithologic unit, and initial state of ground stress.
4. Identification of Appropriate Subsurface Investigation Methods: These might include test sections.
5. Determination of Optimum Program: This would consist of the development of several sets of curves as shown on Figure 5. Curve 5a represents a relative comparison of the cost of problems developing (projected cost times the probability) and the cost of increased conservatism (such as flatter slopes, lower bearing values, etc.) for a given level of information (uncertainty). This would be done for the intial level of uncertainty which would require very conservative design concepts to reduce the level of risk. Additional curves developed would relate to a modest, comprehensive, and very comprehensive investigation program. The cost of the optimum design would include the cost of the exploration program. To develop these curves it is necessary to predict the reduced uncertainty produced by a certain level of investigation. With increasing information, the required level of design conservatism decreases but undoubtedly at a decreasing rate. Also regardless of the amount of information there will always be a residual uncertainty necessitating some degree of design conservatism. Each cost-uncertainty evaluation produces an optimum cost. These costs can be plotted as shown on Figure 5b to determine the most cost effective program.

The rigorous implementation of the above procedure can become costly and time consuming and may not be warranted on most projects. However, partial implementation can be very effective in putting the exploration program in perspective, forcing the key issues to be identified and examined early in the program, and minimizing the possibility of expending too much or too little effort obtaining information.

As a specific example of the proposed procedure, consider the cost effectiveness of a cut test section for a portion of a proposed interstate highway in Washington. The general project conditions and proposed procedure include:

1. Project Conditions: The proposed alignment consists of deep cut sections through hard, over-consolidated lacustrine clay. Cutting into similar clays along Interstate 5 through Seattle caused considerable problems consisting of rapid progressive failures along horizontal failure planes resulting in the extensive use of cylinder pile walls (Palladine and Peck, 1972). Preliminary data indicates that cuts into the clay along the proposed project may not require cylinder pile walls.

2. Probable Design Criteria: The behavior of deep cuts into hard

clays is poorly understood. Thus regardless of the number of field explorations and laboratory tests, there may still be a relatively high uncertainty over the behavior of the clay. This may result in requiring extensive use of cylinder pile walls (or equivalent such as slurry walls and/or tie back walls). The premium cost of these walls is estimated to be on the order of seven million dollars.

3. Test Cut Section: Several test sections consisting of preconstruction instrumented cuts into the hard lacustrine clays would significantly reduce the uncertainties about the behavior of the clays. Based on an estimated cost for the test cut sections, existing uncertainty levels, and projected uncertainty levels, the anticipated cost effectiveness of the test sections can be computed. Thus:

   a. Without Test Section - based on current level of uncertainty, cylinder pile walls or equivalent would be required. Premium cost of seven million dollars.
   b. With Test Section - The proposed program might cost about $300,000. Using our current uncertainty ranges, probable results of the test section, and projected decrease in uncertainty we can quantify the probable outcome of the test section program. These outcomes (each with an assigned probability) include no change in the design concept (i.e. cuts fail and/or provide inconclusive data), some reduction in extent of cylinder pile walls or cylinder pile walls (CPW) can be eliminated (i.e., cuts remain stable).

4. Results of Analysis: The hypothetical results of such an analysis might be summarized as follows:

Outcome	Cost Savings (thousands)	Probability of occurring	Expected Cost (probability times cost - thousands)
No design changes	$ 0	20%	0
Minor design changes	$ 600	40%	240
Major design changes	$ 2000	30%	600
Elimination of CPW	$ 7000	10%	700
		Total Expected Savings	$ 1,540

The cost of the program is estimated to be $300,000 plus a cost factor associated with possible scheduling problems, possible public relations problems, and other problems which might be assessed a value of $200,000. Thus the total expected cost effectiveness of the test program is about one million dollars with a range of a $500,000 loss (with a 20% probability) to over a six million dollar savings (10% probability).

Optimizing Slope Design

The design, construction, and maintenance of highway slopes lend itself readily to a risk-decision analysis approach. The fundamental question is what is the optimum trade-off between initial cost (flatter slopes, more drainage, more slope protection) and subsequent maintenance costs.

The general approach would involve:

1. Identify the factors affecting slope performance and evaluate the general significance of each factor; these might include soil conditions, natural slope conditions, occurrence of natural slide features, weather, groundwater conditions, and surface water conditions.
2. Identify the consequences of slope design and performance; this might include capital costs, maintenance costs, possible traffic delays associated with slides, possible environmental effects, and possible safety considerations.
3. Develop a system for evaluating past and future slope performance; this would take into account the key factors and consequences identified in steps (1) and (2) above. In essence, this task would develop a design and performance frequency curve and provide for continual refining of the curve.
4. Based on the frequency curve information, engineering analysis, expert opinion, and cost data, develop a capital cost vs. expected maintenance curve as shown on Figure 6. The maintenance costs would be the actual maintenance cost should a conditions develop times the probability of it occurring. This probability would be strongly influenced by the frequency curves of past slope performance. Several sets of curves would probably be required for different general site conditions and/or different design factors. The results would be used to make cost effective design decisions for new highways and maintenance of existing slopes.

Evaluation of Key Decisions

Occasionally a key geotechnical decision has major impact on the cost and/or feasibility of a project. These decisions might include foundation types, types and/or need for retaining walls, design slope angles, pavement subgrade requirements, construction procedures, and/or contracting methods. Normally faced with key decisions, we tend to make conservative decisions without adequately assessing the cost effectiveness of such "safe" decisions. It is important to realize that few decisions result in absolutely zero problems or, conversely, result in catastrophic failures. Rather each decision results in anticipated construction costs, unanticipated construction costs, and maintenance costs. Use of risk-decision analysis procedures provides a rational basis for key decisions.

The general procedure would include:

1. Identify the appropriate decisions.

2. Identify the possible consequences of each decision and assign probabilities to each outcome.
3. Sum the expected cost of each decision(consequence times probability) to determine the total expected cost of each decision.

As an example, let us consider the choice of cylinder pile walls (CPW) versus conventional retaining walls for the proposed highway project discussed earlier. The procedure and results might include:

1. Decisions: For this example, we will identify four potential decisions:
   a. Extensive use of CPW: use very conservative design approaches requiring CPW at virtually any cut section.
   b. Moderate use of CPW: use a relatively conservative approach requiring CPW whenever the cuts into the clay exceed 15 feet.
   c. Minimum use of CPW; use CPW only where alignment requires moderate cuts into the clay in the side of existing steep hills or very deep cuts in non-hillside cuts (exceeding 30 feet).
   d. Elimination of CPW; use only conventional walls.
2. Consequences: Each decision will have a set of possible outcomes with an associated probability of occurring. The consequences might include:
   a. No construction problems develop with a minimumm of long term maintenance required.
   b. Minor construction problems develop with some minor delays and/or contract extras.
   c. Major construction problems requiring redesign with significant delays and/or contract extras.

   In addition, each alternative will have an estimated initial contract cost (this can also be carried as an uncertainty).

Table I presents the hypothetical results of such a risk/decision procedure. The ultimate decision should be made by the owner based on the results shown on Table I and his preferences. In general, the optimum scheme will probably lie between the two extremes of ultra-conservatism and high risk. It should be realized that any scheme which results in virtually no problems was probably too conservative and not cost effective.

Summary

The purpose of this paper was to demonstrate the use of decision theory and risk analysis in geotechnical engineering. The methodology is certainly not a substitute for sound engineering practices or engineering insight. Nor should the systems analyses technocrats ever be allowed to direct a geotechnical study. However, the decision/risk analysis concepts can provide a useful tool in placing problems in perspective, identifying important factors and alternatives, providing a framework for performing geotechnical evaluations systematically, comprehensively and explicitly, and expressing our conclusions. In essence, it presents our opinions and evaluations in the most honest and useful form enabling the decision maker (owner and/or public) to make the best decision possible with the available information.

Successful applications of the decision/risk concepts are certainly not without difficulties. As with any new technique the initial applications may be met with owner/client misunderstanding, may be inefficient, and even counterproductive. Through experience and owner/client involvement, the initial difficulties should be resolved.

Of major concern, however, is our ability to quantify our uncertainties. We are not very effective at expressing our uncertainties for numerous reasons including:

1. Much of our uncertainty comes from events we do not forsee; thus we end up with ranges that tend to be too narrow.
2. We are paid to know and find it difficult to admit we do not know.
3. We tend to be prouder of our predictions than we should be.
4. Most of us have virtually no idea how to express our degree of uncertainty. As an example of this, E.C. Capen,(1976) gave an uncertainty quiz to almost 1000 AIME and SPE members between 1974-1975. Using ten non-geotechnical questions, such as what is the area of Canada, the participants were asked to bound their estimates with confidence ranges. Some groups were asked for their 98% confidence level, some for their 90% level, etc. Ideally, on the average, asking for the 90% level should result in one wrong out of ten.) The results, as shown on Figure 7, shows how poorly we are at expressing our uncertainties. Regardless of the confidence level asked for (even 98%), the participants consistently got seven wrong.
5. Often the least experienced engineers will have the highest confidence in his predictions; this is of course opposite to what it should be.

Without the ability to express our uncertainty, the basis of the overall risk/decision methodology becomes questionable. However, rejecting such a methodology does not solve the problem of expressing our uncertainties, it only hides the problem Any decision based on a poor understanding of our uncertainties is likely to be a bad decision. Thus, if for no other reason, the risk/decision approach has merit because it forces us to think in terms of uncertainties and level of confidence.

TABLE I

EVALUATION OF ALTERNATIVES FOR CYLINDER PILE WALL (CPW) CRITERIA

Decision (Scheme)	Premium* Construction Cost	Minor** Problems	Major** Problems	Total Expected Premium Cost*
a. Extensive CPW				
Premium cost*	$ 7,000,000	$ 50,000	—	$ 7,005,000
Probability	—	10%	0	
b. Moderate use of CPW				
Premium cost	4,000,000	500,000	6,000,000	4,700,000
Probability	—	20%	10%	
c. Minimum Use of CPW				
Premium cost	2,000,000	500,000	12,000,000	3,950,000
Probability	—	30%	15%	
d. No CPW				
Premium cost	0	750,000	20,000,000	5,300,000
Probability	—	40%	25%	

*Premium cost reflects additional cost of CPW vs. conventional walls.

**Cost reflects problems along entire alignment; probability represents both probability of occurring along entire alignment and/or probable percent of alignment affected.

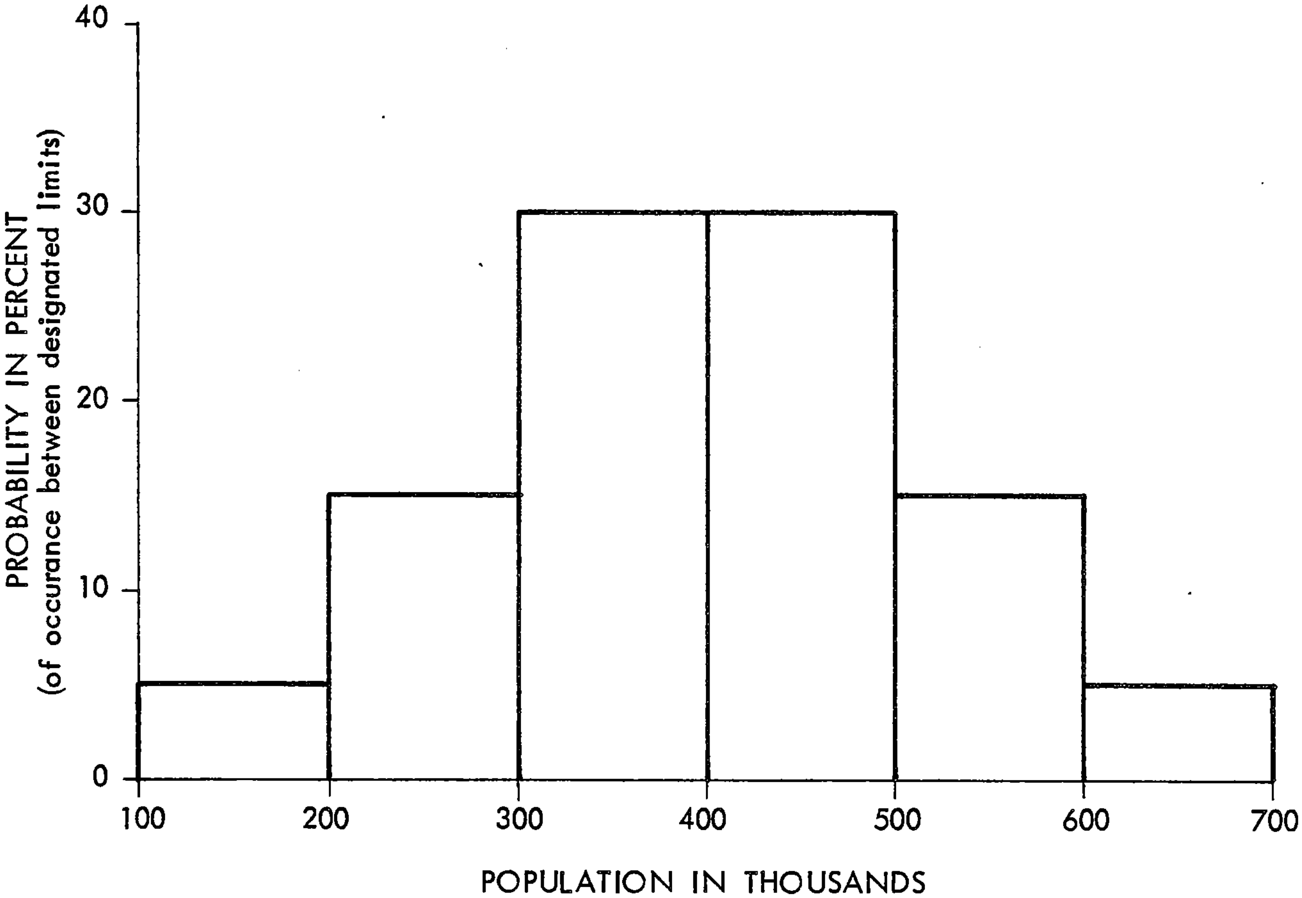

Figure 1 - Uncertainty Curve of Population Estimate of Buffalo, New York
(Based on Author's Confidence Level)

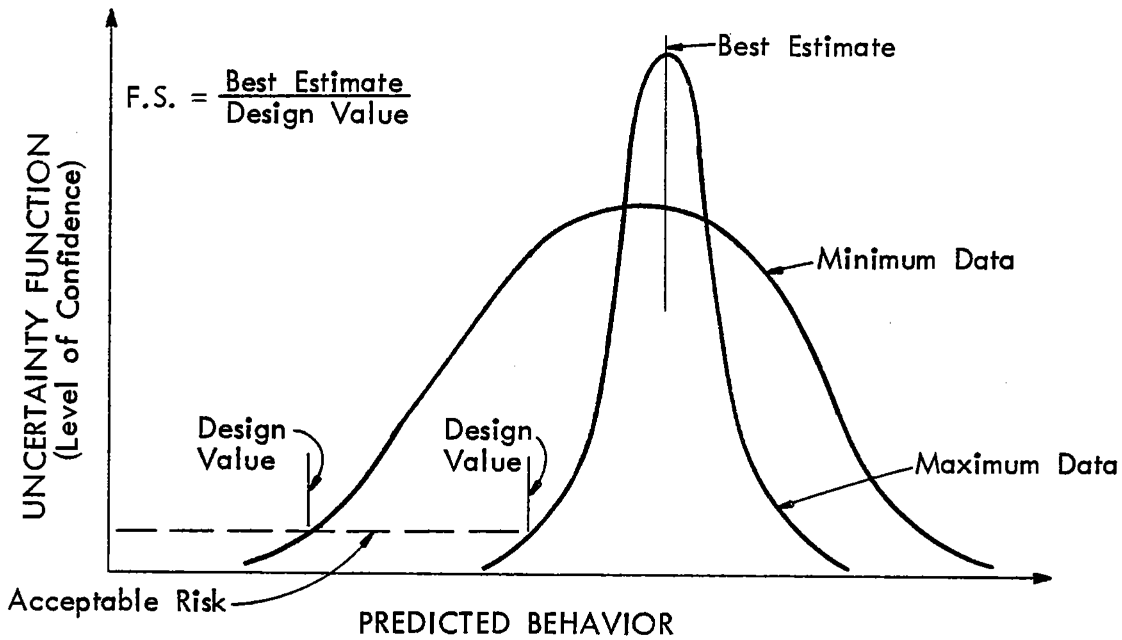

Figure 2a - Relationship Between Uncertainty Distribution and Factor of Safety (F.S.)

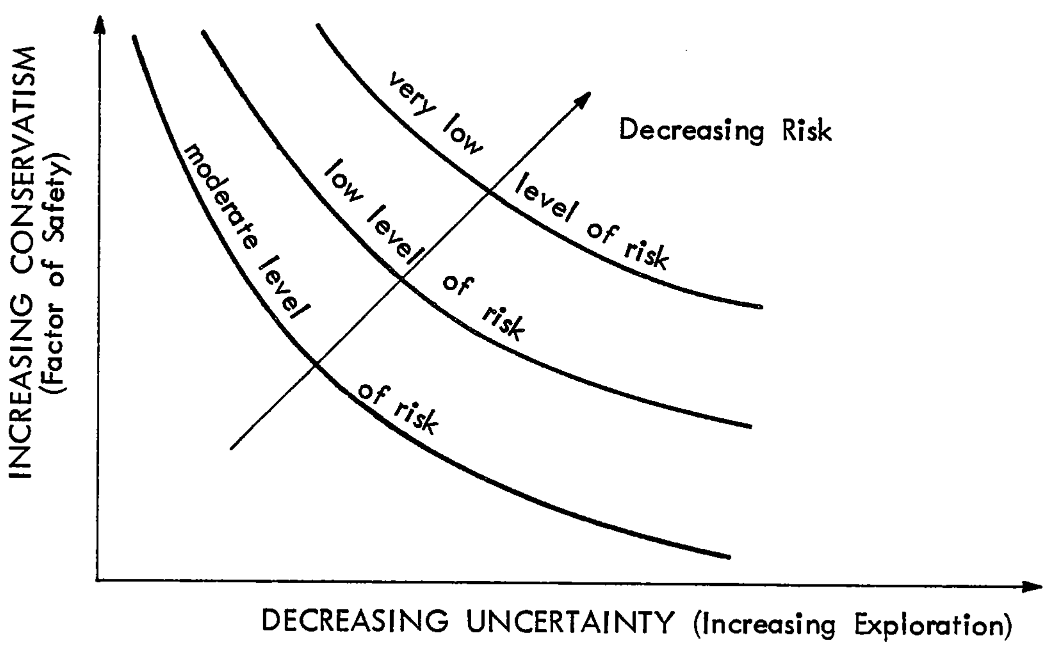

Figure 2b - Relationship Between Factor of Safety, Uncertainty Level, and Risk

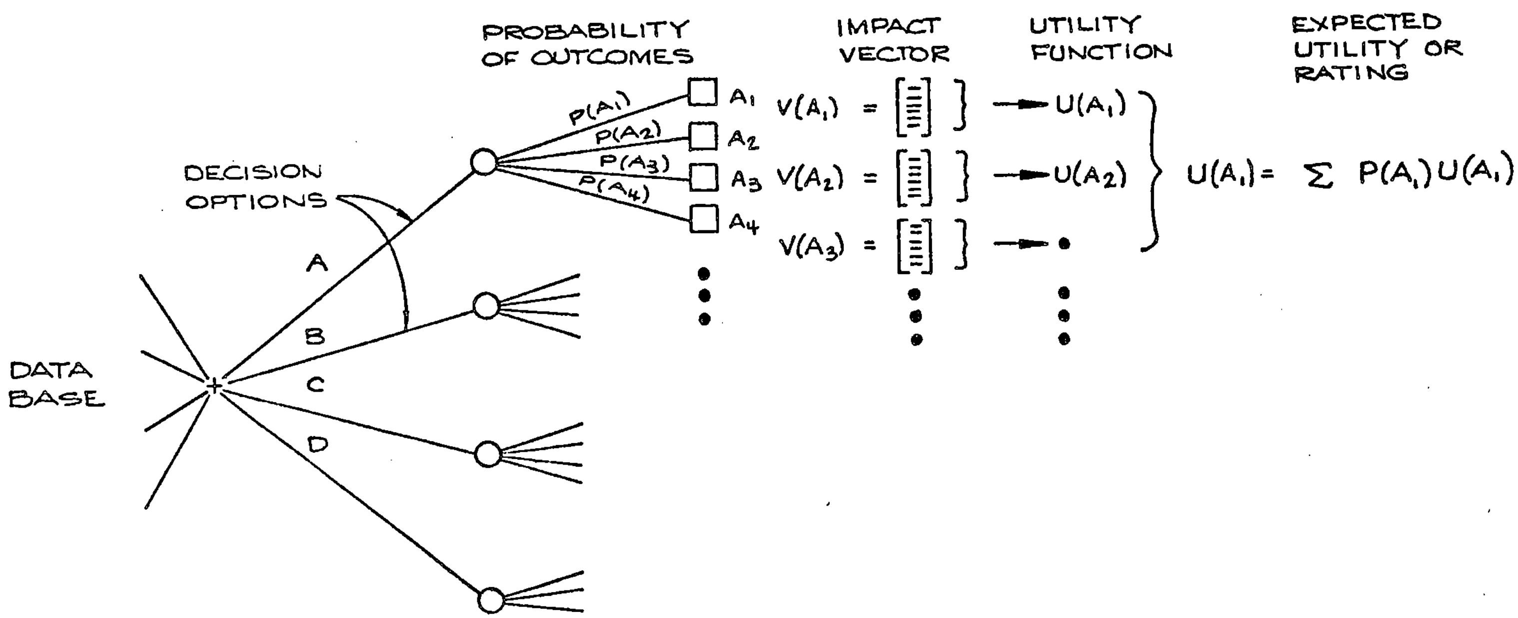

Figure 3 - Decision Theory Framework

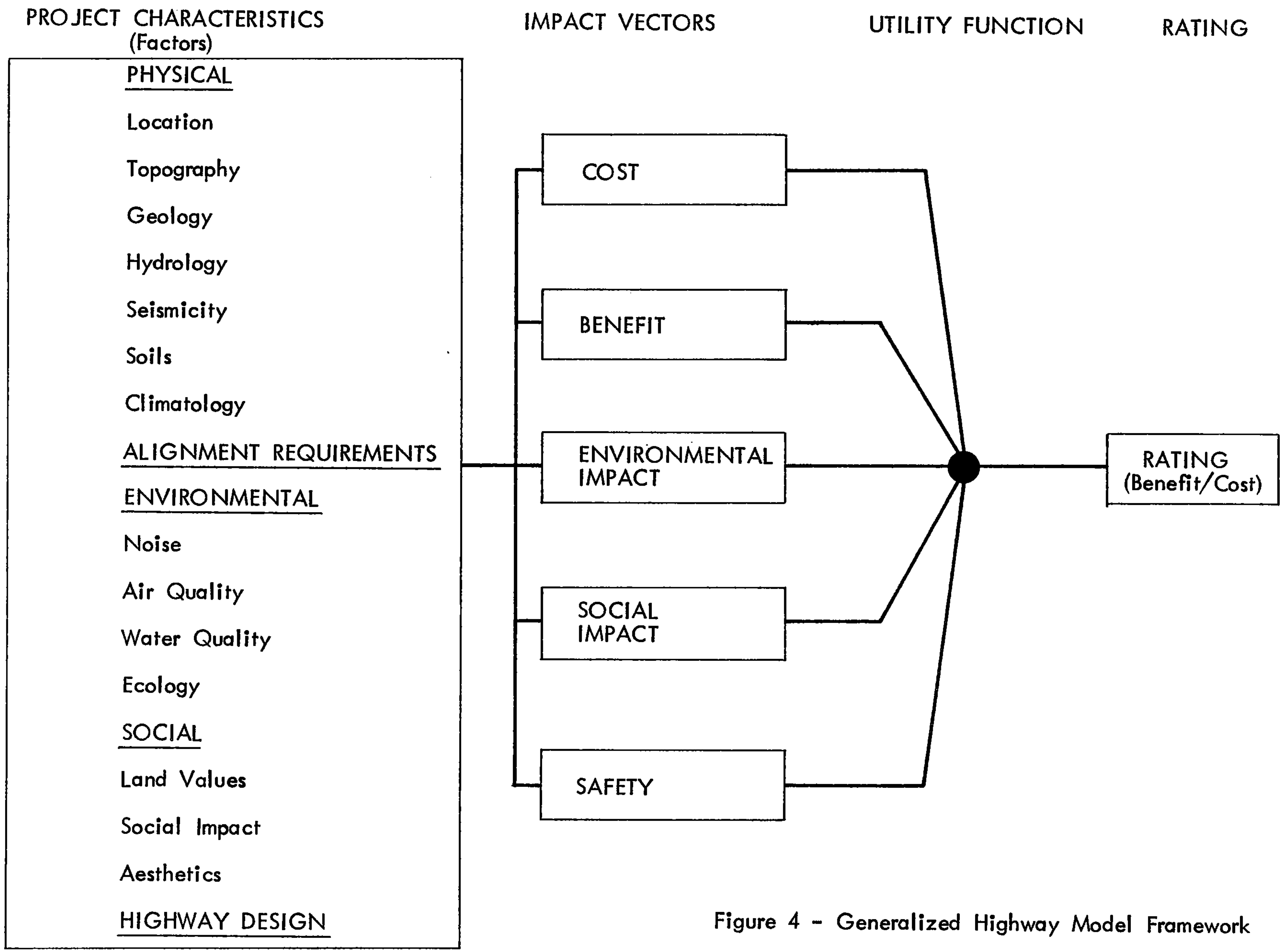

Figure 4 - Generalized Highway Model Framework

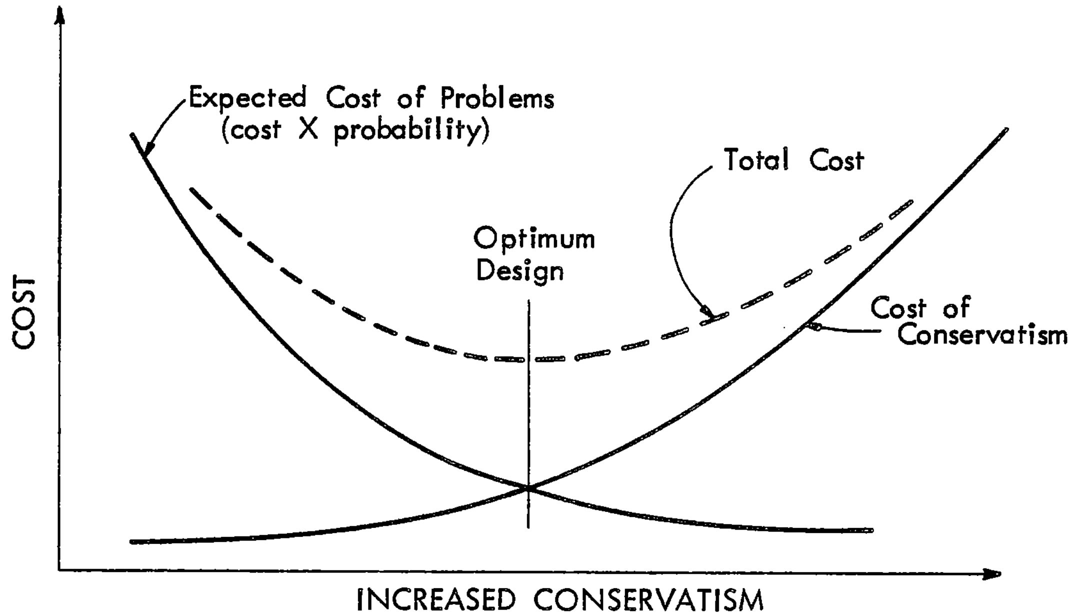

Figure 5a - Relative Comparison of Cost of Problems Developing (failure) and Cost of Increased Conservatism (Based on a Single Level of Information or Uncertainty)

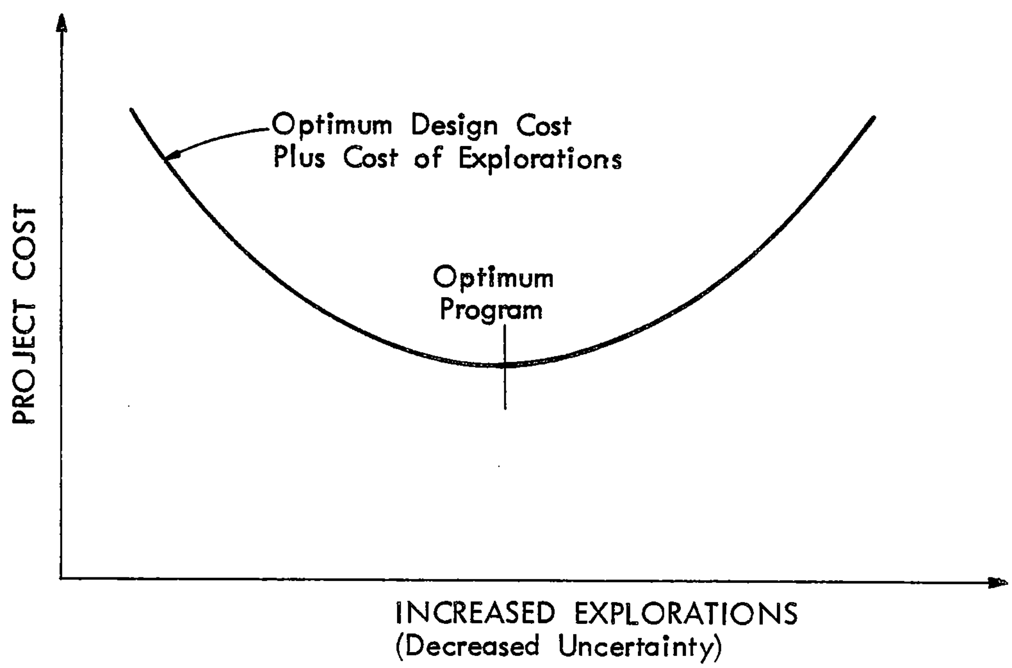

Figure 5b - Cost of Optimum Designs at Various Levels of Information or Uncertainty

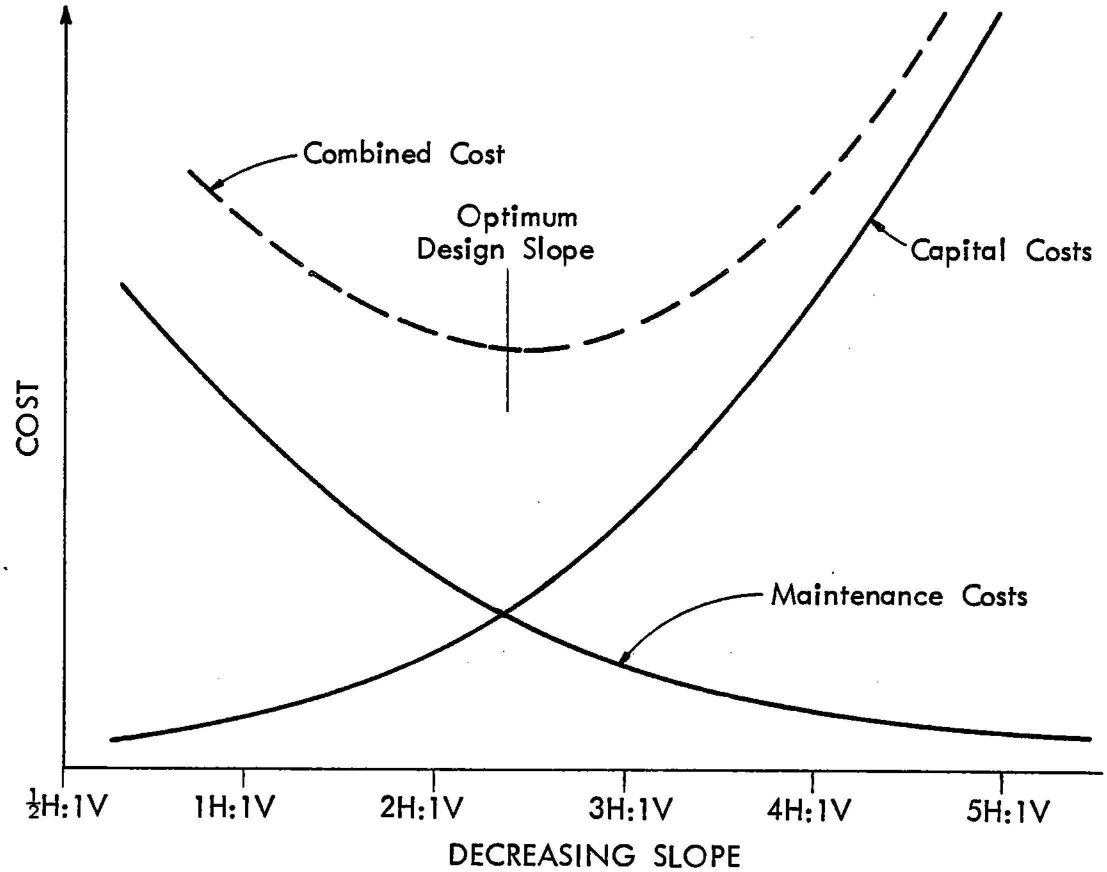

Figure 6 - Relationship Between Capital Costs, Maintenance Costs, and Total Costs for Highway Slopes

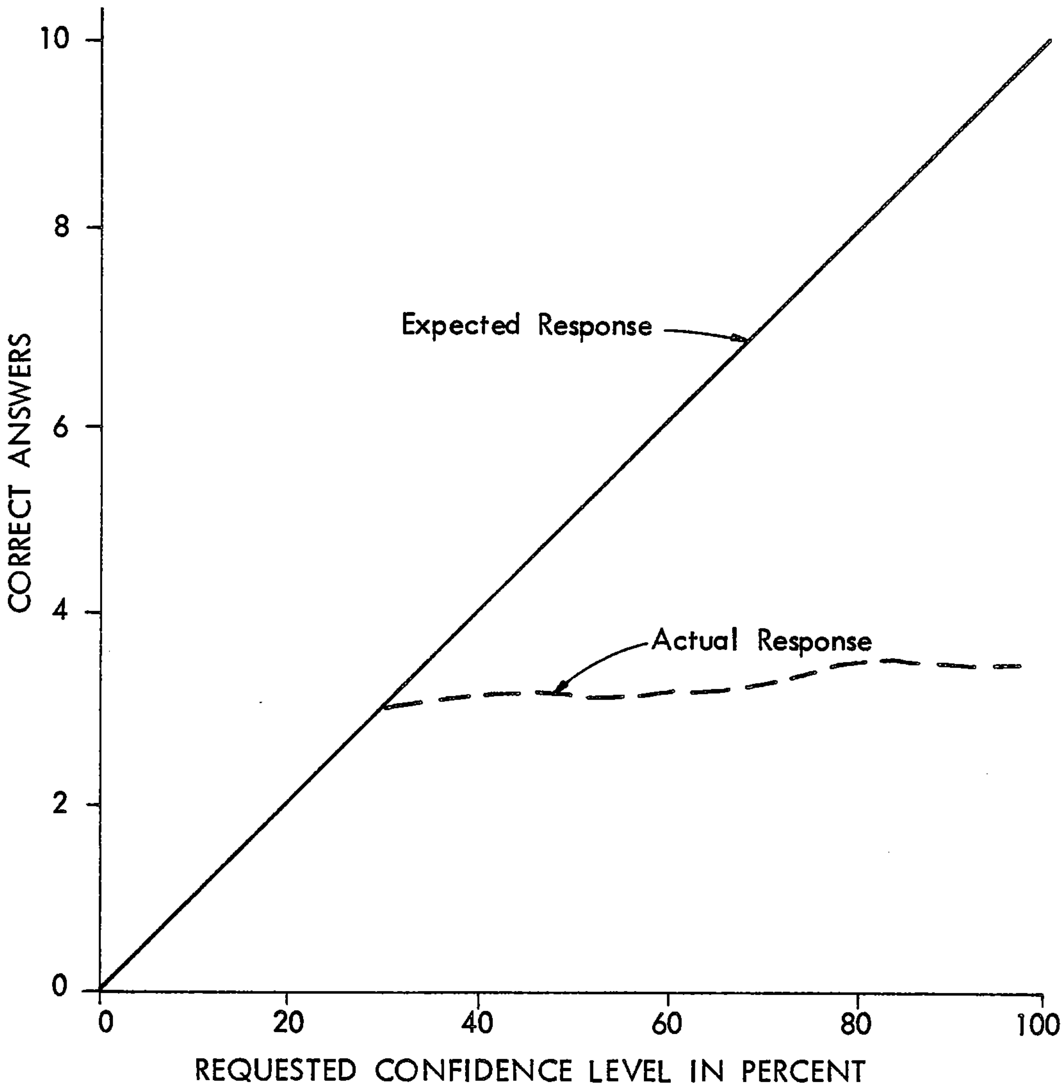

Figure 7 - Results of Uncertainty Quiz (after Capen, 1976)

REFERENCES

1. Benjamin, J.R. and Cornell, C.A. (1970), "Probability Statistics and Decision for Civil Engineers", McGraw Hill, New York.

2. Capen, E.C. (1976), "The Difficulty of Assessing Uncertainty", Journal of Petroleum Technology, August 1976.

3. Keeney, R.L. and Raifta, H. (1976), "Decisions with Multiple Objectives", John Wiley & Sons, New York.

4. Palladine, D.J. and Peck, R.B., (1972), "Slope Failures in an Overconsolidated Clay, Seattle, Washington", Geotechnique 22, No. 4.

5. Schlaifer, R.O. (1969), "Analysis of Decisions Under Uncertainty", McGraw-Hill, New York.

6. Vanmarcke, E. (1979), "Probability and Risk Analysis in Geotechnical Engineering", "Reliability Analysis and Geotechnical Engineering", Session No. 25, ASCE National Convention, Boston, Massachusetts.

STABILIZATION OF THE UPPER PORTION OF THE HAT CREEK LANDSLIDE

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Kuenzli, J.R.[2]
Ramage, John[3]

ABSTRACT

Hat Creek Landslide, located on U.S. Highway 95 near the town of Pollock, Idaho, started moving on January 27, 1974. The slide closed U.S. Highway 95 and the toe impinged into the Little Salmon River. The slide was about 2,300 by 500 feet and involved about 1 million cubic yards of material. Investigation and analysis centered around four alternatives: complete removal; lower slide removal; upper slide removal; and stabilization of the upper slide with groundwater control. The stabilization of the upper slide by groundwater control was selected as the most cost-effective alternative. A construction dewatering system, consisting of 45 eductor well points, was installed to to provide a stable trench excavation. A drain trench, 700+ feet long, was constructed across the head of the slide. The drain trench contained 6-inch perforated pipe and 10-inch collector drain pipe covered by 4-1/2 feet of filter material. The stabilization work was completed in November 1977 and realignment of U.S. Highway 95 in the fall of 1978 for a total cost of $770,000.

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INTRODUCTION

The Hat Creek Landslide is located on U.S. Highway 95 near the town of Pollock, Idaho, about 120 miles north of Boise. (See vicinity map, Figure 1). Movement of the slide was first reported on January 27, 1974. The next day, the toe of the slide had moved across U.S. 95. Two days later the slide had advanced into the Little Salmon River. By January 30, slide movement had slowed permitting the Little Salmon River to re-establish its course slightly to the west of the preslide channel.

Survey lines were set by the State of Idaho Transportation Department and the rate of movement observed. In August 1974, the State conducted seismic refraction surveys to determine the slide geometry and to supplement their subsequent exploration program.

The Department installed four dewatering wells and constructed a shallow drain trench in the upper portion of the landslide to drain water from the slide. Continued movement of the slide destroyed the drain trench and was endangering the dewatering wells.

CH2M HILL was retained by the Department to investigate the landslide, provide recommendations for the correction of the landslide and technical assistance during the construction.

SLIDE DESCRIPTION

Surface Features

The Hat Creek Landslide occurred in a ravine above the east bank of the Little Salmon River. Prior to the slide, vegetation on the hillside consisted of grasses, sagebrush, scattered conifers, and phreatophytes around springs and the ravine bottom. The slopes of the hillside prior to the slide were moderately steep, ranging from 25 to 50 percent. There were no roads or trails in the area prior to the slide. The hillside was mainly used for rangeland by local ranchers.

The slide is similar to the numeral 8 with a narrow constriction, the throat area, separating the slide into an upper and lower section. The upper slide is about 500 feet across and 500 feet in length. The lower slide is also about 500 feet across, but nearly 1,400 feet in length.

The narrower throat area is about 50 feet wide, and constricted by two rock outcrops. Surface features are shown on the photograph in Figure 2. Profiles of the slide are shown on Figures 4, 5, and 6.

The head of the slide is delineated by scarps 15 to 40 feet high. The upper scarp area slopes steeply to the middle of the upper slide. The head of the slide and upper scarp area were stable during the investigation. However, scarps at the lower end of the upper slide were active. Cracks opened at rates sometimes ranging from 2 to 12 inches per day. Water was ponding in many of these active scarp openings.

Below the throat the ground surface of the lower slide flattens to form a bench area, a gentle slope in the middle of the lower slide. Below the bench area the slope of the ground surface increases to about 60 percent forming the toe of the landslide.

<u>Subsurface Profile</u>

The slide mass generally consists of a mixture of cobble-boulder colluvium and weathered rock overlying metamorphic bedrock. Boundaries between the colluvium, weathered rock, and unaltered rock are distinct in some locations, while in other locations the profile is one of gradual change from colluvium to unaltered bedrock.

The contact between the slide mass and the underlying bedrock appeared to be the shear zone. The thickness of this zone varies from a few inches to as much as 3 to 4 feet. The shear zone was exposed along the flanks of the slide and at a bedrock exposure in the toe of the lower slide.

In the undisturbed area above the slide, there are 50 to 60 feet of colluvium overlying 20 to 25 feet of weathered rock. Because of differences in lithology, degree of weathering, and variations in attitude, the weathered rock and unaltered rock have nonuniform characteristics.

Groundwater

The depth of groundwater was measured in the observation wells, slope inclinometer installations and in the borings at the completion of drilling. Groundwater depths ranged from the ground surface to greater than 30 feet deep in some portions of the slide.

Apparent artesian pressures were encountered in some of the borings. Soils were generally moist to damp except for a wet, saturated zone about 5 feet thick overlying the bedrock surface. When borings penetrated this wet zone, water levels in the borings would rise 4 to 6 feet above the saturated zone.

AREA GEOLOGY

The landslide is located in a zone of metamorphic rocks that were highly distorted and disturbed during the emplacement of the Idaho Batholith. This major unconformity, accompanied by east-west thrust faulting, has placed older meta-sediments over the Seven Devils Volcanics to the west.

Because of metamorphism and the series of thrust faults in the area, stratigraphic and structural relationships are complex and local anomalies are common. In the slide area, the foliation planes have general strike attitudes of north 20 to 30 degrees east and dips of 40 to 50 degrees southeast. It should be noted that attitude is referenced to foliation and has no necessary relationship to bedding. Field observations show that individual units grade laterally and stratigraphic relationships are difficult to establish.

In the slide area, much of the schist is talc rich. These rocks are weak and, when weathered, behave like a cohesive soil. Most of the rocks are schistose and exhibit marked differences in strength depending on lithologic character and the direction of applied load. The probability of minor cross faulting and shearing also exists at, and adjacent to the slide area.

EXPLORATION AND TESTING PROGRAM

The subsurface exploration consisted of nine borings to determine the subsurface profile and obtain samples for testing; and 15 seismic traverses and 13 solid point penetrometer probes to determine the bedrock profile. Five slope inclinometer installations and five observation wells were used to measure movement and groundwater levels within the slide mass. Boring locations are shown on Figure 3 and boring logs on Figures 7 and 8.

Direct shear tests were performed on specimens of the bedrock, bedrock-slide material contact, the slide material, and a mixture of the bedrock and slide material.

Results of the direct shear tests are summarized on the following page (Reference 1):

SUMMARY OF DIRECT SHEAR TESTS

Sample Description	ø	C(psf)	Remarks
Talc schist	27°	550	Dry peak strength
Talc schist	22°	300	Dry residual strength
Talc schist	20°	200	Wet residual strength
Talc schist	33°	0	Dry residual strength
Talc schist	23°	100	Wet residual strength
Talc schist - slide material contact	20°	200	Saturated
Slide material	34°	0	Remolded to 110 pcf at 12.5 percent moisture content
Mixture of slide material and talc schist	25°	0	Remolded to 110 pcf at 12.5 percent moisture content

STABILITY ANALYSIS

The stability analysis was performed using a computer program developed at Purdue University for the Indiana State Highway Commission. The program, STABL, performs general two dimensional limiting equilibrium analyses by the method of slices for general slope stability problems.

Preslide Conditions

The stability of the hillside prior to failure was analyzed to evaluate strength properties of the failure zone and to compare them with results of laboratory tests.

Results of this analysis indicate that with preslide topography and groundwater at the ground surface, the factor of safety is 1.01 for strength properties of $\phi = 25°$ and C = 1,000 psf. These properties are slightly higher than strength properties determined in the laboratory. Using laboratory determined strength parameters and groundwater at ground surface, the factor of safety is 0.82. Lowering the groundwater to 20 feet below the ground surface increases the factor of safety to 1.12.

Post Slide Conditions

The stability of the slide area in the post slide configuration was analyzed. The stability of the upper, lower, and entire slide was determined as well as the effects of dewatering and construction on the stability of the slide.

The following soil properties, based on the results of laboratory tests, field tests, and preslide analysis, were used.

Saturated unit weight	γsat = 115 pcf
Moist unit weight	γt = 110 pcf
Intact strength of talc schist	$\phi = 27°$, C = 550 psf
Residual strength of talc schist slide material contact	$\phi = 20°$, C = 200 psf
Peak strength of slide material	$\phi = 34°$, C = 0

The results indicate these conditions:

- The stability of the slide area in its post slide configuration is sensitive to groundwater fluctuation

- The upper slide is less stable than the lower slide

- With groundwater at the bedrock surface (i.e., the slide mass completely dewatered), the factor of safety for the entire slide is 1.26

- The existing slide mass is unstable (factor of safety = 0.96) using the residual strength of the talc schist-slide mass contact (Ø = 20°, C = 200 psf) with groundwater at a depth of about 20 feet

- The strength properties required to stabilize the slide are Ø = 27° and C = 550 psf with the groundwater at ground surface. These are approximately the strength properties of the intact talc schist bedrock. In other words, with the groundwater at the ground surface, the failure zone must regain strength properties comparable to the strength of the intact talc schist bedrock in order to stabilize the slide in its existing configuration.

STABILIZATION ALTERNATIVES

Four alternatives were evaluated for the stabilization of the landslide: complete removal, lower slide removal, upper slide removal, and stabilization of the upper slide by groundwater control.

Complete Removal

Complete removal of the slide would involve excavating about 800,000 cubic yards of slide material. Material removed from the slide area could be placed in waste areas immediately north and south of the slide along the existing U.S. Highway 95.

Lower Slide Removal

Removal of the lower slide would involve excavating about 495,000 cubic yards of material. Regrading of the upper portion of the slide and installation of surface drains at existing springs would be required to intercept water entering the upper slide.

Upper Slide Removal

Removal of the upper slide would involve excavating about 306,000 cubic yards of material in addition to removing about 20,000 cubic yards of the toe of the slide for highway realignment. Regrading of the lower slide and installation of surface drains would be required to intercept water entering the lower slide.

Stabilization of the Upper Slide by Groundwater Control

a. Drain Trench

The interceptor drain trench, excavated to the bedrock contact, would be located in the upper slide as shown on Figure 9. The maximum depth of the drain trench would range between 25 and 30 feet at the center of the slide. A filter gravel blanket was recommended in the bottom and on the uphill slope of the excavation to intercept seepage.

b. Horizontal Drains

The horizontal drains alternative is similar to the drain trench alternative except that horizontal drains would be used to intercept the groundwater. The horizontal drains are expected to intercept seepage flowing into the slide by tapping the zone of slide mass above the bedrock contact.

A preliminary cost estimate was prepared for the alternatives and is summarized below:

Complete Removal	$1,258,400
Lower Slide Removal	$1,030,400
Upper Slide Removal	$1,012,200
Stabilization of the Upper Slide	
a. Drain Trench	$ 681,400
b. Horizontal Drains	$ 436,700

RECOMMENDED SOLUTION

Stabilization of the upper slide by controlling groundwater with a drain trench was recommended to the State of Idaho. Although complete removal was considered the most reliable means of correcting the slide, the drain trench scheme was recommended:

- It was considerably less costly than any of the removal schemes ($681,000 estimated)
- It provided similar benefits as the upper slide removal alternative with some risk, but at a much lower cost
- The volume of excavated material and the areas required for disposal were considerably less than for the other alternatives

- All existing springs could readily be intercepted by the drain trench

- Minimal maintenance would be required following construction

DRAIN TRENCH CONSTRUCTION

The stability analyses indicated that construction dewatering would be required to complete the drain trench excavation. An eductor system was selected rather than conventional well points or deep wells for two reasons:

- The 30-foot pumping head would require a staged well point system

- The relatively small discharges are not suited to deep well installations

The eductor system was designed to lower the groundwater levels in the area of the drain trench permitting a stable excavation. Based on the subsurface model shown on Figure 10A, analysis indicated that eductors spaced at 20-foot centers would lower the groundwater levels sufficiently to stablize the excavated slope. In areas of concentrated groundwater flows, the eductor spacing would be decreased to 10 feet.

The construction dewatering system was installed in February and March 1977. The dewatering system consisted of two lines of eductor well points surrounding the proposed trench excavation. The eductors were spaced at about 20-foot centers on the uphill side of the drain trench and on about 50-foot centers on the downhill side. The eductor spacing was decreased to 10-foot centers in areas of groundwater concentration. All eductor wells were drilled 2 feet into bedrock and surrounded with filter sand.

Discharge from the dewatering system during the initial start-up was about 25 gallons per minute (gpm), but dropped to about 3 gpm within one week. Drawdown observed in the observation wells was erratic. The erratic nature of the drawdown was partially attributed to both pump downtime and surface runoff entering the observation wells.

Production of the dewatering system was less than the initial estimate. The initial estimate was based on groundwater flow on top of the bedrock under the entire slide mass. The eductor system was designed to intercept this flow.

The major flow of water was not found on top of the bedrock, but in two fractured zones near the south flank. (See Figure 10b.) As such, water was not available to all eductors, only those drilled into the fractured zones. Of those in the fractured zone, only those penetrating a water-bearing fracture were capable of producing water. Therefore, only a few of the eductors were likely to intercept significant quantities of water.

The trench was installed in fall 1977. The trench consisted of an open cut excavation to bedrock. The bottom of the drain trench was generally excavated a minimum of 2 feet into the bedrock penetrating the impervious gouge zone. Two drain pipes, one perforated and one nonperforated were placed in the bottom of the trench. Granular backfill was placed around the drain pipes and extended up the uphill slope.

Near the boundary of the slide, the soils were dry and the excavation proceeded rapidly. Once within the slide boundary soils became wetter and the rate of excavation slowed somewhat.

In two areas near the middle of the slide the uphill slope of the excavation became unstable. Cracks 2 to 4 inches wide were observed on the slope. Excavation was performed as rapidly as possible and the slope was buttressed soon after the trench was backfilled to prevent a major slope failure. With the exception of these two areas, the trench excavation remained stable.

Two water-bearing fractured zones were encountered. Water encountered in these areas surfaced through fractures in the side of the drain trench or through fractures on the bedrock surface uphill of the drain trench. Where possible, water was tapped at the source by collecting water into laterals from the drain pipe.

The total cost to complete the installation of the eductor dewatering system and drain trench was $369,000. The eductor dewatering system consisted of 45 eductors; a total drilling footage of 1,012 linear feet; and headers, pumps and miscellaneous piping for a total cost of $143,000. The drain trench consisted of about 35,000 cubic yards of excavation; 2,100 linear feet of 6-inch drain pipe; 440 linear feet of 10-inch drain pipe; 2,400 linear feet of horizontal drains; and miscellaneous piping and appurtenant structures for a total cost of $226,000.

The realignment of U.S. 95 at the toe of the landslide was completed in fall 1978. About 250,000 cubic yards of material were removed from the lower slide. Cost of the realignment was $400,000. The total cost of the stabilization was about $770,000 as compared to the preliminary estimate of $681,000.

The State Transportation Department has been monitoring the landslide since the completion of the realignment and has observed no movement in the lower portion near the toe of the slide, nor in the upper slide near the drain trench.

REFERENCES

1. Preliminary Design Recommendations for Stabilization of the Hat Creek Landslide near Pollock, Idaho. CH2M HILL, Boise, Idaho. June 1, 1976.

2. Hat Creek Landslide, Supplement. CH2M HILL, Boise, Idaho. February 8, 1978.

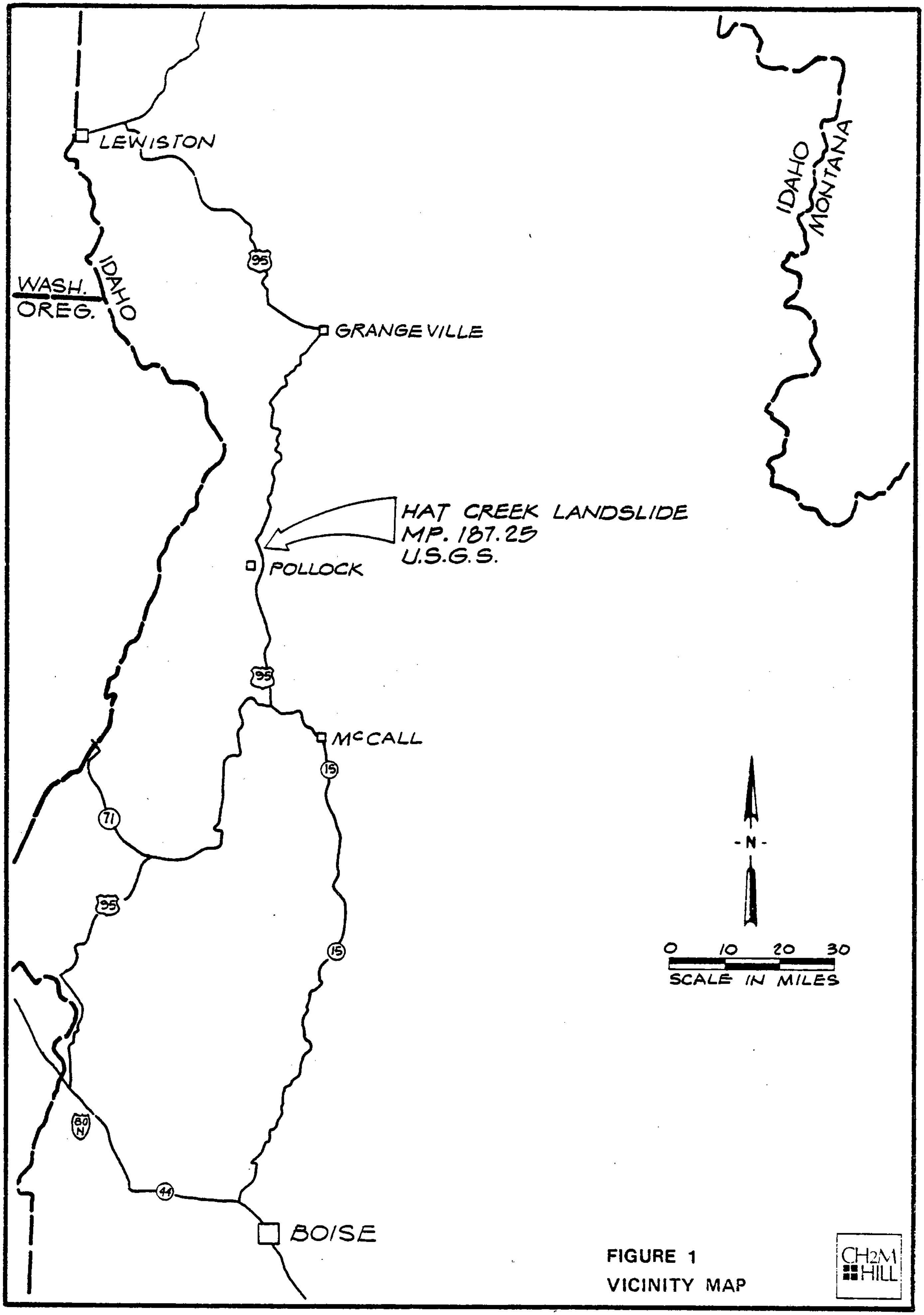


FIGURE 1
VICINITY MAP

CH2M HILL

FIGURE 2
HAT CREEK LANDSLIDE

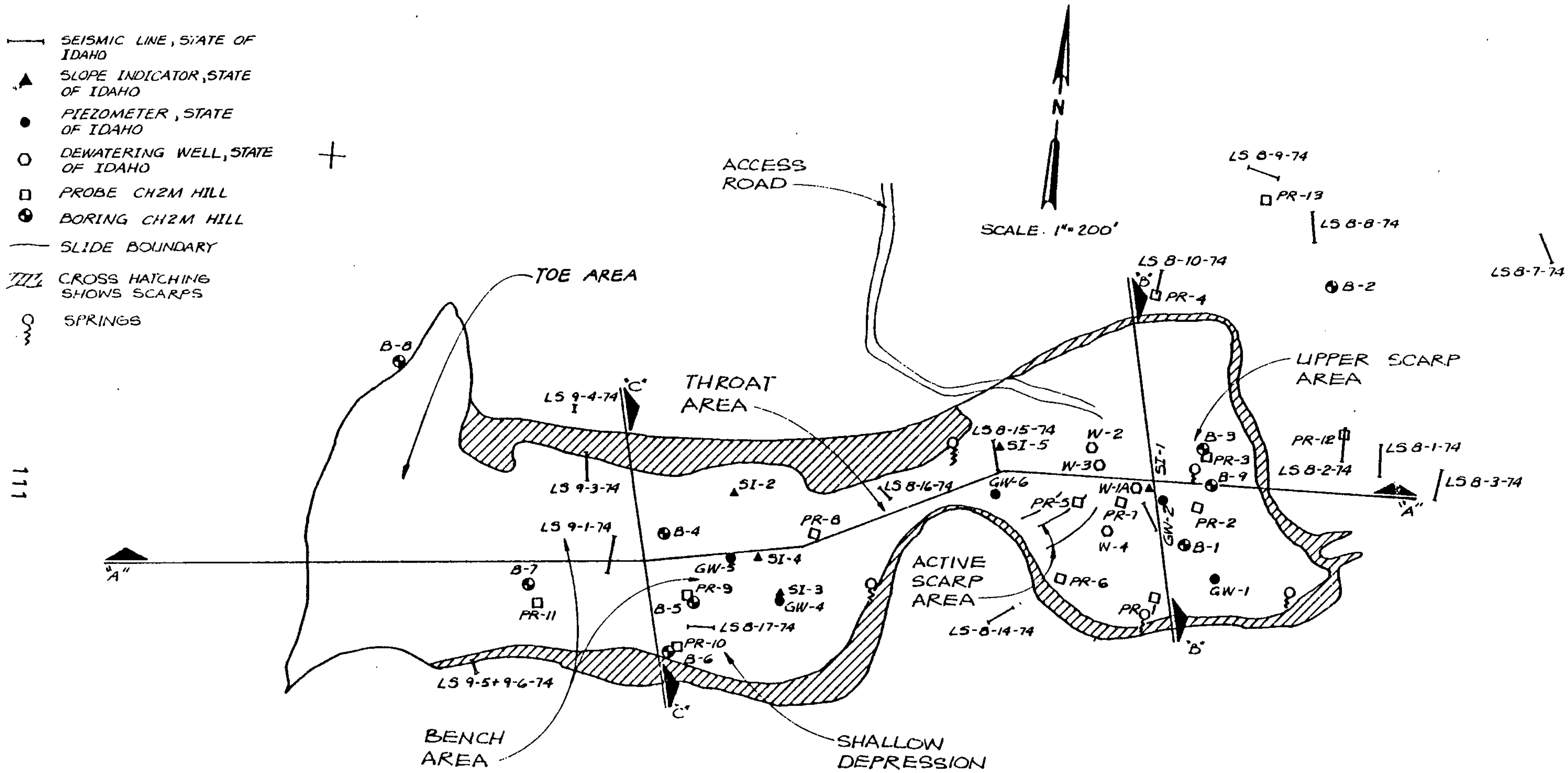


FIGURE 3
SAMPLING LOCATIONS

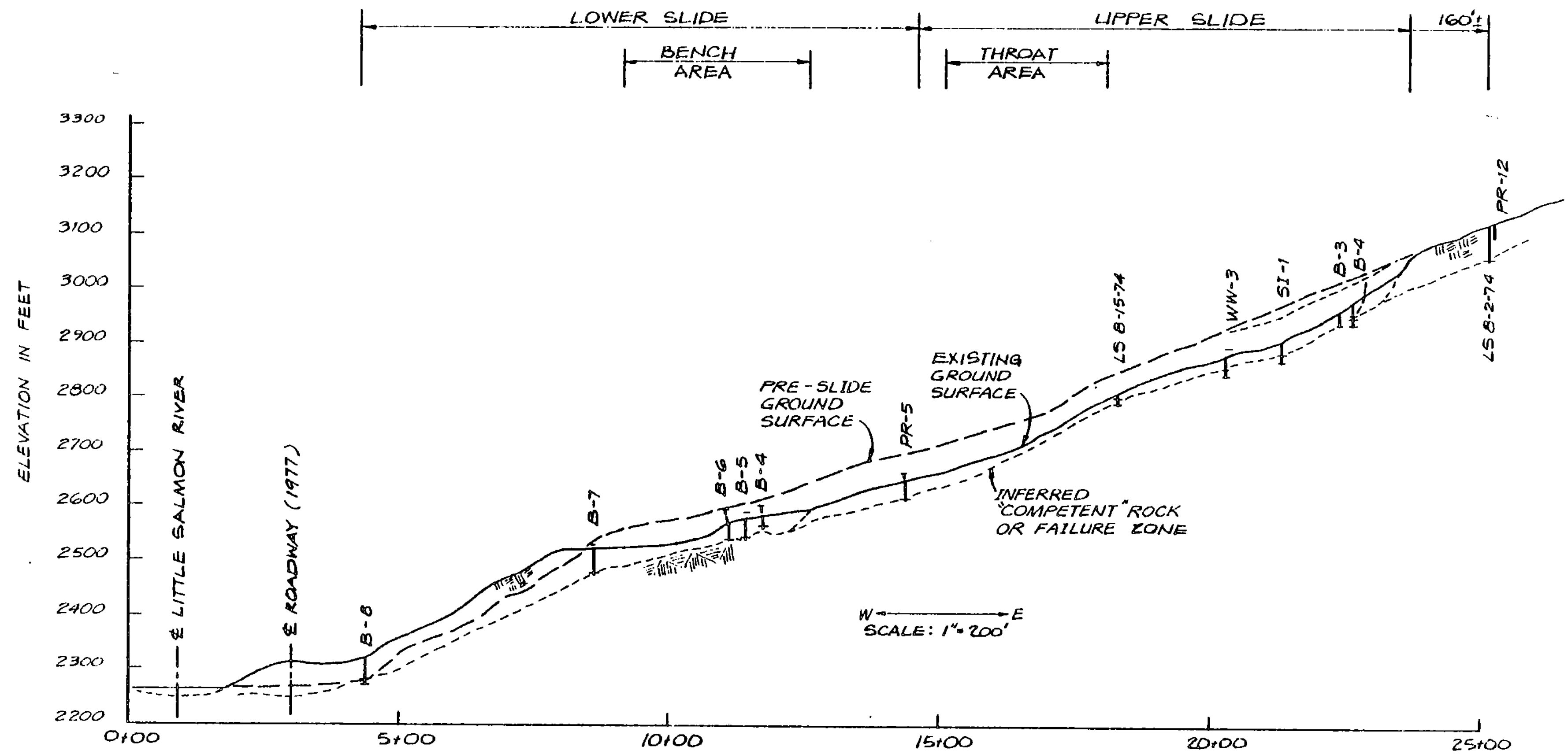


FIGURE 4
PROFILE A – A'

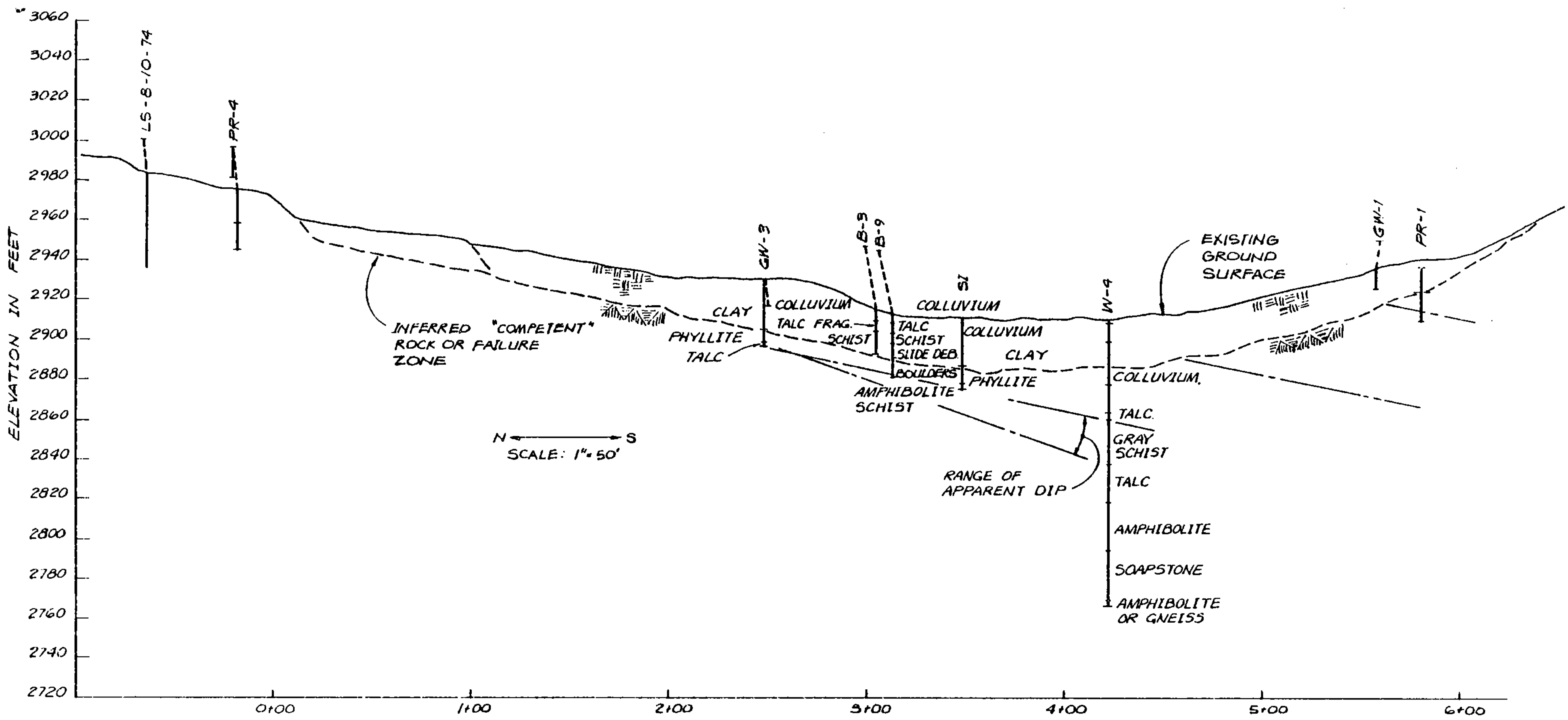


FIGURE 5
PROFILE B – B'

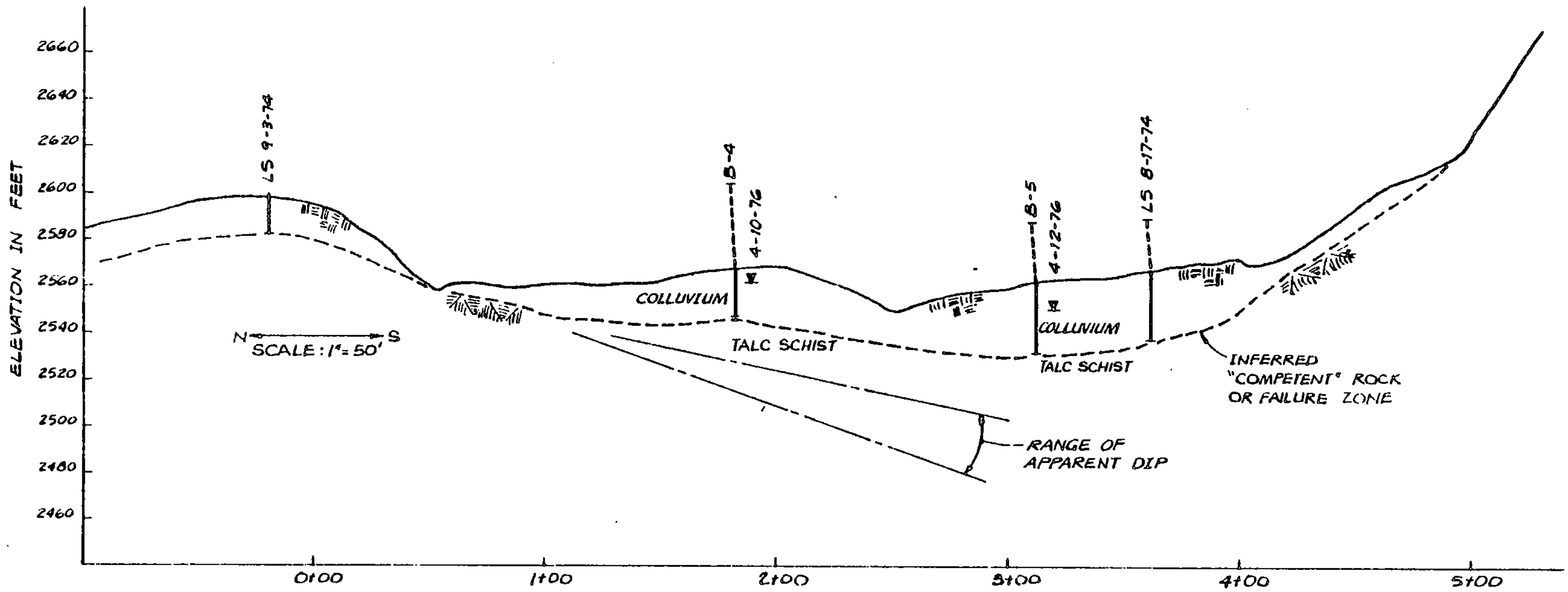


FIGURE 6
PROFILE C -- C'

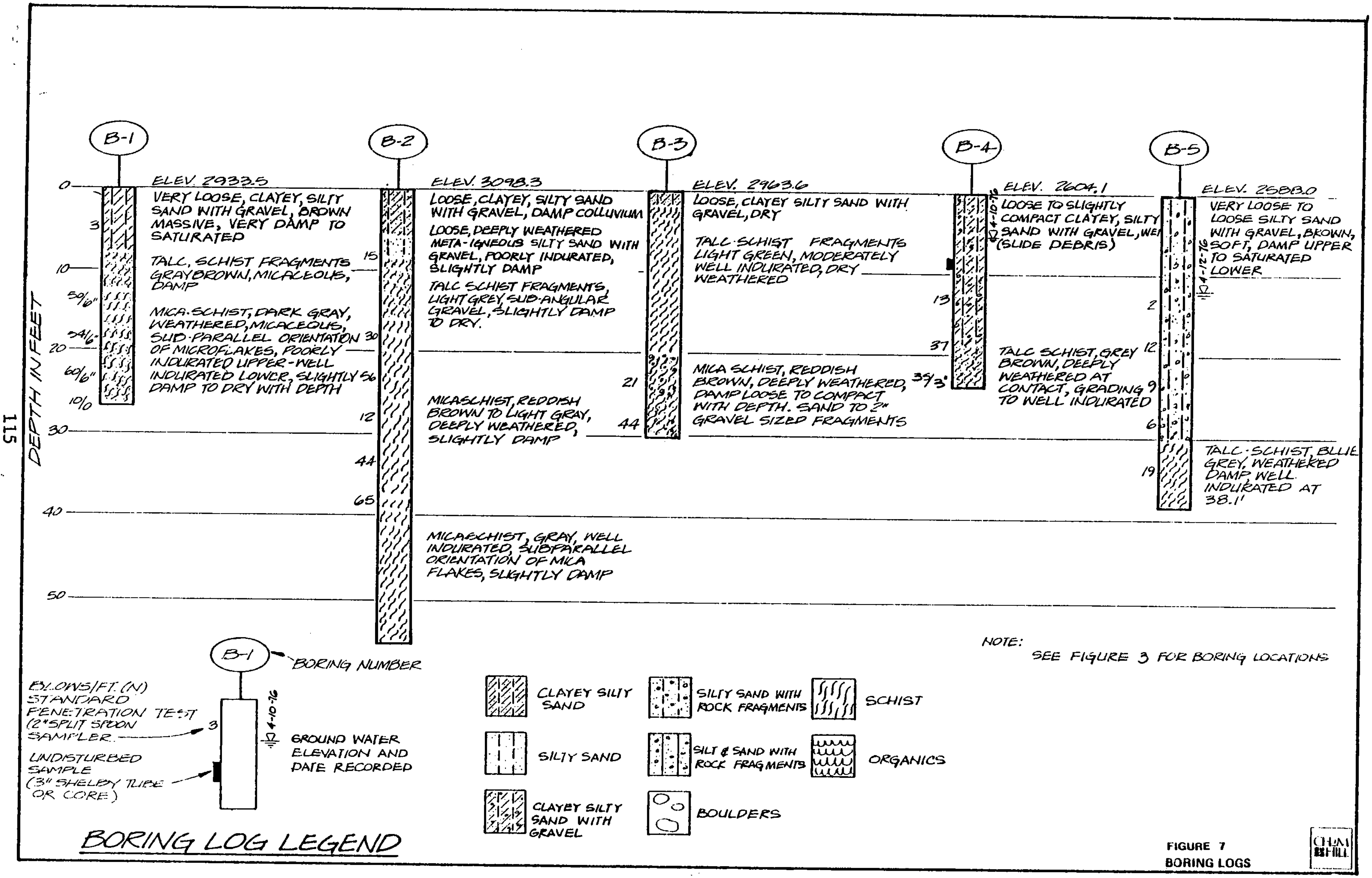
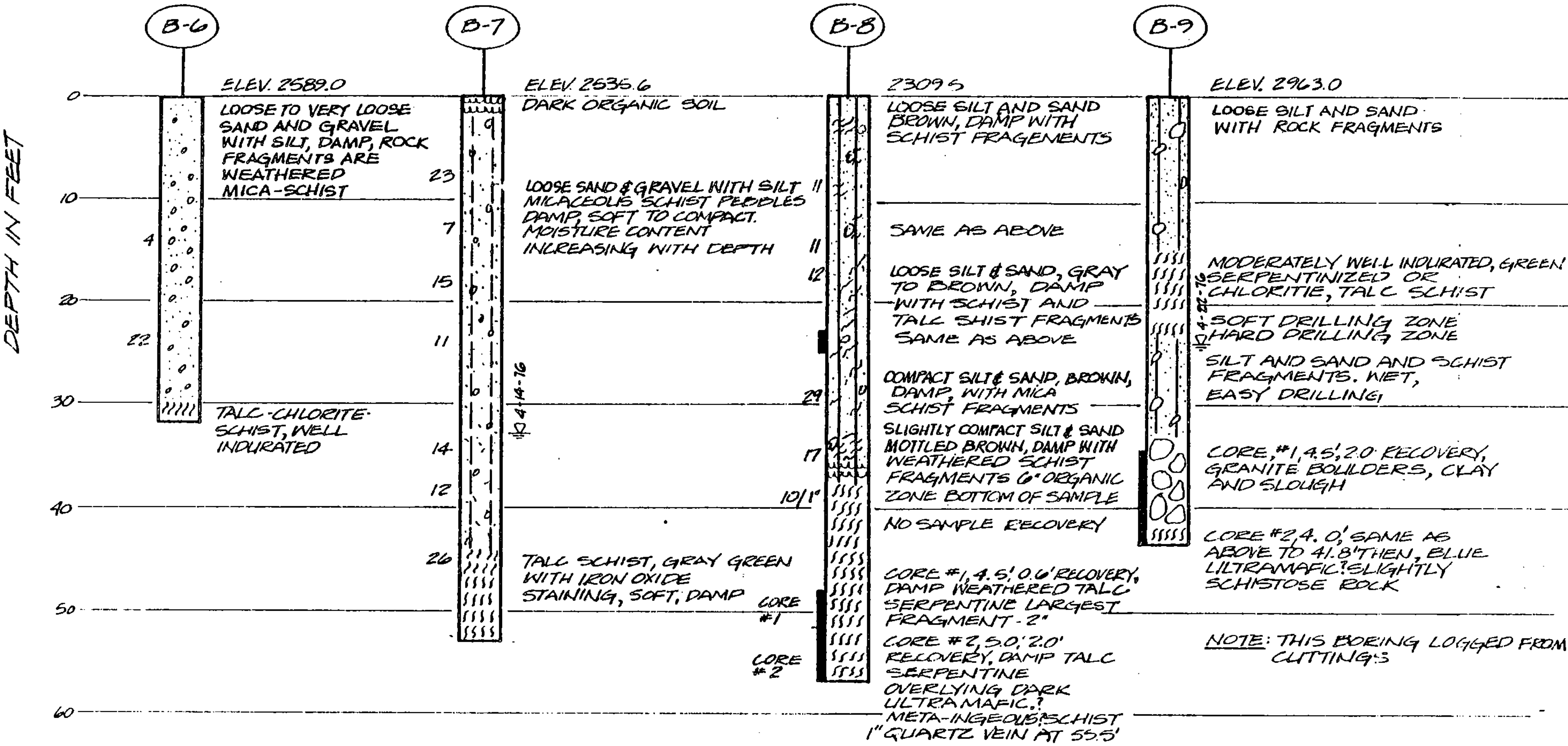


FIGURE 7
BORING LOGS



NOTES:

1. SEE FIGURE 7 FOR LEGEND.
2. SEE FIGURE 3 FOR BORING LOCATIONS

FIGURE 8
BORING LOGS

CH2M HILL

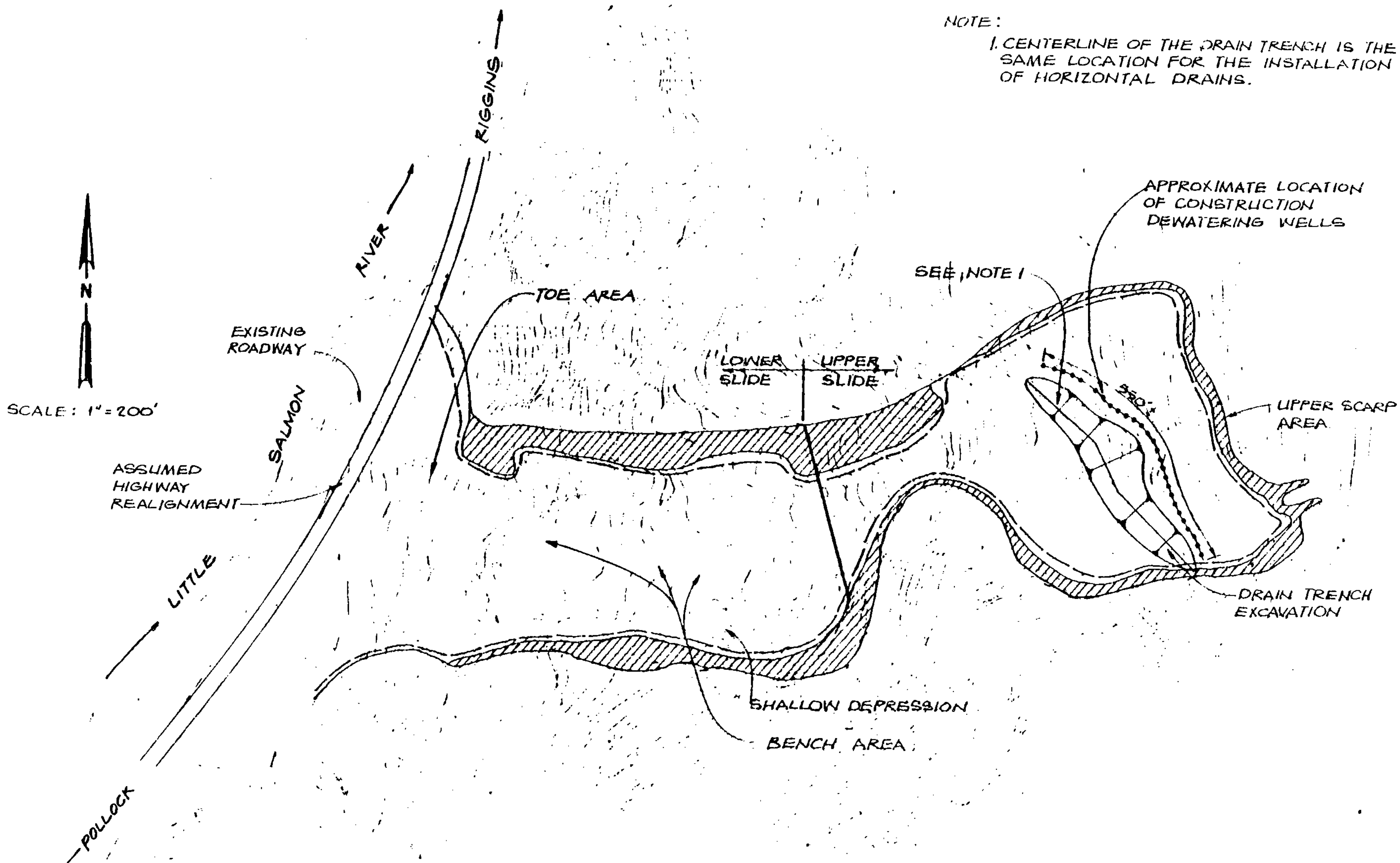
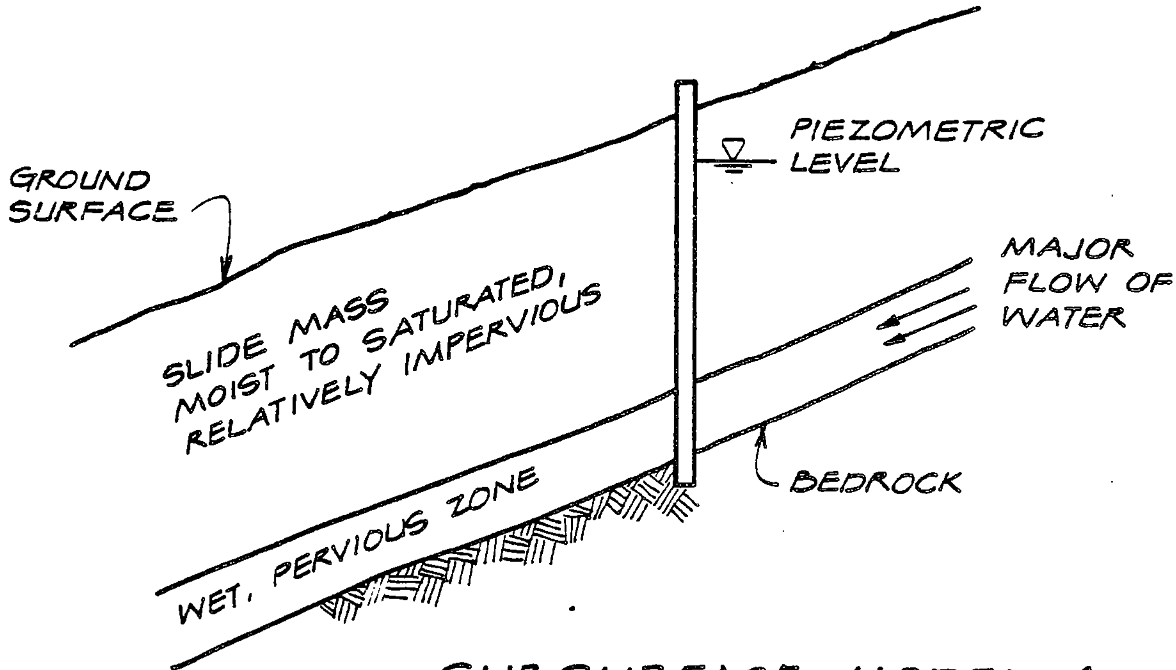
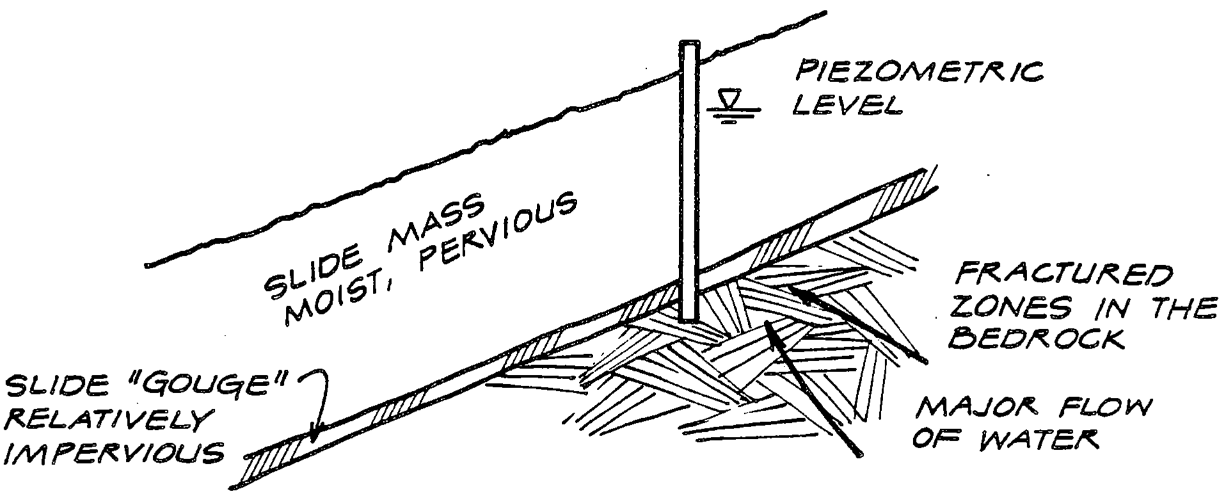


FIGURE 9
PLAN OF SLIDE AREA



SUBSURFACE MODEL-A

USED IN ANALYSIS



SUBSURFACE PROFILE-B

ENCOUNTERED DURING
DRAIN TRENCH EXCAVATION

FIGURE 10
SUBSURFACE MODEL

CH2M HILL

LANDSLIDE REMEDIAL MEASURES

By

David L. Royster
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Tennessee Department of Transportation

ABSTRACT

Landslide remedial measures may be divided into four major categories: drainage, removal, restraint, and relocation. The measure chosen is dependent on a number of factors: the type of movement (fall, slide, flow, etc.); the kinds of materials (rock, soil, debris) involved; size and location of the failure; the processes or agents that precipitated the movement; the place or thing affected by or situation created as a result of the failure; potential for enlargement; available resources; etc.

There are also a number of levels of effectiveness and levels of acceptability which may be applied in the use of these measures. For example, while one slide may require an immediate correction that is absolutely fail-safe, another may only require minimal control for an indefinite period through a planned maintenance program. The Tennessee Department of Transportation has utilized all of these measures individually and in various combinations to solve their landslide problems over the past several years.

LANDSLIDE REMEDIAL MEASURES

By

David L. Royster
Director, Division of Soils and Geological Engineering
Tennessee Department of Transportation

Introduction

The term landslide has been defined as the downward and outward movement of slope-forming materials: natural rock, soils, artificial fills, or combinations of these materials (Varnes, 1958). This is a simple, straight-forward definition of what quite often is a very complicated phenomenon; one that is complicated not so much by the causes and mechanism of the failure, but by the results of the failure and the considerations involved in its correction. This is not to say that the causes and movement mechanism are uncomplicated, for certainly they may very well be, and they must be understood if the proper remedial measures are to be devised, but the greatest benefit in understanding causes, mechanisms, and landslide-producing processes lies in the use of this understanding to anticipate and devise measures to minimize and prevent major slope movements. The term major should be underscored here because it is neither possible nor feasible, nor even desirable, to prevent all slope movements. There are many types of landslides that can be handled more effectively and at less cost after they occur. The consideration that landslides must be avoided at any cost may not always be the best approach. The key is in knowing when to use preventive measures and when to develop contingency remedial measures.

Landslide avoidance through selective locationing is obviously desired (even required) in many cases, but the dwindling number of safe and desirable construction sites may force more and more the use of landslide-susceptible terrain. This is especially true with new highway locations. It is ironic but a fact that 25 to 30 years ago when there were virtually no restrictions on new highway locations, and when many major landslides could have been avoided through selective locationing, the highway industry as a whole did not have the expertise to recognize the need for avoidance. Now that the industry is aware of potential problem areas, the option of avoiding them is not always available. Hence, we must do a better job of tailoring and adapting our highways and other construction projects to these less desirable sites. This is not only true of new highways but also of new housing developments. More and more developers are seeking locations with "a view." These, unfortunately, are frequently along ridges and the sides of hills where landslides are most likely to occur. What must be done in these cases is to make certain through zoning laws and building and grading codes that the planners, developers, and constructors are fully aware of the problems of such areas and insure that they do not trigger major landslides or, even worse, alter the site conditions in such a way that landslides develop long after they have departed and the site becomes the sole responsibility of the unsuspecting home owner. These so-called delayed landslides are a common occurrence, and when they happen their repair costs can be prohibitive.

But what about landslides that do occur as a result of construction activities? How can we be sure that we have selected the most appropriate remedial measure? Still further, what is the definition of appropriate remedial measure? In a broad sense, it is that measure used to solve or ameliorate a landslide problem within a certain required or accepted standard. The standard established for a given situation, i.e., safety factor, degree of effectiveness, or some other less precise requirement, is usually controlled by economics, but other factors or considerations may also be involved. This paper explores, principally through the use of actual cases, some of the factors involved in selecting remedial measures, i.e., the type of landslide, its topographic and geologic setting, the circumstances surrounding its development, potential for enlargement, etc.

Landslide Types

A number of classifications have been suggested over the years for landslides, but probably the one that has been referred to more than any other in the recent past is the one developed by Varnes (1958). It is based on two main variables: type of material and type of movement. The materials are classed as bedrock or soils, and the types of movement are divided into three principal groups--falls, slides (rotational and translational), and flows. A fourth group, which is listed as complex, is any combination of any or all of the other three types of movement. In 1978 Varnes updated his classification and added two new modes of movement--topples and lateral spreads. In this version, Varnes refrains from using the term "landslide," substituting what he believes to be a more appropriate, all-inclusive term, "slope movement." He has done this principally to incorporate such descriptive terms as "creep," "toppling," and "spreading" which are not viewed in the strict sense as sliding movements. Varnes' new classification also adds a third material term, "debris," which is helpful in differentiating the material that lies gradationwise between "rock" and "earth" ("soil" was the term Varnes used in an earlier report in 1958).

Since the principal thrust of this paper is to describe remedial measures, no attempt will be made to discuss in detail Varnes' classification system. It will be the system most referred to, however, in describing the type of movement and type of material involved in each landslide.

It has been the experience of the writer that the majority of the larger slope movements, at least in Tennessee, can be classified as slides; more precisely, translational slides, and more often than not the material involved is debris. Most of the slides that have occurred along the Cumberland Plateau escarpments in recent years--those along Interstate-40 near Rockwood, for example--fall into this category (Royster, 1973, 1974, and 1977). The next most prevalent type is flows, but many of these may also have slide or slumping characteristics that would place them in Varnes' complex category. Topples and lateral spreads are the least common of the five major groupings.

"All true slides," writes Varnes (1978), "involve the failure of earth materials under shear stress." "The initiation of the process," he continues, "can therefore be reviewed according to (a) factors that contribute to high shear stress and (b) the factors that contribute to low shear strength."

Translational slides, which, as stated, constitute the majority of slope movements in Tennessee, involve shear displacement along a more or less planar or gently undulatory surface. They are usually controlled by discontinuities or surfaces of weakness such as bedding planes, formation contacts, faults, joints, etc. Figures 1 through 6 are examples of translational slides involving three different types of materials. Figures 1 and 2 depict a translational slide in colluvium and shale. Movement resulted when the toe-support was excavated. Note that the surface of failure essentially parallels or corresponds to the colluvium-shale interface. Figures 3 and 4 illustrate a slide

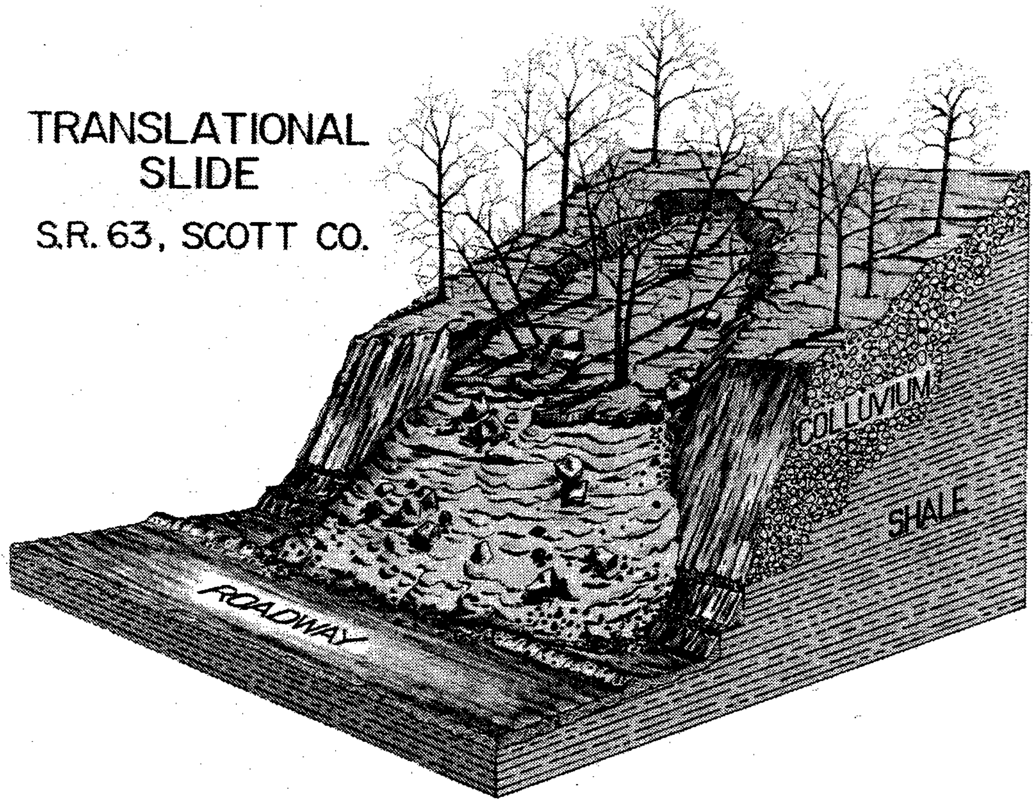
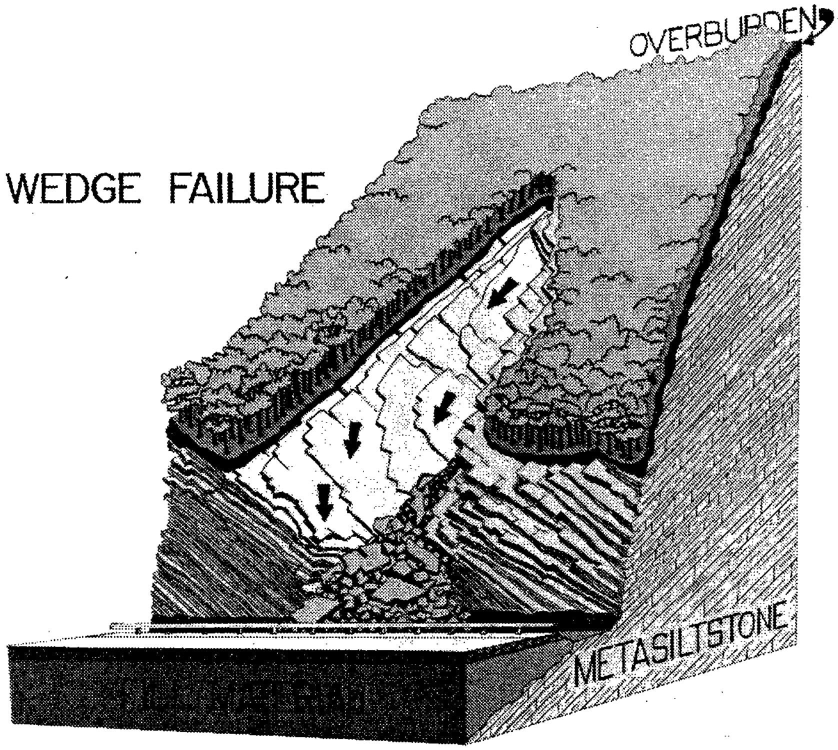


Figure 1.

Figure 2. This translational slide developed during construction in the winter of 1976.

in steeply dipping (40°-50°), highly jointed strata of metasiltstone and metasandstone. There are two directions of jointing in this case; one that is slightly skewed to the strike of the strata and the other at essentially right angles. A series of wedge-shaped slides have developed where these structural discontinuities intersect. Figures 5 and 6 depict a classic block glide failure in somewhat jointed and moderately dipping (20°-25°) sandstone with thin shale interbeds. The major joint sets in this illustration tend to parallel dip, with minor jointing normal to dip.

Of the more common type failures, translational slides probably rank second to flows in terms of analysis and evaluation difficulty, and also second in terms of repair difficulty. The analysis difficulty is that it is virtually impossible to locate and thus quantify, especially prior to failure, all of the sometimes obscure and minute discontinuities that may affect stability. The repair difficulty is related to the fact that translational slides almost always enlarge, sometimes quite substantially, during correction. As stated by Varnes (1978), "a translational slide may progress indefinitely if the surface on which it rests is sufficiently inclined and the shear resistance along this surface remains lower than the more or less constant driving force." It is imperative, therefore, that the designer look well beyond the slide mass itself when formulating corrective



I-40, COCKE COUNTY

Figure 3.

Figure 4. Aerial view of the slide shown in Figure 3 as it appeared two days after it occurred in July 1976.

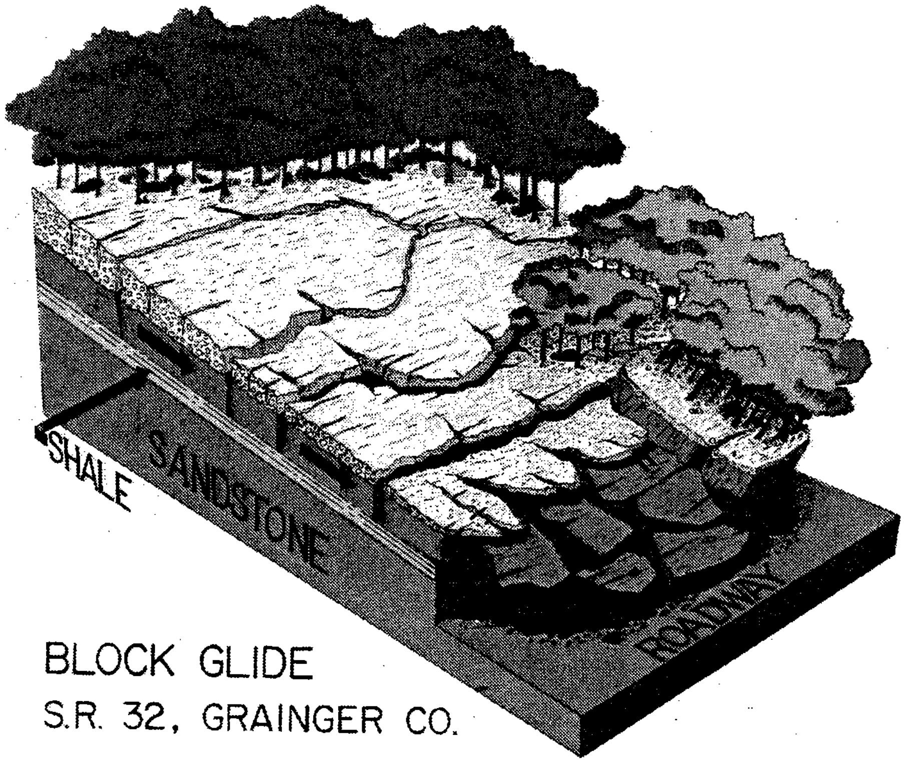


Figure 5.

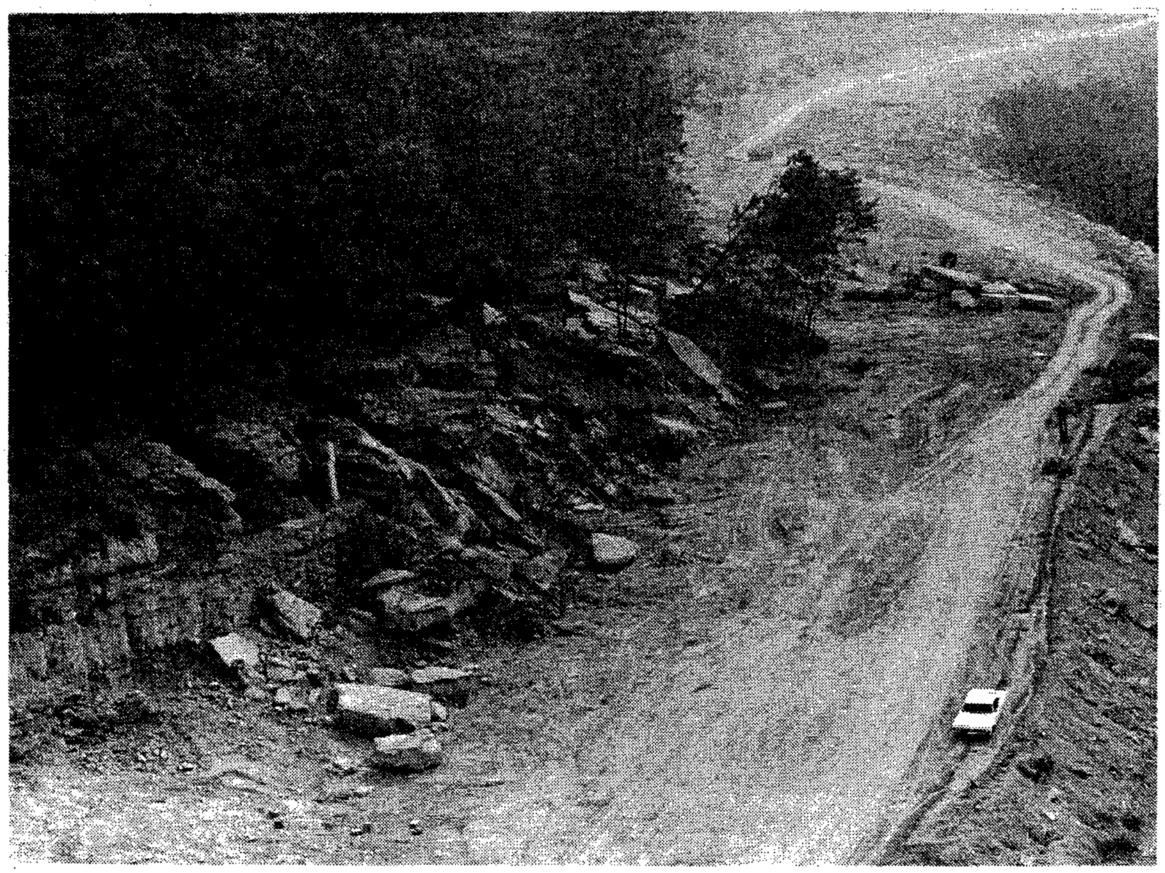


Figure 6. The strata in this cut which failed during construction are inclined approximately 22 degrees toward the roadway.

measures for translational slides.

A common but not necessarily major type of slope movement, in terms of size and cost, is the rotational slide (Figures 7 and 8). It differs from the translational slide in that the movement is more or less rotary about an axis that is parallel to the slope (Varnes, 1978). The failure is typically spoon-shaped and usually consists of a series of units or segments that are relatively undeformed. The scarp at the head of most rotational slides is commonly quite steep, being in many cases almost vertical, and the top surface of each slumped unit tends to tilt backward toward the slope. Rotational slides are not normally as difficult to analyze as translational slides because they frequently involve materials that are more homogeneous and, therefore, easier to sample and test. This results in data that is more accurate and reliable.

Flow, as defined in AGI's Glossary of Geology, is "a mass movement of unconsolidated material that exhibits a continuity of motion and a plastic or semifluid behavior resembling that of a viscous fluid." Varnes (1978) goes somewhat beyond this definition and classifies some forms of bedrock movement as flow. "Flow movements in bedrock," he writes, "include deformations that are distributed among many large or small fractures, or even microfractures, without concentration of displacement along a through-going fracture." "The movements," he continues, "are

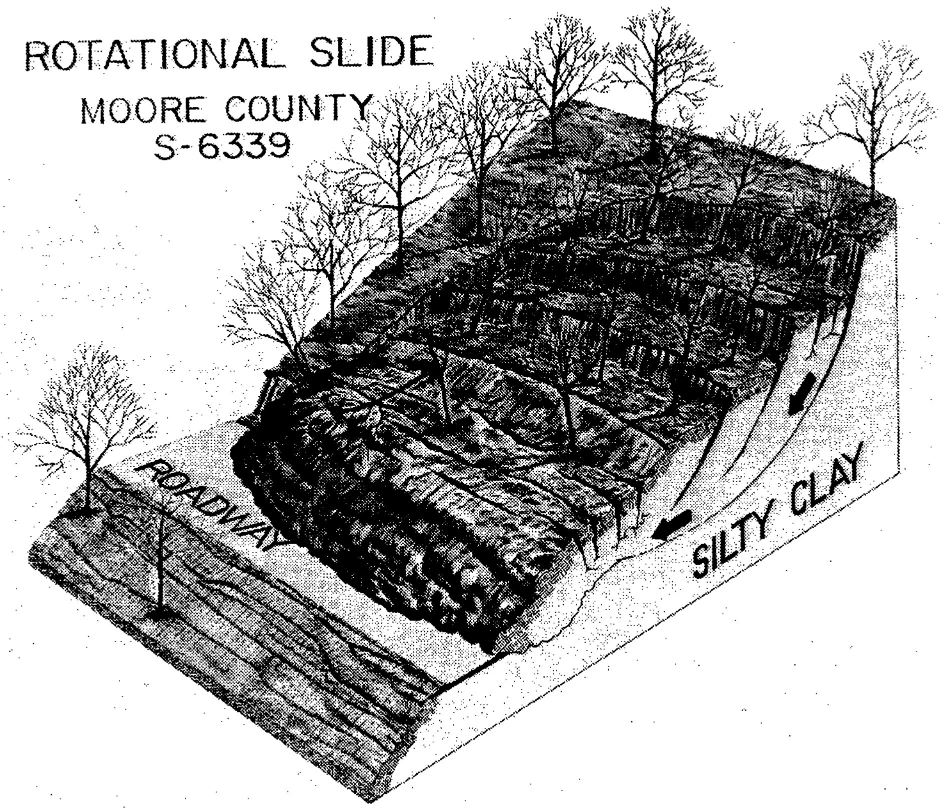


Figure 7.

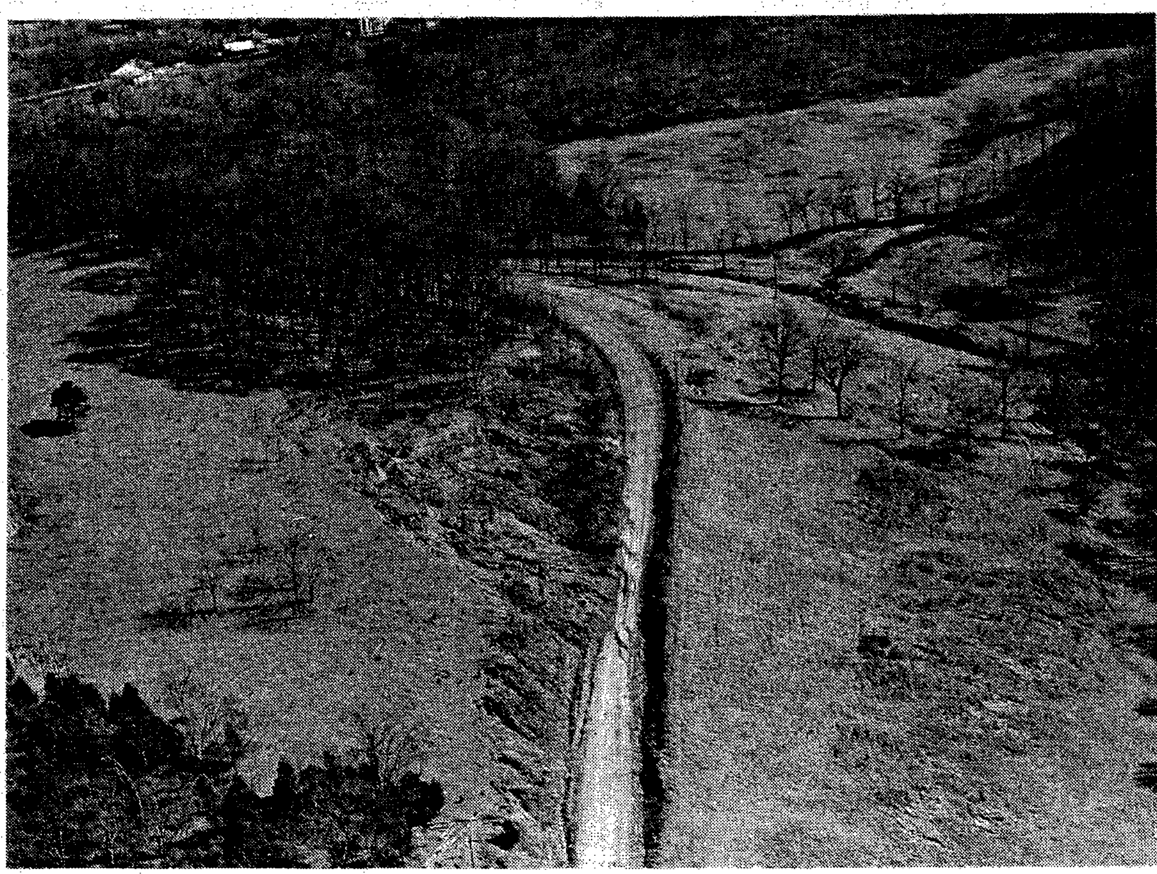


Figure 8. This failure began as a rotational slide but had deteriorated to a slump-flow by the time this photograph was taken.

generally extremely slow and are apparently more or less steady in time, although few data are available."

All of the significant flow-type failures experienced along Tennessee's highways, and the ones that will be described in this paper, are composed of unconsolidated materials. Two types of flows are discussed: one, which may be termed as "sheet flow" and the other as "channel flow." Two of the three examples of sheet flow used in this paper (Figures 9-12) developed on steep-sided slopes where the overburden was relatively thin (1 m to 3 m) and where much of the vegetation had been removed during construction. Most of the movement in both cases occurred during or following heavy and prolonged periods of precipitation, which means that liquification of the materials resulted more from direct surface infiltration than subsurface migration; although groundwater seepage was a contributing factor in the S.R. 53, Clay County failure (Figures 9 and 10). The other example of sheet flow began as a series of small slumps in a relatively deep cut (19 m) in cherty clay residuum. Failure was the result of a gradual oversteepening of the slope through 15 to 20 years of maintenance. The oversteepening occurred due to periodic excavations of the lower slope and ditchline following the development of small slump-flows along the entire 244-meter length of the cut.

In the two examples of channel flow, one involved colluvium flowing in a pre-existing trough (Figure 13) and the other

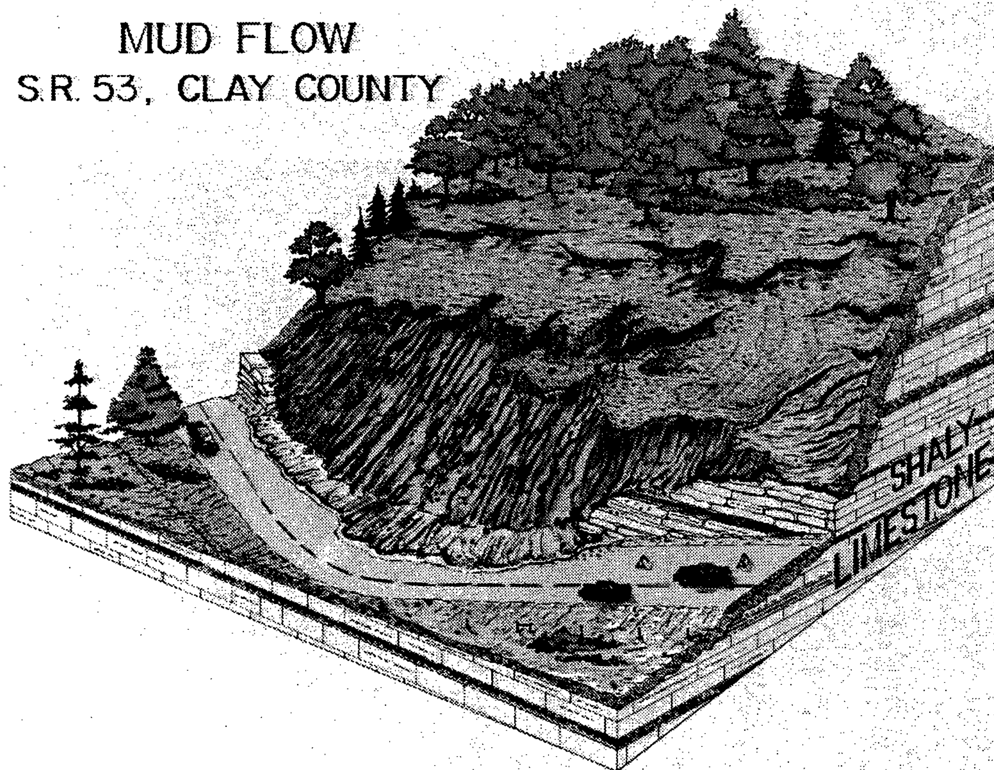


Figure 9.

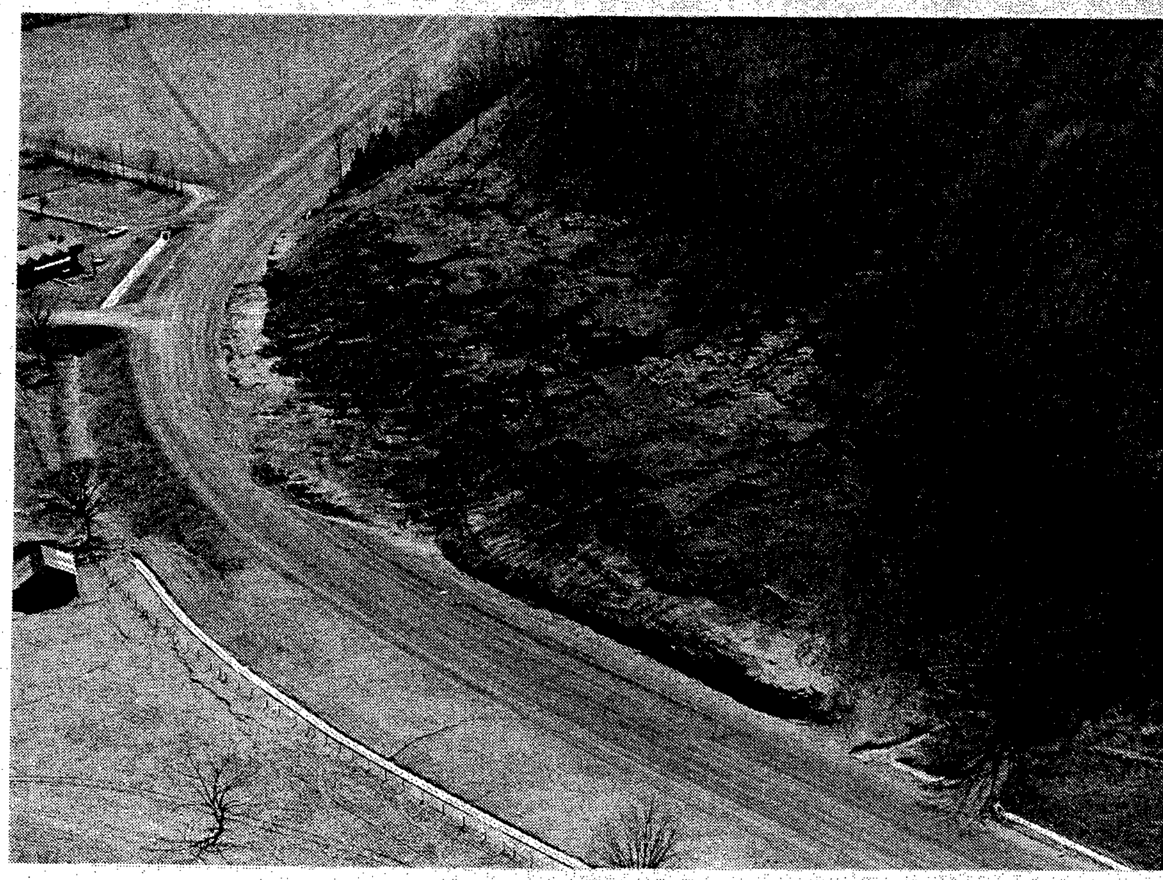


Figure 10. This failure occurred when 9 inches of rain fell during a 24-hour period in March 1975.

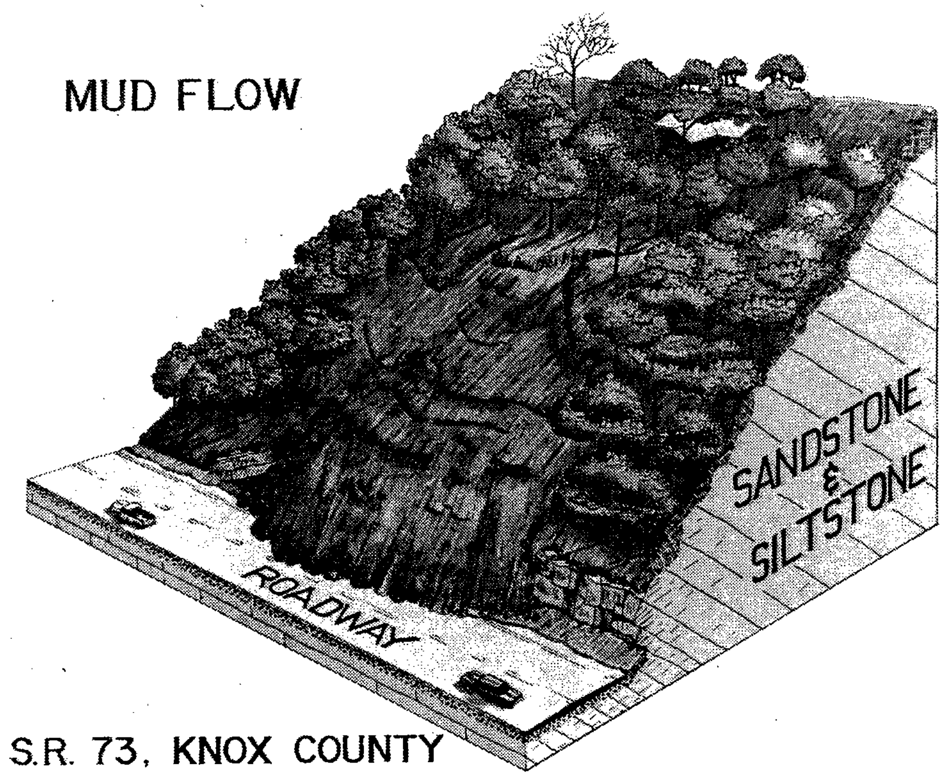


Figure 11.

Figure 12. An aerial view of the mud flow depicted in Figure 11 as it appeared in January 1974.

involved residual soil that failed originally as a rotational slide, and then retreated upslope as a translational slide while at the same time advancing downslope as a flow that developed as a result of increased surface water infiltration (Figures 14 and 15).

As indicated earlier, flows are probably the most difficult of all slope movements to analyze and correct. The analysis difficulty is related to the fact that, when active, they are virtually impossible to drill, sample, and instrument, which means that assumptions, observations, and experience must be quite heavily relied on in establishing failure surfaces, strength parameters, and other factors.

Relocation, excavation, and drainage are the three most widely used techniques in dealing with flow movements. Restraining structures are rarely used except in the case of small failures. Excavation may be difficult simply because of accessibility. Drainage, one of the most effective methods, must nearly always be installed during the dry season when all movement has essentially stopped.

"In falls, a mass of any size is detached from a steep slope or cliff, along a surface on which little or no shear displacement takes place, and descends mostly through air by free fall, leaping, bounding, or rolling" (Varnes, 1978). This category includes rock falls, debris falls, and even earth falls, such as the loess falls

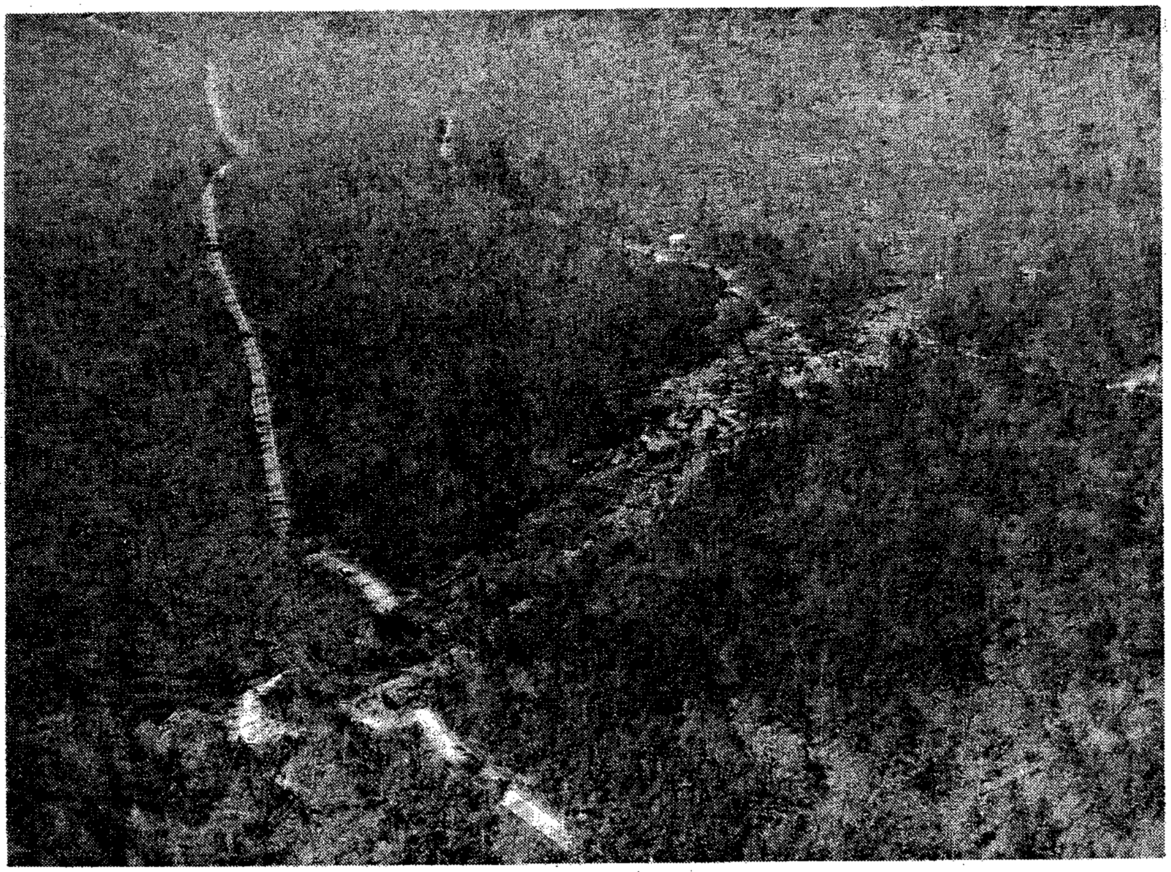


Figure 13. This failure, which completely severed Secondary Route-4412 in Marion County, involved colluvium flowing in a pre-existing trough in highly weathered shale (see Figure 18 for the repair measure).

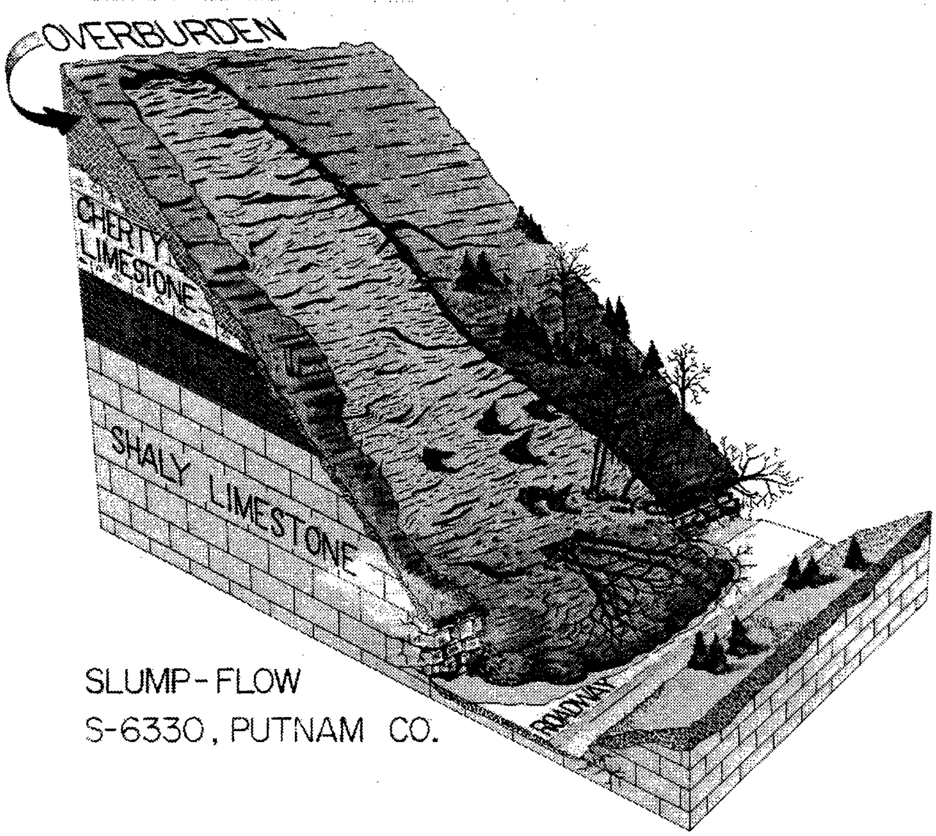


Figure 14.

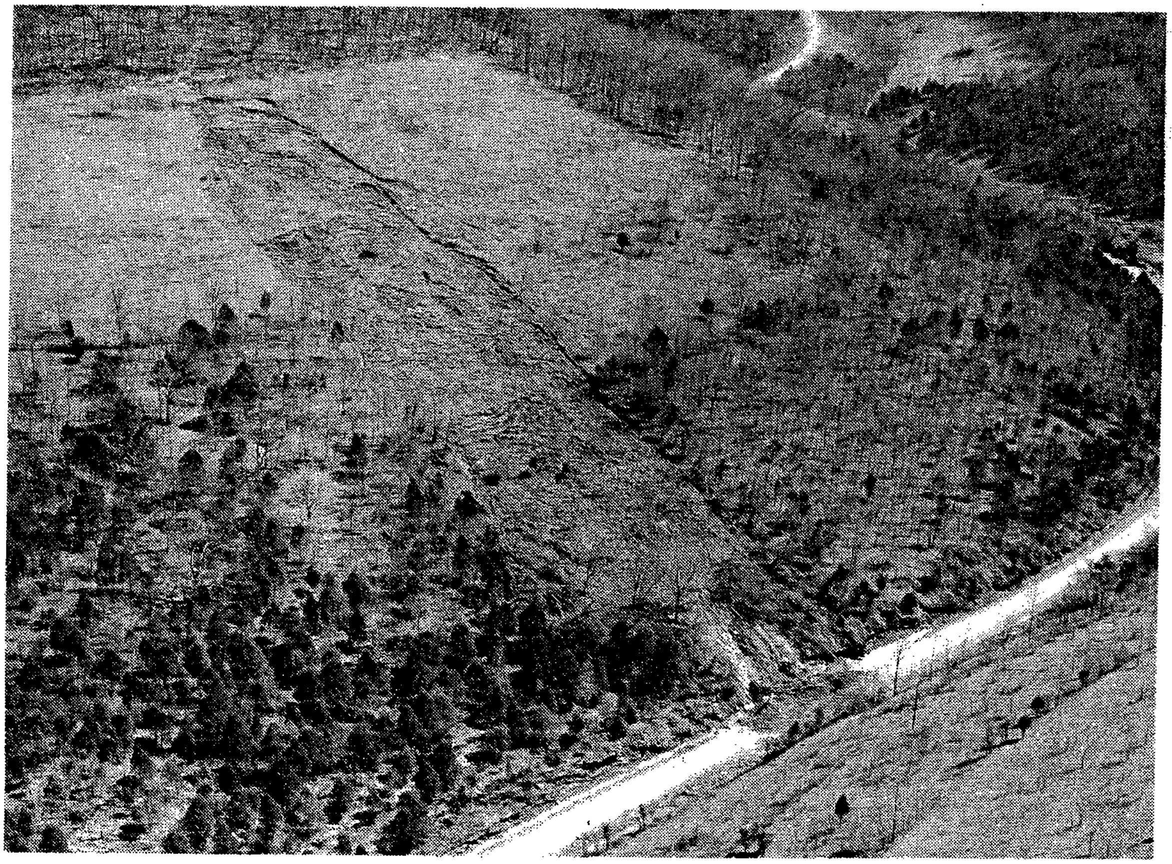


Figure 15. Aerial view of the slide depicted in Figure 14 as it appeared in March 1975.

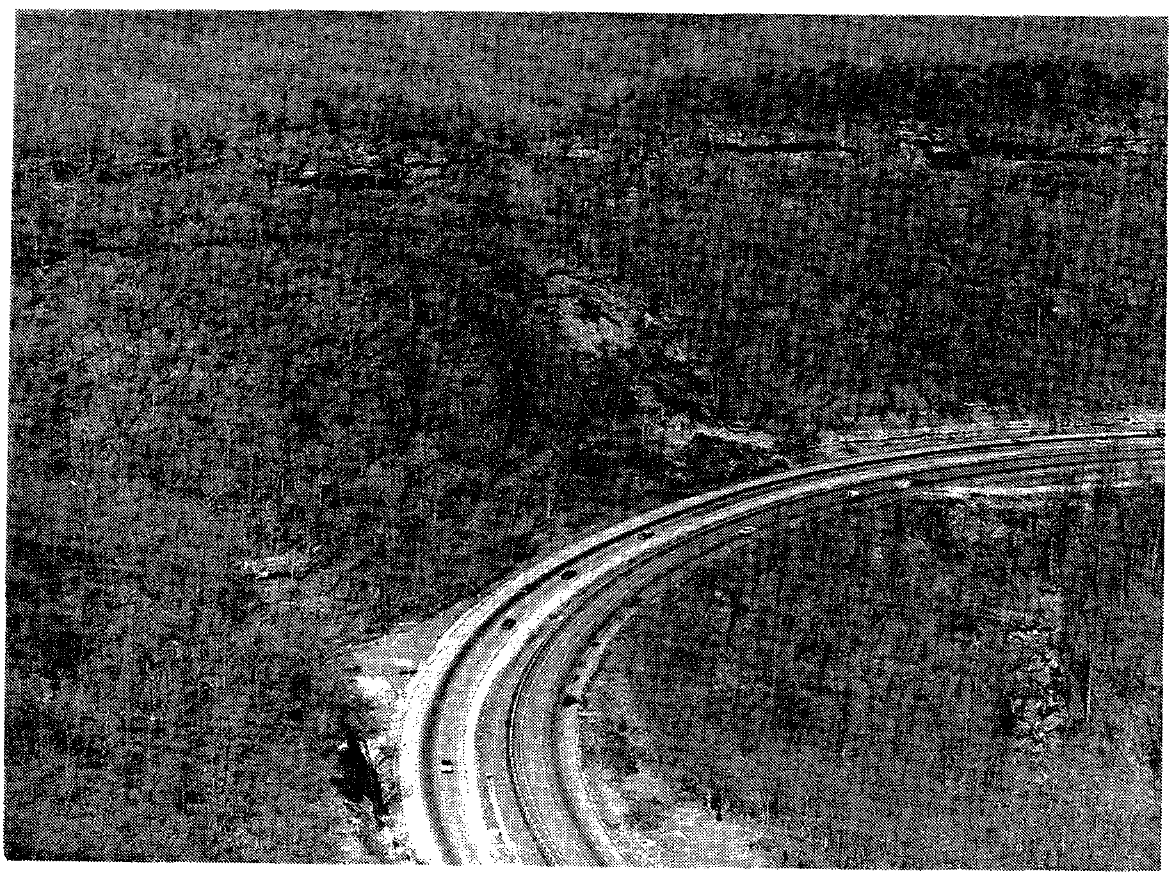


Figure 16. A translational slide in colluvium along Interstate-24 near Monteagle in Grundy County.

described by Varnes that occur along the bluffs of the lower Mississippi River valley.

The only falls that will be dealt with in this paper are rock falls where newly detached masses of rock have fallen from roadway cuts.

Remedial Measures

Terzaghi (1950) has written that "if a slope has started to move, the means for stopping movement must be adapted to the processes which started the slide." It follows, then, that since the processes that started the failure are related to the type of failure that develops, the corrective measure must also be adapted to the type of failure, as well as the type of material involved in the failure.

Remedial measures may be divided into four major categories: drainage, removal, restraint, and relocation. Even those rarely used and somewhat exotic methods described by Hutchinson (1977), such as grouting, chemical stabilization, freezing, heating, vegetating, blasting, and electro-osmosis relate one way or the other to either one or a combination of these four categories. Vegetating, or the planting of trees, for example, may serve as a form of drainage by removing moisture through evapo-transpiration. Likewise, grouting may be viewed as a restraint measure, as well as a means to alter or control drainage. Even berms, stabilizing fills, and toe counterweights, though viewed by some as a separate and distinct category, are also considered a form of restraint.

Just as there are a number of available remedial measures, there are also a number of levels of effectiveness and levels of acceptability that may be applied in the use of these measures. We may have a slide, for example, that we simply choose to live with; one that poses no significant hazard to the public, but one also that requires periodic maintenance, through removal, due to occasional encroachment onto the shoulder or roadway. Another may involve a portion of the outside lane along a sidehill fill that slumps or settles during the rainy season, but which can also be lived with by periodic patching. Still another may pose a somewhat greater threat, and while total correction cannot be justified because of budgetary constraints, it can also be lived with if it can be essentially stable 95% of the time by the installation of some type of dewatering system.

There may be still other slides that do not require correction. One such slide occurred along Interstate-24, just north of Monteagle in the spring of 1974 (Figure 16). It involved a thin layer (1 m-3 m) of colluvium moving over shale and interbedded limestone. The slide, a combination translational-flow failure, occurred during a heavy rainstorm and extended upslope for about 75 meters. The foot of the slide bottomed-out on a narrow limestone ledge with some of the material flowing over the ledge, toeing out near the roadway ditchline. The foot, in effect, was supported by the limestone, and since the slide material consisted mainly of coarse debris, the infiltrating surface water which triggered the failure flowed on through virtually unimpeded. Thus, the slide became essentially stable as soon as the stresses were relieved and the pore pressure that had built up quite suddenly at the colluvium-inplace material interface had dissipated. The slide has remained stable and virtually unchanged since that time.

Most slope movements, however, must eventually be dealt with. How they are handled, as implied previously, depend on the type movement, the processes that precipitated the movement, the kinds of materials involved, the location of the slide, the place or thing affected by or situation created as a result of the slide, and available resources. Also important is the time of year in which the slide occurs. Most slides in Tennessee occur during the spring months, following the spring thaw and when precipitation is most abundant. This is also the time of year, unfortunately when the slide areas are the least accessible. Depending on the situation, most corrective measures, once underway, should be carried through to completion with as little delay as possible. Unless absolutely necessary, they should never extend beyond one construction season. In Tennessee, the ideal time for slide repair is between about June 15th and October 15th. This is the period when precipitation is usually at a minimum, the groundwater tables are at their lowest, and there are more hours of daylight for optimum drying conditions, as well as for carrying out the work itself. In planning and scheduling remedial treatments, therefore, these factors must be considered along with all the others in determining the best approach to the problem.

Hutchinson (1977) has indicated that drainage is the principal measure used in the repair of landslides, with removal, or as he expresses it, "the modification of the slope profile by cutting and filling," the second most used method. These are also generally the least costly of the four major categories. It has been the experience of the writer, however, that while one remedial measure may be dominant, most slide repairs involve the use of a combination of two or more of the major categories. For example, while restraint may be the principal measure used to correct a particular slide, drainage and removal, to some degree and by necessity, are also utilized. Similarly, where removal is the dominant measure, drainage may also be included along with some type of erosion control, i.e., sodding, reforestation, etc.

Flows are probably the most difficult type of slope movement to analyze and control. As a result, removal and relocation are the most widely used remedial measures. Removal (Figure 17) was the measure applied in the failure depicted in Figures 9 and 10. Had the overburden deepened or thickened upslope beyond the treeline, some type of restraint structure would probably have also been needed. Had it been needed, it would probably have been founded on the wide rock ledge about two-thirds of the way up the slope. Failure occurred on March 13-14, 1975 when more than 9 inches of rain fell during an approximate 24-hour period. Removal was accomplished in July and August, during which time the total rainfall was only about 4 inches. This is a departure from normal of about 4 inches for those two months for this area, which made for even more ideal construction conditions than would ordinarily be expected. This is a good example of the point made previously that time-of-year must be considered when planning and designing landslide remedial measures. Vegetation has now been re-established on much of the slope, and it is not nearly as barren at present as it appeared in August 1975 when the photograph (Figure 17) was taken.

Another failure that occurred during this time (March 12-14, 1975) was the "channel flow" depicted in Figure 13. This slide actually began several years before as a translational slide and involved colluvium moving over weathered shale in a trough-like depression. It is typical of the kinds of slides that develop along the escarpment areas of the Cumberland Plateau (Royster 1973). Since the slide was along a low-volume road (140 ADT in 1975), and since it could be kept open with a reasonable amount of maintenance, a major repair effort could not be justified. The more than 6 inches of rain that fell in the area during this period, however, resulted in the development of a flow failure that severed the road completely. A field investigation revealed that the colluvium ran too deep and covered too wide an area to be controlled by a buttress or a similar restraint structure. Removal was ruled out because of the large volume of material involved in the slide mass itself, and because of the tendency for these kinds of failures in this type of environment to greatly enlarge during excavation. Drainage was not considered to be an acceptable alternative because of the virtual impossibility of preventing further heavy surface infiltration, and the equally impossible likelihood of being able to drain the subsurface with horizontal or trench drains. Relocation, as it turned out, was the only reasonable alternative. It involved realigning the roadway for a distance of 0.56 km (0.35 miles) around the toe of the slide (Figure 18). In situations of this type extreme care must be exercised in relocating the roadway because of the danger of triggering another slide on the opposite side of the valley. In other words, the location as well as the design must be precisely tailored to fit the soil and geologic conditions in the area. This was done in this case and to date no additional problems have developed. The cost of relocation, excluding right-of-way and design, amounted to slightly more than $264,000.00.

The slump-flow shown in Figures 14 and 15 also occurred during the heavy rains of March 12th through the 14th. The rainfall for this period, measured at a nearby station (Cordell

Figure 17. Total removal was the measure used to correct the mud flow shown in Figures 9 and 10.

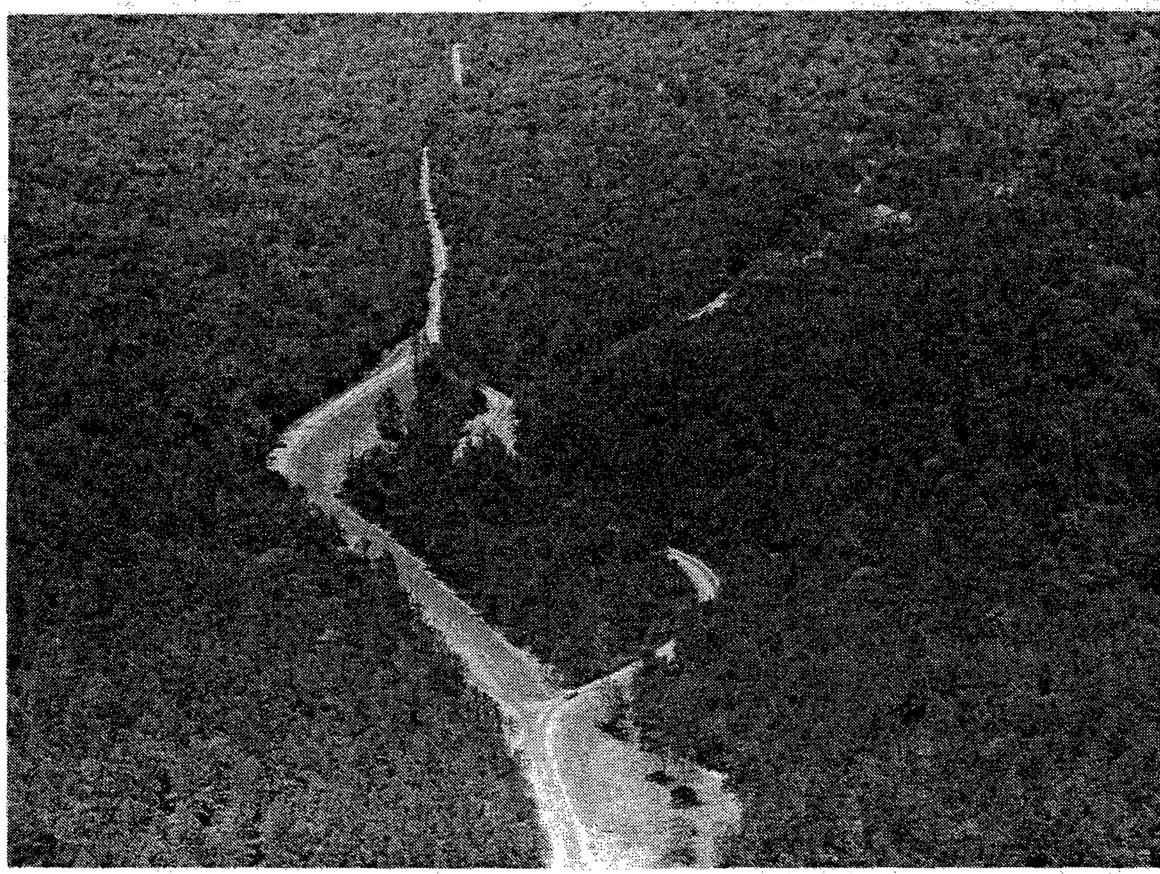


Figure 18. Relocation was the means used to correct the problem shown in Figure 13. The new alignment is at the left.

Hull Lock-Dam), amounted to 7.3 inches. As can be seen or inferred from the illustrations, the length of the slide (168 m (550') is somewhat out of proportion relative to the width (53 m (175') and depth (2 m-5 m (8'-15') for this type of terrain and geologic conditions. The movement actually began as a slump sometime after the lower part of the hillside had been cleared of vegetation for agricultural or logging purposes. It then rather steadily but slowly reverted to a translational slide as it retreated upslope before developing into a flow, principally due to the heavy March rains. It advanced over the limestone outcrop along the lower slope and onto the roadway.

Three remedial methods were explored: restraint with a rock buttress, total removal, and relocation. Restraint was ruled out because it would have involved the placement of approximately 12,000 to 15,000 cu/meters (16,000-20,000 cu/yds.) of buttress rock along an inplace rock ledge above the roadway. This involved an accessibility problem, and there was the possibility that slide enlargement would result in material flowing over the buttress. Total removal was rejected because it would have meant finding a place to dispose of the more than 38,000 cu/meters (50,000 cu/yds.) of slide material, plus whatever additional material would have resulted from enlargement. Furthermore, since this failure was not the result of roadway construction or maintenance, there was some reluctance to encroach on private property due to possible future liability resulting from slide enlargement. Similar to the case with the Marion County slide, relocation was the selected alternative. It involved realigning the roadway a distance of .38 km (.236 miles) along a raised gradient around the toe of the slide (Figure 19). The repair measure also involved re-routing a small stream through a culvert along the old roadbed beneath the toe of the slide and providing for an overflow outlet in case of flooding along the stream. The cost of the project, again excluding design and a minimum amount of additional right-of-way, amounted to about $200,000.00.

The mud flow shown in Figures 11 and 12 is one of the few flow type failures in which restraint was used as the principal repair measure. This particular failure had plagued the Department of Transportation from 1969 until it was finally repaired in 1974. It involved the periodic liquification of a cohesionless to slightly plastic soil. Failure resulted when the soils along the lower slopes liquefied during periods of heavy rainfall and began to flow. This loss of support in the lower slope resulted in upslope slumping. As the rain continued, the slumped materials in the upper slope also liquefied and they too began to flow. As is usually the case with cohesionless materials, the slope regained a relatively high degree of stability soon after the rain stopped. However, the loss of vegetation and the creation of new slumped areas resulted in the cycle beginning again with each subsequent

Figure 19. Relocation and re-routing of surface drainage were the methods used in the correction of this slide in Putnam County (see Figures 14 and 15).

Figure 20. Partial removal and a revetment-like buttress were utilized in the repair of the mud flow shown in Figures 11 and 12.

rainfall. The slope was repaired with a revetment-like rock buttress at a cost of $176,000.00. The buttress in this case serves more as slope protection than as restraint structure (Figure 20).

The other example of sheet flow, which was mentioned previously, involved, not only flow, but erosion and slumping in a 19-meter (62') deep cut in cherty clay, a residual soil of the Knox Dolomite. As indicated before, the failure developed as a result of long term slope deterioration where there was a gradual oversteepening of the slope due to erosion, shallow flows, and minor slumping. The remedial measure involved removal, slope flattening, sodding, and drainage (Figure 21). A slope ratio of 2:1 was utilized and a 6-meter (20') wide bench was constructed at the midpoint in the slope to reduce erosion. The cost of construction amounted to $190,000.00.

Another failure that began as a slump and then deteriorated to a flow affected a section of Interstate-24 near Beech Grove in Bedford County. It had been "lived with" for a number of years by periodically removing slumped material from the lower slopes and keeping the ditches clean. By the spring of 1976, however, it became obvious that a more positive solution would be necessary (Figure 22). The failure involved movement of residual clay and silt through a large solution channel in the limestone that crops out at roadway level. Such channels are locally referred to as

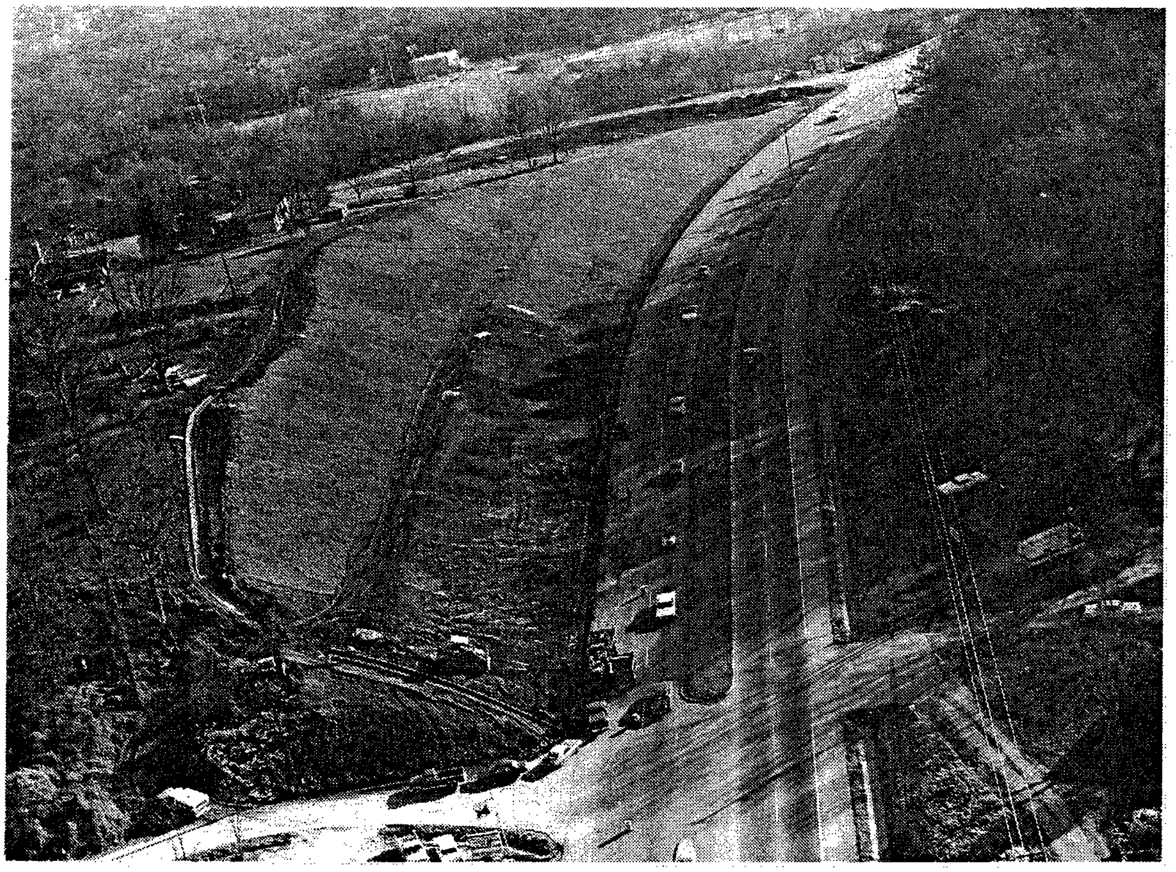


Figure 21. Slope flattening, benching, surface drainage, and sodding were used in the correction of a failure along State Route 33 in Knox County.

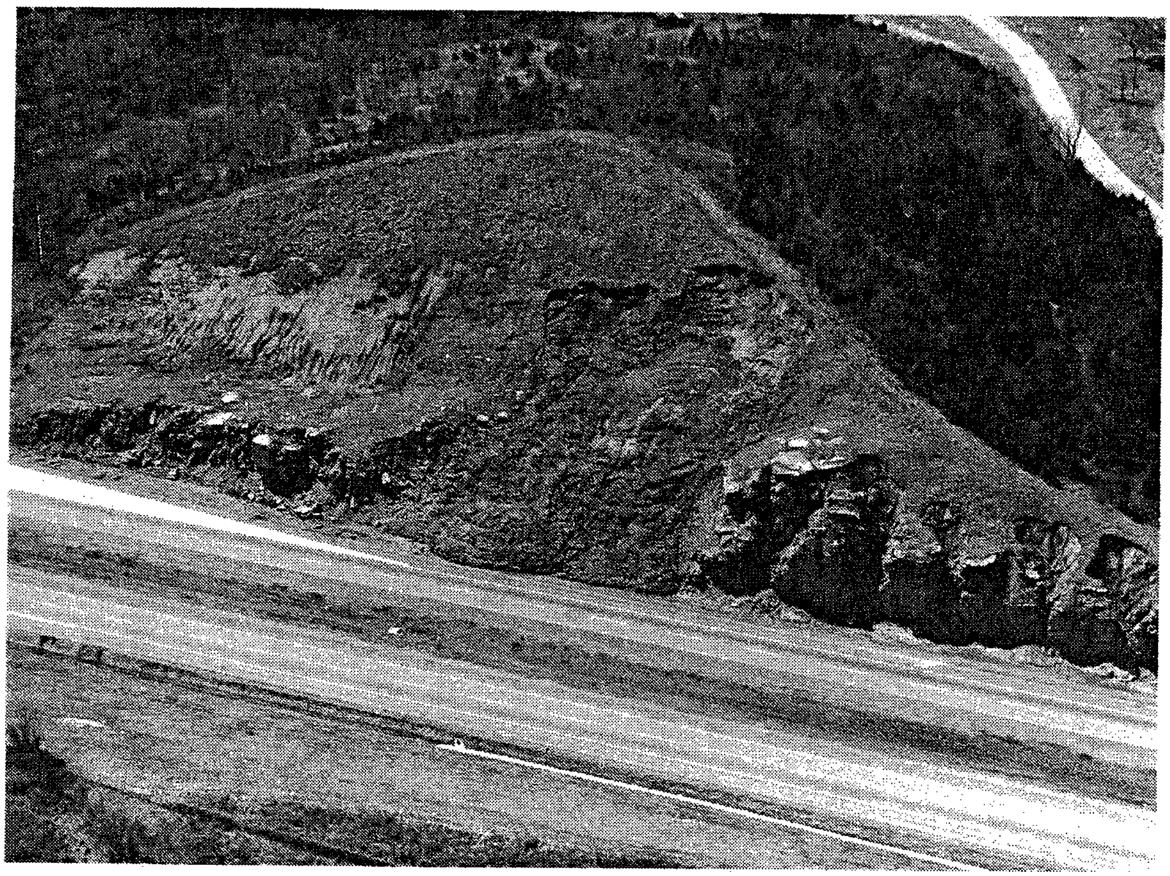


Figure 22. This slide developed in an enlarged soil-filled solution channel ("mud cutter") along Interstate-24 near Beech Grove.

Figure 23. Removal and restraint with a rock buttress were the corrective measures utilized (see Figure 22).

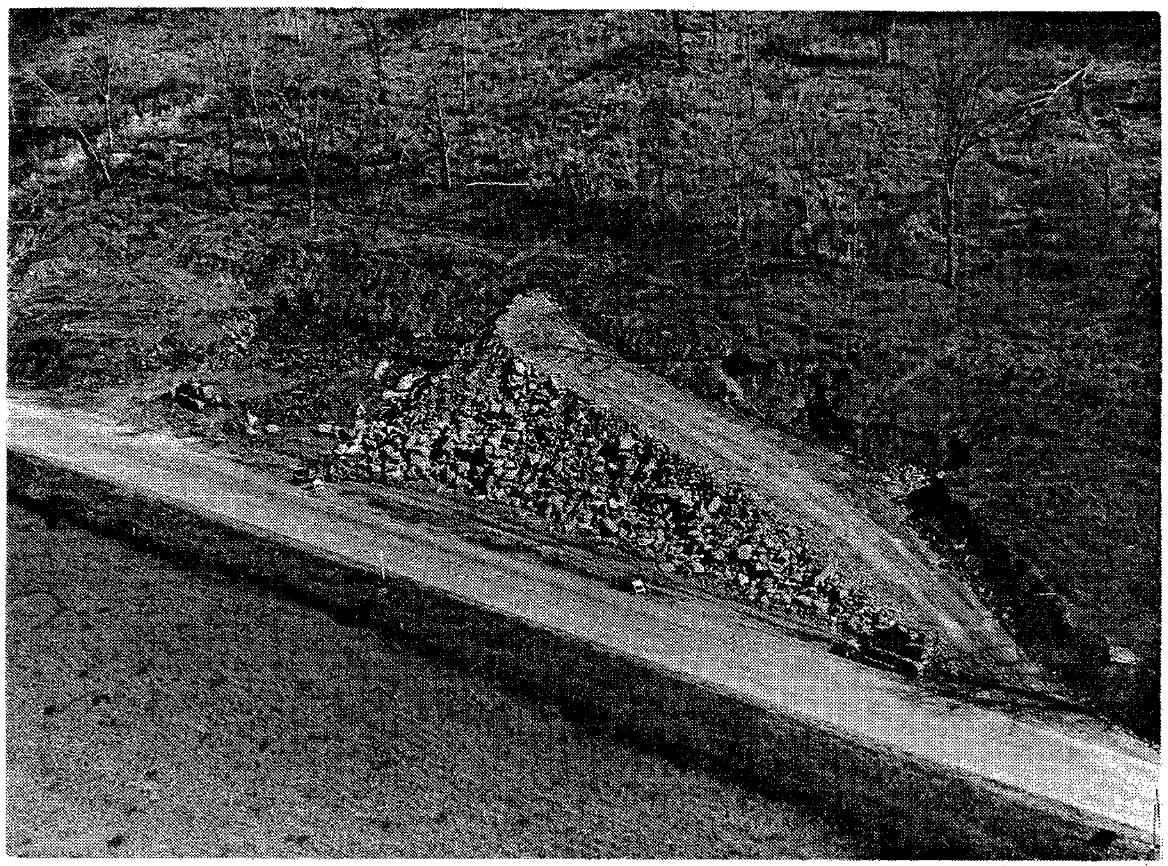


Figure 24. Some slide repairs should be made in segments, beginning at one flank and extending to the other.

"mud cutters." They frequently develop along faults, joints, and fractures in carbonate rocks, and they can be especially troublesome when they are overlain by weaker rocks, such as shales, or thick accumulations of overburden. The repair measure involved both removal and restraint. Total removal was not feasible due to the large amount of material that would have to be wasted (57,000+ cu/meters (75,000 cu/yds.), and the fact that some enlargement would definitely have occurred. Drainage was ruled out because of the impossibility of controlling surface infiltration and the likelihood that subsurface drainage in such fine-grained soils would be ineffective.

The final design called for the removal of approximately 14,000 cu/meters (18,000 cu/yds.) of the failed material, and then buttressing the remainder with nearly 20,000 cu/meters (26,000 cu/yds.) of nondegradable buttress stone. The specification called for the buttress stone to have a maximum size of .9 meters (3 feet) in the longest dimension. At least 50% of the material by volume was to have been uniformly distributed between .3 and .9 meters (1'-3'), with no greater than 10% (by volume) passing the 51 mm (2") sieve. It was also to be totally devoid of soil and shale materials.

The project was let to contract in September 1976, being completed in approximately three months thereafter at a cost of $165,000.00. Except for minor slumping in the slope above the buttress and some deterioration of adjacent "mud cutters," no additional problems have developed (Figure 23).

Since rock buttresses are commonly used in the repair of landslides in Tennessee, a few comments regarding their construction are needed. First of all, it is important that the aforementioned gradation be met. Experience has shown that this gradation, i.e., a maximum size of .9 meters (3 feet) in the longest dimension, with at least 50% (by volume) of the material uniformly distributed between .3 and .9 meters (1'-3'), and no greater than 10% (by volume) passing the 51 mm (2") sieve, is needed to insure an angle of friction (ϕ) of $40^{\circ} \pm$ and to also insure that the buttress will be essentially free-draining. The larger pieces of rock should be roughly equidemensional; that is, slabby and elongate pieces should not be accepted. Furthermore, the material must be non-degradable. For example, limestone that contains shale partings, or sandstone that is friable or crumbly, should not be accepted. To insure an acceptable gradation, the rock should be sieved, but since this is expensive and not always practical, a good approximation of the gradation can be obtained with the use of a loader equipped with a slotted bucket. Though production may be somewhat reduced, most of the "fines" will be shaken through the slots in the loading process.

Wear and tear of equipment, of course, is a factor in handling material of this size. The chassis, beds, and suspension systems of on-road trucks are especially affected, as well as the buckets and hydraulic systems of loaders. This can be a major problem for the small, inexperienced contractor, especially if he has cut his profit margin to the bare minimum in the bidding process. This factor is mentioned here because once the contractor becomes aware of the problems in handling this material, he may be forced to make adjustments in his operation that may adversely affect the quality of the buttress stone. And since the gradation of the stone is more or less visually controlled, the problem of specification enforcement may become even more difficult.

Where a more aesthetically pleasing slope is desired, the buttress can be covered with top soil and sodded or seeded. The lower portion, however, should be left uncovered to insure that drainage may move through freely. Filter cloth should be placed between the buttress material and top soil to prevent contamination and silting of the buttress stone. The area below the buttress should also be graded to drain. Water that is permitted to collect or pond around the toe area may soften the foundation to the point of failure. Pines, hemlocks, or other evergreen trees or shrubs may be planted to screen that portion of the buttress left uncovered.

It is also important in the repair of landslides to know where to begin; hence, the construction sequence and procedures must be pre-planned. Too many repair measures are developed without giving this factor due consideration. Frequently, the designer leaves many of the decisions regarding sequence and procedure to the contractor, when, in many cases, these should be spelled out in the plans and then coordinated in the field by the engineering geologist or geotechnical engineer responsible for the design concept.

Some slides should be worked from the crown; others from the toe; and still others from the flanks. How they are worked depends, again, on the type of slide, the kinds of materials involved, topographic configuration, potential for enlargement, etc. Translational-flows that are to be controlled with a rock buttress, for example, frequently have to be worked in sections, beginning at one flank, or both flanks simultaneously, and extended across the toe or foot area. Construction is carried out in, say, 15 meter (50 foot) sections with the buttress rock being placed in the first section as excavation of the next proceeds. This procedure reduces the likelihood of lateral, as well as headward migration (Figure 24).

A translational-slump-flow along State Route 61 in Anderson County was corrected by using a combination of removal and restraint with a small toe buttress. The failure originally developed in construction as a result of undercutting relict bedding which dipped into the roadway at an angle of about 27°. Stability was temporarily achieved by flattening the slope to $1\frac{1}{2}$:1 and establishing a good vegetative cover. Over the years, however, further deterioration developed in the form of slumps and flows which worsened with each significant rainfall (Figure 25). More than 5 inches fell in this area on March 12-13, 1975 and produced flows that temporarily closed the road and forced the development of a permanent repair measure. The failure was repaired by removing 53,500 cu/meters (70,000 cu/yds.) of material and supporting the toe area with 11,500 cu/meters (15,000 cu/yds.) of buttress stone (Figure 26). The backslope was flattened to a 2:1, or approximately that of the dip of the relict bedding and, to prevent or minimize lateral migration, the flank slopes were cut on a 3:1 ratio. Except for some minor erosion prior to the establishment of a vegetative cover, the area has remained stable. Repair costs amounted to $219,226.00.

Slope movements in the form of translational slides and rock falls have been an almost constant problem along a 15 km (9-mile) section of Interstate-40 in Cocke County from the time construction began in 1962 through the present. The alignment travels along the Pigeon River Gorge through a somewhat complex assemblage of Cambrian and Precambrian sediments and metasediments. The slides are controlled for the most part by geologic structure such as that depicted in Figure 3.

One of the larger slides occurred near the Waterville Interchange in the spring of 1972 (Figure 27). It was actually the reactivation of a slide that had begun to develop in 1968 when

Figure 25. This failure, a translational-slump-flow, developed along relict bedding that dipped directly toward the roadway.

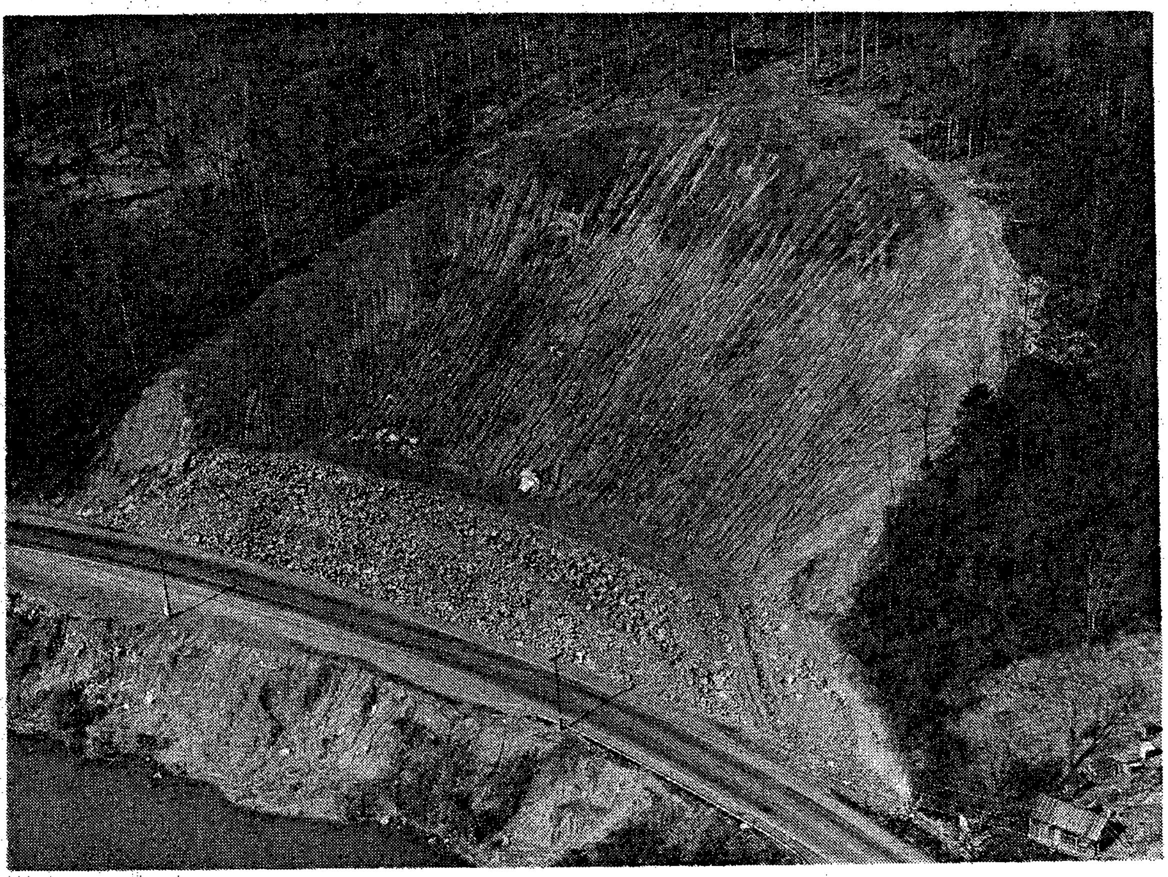


Figure 26. The repair measure utilized removal on a 2:1 slope and restraint with a toe buttress.

the river channel was widened in connection with the placement of slope pavement. The initial failure developed during excavation of the river channel when the toe of a wedge-shaped deposit of soil and weathered rock was undercut near the river level. Minor movements, resulting in a slow but perceptible migration of material into the river, continued from 1968 through the winter of 1971-72. Then, in the spring of 1972, heavy rains caused a large segment of the mass to slide into the river, constricting the channel and causing the stream to be diverted into the fill slope on the opposite side. This resulted in a lengthy section of the slope pavement being washed out. While this was being repaired in the fall of 1972, another flood occurred that caused further movement and the loss of most of the remaining slope pavement. In 1973, another contract was awarded to remove the slide, repair the damage to the fill slope, and to replace the slope pavement with boulder rip-rap (Figure 28). Restraint was not considered feasible because of the large quantity of material that would have had to be supported and because of the poor foundation conditions along the river's edge. Drainage was ruled out because of the infeasibility of controlling surface infiltration, as well as the improbability of removing enough water from the failure zone with horizontal drains to prevent further movement during periods of heavy rainfall. Approximately 124,000 cu/meters (162,000 cu/yds.) of slide material was removed at a cost of $2.88

Figure 27. Eastward view of the Waterville slide along Interstate-40 in Cocke County as it appeared in February 1973.

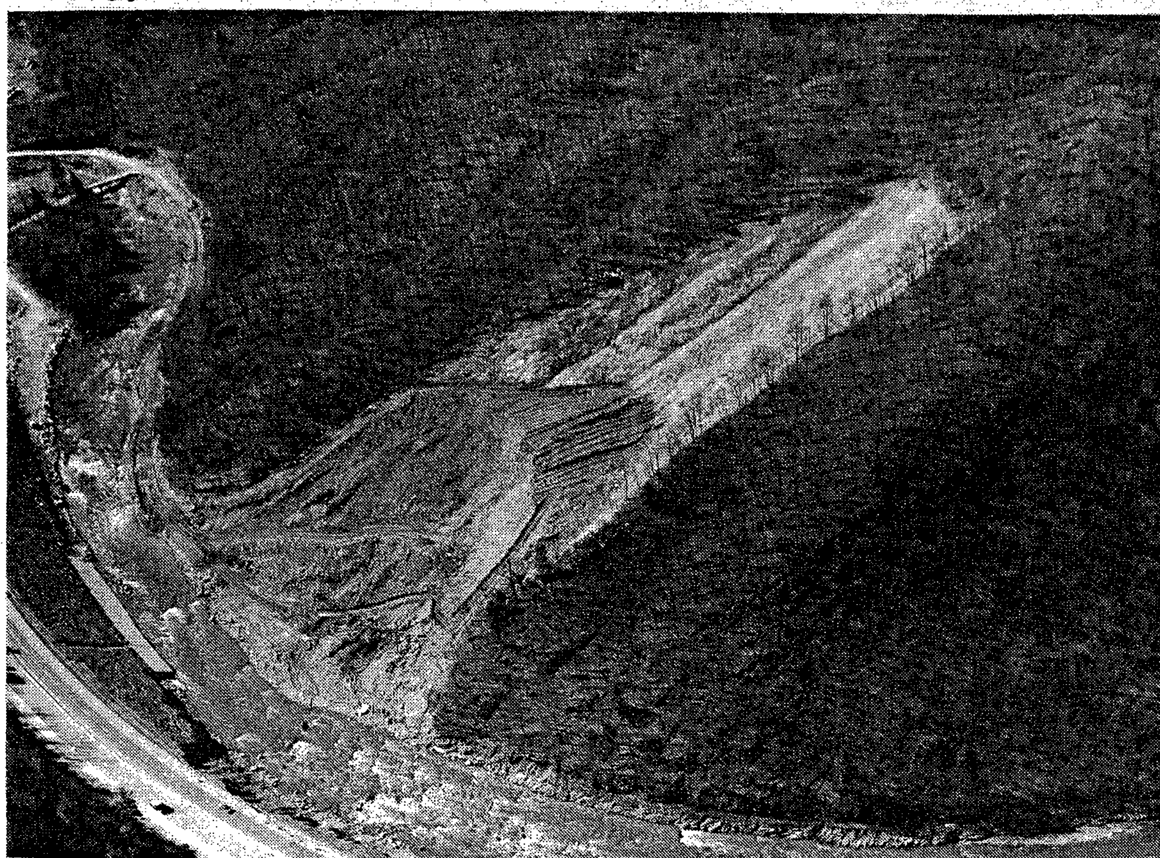


Figure 28. The remedial measure involved total removal. The project was approximately one-half complete when this photo was taken in February 1974.

per cu/meter ($2.20 per cu/yds.). The total project cost, including the buttress stone and boulder rip-rap used for fill slope protection, amounted to $2.25 million. No additional problems have developed since completion of the project in June 1974 (Figure 29).

Another slide in this area, and one that has been an almost continuous problem since 1964, is the "Hartford Slide." This slide was discussed in a paper presented by the writer at the 28th Annual Highway Geology Symposium in Rapid City, S.D. in August 1977 and subsequently published in the proceedings of that meeting. It will not be discussed further in this paper, however, because there is presently litigation pending regarding certain construction problems encountered in its repair. It is only mentioned here because it is one of the few examples of a major translational slide in which drainage will be utilized as the principal repair measure (Figures 30 and 31).

Translational slides similar to the one shown in Figures 3 and 4 developed during the construction of Secondary Route-2422 in Sevier County near Gatlinburg in the winter of 1976-77 (Figure 32). The slides resulted from undercutting steeply dipping (45^{o}+), highly weathered beds of Precambrian metasiltstone. The repair measure was somewhat unique in that the slide debris was actually used to raise the gradient of the roadway 3 m to 4.5 m (10'-15'), thereby serving as a restraint against movement of beds that had been exposed along the original gradient. In addition, a trench

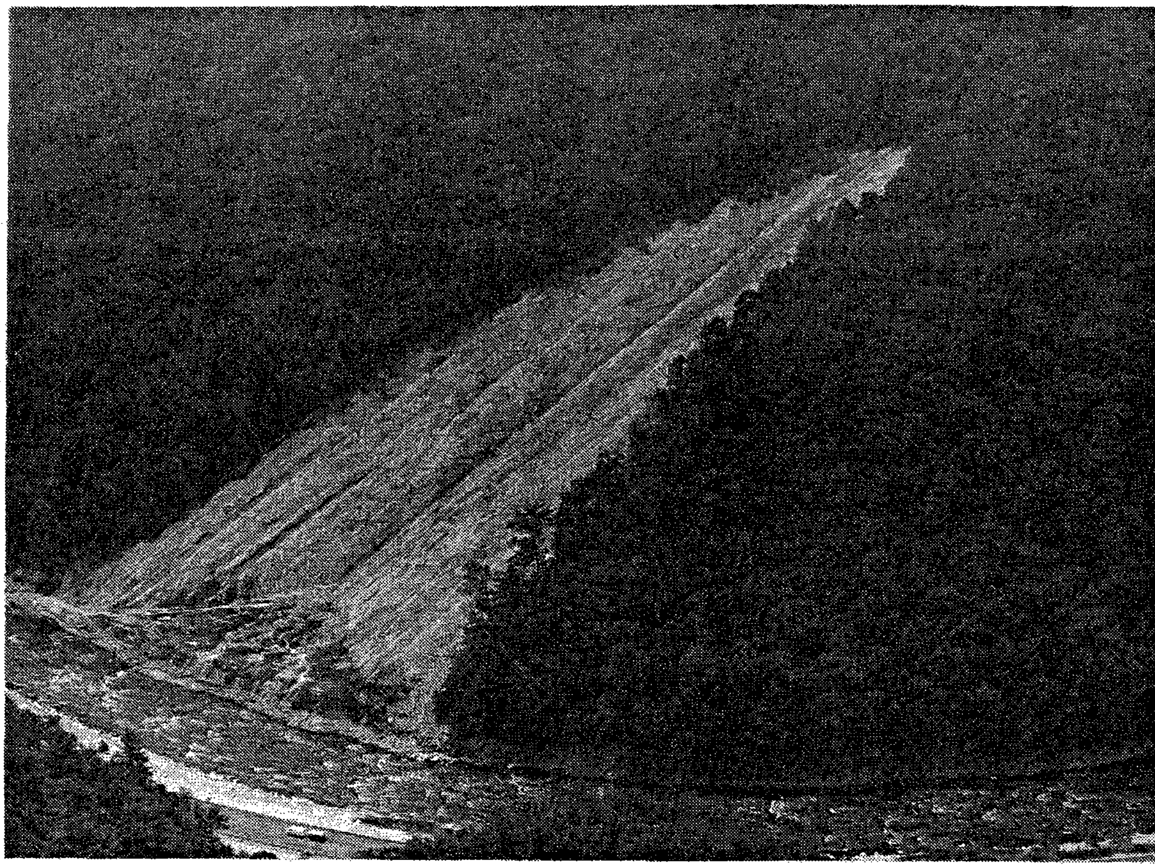


Figure 29. The Waterville slide as seen in June 1975. The view is toward the west. Note that the slope pavement has been replaced by boulder rip-rap.

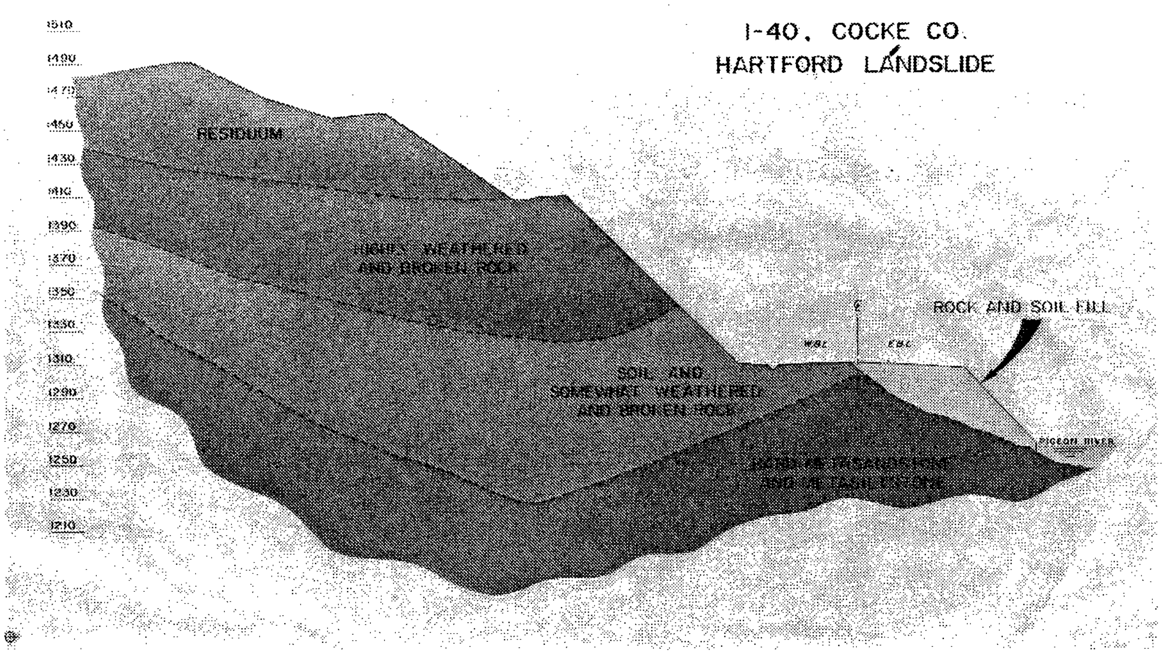


Figure 30. This translational slide near the Hartford Interchange along Interstate-40 involved soil and weathered and broken rock moving over essentially unweathered metasediments.

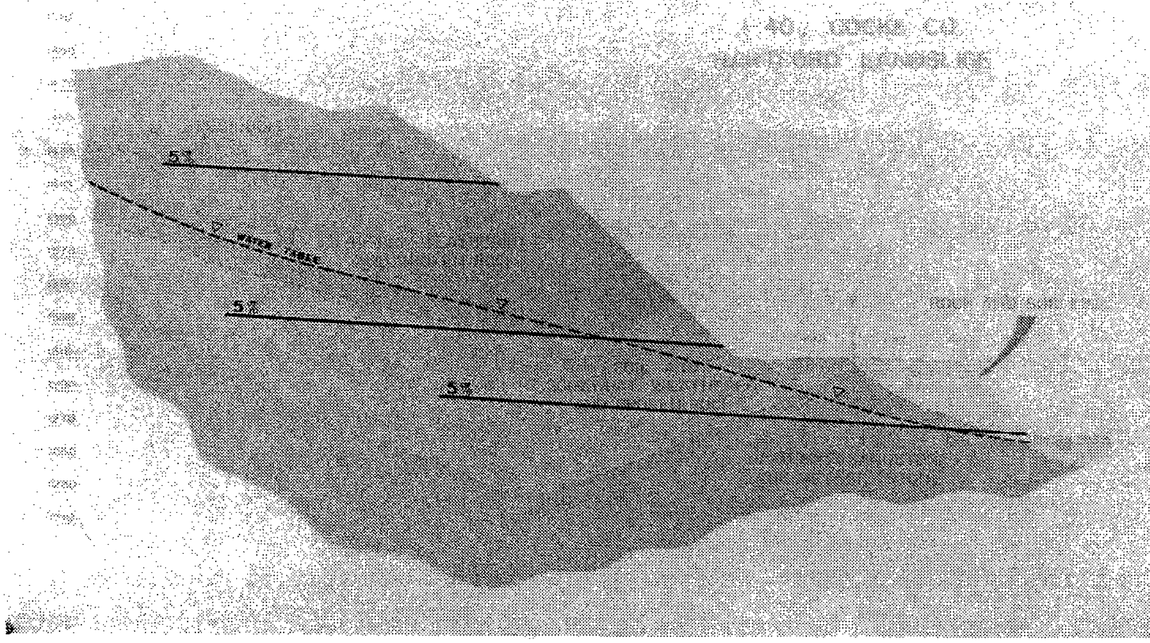


Figure 31. Schematic showing the drainage levels planned for the Hartford slide.

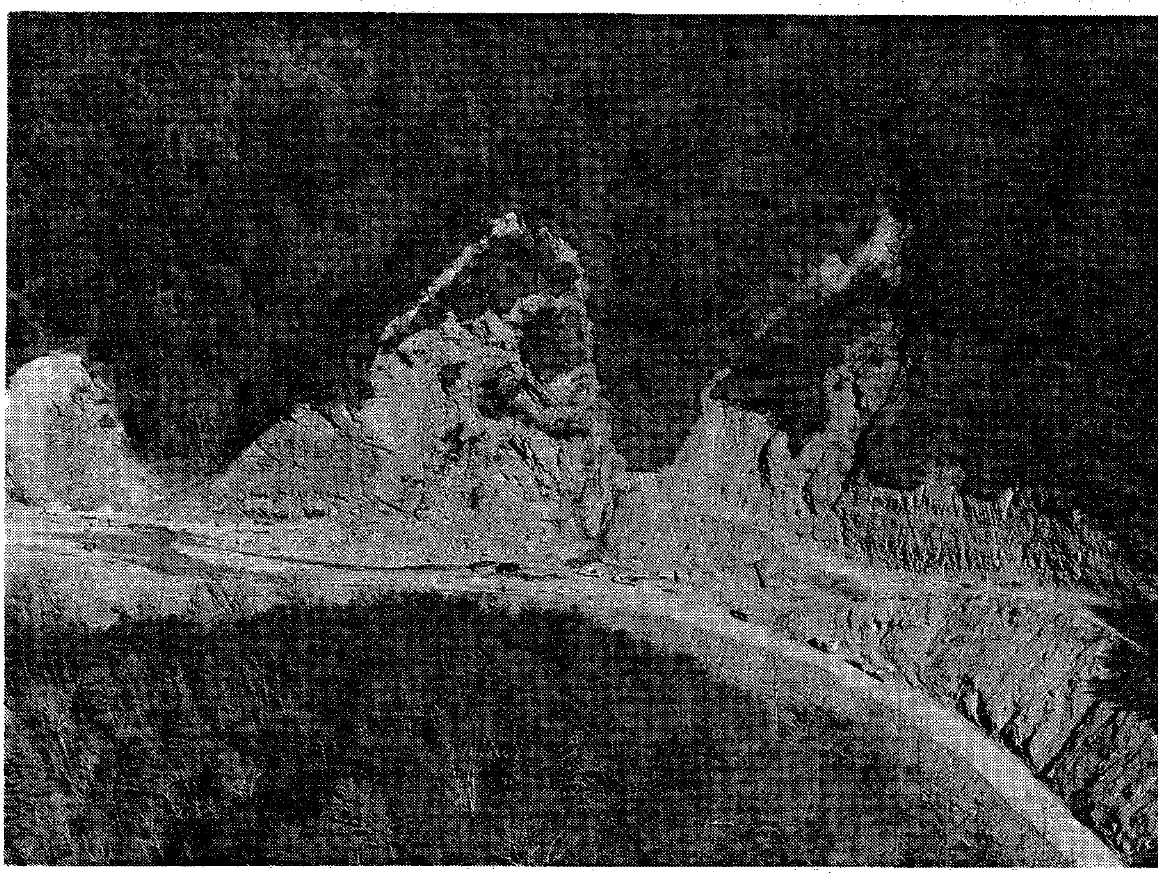


Figure 32. Translational slides (wedge failures) developed during construction of Secondary Route-2422 in Sevier County.

drain was installed that extended to a depth just below the original ditchline. It was designed to collect and remove water migrating along the bedding planes of the cut slope, which otherwise would have been blocked by the added fill. The revised design also resulted in an increase in the width of the fallout area from 4.5 m 7.5 m (15' to 25') (Figure 32).

A translational slide along State Route 56 near Beersheba Springs in Grundy County was corrected by supporting the area above the main scarp with an H-pile and timber restraint structure. The slide involved colluvium and fill material moving over weathered shale along a 46-meter (150') section of roadway supported partially by inplace sandstone and partially by fill. The main scarp displaced the roadway from about the middle of the outside lane, coinciding in its downward projection with the weathered shale-colluvium contact (Figure 34). The roadway was kept open for many years by periodically filling in the slumped area with "shot rock" and capping off with a bituminous plant mix, and since the traffic volume through the area was relatively low (1000± ADT in 1976), a major repair effort could not be justified. Such a repair, using reinforced earth, a gabion wall, or crib wall probably would have exceeded $200,000.00 because of the depth required to reach an adequate foundation. Relocation into the cut slope was not possible because of a number of buildings along the upper slope.

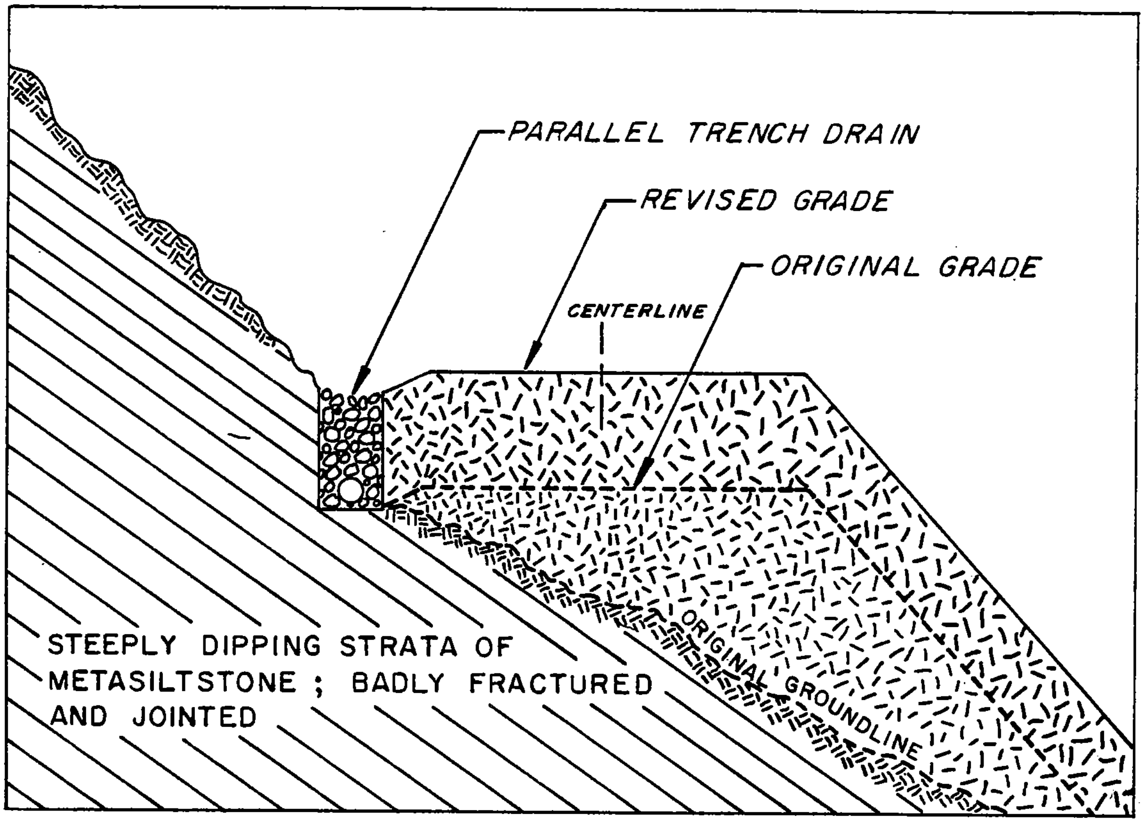


Figure 33. The repair measure for the slide shown in Figure 32 involved drainage and buttressing the slope by raising the gradient of the roadway.

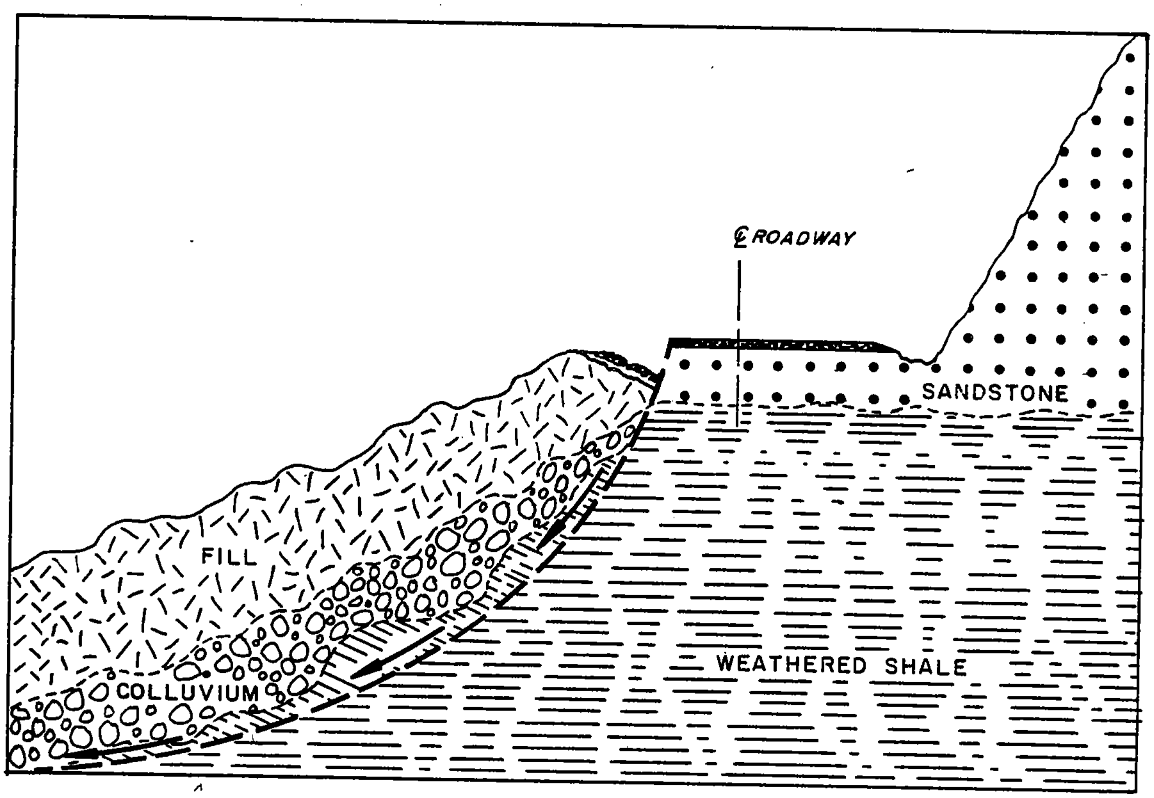


Figure 34. Schematic of the translational failure along State Route 56 in Grundy County.

It was determined that the most economical approach would be to support the fill portion of the roadway with the H-pile and timber structure shown in Figure 35. The 254 mm (10") piles were placed on 1.37 m (4.5') centers to a depth of 9 m (30') in 457 mm (18") drill holes and backfilled with concrete. The piles were placed in drilled holes rather than being driven to avoid shearing and fracturing the weathered shale and to insure a more uniform alignment. The 457 mm (18") sized holes were necessary to compensate for sidewall irregularities that might prevent a plumb seating of the piles. The 7.6 cm X 20.3 cm (3" X 8") treated timbers used for the horizontal support members were attached with nails driven into the boards and bent around the flanges of the piles. Further support against slippage of the boards was added by the weight of the granular backfill. A 38 mm (1.5") spacing was held between each timber to insure drainage of the backfill. The cost of the project, including forty-two 9 m (30') piles, concrete for backfill, 6,000 board feet of timber, 360 cu/meters (470 cu/yds.) of excavation, 360 cu/meters (470 cu/yds.) of granular backfill, repaving, etc., amounted to $79,500.00. In the nearly two years since completion, no additional problems have been observed.

The State Route 56 slide is an excellent example of the use of imagination and creativity to solve a vexing problem within the constraints of a very tight budget. The use of piling to correct landslides has never been looked upon with favor by most engineering geologists and geotechnical engineers (Gedney and Weber, 1978; Zaruba and Mencl, 1969; Baker and Marshall, 1958; and Root, 1958); nevertheless, there are situations where they can be used with good results. The key lies in being able to recognize those situations.

Figures 5 and 6 typify the problem of adverse dip in roadway excavations. This particular situation exists along most of the 5.49 km (3.39 mile) section of the new alignment of State Route 32 that traverses the south slope of Clinch Mountain in Grainger County. The project was let to contract in July 1976, and as of this writing (summer 1978), is still under construction. The Clinch Sandstone, which underlies the area, dips at an angle of about 22^{o} directly into the roadway along the entire alignment. In many places it contains thin beds of shale sandwiched between the .3 m-1 m (1'-3') thick strata of sandstone. It also contains minor faulting (Figure 36) and is moderately jointed, with joint sets running both parallel and normal to dip.

The adverse dip problem was recognized in the planning and location phases, but since there was no reasonable location alternative--that is, no area within the corridor where this condition did not exist--it was determined that the problem would have to be handled in the design and construction phases. To remove or minimize the threat of sidehill fill failures, the foundation areas were stripped of all overburden and the lower

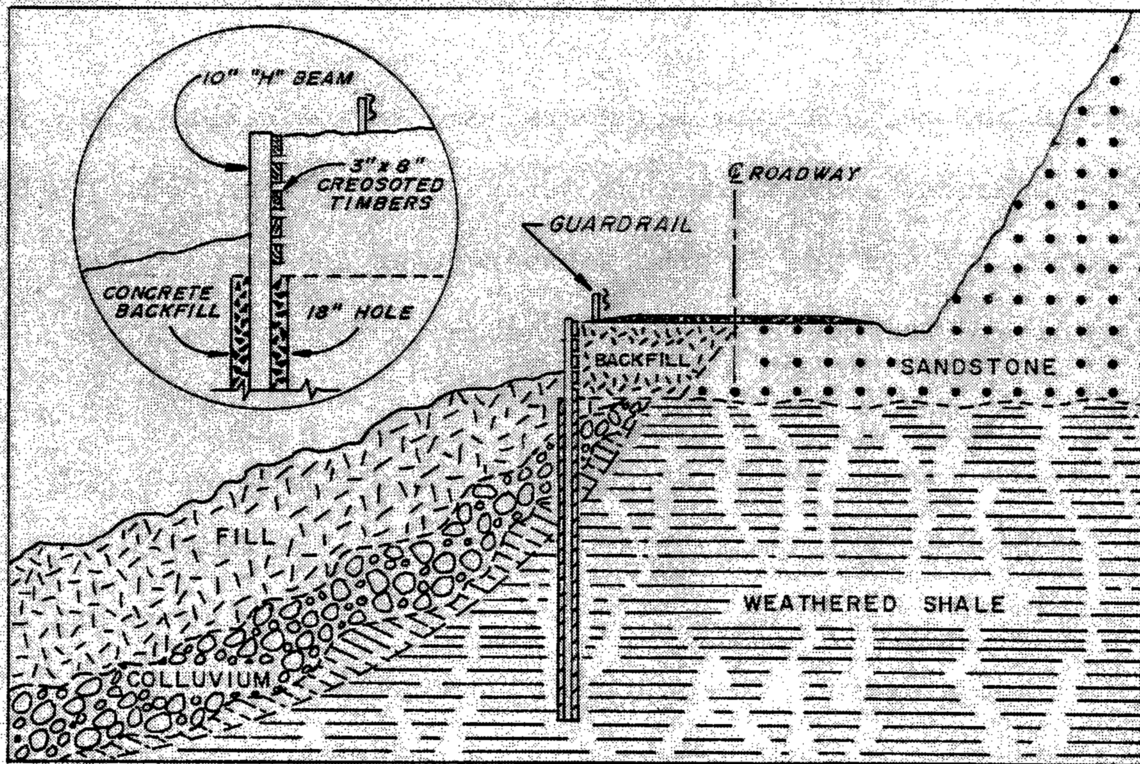


Figure 35. The repair measure involved restraint with a steel pile and timber structure.

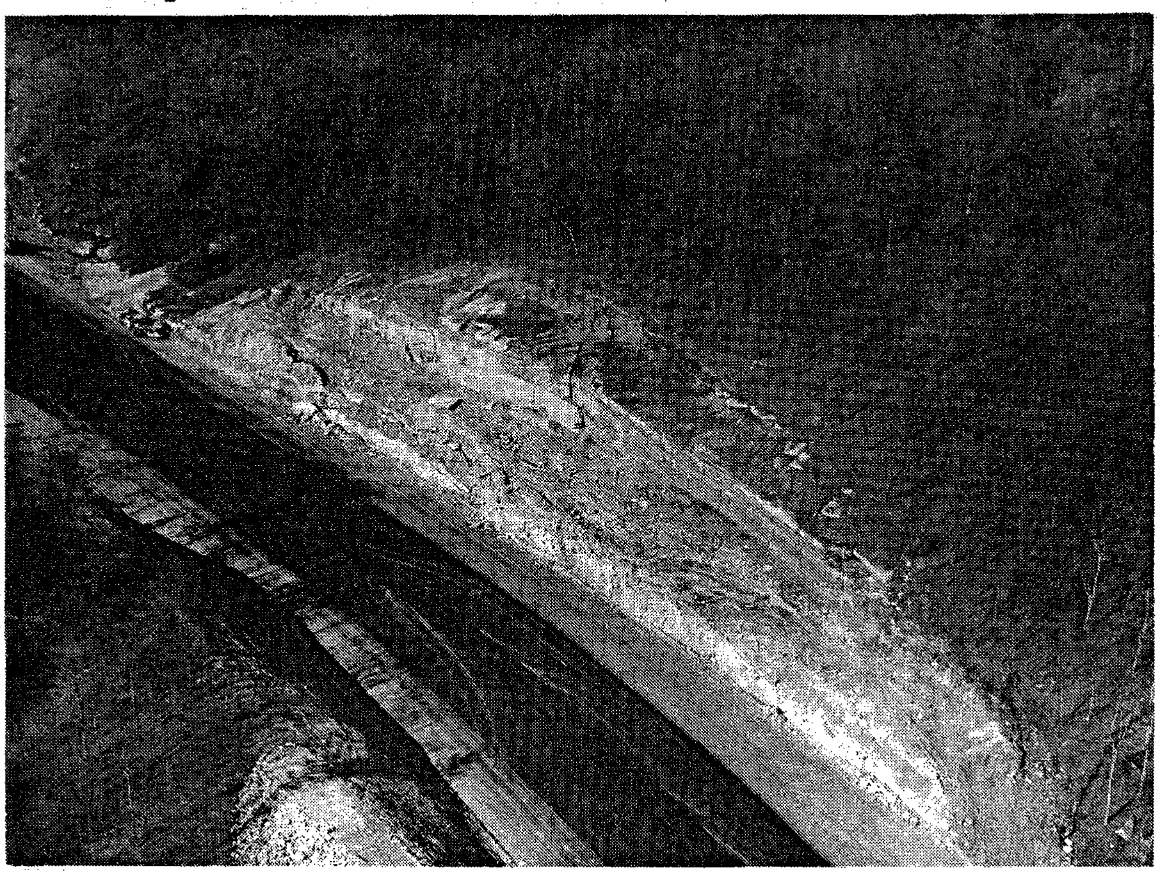


Figure 36. The beginning of failure of a cut slope traversed by a fault along State Route 32 in Grainger County.

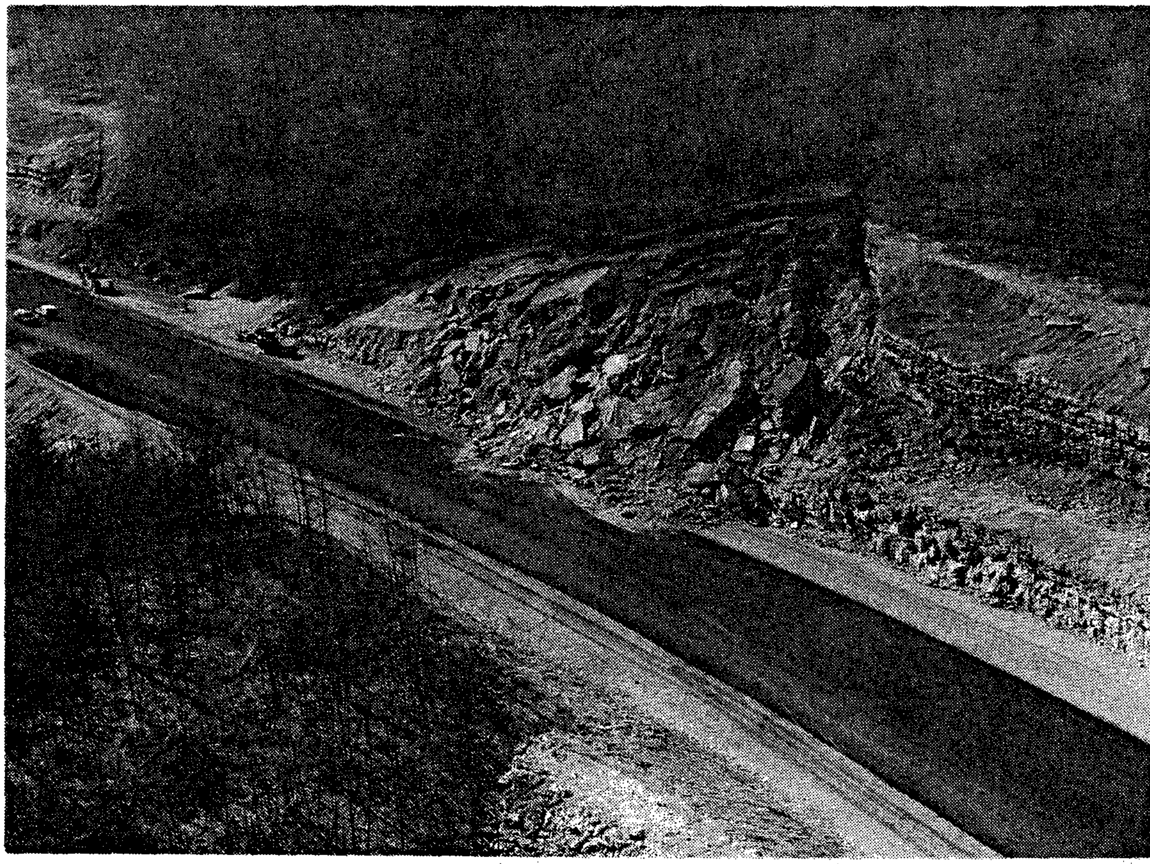


Figure 37. The same slope approximately two months later. Note that the main scarp coincides with the strike of the fault.

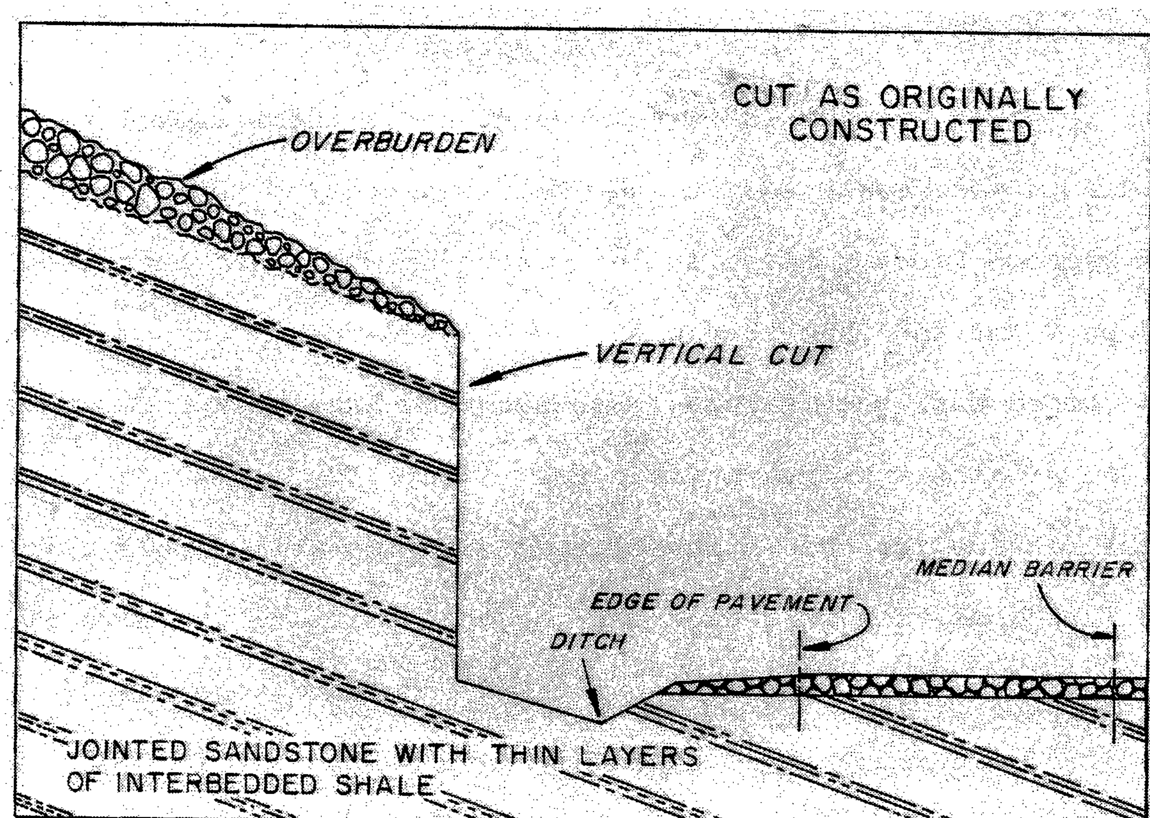


Figure 38.

portions of the fills constructed of nondegradable rock. In the cut sections consideration was given to the use of bolts, anchors, or dowells to increase shearing resistance along the inclined beds. Some would have been installed above the cutlines prior to excavating, with additional installations being made as needed after excavating. This was ruled out, however, because of the uncertainties concerning rock breakage during blasting. It was determined that the risk of loosening the bolts and dowels installed prior to blasting was too great for the expense involved. The final design called for those cuts of approximately 30 meters (100') to be pre-sheared vertically and to have 12-meter and 9-meter (40' and 30') benches, respectively, 6 meters and 18 meters (20' and 60') above grade. Those cuts 18 meters (60') and less in height required 6-meter (30') benches at about their midpoints. This design was developed with the full expectation that there would be some bedding plane failures, but it was hoped that they would be minor and that they could be handled during construction. As it turned out, the failures that occurred have been more troublesome and significant than anticipated (Figure 37). There are several reasons for this: first, some areas contained more thin shale interbeds than were expected; second, jointing was more of a factor than was anticipated; and third, the sandstone itself proved far more friable and susceptible to weathering in some places than it was predicted to be. But in retrospect, and in spite of this new information, the design probably would not have been changed to any great extent from what it was originally. There are simply few alternatives when there is an adverse dip situation such as this that extends for nearly 5.5 km (3.4 miles). Furthermore, the philosophy still prevails--right or wrong--that greater risks can be taken in these kinds of situations in rural areas because of the availability, where needed, of additonal rights-of-way.

Something that is being tried in the stabilization of some of these failures (Figures 37 and 38) is what might be termed a "shot-in-place" buttress (Figures 39 and 40). The idea is to "relax the slope" by breaking up the bedding planes along which sliding is taking place and allowing the "shot rock" to act as a buttress against further upslope movement. Apparently the initial results of this treatment are effective, but only time will provide the true test.

Rock buttresses are not only structures that have been used by the Tennessee Department of Transportation in the repair of translational slides. Gabion walls (Figure 42) and a Reinforced Earth Fill (Figure 43) were used with excellent results on colluvial slides that occurred along Interstate-40 near Rockwood (Royster 1973 and 1974). Concrete and metal crib walls have not been used by the Department in the repair of landslides, but they have been used with good results by other governmental agencies and several

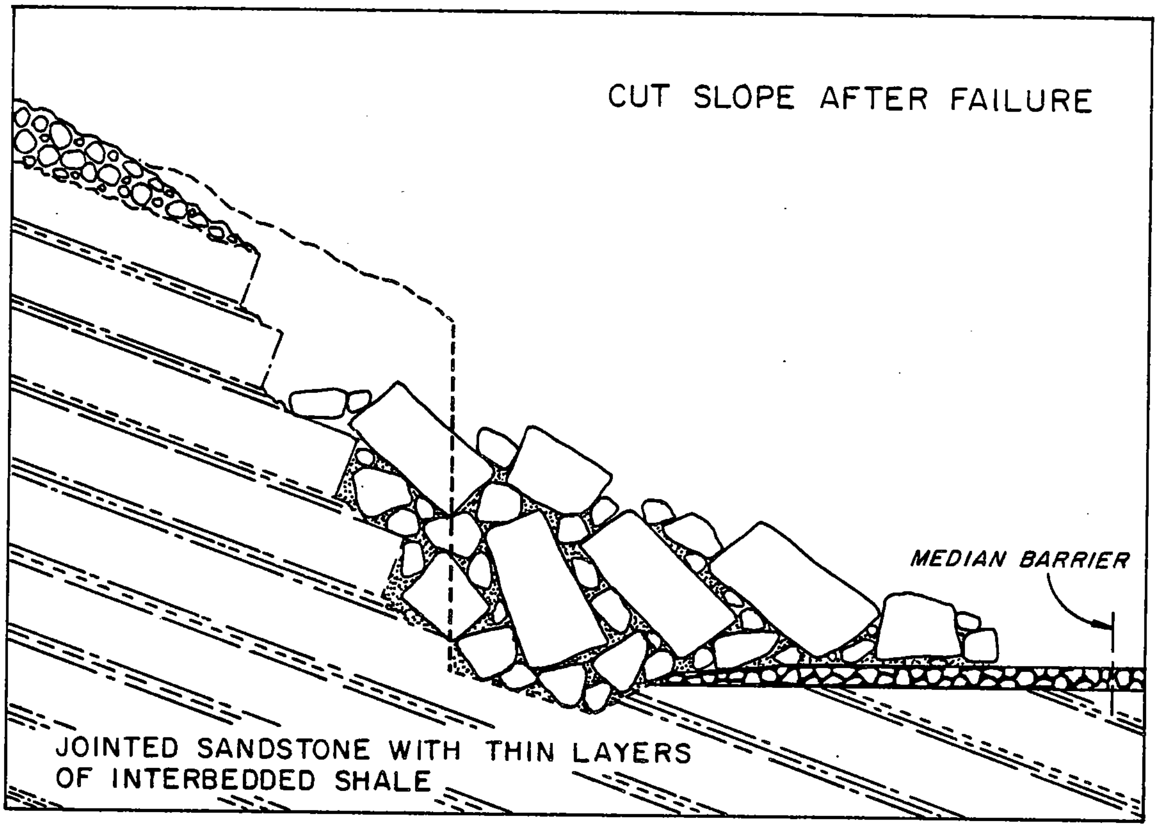


Figure 39.

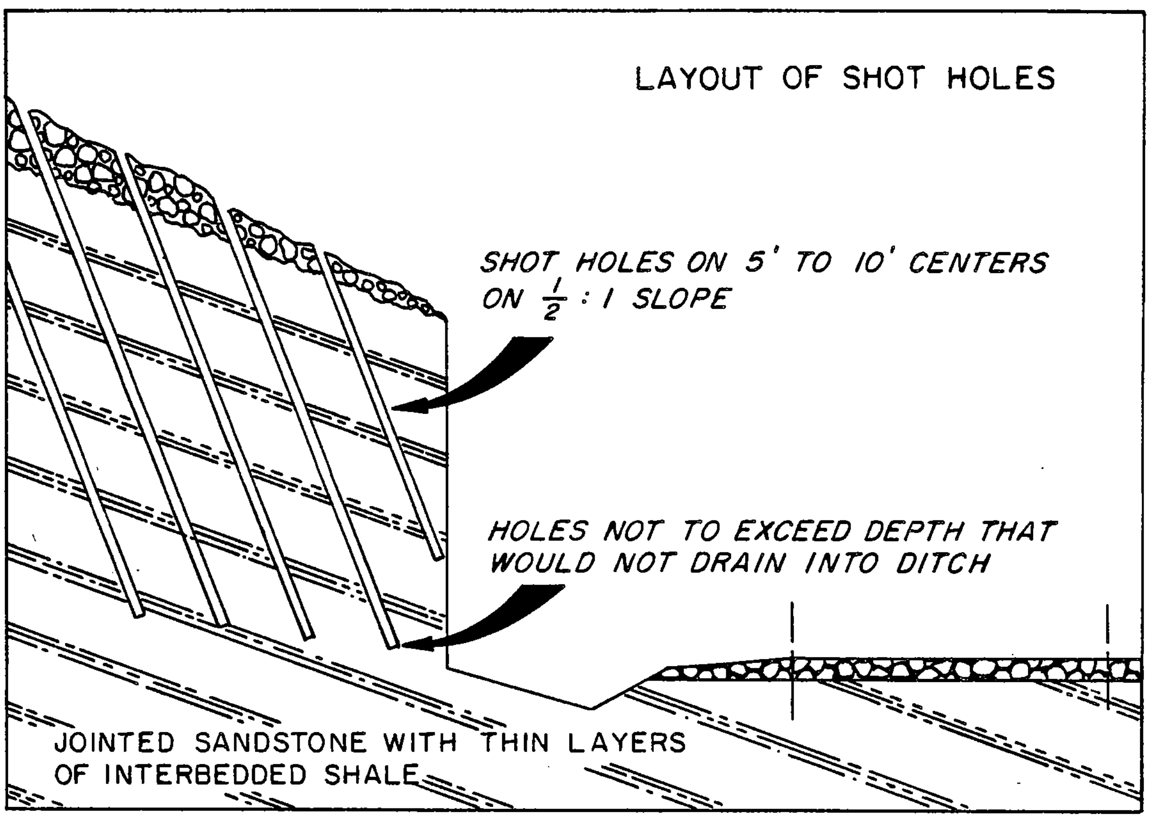


Figure 40.

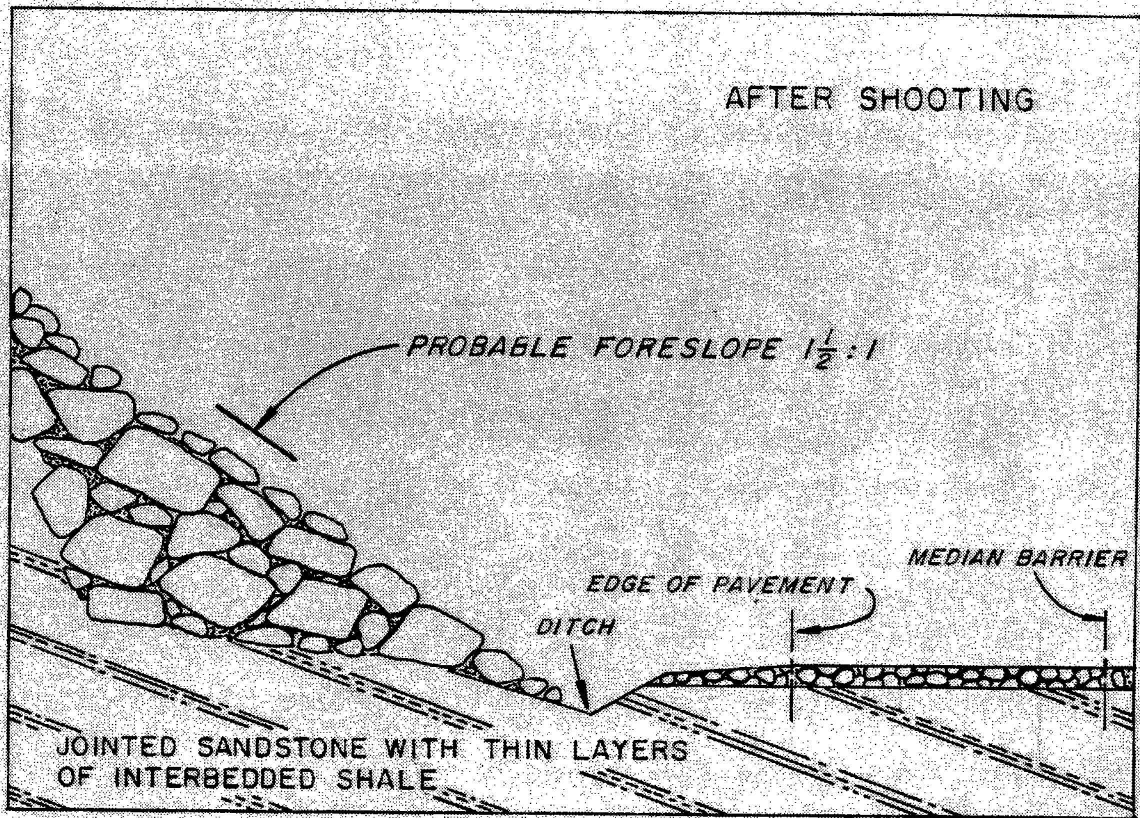


Figure 41.

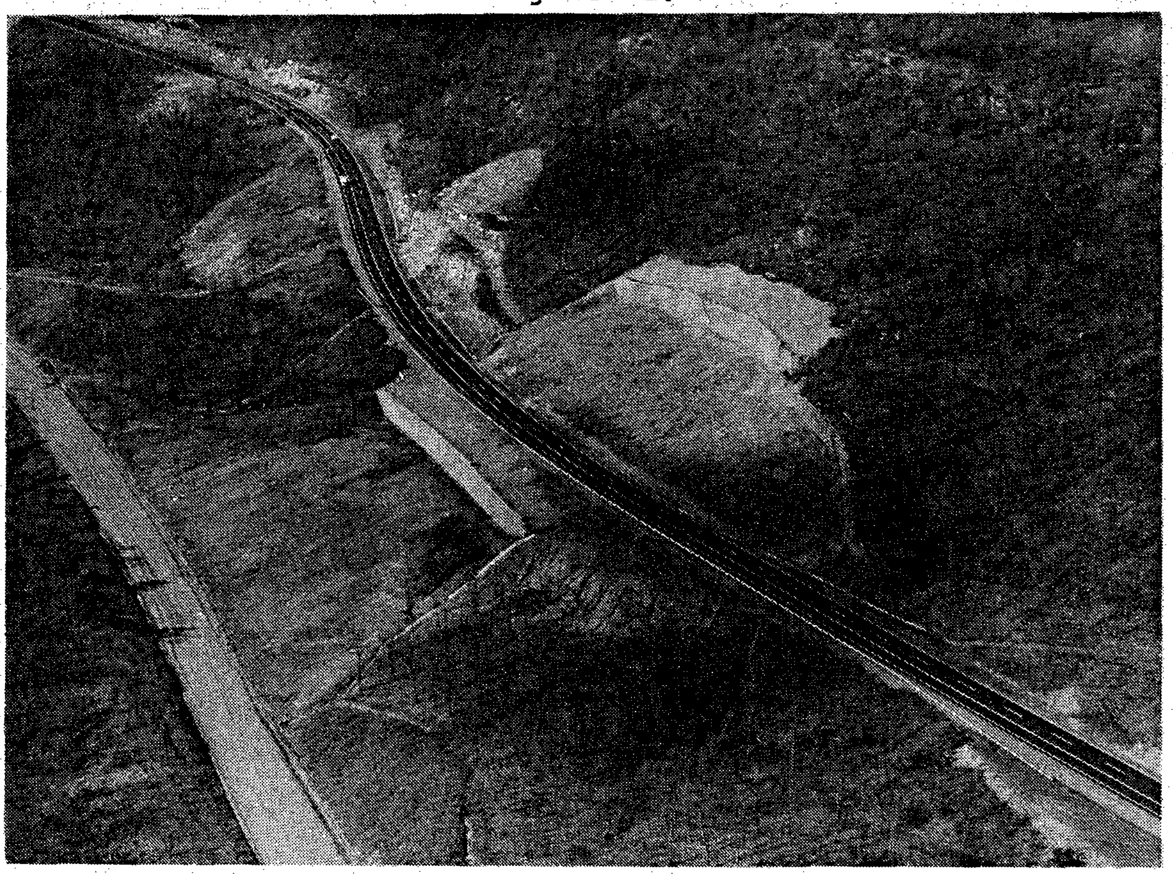


Figure 42. Gabion walls were one of the types of structures used in the repair of slides along I-40 near Rockwood.

firms in the private sector in the state on small slides. A translational slide that developed in a plunging syncline near Chickamauga Dam in Hamilton County was controlled successfully by a small timber wall constructed by the Hamilton County Highway Department (Figure 44). Such structures would seem to have excellent potential for small slides in rural areas along low-volume roads (Schuster, et al, 1973 and 1975).

As stated previously, and as indicated in most of the examples given thus far, most slide repairs involve the use of a combination of two or more of the four major categories. Figure 45 illustrates a repair measure that has been utilized with considerable success to correct side-hill fill failures in Tennessee. It involves both restraint and drainage. Several failures along I-40 near Rockwood and I-75 in Campbell County were repaired using similar designs (Royster, 1973 and 1977).

Rock falls are becoming an increasing problem for departments of transportation all across the country. The reasons are many, but principally it is because of the excavation of deeper and deeper cuts through various types of geologic materials and conditions, and also because those cuts that were excavated, say, 2 to 5 years ago may only now be showing the effects of weathering. In recent years the problem has been compounded by what seems to be a trend toward more liberal settlements to motorists claiming damages from falling rocks. While the states still invoke the

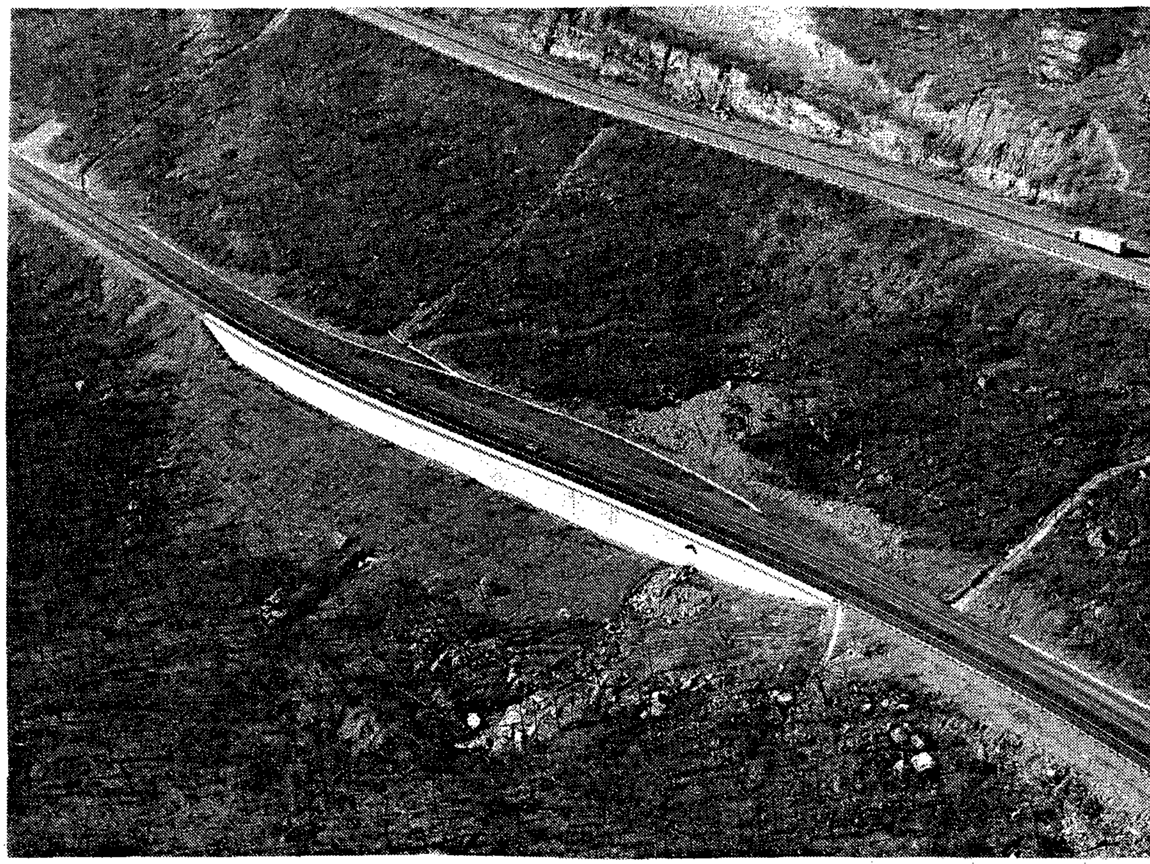


Figure 43. A Reinforced Earth structure was used to correct a major embankment failure on the same project.

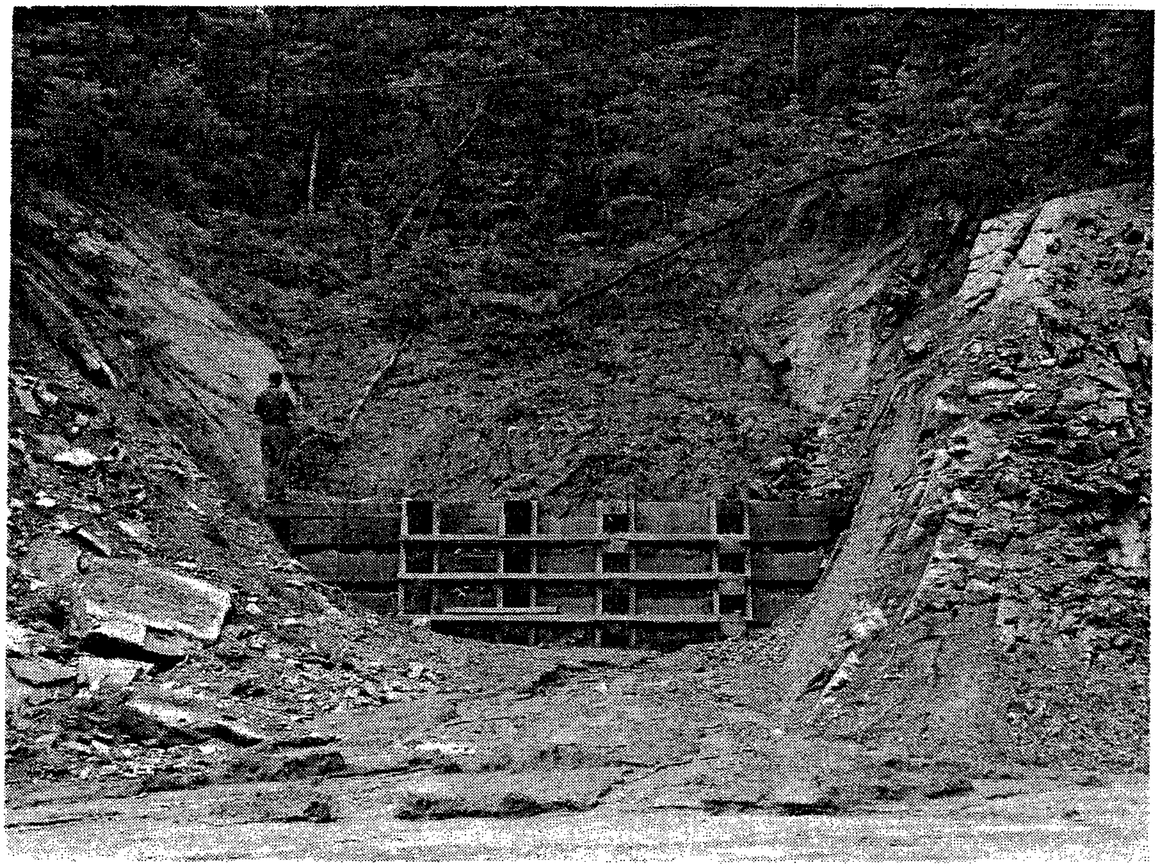


Figure 44. A small timber structure used to control a translational slide that developed in a plunging syncline along a rural road in Hamilton County.

protection of sovereign immunity, more and more cases are being heard by such agencies as the States' Boards of Claims, and it appears that with the input of some overly zealous and perhaps opportunistic members of the legal profession precedents are being established that may force designers to go well beyond the realm of reason in their future rock slope designs. As ridiculous as it may seem and as costly as it may be, it would appear that designers of the future are going to have to attempt to develop slope designs that are "totally" fail-safe or else be prepared to spend a large portion of their time in court defending themselves against claims, some of which, no doubt, will border on the ludicrous and absurd.

Rock falls have not been a major problem in Tennessee, but like most states with high rock slopes along their roadways there are areas of concern. One such area is along Interstate-24 south of Monteagle in Marion County. Here, there are cuts up to 46 meters (150') through limestone and sandstone formations that contain interbeds of shale. The shale, being softer and much more susceptible to weathering, tends to break down quite rapidly when exposed, undermining the more resistant limestone and sandstone strata (Figures 46 and 47). While few of these falls have reached the traveling lanes, there are zones that are potentially hazardous. To remove or reduce the hazard, the Department let to contract a project in April 1977 to stabilize these zones

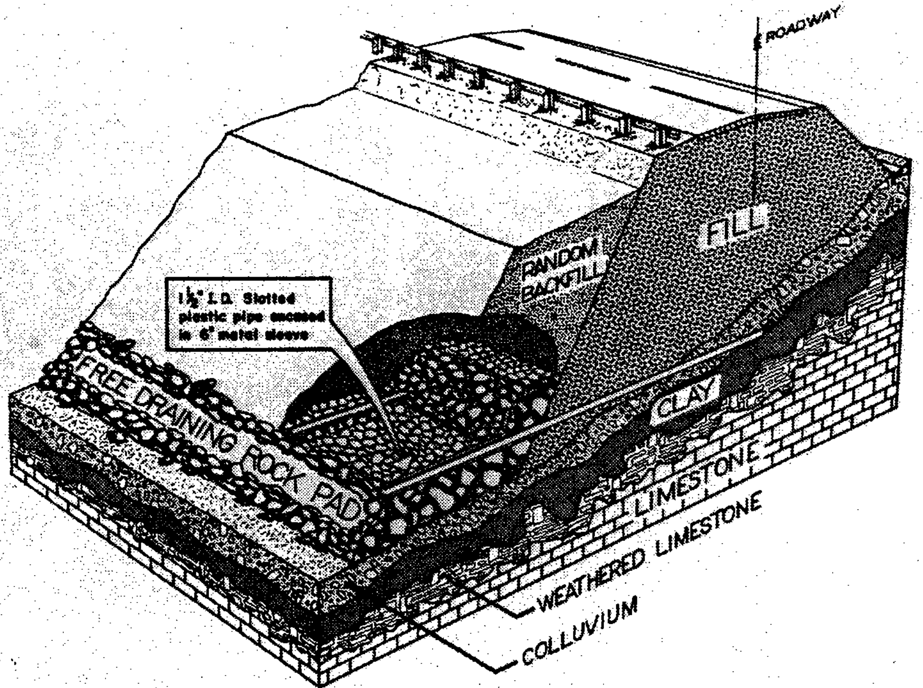


Figure 45. Method used to control side-hill fill failures along I-40 near Rockwood and I-75 in Campbell County.

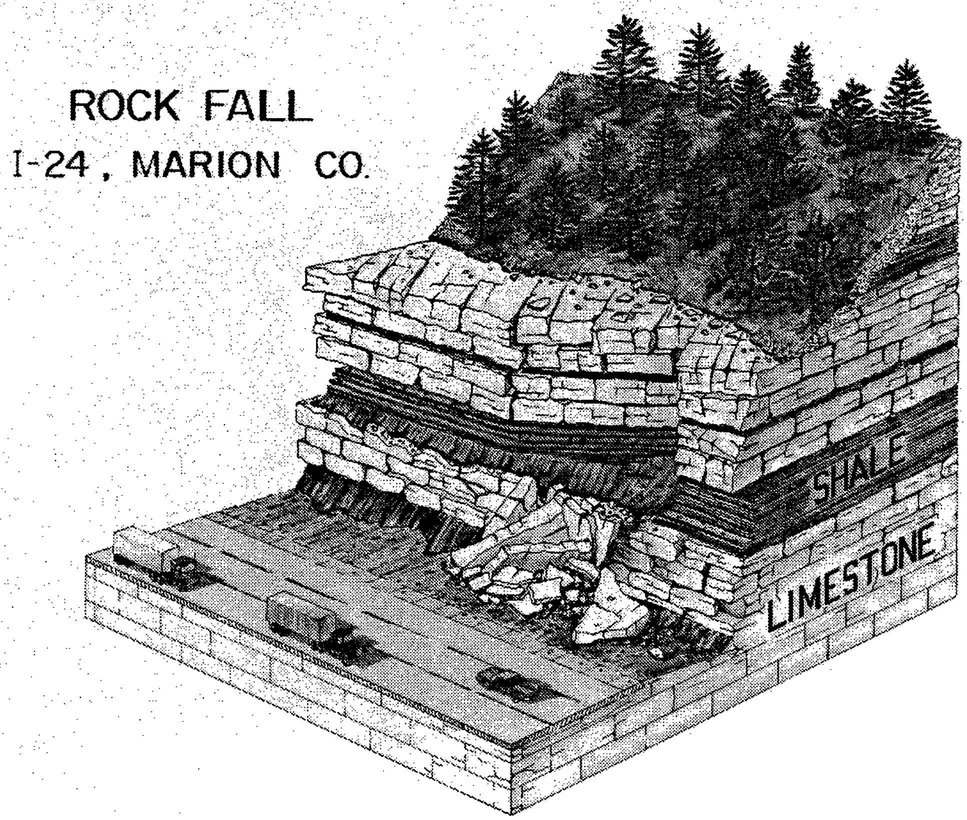


Figure 46.

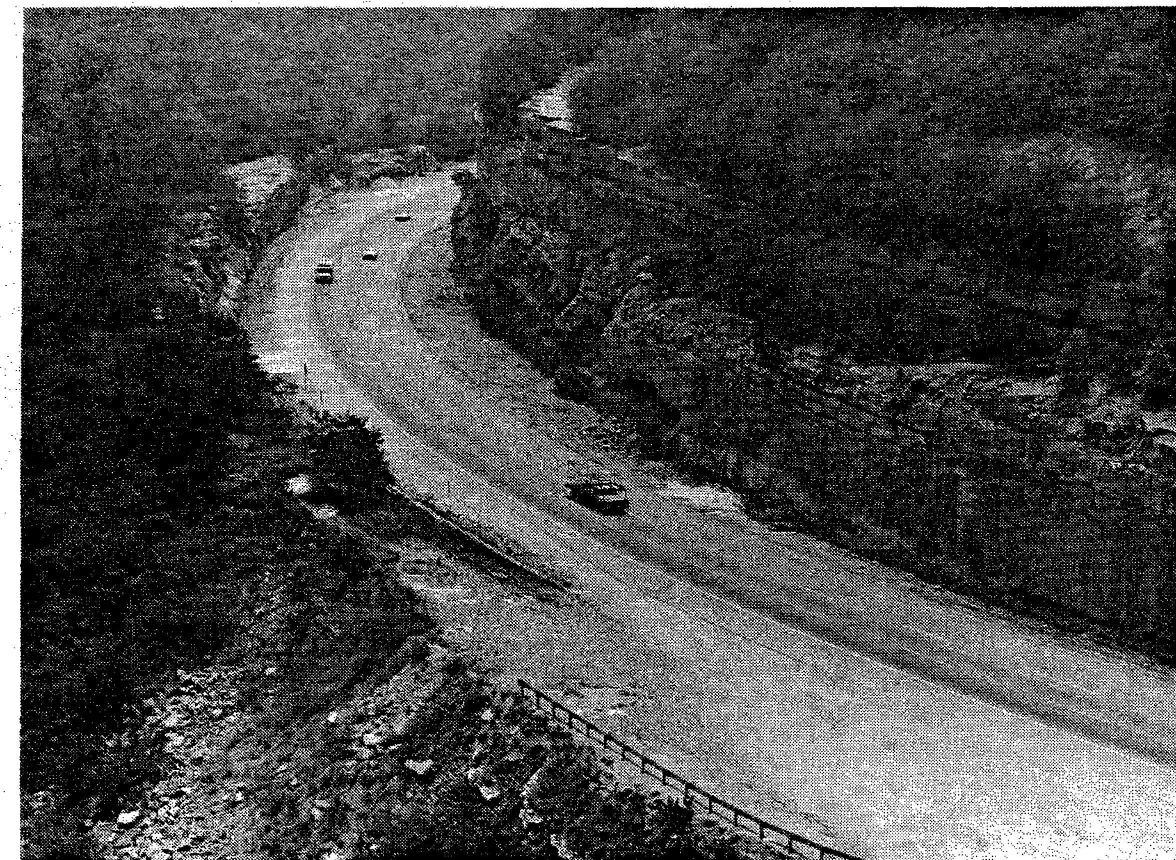


Figure 47. Aerial view of one of the rock fall problem areas along I-24 in Marion County.

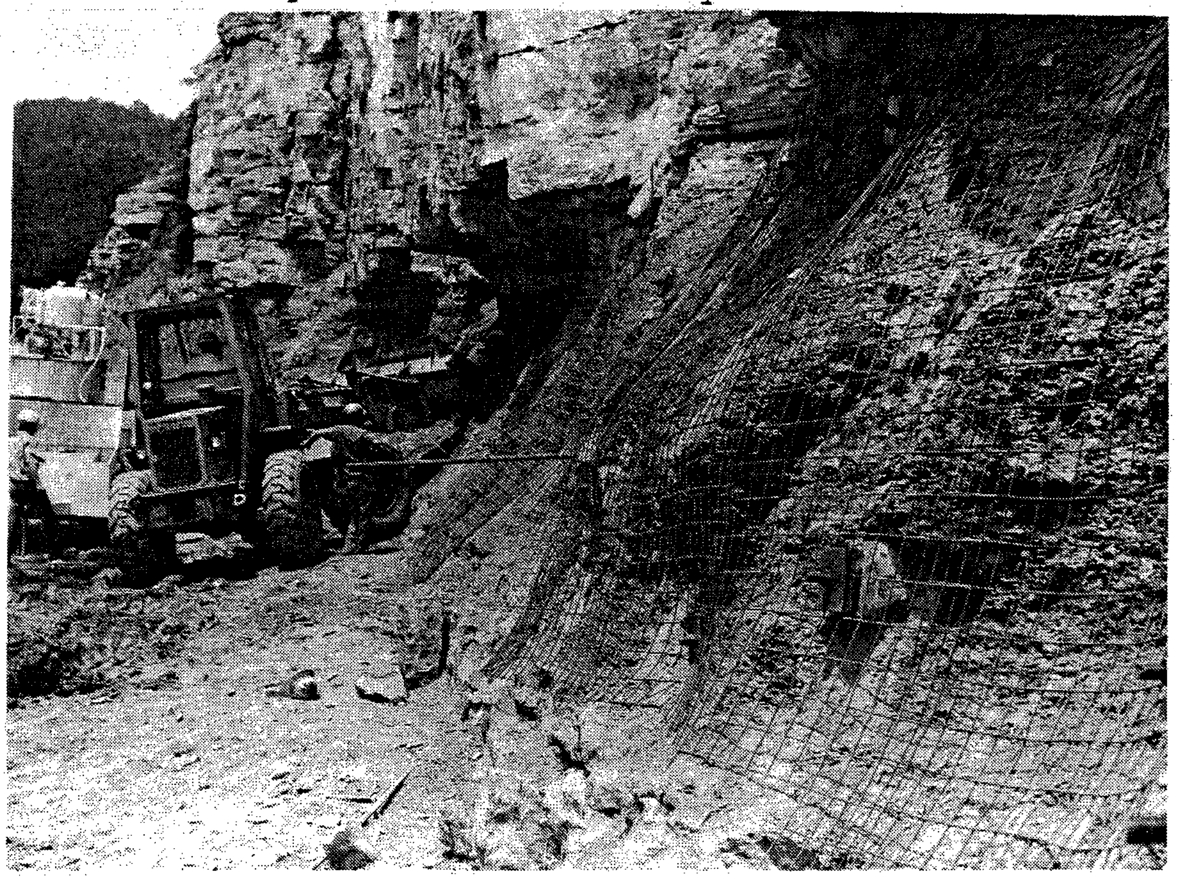


Figure 48. Wire mesh being bolted to a shale zone along I-24 prior to applying shotcrete.

with shotcrete. The wire mesh on which the 76 mm+ (3") thickness of shotcrete was applied was held in place with rock bolts 3 meters (10') in length and spaced on 1.5 m (5') centers. Weep holes cased with schedule 40 PVC pipe were randomly placed throughout the facing (Figures 48 and 49). In addition, and to insure more positive drainage, 3 m (10') sections of perforated PVC pipe were installed in horizontal holes drilled with percussion drills in the obvious seepage areas along joints and fractures at or just above the limestone-shale and sandstone-shale interfaces. The purpose was to intercept excesses of downward and laterally migrating water before it entered the stabilized zone. While considerable slope deterioration had taken place prior to the stabilization--and even though some areas must be placed in the too little-too late category--the project as a whole can be considered a success. The method will no doubt be used in similar situations in the future.

As evidenced by recent additions to the literature, i.e., Deere and Patton, 1971; Piteau, 1972; Hoek and Bray, 1974; Jaeger and Cook, 1976; Attewell and Farmer, 1976; Piteau and Peckover, 1978; it is obvious that solving problems relative to rock slopes is changing from what has been predominantly an art to what may be termed an art-science. This change is due in part to geologists and engineers combining their expertise and interest in solving these problems. D. R. Piteau (1972) expresses

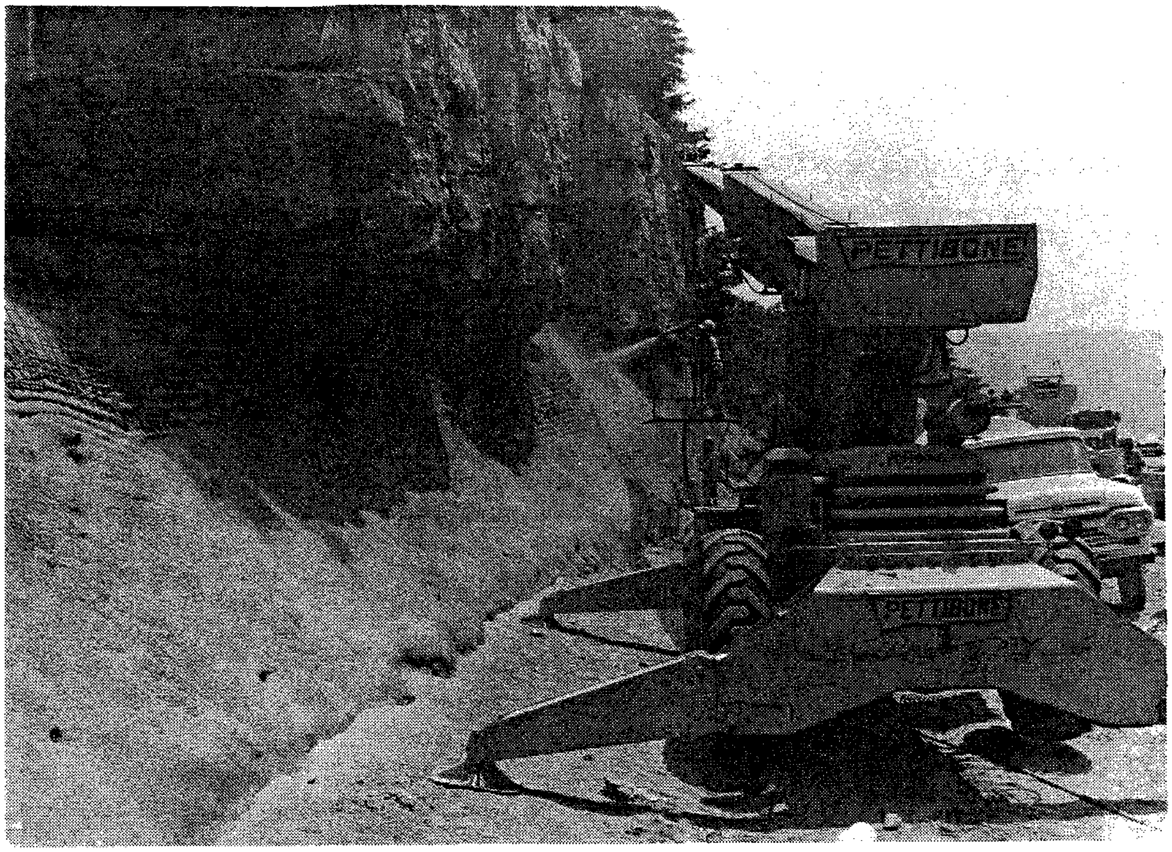


Figure 49. Shotcrete being applied in a 76 mm+ (3"+) thickness on the Marion County project.

the need for this relationship as follows:

> "The stability of a slope cut into rock is basically a problem of engineering geology--geology, because it involves a sound understanding of structure, water flow, weathering and other natural conditions relating to the geological environment, and engineering, because calculations are involved, these being based on mechanics and relating to the strength of the materials and to the forces causing instability of the slopes. The engineering geologist provides the basic data on which the engineering calculations are carried out and, for a successful outcome, the closest association must exist between the engineering geologist and the civil and/or mining engineer, with full appreciation and understanding of the contributions made by each. The accuracy of the final answer can only be as accurate as the geological data at hand, and these must be relevantly related both to existing and to future geological conditions. The problem properly resolves itself into one where engineering and geology complement one another, combining good knowledge of precedent with the arts of estimation and judgment."

Rock slope engineering in actual practice may always be more of an art than a science; nevertheless, the continual collaboration and sharing of experiences by geologists and civil and mining engineers will no doubt move the field as a whole closer toward the science end of the art-science spectrum than it is at present. A recent publication (Piteau and Peckover, 1978) covers in some detail the latest measures employed in the prevention and correction of slope movements involving rock slopes.

Reaction to the Failure

One factor that is rarely mentioned in the literature and in discussions concerning landslides, but one that is almost always in evidence is the psychological reaction to the failure.

Zaruba and Mencl (1969) describe their view of this reaction as follows:

> "Landslides are of serious concern to the persons involved; the intensity of the natural phenomenon creates a depressive atmosphere and anxiety arises to whether the collapsing slope can be stabilized at all. The question of guilt as to whether or not the failure could have been prevented, also arises."

The initial reaction might also be described as a mixed feeling of anger, embarassment, and helplessness, and even despair when a landslide of significant magnitude is viewed for the first time, especially if it developed during the latter stages of construction. The reaction must be somewhat like that of the baseball pitcher who gives up the lead run via a homerun in the top of the ninth inning, the doctor whose patient develops a blood clot after a successful operation, or even the home owner who watches as a torrential rain douses the roof he has just finished painting. The individual concerned with the landslide, however, like the pitcher and the doctor, must view the situation philosophically as a temporary setback that can be overcome in due time with the correct analysis and treatment or adjustment. It should also be viewed as a learning experience; one in which all of the observations and data are thoroughly recorded so that they might be used to prevent a similar situation in the future.

Conclusion

Landslides may be corrected or controlled by any one or any combination of four principal measures: drainage, removal, restaint, or relocation. The measure chosen is largely dependent on cost, but type of failure, location of the failure, potential for enlargement, etc., may actually dictate the method to be used.

There are also a number of levels of effectiveness and levels of acceptability that may be applied in the use of these measures. While one slide may require an immediate and absolute long-term correction, another may only require minimal control for a short period. Whatever the measure chosen, and whatever the level of effectiveness required, the Engineering Geologist or Geotechnical Engineer must also consider what might have been done to prevent the problem in the first place. Hindsight, while of little benefit in solving the immediate problem, can be of great benefit in avoiding similar occurrences in the future.

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References

Attewell, P. B., and Farmer, I. W., 1976, Principles of Engineering Geology: Chapman and Hall, London; distributed in the U.S.A. by Halsted Press, 1045 pp.

Baker, R. F., and Marshall, H. C., 1958, "Control and Correction." In Landslides and Engineering Practice (Eckel, E. B., ed.): HRB Special Report 29, pp. 150-188.

Deere, D. U., and Patton, F. D., 1971, "Slope Stability in Residual Soils." Reprinted from the Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, San Juan, Puerto Rico, pp. 87-170.

Gedney, D. S., and Weber, Jr., W. G., 1978, "Design and Construction of Soil Slopes." In Landslides: Analysis and Control (Schuster, R. L. and Krizek, R. J., eds.): TRB Special Report 176.

Hoek, Evert, and Bray, John, 1974, Rock Slope Engineering: The Institution of Mining and Metallurgy, London, 309 pp.

Hutchinson, J. N., 1977, "Assessment of the Effectiveness of Corrective Measures in Relation to Geological Conditions and Types of Slope Movement." Bulletin of the International Association of Engineering Geology, Number 16, pp. 131-155.

Jaeger, J. C., and Cook, N. G. W., 1976 (Second Edition), Fundamentals of Rock Mechanics: Chapman and Hall, London; distributed in the U.S.A. by Halsted Press, 585 pp.

Piteau, D. R., 1972, "Engineering Geology Considerations and Approach in Assessing the Stability of Rock Slopes." In Bulletin of the Association of Engineering Geologists, Volume IX, Number 3, pp. 301-320.

Piteau, D. R., and Peckover, F. L., 1978, "Rock Slope Engineering." In Landslides: Analysis and Control (Schuster, R. L. and Krizek, R. J., eds.): TRB Special Report 176.

Root, A. W., 1958, "Prevention of Landslides." In Landslides and Engineering Practice (Eckel, E. B., ed.): HRB Special Report 29, pp. 113-149.

Royster, D. L., 1973, "Highway Landslide Problems Along the Cumberland Plateau in Tennessee." Bulletin of the Association of Engineering Geologists, Volume X, Number 4, pp. 255-287.

Royster, D. L., 1974, "Construction of a Reinforced Earth Fill Along Interstate-40 in Tennessee." Proceedings of the 25th Annual Highway Geology Symposium, Raleigh, North Carolina; (Also published as a case history by the Reinforced Earth Company of Washington, D. C.), 11 pp.

Royster, D. L., 1977, "Some Observations on the Use of Horizontal Drains in the Correction and Prevention of Landslides." Proceedings of the 28th Annual Highway Geology Symposium, Rapid City, South Dakota, 55 pp.

Schuster, R. L., Jones, W. V., Smart, S. M., and Sack, R. L., 1973, A Study and Analysis of Timber Crib Retaining Walls. Civil Engineering Report 73-1: Prepared for the U. S. Forest Service by the College of Engineering, University of Idaho, 186 pp.

Schuster, R. L., Jones, W. V., Smart, S. M., and Sack, R. L., 1975, "Timber Crib Retaining Structures." In Low-Volume Roads: TRB Special Report 160, pp. 116-27.

Terzaghi, Karl, 1950, "Mechanism of Landslides. In Application of Geology to Engineering Practice, Berkey Volume: Geological Society of America, New York, pp. 83-123.

Varnes, D. J., 1958, "Landslide Types and Processes." In Landslides and Engineering Practice (Eckel, E. B., ed.): HRB Special Report 29, pp. 20-47.

Varnes, D. J., 1978, "Types of Slope Movements." In Landslides: Analysis and Control (Schuster, R. L. and Krizek, R. J., eds.): TRB Special Report 176.

Zaruba, Q., and Mencl, V., 1969, Landslides and Their Control: Elsevier, New York, and Academia, Prague, pp. 157-158.

Petrographic Examination of Aggregates Used in Bituminous Overlays for Indiana Pavements as Related to their Polishing Characteristics

by

A. Shakoor and T.R. West*

Abstract

As part of an on-going research project performed in conjunction with the Indiana State Highway Commission, detailed petrographic analysis was conducted on 84 samples from bituminous pavement surfaces throughout the state. In addition to the petrographic analysis, polishing characteristics of these aggregates were determined through measurement of polish values on coarse aggregates in the laboratory using the "Accelerated Polish Test for Coarse Aggregates", Texas Standard - 438-A. Megascopic and microscopic techniques were performed in the petrographic study.

Petrographic analysis shows that both gravel and crushed stone comprise the aggregates used. The crushed stone is angular to subangular and consists of limestone and/or dolomite with appreciable quantities of chert. Gravel is generally subangular to sub-rounded and consists of a large variety of rock types including carbonates, sandstone, siltstone, shale, schist, quartzite, amphibolite, granite, diorite, andesite, gabbro, basalt, quartz and chert. Sandstone and granite generally show a slight to moderate degree of weathering. Blast furnace slag was also used in several of the pavement overlays.

Preliminary comparisons between petrography and polish values suggest a close relationship between mineral composition and the polishing aspects. The limestones studied exhibited slightly higher levels of resistance to polishing than did dolomites as indicated by the accelerated polish test. Also, the dolomites polish faster than the limestones and hence, their "Wear Factor" was greater. Argillaceous limestones polished somewhat more than did the purer limestones.

For coarse gradations, crushed limestone aggregates appear to polish to about the same extent as do coarse-sized slags. For the sand mixes, slags have a higher initial polish value than do quartz-rich and carbonate-rich samples but they polish faster so that their final polish value is the lowest of the three. Subrounded to rounded carbonates (gravels) show lower initial polish values but they also have low wear factors. The final polish value is similar to that for crushed carbonates, which began as angular fragments.

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Introduction

Statistics indicate that a large number of accidents are related to skidding on wet or otherwise slippery pavements. This results not only in loss of life and property but in legal complications as well (2).*

Because of the significance of this problem the Indiana State Highway Commission is actively engaged in research on pavement skid resistance. The purpose is to maximize the skid resistance for bituminous overlays in the State.

In Indiana most of the interstate roads, other federal highways and many state highways were built as concrete pavements. The pavement overlays, placed to recondition deteriorated concrete pavements, however, are bituminous. Control of future maintenance costs depends upon obtaining the best bituminous overlays both in terms of long life and skid resistance. In the past bituminous roads were resurfaced when extensive cracking, ravelling or other obvious failures had occurred. Shortly, we may be faced with resurfacing roads when the skid resistance drops below a minimum critical value.

As part of the larger research program undertaken by the Indiana State Highway Commission, the authors conducted petrographic analysis of 84 samples from bituminous pavement surfaces throughout the State. Samples were taken from 14 locations which included state roads, U.S. highways and

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* Numbers in ( ) refer to the list of cited references at the end of this paper.

interstate routes. Each location was subdivided into as many as six independent sampling points distributed over a distance of about five miles. The locations were numbered consecutively and each individual sample assigned numbers 1 through 6 plus a direction indicator, designating whether it was north, south, east or west of the original selection point. This detailed sample information is included in this report only for the few locations supplied as examples (Tables 1 and 2).

In addition to the petrographic analysis, polishing characteristics of these aggregates were determined both in the laboratory as well as in the field. The petrographic analysis is then correlated with polish values to establish the influence of mineral composition, particle shape, grain size, and other textural parameters on polish resistance. These data may be useful to determine relationships between aggregate polish resistance and pavement skid resistance. This paper presents the preliminary results on petrographic analysis and polish resistance based on the 84 samples examined in this study.

Natural aggregates in Indiana are either 1) crushed stone from limestone or dolomite quarries or 2) glacial stream gravels and sands. The gravels are rounded to sub-rounded rocks in which 50 to 60% are carbonates (limestone and dolomite) plus some chert, sandstone, siltstone and shale, along with various igneous and metamorphic rocks, i.e. granite, basalt, andesite, diorite, schist and quartzite. In some cases the gravels are crushed to produce aggregates with pieces containing some angular faces whereas others maintain the rounded shape.

Data Collection

Cores measuring 6" in diameter and 3-4" deep were drilled in the bituminous overlays at the various field sites to obtain the samples for study. The aggregate was extracted from these cores by dissolving the bituminous binder according to ASTM standard D2172, Method B. The apparatus used is shown in Figure 1 and the solvent can be either trichloroethylene, 1,1,1-trichloroethane or benzene.

The extracted aggregates were studied petrographically using both megascopic and microscopic techniques. Detailed descriptions of the mineralogy and texture were provided at this juncture. Further information about the procedure is supplied in a later section of this paper.

Representative pieces of the large fraction of the aggregate (½" size to No.4 size) are mounted in small coupons using a polyester bonding agent to hold them in place. The placement of the aggregates in the steel mold for the coupons is shown in Figure 2.

Polish values were obtained on these aggregate coupons using the Accelerated Polish Test for Coarse Aggregates, Texas Standard 438-A, described by Patty, 1973 (15). This

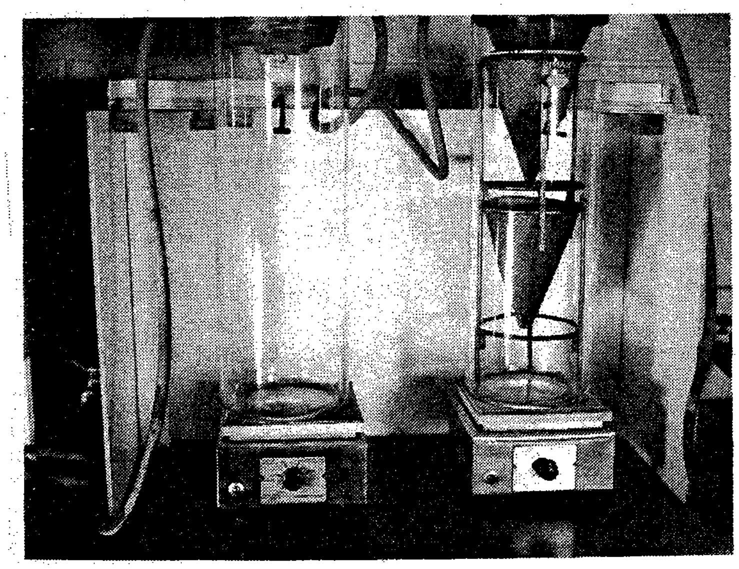

Figure 1 Extraction Equipment to Remove Bituminous Binder

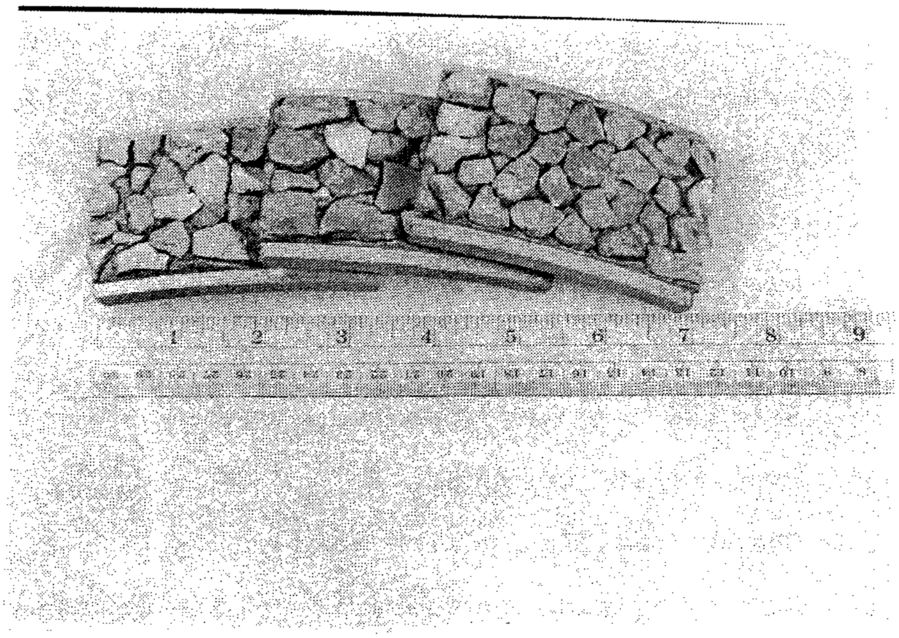

Fig. 2 Aggregate coupons showing aggregates held in place by epoxy.

test has been in use since 1972 and is commonly referred to as the "polish test".

The polish test involves two pieces of equipment: 1) the British Portable Tester (Figure 3) which determines the skid resistance of the sample before and after the coupons are polished and 2) the British Accelerated Polishing Machine (Figure 4) which does the polishing of the coupons.

Hence the procedure for laboratory testing is as follows:

1. Obtain cores from the pavement.
2. Remove aggregates from bituminous binder.
3. Perform the petrographic analysis.
4. Prepare the coupons.
5. Measure initial polish value.
6. Run coupons on accelerated polish machine.
7. Measure final polish value.
8. Subtract the final polish value from the initial one yielding a number known as the wear factor.

Two different aggregate gradations for bituminous overlays used to resurface roads in Indiana were included in the study. The first is a coarse aggregate gradation with a maximum size of 3/4" ranging to minus No. 100 size material. The other is a finer size known as a sand mix aggregate with the maximum size of No. 4 (or sometimes 3/8") ranging to a minus No. 100 size.

Petrographic Analysis

Petrographic analysis of aggregates as related to construction has been considered by researchers for some years

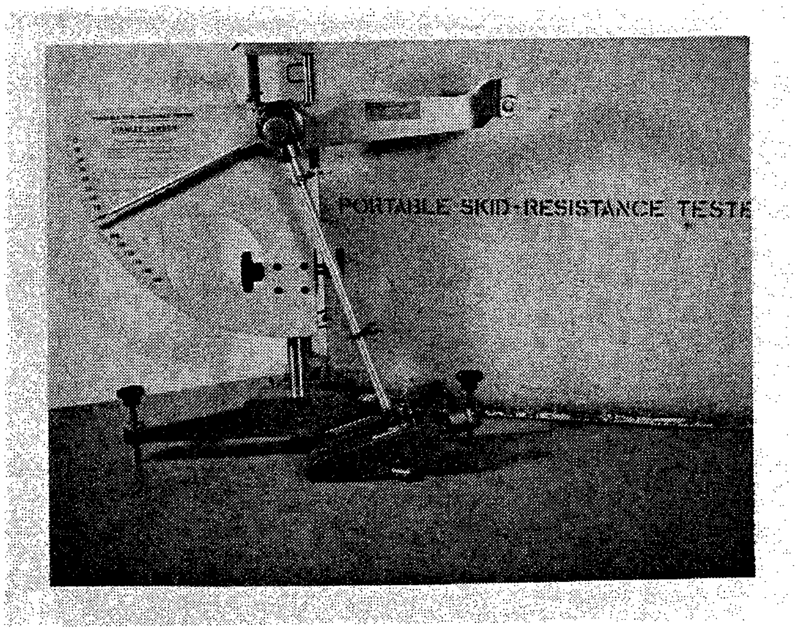

Fig. 3 British Portable Tester (BPT).

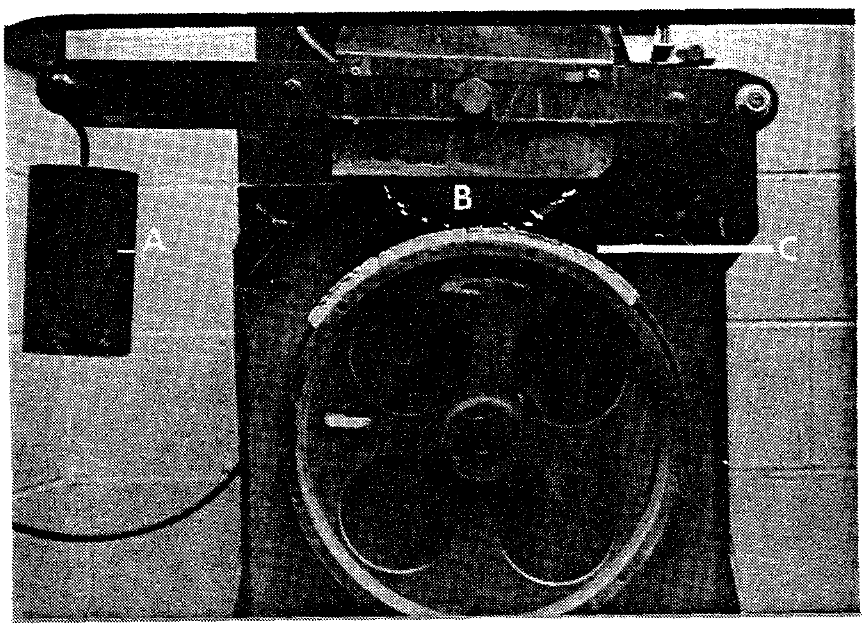

Fig. 4 British Accelerated Polishing Machine. A, designates the weight which forces the polishing wheel B, against the aggregate coupons, C.

including work accomplished at Purdue University. Early studies by Mielenz and his coworkers (11,12,13,14) and by Mather and Mather (10) demonstrated the value of petrographic examination in regard to aggregate use. Later, work by Gillott (4) and by Hadley (7) related textures and compositions of carbonate rocks to their behavior in concrete. At Purdue University early studies were accomplished by Sweet and Woods (24) followed by that of Shupe and Lounsbury (22) and Shupe and Goetz (21) who worked with aggregates for bituminous mixes, and later that of Lounsbury and West (9) on concrete aggregates of Indiana. Subsequently, textural analysis of aggregates relative to abrasion resistance and degradation was studied by West and his coworkers (26,27,28). Recently (29) detailed petrographic examination has been applied to gravel pit evaluations to discern differences in aggregate quality.

In the petrographic study for the Indiana State Highway Commission, the samples with coarse aggregate gradation were separated into the eight size fractions as shown below:

3/4" - 3/8"
3/8" - No.4
No. 8 - No.16
No.16 - No.30
No.30 - No.50
No.50 - No.100
Minus No.100

The first five size fractions, i.e., through sieve No.30, were subdivided according to individual rock constituents using hand lens and binocular microscope, and the percentage by weight of each constituent was determined. Included in the petrographic description of each rock type was the particle shape, crystallinity and size of crystals, degree of weathering, cementing material and the impurities present. These were considered because of their importance in determining skid resistance. The distinction between limestone and dolomite was made by the use of 10% HCl - those pieces which resulted in brisk effervescence were taken as limestone while those which showed slow effervescence (or produced effervescence only when scratched) were considered as dolomite. However, this distinction was not considered to be sufficiently accurate for the size fractions from No.8 - No.16 and No.16 - No.30, in which case the percentage of the carbonate fraction was considered as a group.

For bituminous pavements containing coarse aggregate, it is generally accepted that the coarse aggregate portion largely determines the skid resistance. Hence, fractions smaller than No.30 were not included in the study of individual constituents. However, they were examined under the binocular microscope and it was generally observed that the amount of quartz increased with decreasing grain size.

Finally, thin sections were made for the coarsest size pieces (3/4" - 3/8") of limestone or dolomite - whichever was more prevalent in the sample. These thin sections were studied in detail under the optical microscope to determine the composition and texture of the carbonate fraction. Where limestone and dolomite were present in more or less equal proportions for the aggregates, thin sections were made for each. Carbonates were selected for the microscopic study because, in most cases, they formed the main portion of the coarse fraction.

For the sand mix bituminous overlays, the size fractions studied were:

No.4 - No.8
No.8 - No.16
No.16 - No.30
No.30 - No.50

These were analyzed in detail and described according to rock type and texture as previously discussed. Petrographic information for coarse aggregate gradation is shown in Table 1 and for a sand mix sample in Table 2. The different rock types are identified and described; this shows abbreviated descriptions. Included also is the particle shape, with angular, rounded, subangular, subrounded and flat varieties. Angular and subangular are typical of crushed stone whereas rounded and subrounded are typical of gravel pieces. This illustrates some of the degree of detail in the petrographic study.

Results of Petrographic Analysis

Petrographic analysis indicates that gravel, sand, crushed stone, and slag comprise the aggregates present in pavements studied. These have been used either as a coarse mix with a maximum size of 3/4", or as a sand mix with a maximum size of No.4 or sometimes 3/8" material. Crushed stone is angular to subangular and consists of limestone

TABLE 1. PETROGRAPHIC ANALYSIS FOR DIFFERENT SIZE FRACTIONS OF AN AGGREGATE

Sample No. 5-5W

Location: ____________

Rock Types	Description of Rock Types	Amount (% by weight) and particle shape for each size fraction									
		3/4"-3/8"		3/8"-No. 4		No. 4-8		No. 8-16		No. 16-30	
		Particle Shape	%	Particle Shape	%	Particle Shape	%	Particle Shape	%	Particle Shape	%
Limestone	light grey, fine to medium grained, crystalline; some pieces are dolomitic	A	92.9	A-SA	98.0	A	62.6				
Limestone	grey, fine grained; some pieces are slightly weathered	-	-	-	-	SA-SR	15.1	SA-SR	39.8	SA-SR	34.2
Dolomitic Limestone	light grey to grey, fine grained	-	-	-	-	SA-SR	5.8				
Sandstone	fine to medium grained, weathered	-	-	-	-	SA	2.2	SA-SR	11.1	SA	8.6
Siltstone	fine grained, slightly calcareous, somewhat schistose in finer fractions	-	-	-	-	SA-SR	2.6	F-SA	11.1	F-SA	16.3
Shale	dark grey with well developed fissility	-	-	-	-	F-SA	2.2	F-SA	6.3	-	-
Quartzite	fine grained, hard and massive	-	-	-	-	SA-SR	2.2	SA	7.9	SA	-
Granite	medium grained, hard and massive	-	-	-	-	SA	1.9	SA-SR	6.3	-	5.0
Diorite	medium grained, hard and massive	-	-	-	-	-	-	SA-SR	3.2	-	5.0
Basalt	black, fine grained, polished	-	-	-	-	A	0.2	-	-	-	-
Quartz	aggregate of small grains as well as individual grains	-	-	-	-	SA-SR	1.3	SR	9.5	SA-SR	28.0
Chert	fine grained, hard and massive; conchoidal fracture	A	7.1	A	2.0	A-SA	3.9	A-SA	4.8	A-SA	7.9
			100.0		100.0		100.0		100.0		100.0

A - Angular R - Rounded F - Flat or disc shaped SA - Sub-angular SR - Sub-rounded

TABLE 2. PETROGRAPHIC ANALYSIS OF THE DIFFERENT SIZE FRACTIONS OF AN AGGREGATE SAND MIX
(SAMPLE NO. 8-1s)

ROCK TYPES	DESCRIPTION	AMOUNT (% BY WEIGHT) & PARTICLE SHAPE FOR EACH SIZE FRACTION							
		No. 4-8		No. 8-16		No. 16-30		No. 30-50	
		PARTICLE SHAPE	%	PARTICLE SHAPE	%	PARTICLE SHAPE	%	PARTICLE SHAPE	%
LIMESTONE	LIGHT GREY, FINE GRAINED, COMPACT	SA-SR	44.1	SA-R	65.4	SR	62.5	SA-SR	32.2
DOLOMITE	"	SA-SR	19.6						
SANDSTONE	FINE TO MEDIUM GRAINED, SLIGHTLY WEATHERED	SA-SR	9.8	SA	4.7	SA-SR	3.1	SA	3.2
SILTSTONE	FINE GRAINED, SLIGHTLY WEATHERED	F-SA	9.8	F-SA	8.4	F-SA	7.8	F-SA	6.4
GRANITE	MEDIUM GRAINED, HARD & MASSIVE, WEATHERED	SA	2.9	SA	4.7	SA	1.6	-	-
DIORITE	LIGHT GREY, MEDIUM GRAINED, MASSIVE	SA	1.0	-	-	-	-	-	-
QUARTZ	MASSIVE, SLIGHTLY FRACTURED	SA	4.9	SA	10.2	SA	21.9	SA-SR	45.2
CHERT	FINE GRAINED, CONCHOIDAL FRACTURE	A-SA	7.8	A-SA	6.5	A-SA	3.1	A-SA	13.0
			99.9		99.9		100.0		100.0

A=ANGULAR SA=SUB-ANGULAR F=FLAT OR DISC SHAPED
R=ROUNDED SR=SUB-ROUNDED

and/or dolomite with appreciable quantities of chert. Gravel is generally subangular to rounded and consists of a large variety of rock types including carbonates, sandstone, siltstone, shale, schist, quartzite, amphibolite, granite, diorite, andesite, basalt, gabbro, quartz and chert. Except for a few sand mixes which consist entirely of siliceous rocks or slag and those overlays which contain slag as the coarse fraction, carbonates comprise the major portion of the aggregate in the overlays. Microscopically, the limestones vary from almost pure $CaCO_3$, through dolomitic limestone, to argillaceous limestone, and they are fine grained, crystalline to cryptocrystalline. The dolomites are somewhat coarser in grain size and consist of euhedral, well-interlocking rhombs. Those dolomites which appear as crushed stone typically contain an abundance of solution pores. Sandstone and granite generally show a slight to moderate degree of weathering. Shale in all cases is soft and highly fissile. Rocks such as andesite, basalt and gabbro were found in only a few aggregate samples.

Slag, used in both the coarse mix and sand mix, is grey, light-weight, rough-surfaced, and a highly vesicular (scoraceous) variety.

Discussion - Petrography, Polishing and Skid Resistance

Mineral composition, and to a lesser degree, grain size and particle shape have the greatest influence on aggregate polishing and therefore skid resistance of the pavement as far as the aggregate is concerned (1,3,5,16,17, 18,19,23,25). Previous work at Purdue University had established that the composition of the aggregate had an important effect on skid resistance of the pavement (6,8,20,21,22,23). An effort was, therefore, made to correlate these petrographically-determined parameters with the roughness characteristics of the aggregate. Because of the extensive detail involved in the current study it was hoped that a significant relationship between polishing characteristics and aggregate petrography could be found.

A summary of the data obtained in this study is presented in Table 3. Ten different categories of aggregates are included with these subdivisions based on mineral constituents and sample gradations.

Mineral Composition: Preliminary comparisons between petrography and polish value suggest an important contribution regarding mineral composition. Limestones exhibit higher numbers for the final polish value than do dolomites. Where limestone and dolomite were used in equal proportions, the final polish values are generally lower than cases where limestones predominate, but higher than the dolomite-rich samples. Dolomites also polish more easily and hence their wear factor is higher. This indicates that even though the initial polish value of dolomites may be as good or even better than limestones, dolomites seem to become more slippery when applying equal polishing effort.

TABLE 3. Average of Polish Values for Different Aggregates in Test Samples, Indiana Study.

MAJOR CONSTITUENTS OF THE COARSE FRACTION	SOURCE	NUMBER OF SAMPLES	PARTICLE SIZE	PARTICLE SHAPE	AVERAGE POLISH VALUE Initial	AVERAGE POLISH VALUE Final	AVERAGE* WEAR FACTOR
1. Sand mixes with limestone & dolomite as major constituents		6	No.4-8	SA-SR	31.0	26.8	4.2
2. Sand mixes with slag		6	3/8"-No.4	A-SR	39.2	24.8	14.3
3. Sand mixes with sandstone, quartzite, quartz & chert		6	No.4-8	SR-R	36.0	26.8	9.2
4. Slag		12	3/4-3/8"	A-SA	40.5	29.3	11.3
5. Limestone	G&C.S.	13	3/4-3/8"	A-SR	36.6	29.3	7.7
6. Argillaceous limestone	C.S.	6	3/4-3/8"	A-SA	37.0	29.0	8.0
7. Dolomite	C.S.	11	3/4-3/8"	A-SA	39.4	27.5	11.9
8. Limestone and dolomite	G	6	3/4-3/8"	SR-R	31.7	28.8	2.8
9. Limestone and dolomite - partly crushed	G	12	3/4-3/8"	SA-R	34.6	30.8	3.8
10. Limestone and dolomite - completely crushed	G	9	3/4-3/8"	A-SA	34.6	26.3	8.3

G = Gravel C.S. = Crushed Stone

* The average wear factors given in this table are not the difference between the average initial polish value and average final polish value shown but instead are the average of the sample values themselves for each category.

The crushed limestones which are argillaceous in nature have lower initial polish values (from 35-38) and higher wear factors (from 7-10) than do most other limestones. A combination of very fine grained texture and high clay content seem to cause this problem. Because of this, they tend to have very smooth surfaces even in the crushed state. Clay content in this limestone was estimated petrographically at 10%.

After carbonates, slag is the next most common material used in the bituminous overlays studied. Slag is believed to be one of the most skid resistant materials (16). For the samples in this study, when used as 100% coarse aggregate material, the polishing characteristics of slag are only slightly better than those of limestone (Table 3), as the slag has a higher initial value but the same final polish value. However, where the slag has been used as a sand mix the final polish values are the lowest of the sand mixes (Table 3, items 1-3). In general, all the sand mixes showed a lower final polish value than most other aggregates.

One characteristic feature of slag is its high wear factor (equal to 11.3). It is interesting to note that the wear factor is even greater when the slag is used as a sand mix than it is when used as a coarse mix (Table 3, item 2 vs. item 4).

The importance of mineral composition in determining polish resistance is well demonstrated by the marked difference between initial polish values for sand mixes in which carbonates are the major constituents (Table 3, item 1) versus sand mixes with slag (item 2) and those with sandstone, quartzite, quartz and chert (item 3). The carbonates have a lower initial polish value but their final value is as good or better than the others.

Aggregate Size: On the basis of the particle size, the aggregate material from Indiana can be divided into three categories:

1. Those in which the largest size is 3/4"-3/8".
2. Those in which the largest size is 3/8"-No.4 but only a small part of the total aggregate falls in this size range.
3. Those in which the largest size is No.4-No.8 These are sand mixes.

A review of Table 3 indicates that sand mixes with carbonates (item 1) have low initial polish values (average 31) and low wear factor (average 4.2) as compared with the coarse grained carbonate aggregates (item 5,6 and 7). The sand mixes which contain slag as the main constituent, have relatively high initial polish values (average 39.2). Also, these sand mixes contain a small proportion of 3/8"-No.4 material. However, as the wear factor is the highest (average 14.4) for these sand mixes, it is not certain if the sand mixes with slag will maintain their high degree of roughness over a long period of time or not. The sand mixes composed of slag have markedly lower polish values (both initial and final) than do the coarse-sized slags. The wear factor is also higher for the sand mixes.

An overall comparison of the sand mixes with the coarse grained aggregates actually suggests that overlays with coarse grained limestone aggregate will have a greater resistance to polishing over a longer period of time than will sand mixes. However, the value of the laboratory polish test as a predictor of the polish resistance of sand mixes in service is open to question.

Particle Shape: The effect of particle shape on polish value can be discerned by comparing carbonate aggregate samples of gravel (Table 3, item 8) partly crushed gravel (item 9) and fully crushed gravel (item 10). The relatively low initial polish values (average 31.7) for item 8 are apparently due to the abundance of subrounded to rounded particles. Both items 9 and 10 show higher initial polish values presumably because of the angular nature of some or all of the particles. As would be expected the more angular sample (item 10) has the highest wear factor, a result of the rounding of the particles during polishing. Its lower final polish value (equal to 26.3) is likely due to the somewhat argillaceous nature of this aggregate.

Polished Grains: The group of sand mix samples containing sandstone, quartz, etc. (item 3, Table 3) deserve special attention. All the grains were subrounded to rounded and showed a moderate to high degree of polish prior to the onset of testing. Because of the abundance of rounded and polished grains and the sand-mix nature of the aggregates, it was expected that the samples would have low initial polish values and low wear factors. Instead, the wear factor is reasonably high, the initial polish value is higher than for angular carbonates in sand mixes, and the final polish value is as high as for any of the other sand mixes. The effect of the rounded and polished nature of the grains seems to have been subdued by the siliceous composition. The difference in hardness between sandstone and quartzite may improve the surface texture resulting in a higher resistance to polishing.

At this stage of the research is appears that sand mixes with a high percentage of insoluble residues (hence non-carbonate rocks) tend to be more polish resistant than carbonate sand mixes even though the siliceous grains of the first group may be well-polished at the outset.

Results and Conclusions

Based on laboratory polish tests of the aggregate samples studied in this research project which were extracted from selected bituminous overlays of Indiana highways:

1. The aggregates involved a wide assemblage of rock types including limestone, dolomite, chert, sandstone,

siltstone, shale, granite, quartz, basalt, andesite, diorite, schist and quartzite, plus blast furnace slag. Limestone and dolomite were the prevalent constituents for most of the samples.

2. Limestones are slightly less prone to polishing than are dolomites.

3. Argillaceous limestones are more prone to polishing than are purer limestones. No argillaceous dolomites were included in the study.

4. Coarse slag had essentially the same resistance to polishing as did coarse limestones although the slag had a higher initial polish value.

5. Both coarse slag and the sand-mix sized slag showed the highest wear factors (highest wear rate) as compared to other aggregate constituents.

6. Angular particles have higher wear factors than subrounded to rounded particles for aggregates of similar composition.

7. Sand mixes in general have a lower laboratory polish value than do coarse mixes for aggregates of similar composition.

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References Cited

1. American Association of State Highway and Transportation Officials, Task Force for Pavement Design-Operating Subcommittee on Design, 1976, "Guidelines for Skid Resistant Pavement Design", Washington, D.C., 20 pages.

2. Carlson, R.F., 1974, "A Review of Case Law Relating to Liability for Skidding Accidents", Transportation Research Record, 263, pp. 49-60.

3. Furbush, Malcolm A. and Styers, Karen E., 1972, "The Relationship of Skid Resistance to Petrography of Aggregates", Final Report, Bureau of Materials, Testing and Research, Pennsylvania Dept. of Transportation, 34 pages.

4. Gillott, J.E., 1963, "Petrology of Dolomitic Limestones, Kingston, Ontario, Canada".; Bull. Geol. Soc. Am., 74: 759-777.

5. Gramling, Wade L. and Hopkins, John G., 1974, "Aggregate Skid Resistance Relationship as Applied to Pennsylvania Aggregates", Bureau of Materials, Testing and Research, Pennsylvania Dept. of Transportation.

6. Grumau, D.L. and Michael, H.L., 1955, "Skid Characteristics of Pavement Surfaces in Indiana", Proceedings of the 41st Annual Purdue Road School, Extension Series No. 88, vol. 39, no. 5, pp. 73-96.

7. Hadley, D.W., 1961, "Alkali Reactivity of Carbonate Rocks - Expansion and Dedolomitization. Proc. Highway Research Board", 40: 562-474.

8. Herrin, M. and Goetz, W.H., 1954, "Effect of Aggregate Shape on Stability of Bituminous Mixes", Proceedings of Thirty-Third Annual Meeting of the Highway Research Board, vol. 33, pp. 293-308.

9. Lounsbury, R.W. and West, T.R., 1965, "Petrography of some Indiana Aggregates in Relation to their Engineering Properties". Proc. 16th Annual Highway Geol. Symposium, pp. 24-29.

10. Mather, K. and Mather, B., 1950, "Method of Petrographic Examinations of Aggregates for Concrete".; Proc. Am. Soc. Testing Mats. 50: 1288-1312.

11. Mielenz, R.C., 1946, "Petrographic Examination of Concrete Aggregates"; Bull. Geol. Soc. Am. 57: 309-318.

12. Mielenz, R.C., 1954, "Petrographic Examination of Concrete Aggregates". Proc. Am. Soc. Testing Mats. 54: 1188-1218.

13. Mielenz, R.C., 1966, "Petrographic Examination". Am. Soc. Testing Mats. Special Technical Publ. (STP) no. 169-A, pp. 381-403.

14. Mielenz, R.C. and Witte, L.P., 1948, "Tests Used by the Bureau of Reclamation for Identifying Reactive Concrete Aggregates". Proc. Am. Soc. Testing Mats. 48: 1071-1096.

15. Patty, Tom S., 1973, "Accelerated Polish Test for Coarse Aggregates", Proceedings of the 24th Annual Highway Geology Symposium", Wyoming State Highway Department, 13 pages.

16. Patty, Tom S., 1975, "Petrography as Related to Potential Skid Resistance of Paving Aggregates used on Texas Highway Projects", pp. 127-134.

17. Rice, J.M., 1979, "Seasonal Variations in Pavement Skid Resistance", Public Roads, March vol. 40, no. 4, pp. 160-166.

18. Ryell, J., Corkill, J.T., and Musgrove, G.R., 1979, "Skid Resistance of Bituminous Pavement Test Sections on Highway 401, Toronto By-Pass", Presented at the 58th Annual Meeting of the Transportation Research Board, Washington, D.C., 28 pages.

19. Sherwood, W.C. and Mahone, D.C., 1971, "Predetermining the Polish Resistance of Limestone Aggregates", Virginia Highway Research Council, Univ. of Virginia, Bull. No. 79, 10 pages.

20. Shupe, J.W. and Goetz, W.H., 1958, "A Laboratory Method for Determining the Skidding Resistance of Bituminous Paving Mixtures", Proceedings of the American Society for Testing Materials, vol. 58, pp. 1-24.

21. Shupe, J.W. and Goetz, W.H., 1960, "A Laboratory Investigation of Pavement Slipperiness", Bull. 219, Highway Research Board, Washington, D.C., pp. 56-73.

22. Shupe, J.W. and Lounsbury, R.W., 1959, "Polishing Characteristics of Mineral Aggregates", Proceedings of the First International Skid Prevention Conference, pp. 509-537.

23. Stephens, J.E. and Goetz, W.H., 1960, "Designing Fine Bituminous Mixtures for High Skid Resistance", Highway Research Board Proceedings, vol. 39, pp. 173-190.

24. Sweet, H.S. and Woods, K.B., 1942, "A Study of Chert as a Deleterious Constituent in Aggregates". Engineering Bull. of Purdue University: 26, no. 5, 111 pages.

25. Van Till, C.J., Carr, B.J., Vallerga, B.A., and Hilliard, J.M., 1976, "Guidelines for Skid-Resistant Highway Pavement Surfaces", Research Results Digest 89, November.

26. West, T.R. and Johnson, R.B., 1966, "Analysis of Textural and Physical Factors Contributing to Abrasion Resistance of Some Indiana Carbonate Aggregates". Proc. Indiana Acad. Sci. 75: 153-162.

27. West, T.R., 1969, "Textural Analysis and Engineering Testing of Degrading Base Coarse Aggregates", Proc. 7th Annual Engr. Geol. and Soils Engr. Symposium, pp. 247-270.

28. West, T.R., Johnson, R.B., and Smith, N.M., 1970, "Tests for Evaluating Degradation of Base Coarse Aggregates". SBN 309-01885-4, National Cooperative Highway Research Program (NCHRP), Report 93, Highway Research Board, Washington, D.C. 92 pages.

29. West, T.R. and Okagbue, C.O., 1980, "Grain Size Analysis and Petrographic Examination of a Gravel Deposit Relative to Engineering Quality"; North-central section, Geol. Soc. of America, Abstracts of Programs, p. 260.

GEOPHYSICAL INVESTIGATION

PROPOSED I-664

TUNNEL BRIDGE ROUTE

HAMPTON ROADS CROSSING

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GEOPHYSICAL INVESTIGATIONS OF HAMPTON ROADS

FOR CROSSING OF ROUTE I-664

The paper deals with an account of a geophysical and sediment sampling project conducted for a proposed tunnel/trestle crossing of Hampton Roads, Virginia between Newport and Portsmouth. The investigation was sponsored by the Virginia Department of Highways and Transportation.

The purpose of the studies was to explore the nature of the marine sediments in the areas proposed for possible crossing routes. Geophysical studies included a preliminary investigation in 1972, a detailed survey of the area proposed for the tunnel portion in the fall of 1977 and a third detailed study along the swath proposed for the trestle construction.

The geophysical survey consisted of running continuous seismic reflection profiles throughout the study areas, selected seismic refraction and velocity profiles, and taking numerous sediment core samples.

The geophysical results combined with the core samples made it possible to delineate an old buried river channel which crosses the proposed route, areas of firm sediments, shallow mud patches and regional structure of the marine sediments and stratigraphy.

Presentation will include plan maps of the survey areas, location of seismic lines and core holes, marine structure map of subbottom geologic features and selected profiles.

The geophysical investigation was conducted by the staff of Ocean/Seismic/Survey, Inc. of Norwood, New Jersey, a company with a new name, but direct successors to the Ocean Services Division of Alpine Geophysical Associates, Inc.

INTRODUCTION

Engineering designs for a tunnel under a ship channel and a trestle resting on the sea floor require knowledge of the sediment layering and geologic structures that exist below the sea bed. Such is the case for the proposed highway I-664 to be constructed across a body of water called the Hampton Roads, extending from the southern tip of Newport News, Virginia to Suffolk County. The crossing is to consist of a tunnel under the Hampton Roads Ship Channel with an entrance and an exit on man made islands, and a trestle extending across the longer portion of open water in the Roads. The Route I-664 Tunnel-Trestle Crossing is a project planned and engineered by the Virginia Department of Highways and Transportation.

This paper is an account of selected highlights of a combined geophysical survey and conventional boring program conducted in the area of the proposed crossing of Route I-664 across the Hampton Roads. Three separate geophysical surveys were conducted in the Hampton Roads area; the first in early spring of 1972 to obtain geophysical reconnaissance data which would assist in the selection of a route; the second in the fall of 1977 encompassing the area of the tunnel crossing and the two proposed man-made islands; the third in the fall of 1978 pertaining to the placement and foundation of the trestle over the shallow open body of water of Hampton Roads.

GEOGRAPHIC SETTING

Hampton Roads is a body of water (Roads from Roadstead referring to a relatively sheltered anchorage) lying between the southern boundaries of the Chesapeake Bay to the east and the James River to the west. The Roads connects the city of Newport News to the north with Suffolk County to the south. It spans a water distance of about 4.5 miles. The James River flows into the Hampton Roads from the northwest, then together with the Chesapeake Bay flows into the Atlantic Ocean between Cape Charles, Maryland and Cape Henry, Virginia.

Two similar bridge-tunnel thruways exist in Virginia, one the famous 17 mile Chesapeake Bay Bridge-Tunnel having two tunnels and connecting Cape Charles to the northwest shore of Virginia Beach, the other between the city of Hampton and Norfolk.

The third proposed bridge-tunnel, I-664, will proceed from the southern tip of Newport News across Hampton Roads and terminate on the north shore of Suffolk County, a short distance east of Tidewater Community College, Figure 1.

GEOPHYSICAL LAYOUT

The areas of interest for the tunnel and trestle surveys include a swath about two thousand feet wide extending from Newport News heading south-southeast for about a mile then bending to the south-southwest. The study also included much of the area between the proposed route and the western dike of the Craney Island landfill.

The geophysical survey consisted in running a series of closely-spaced seismic reflection profiles (including continuous bathymetric recordings) at intersecting orientations both along the route swath and in the Craney Island area. Positioning of the survey boat along pre-determined track lines was accomplished by means of an electronic range-range system, the Motorola Mini-Ranger III, Figures 2 and 3.

A deep conventional coring program was in progress simultaneous with the geophysical survey. Cores were taken along the center line of the proposed route at a spacing of 500 feet. Additional cores were taken in the area between the southern portion of the route and Craney Island.

OBJECTIVES

The objectives of the geophysical survey combined with a conventional coring program were 1) to detect and delineate patterns of geologic layering in the subbottom and identify their contents, 2) to determine the types of geological structures along the proposed route and adjacent areas and 3) to determine the possible effect of the nearby Craney Island on the foundation of the Trestle.

Selected topics included in this paper are 1) brief discussion of seismic velocity measurements, 2) geophysical structures pertaining to the overall area, 3) the development of a center-line profile and its associated geophysical features and 4) the relationship of the Trestle route to the Craney Island Dump Site.

SEISMIC VELOCITIES

Seismic velocity measurements were made at selected bore holes, namely B-150, B-158, B-170 and BC-5. Vertical up-hole travel times of seismic waves were measured by an SIE twelve-channel refraction system. Twelve hydrophones were placed on the sea floor, with the first detector within a few feet of the top of the core hole. Other detectors were placed at forty-foot spacing along a line extending away from the holes. Multiple detectors were used to be certain that the seismic arrival used for velocity evaluations was that of the seismic wave front and not an accidentally-caused noise signal. A wave front travelling from a deep source would reach the first few detectors at almost the same time.

Source of seismic energy was dynamite of sixty percent rating. For the deeper shots, a full one-half pound stick was used, but half-sticks and one-third sticks were used for progressively shallower shots. All shots produced arrival signals sufficiently sharp to read within one-half millisecond.

Electric blasting caps used for these tests were high grade seismic blasting caps of zero time delay. A number of these caps were tested independently by a detecting device to measure any possible time delay. No time delay was observed in these independent tests.

Shots were made generally at ten or twenty-foot intervals. The first shot in a test hole was made at the bottom of the core hole. The second shot was ten feet above the first and so on up to the top of the hole. Ten-foot intervals was the planned spacing, but occasionally a partially collapsed hole would prevent the close spacing.

The average vertical seismic velocities observed in holes B-150, B-158 and B-170 ranged between 2483 and 4400 feet-per-second. Interval velocities calculate up to 7500 feet-per-second. The faster values occur at depths of 120 feet and deeper. Average velocities for Hole No. BC-5 ranged between 4643 and 5395 feet-per-second, with interval velocities up to 8100 feet-per-second. The only significant pattern deducible from the observed velocity values was the erratic change from a relatively high velocity to a lower velocity below the high valued layer. Such changes occurred frequently in any given column of measurements.

GEOPHYSICAL STRUCTURES

Old James River

The most prominent geological structure, as interpreted from the seismic reflection profiles, is an old buried river channel which cuts across both the Trestle route and the Craney Island survey areas, **Figure 4.**

An outline of the buried river channel is illustrated on the Structure Plan Map, which encompasses the survey areas of the Trestle and Craney Island. The trend of the buried river channel is approximately along a northwest-southeast line. Because of its proximity to, and alignment with, the present-day James River, this buried river channel might be referred to as the Old James River Channel. Although the present-day James River flows into, and apparently terminates at the Hampton Roads, the buried river channel cuts across Hampton Roads and continues towards the Craney Island dike area. The trend of its southern boundary indicates that the buried channel probably continues through and under a large portion of Craney Island. The width of the buried channel in the area where it traverses the Trestle route, as illustrated on the Composite Profile, is at least 4800 feet, but increases somewhat east of the Trestle route.

Buried Channel

The area called the Old James River Channel is interpreted as a river-like channel, brought about by erosion, which probably occurred at a geological time when the sea level was considerably lower than it is today. The channel, based on cores taken within its boundaries, must have ranged in depth between about 100 and 120 feet below the present-day water level. It was subsequently filled with soft sediments classified according to the core logs as soft silts, soft clays and loose sands. Most of the sediments within the old channel boundaries have relatively low blow counts, and in the middle section, zero blow counts as deep as 120 feet. The soft sediments within the channel are generally uniformly soft horizontally. The stiffer sediments within the channel boundaries are found at depths somewhat greater than 100 feet below water level.

Several shallow seismic reflections appear on the 3.5 Khz records and indicate approximately horizontal bedding of various types of soft sediments, too insensitive to be detected by blow counts. The seismic absorption signal reflector, i.e. the "mud reflector" indicates the depth at which the seismic signal is absorbed and no further seismic energy is reflected from the deeper layers. **Figure 6.**

Seismic signals bordering the channel boundary show a pattern of an erosional surface sloping gradually downward into the channel, but terminating at the beginning of the "mud reflector". The erosional surface probably continues downward following a curved path to the bottom of the channel. Such erosional surfaces are identifiable on some portions of the seismic records made along the north boundary of the buried channel. A trace has been plotted on the Structure Plan Map to represent a former land surface along which an erosional channel bank began to form. On the north side of the buried channel, the distance between the beginning of the channel bank erosional line and the start of the channel mud boundary, ranges between 200 and 300 feet. The north bank erosional surface line is not recognizable everywhere on the seismic records.

The erosional surface trace of the south bank is more clearly defined and traceable throughout the seismic lines within both the Craney Island and the Trestle survey areas as outlined on the Structure Plan Map. The trace is relatively straight and trends along a southeast-northwest lineation. In the vicinity of the Trestle's Center Line the bank erosional line curves somewhat more to the west.

Although the southern boundary of the buried channel has a uniform trend, the northern mud boundary curves easterly and somewhat northeasterly after it passes the Trestle survey area. A second line, labelled as "Inferred Channel Edge", designates the probable northern boundary of the buried channel. However, the gaseous mud obscured the layers in this area, so that its inference is based more on the orientation of the southern boundary line, and the fact that the general trend of the buried channel appears to be in a southeasterly direction.

The curved portion of the northern channel mud boundary may be explained by the presence of a low-lying area adjacent to the channel during the period of deposition when gaseous, soft sediments were deposited within the channel and over the adjacent low-lying terraces.

Channel Mud Boundary

The most continuous and readily identifiable feature of the seismic reflection records is a break or discontinuity in the recordings of subbottom geologic reflecting layers. There is a point or line where the subbottom layers cease to be recorded and only a shallow, poorly defined seismic reflection is recorded. The vertical break between the well-defined sediment reflections and the start of a concave upward reflection, as traced from one seismic profile to the next, defines a line which is referred to as the Channel Mud Boundary.

This outline is not the edge of the silty sediment, but only the point where the gas content of the sediment decreases sufficiently to prevent absorption of the sparker and 3.5 Khz signals. Published data (see references) indicate that a gas content of more than 12 percent will cause complete absorption of a high frequency signal. There may be little or no change in the apparent grain size or general sediment characteristic from one side of the boundary to the other in any given layer. Thickness of the absorbing layer does not even appear critical, only the gas content. The areas outlined on the Structure Plan Map are only more gaseous than those around them. On the records, the edges of the gaseous sediment show the same absorption on the upper surface. Thus, the areas are shown as inverted lobes on the final profiles, or are indicated by a broken line, with dots for the channel mud, and circles for the small mud patches outside the channel.

References:
Kaplan, J.R. ed, 1974. Natural gases in marine sediments. New York, Plenum Press, 324 pp.

Van Overeem, A.J.A., 1978. Shallow penetration, high-resolution subbottom profiling. Marine Geotechnology, Vol. 3, no. 1, pp 61-84.

Whalen, Thomas III, 1977. Acoustical penetration and shear strength in gas-charged sediment. Marine Geotechnology, Vol. 2, pp 147-159.

Shallow Gaseous Mud Patches

The Structure Plan Map shows outlines of areas labelled, "Shallow Gaseous Mud Patch". These areas are located, for the most part, to the south of the James River Channel Mud Boundary; a few overlap the boundary. Outlines of the patches are very irregular. The patches appear within both the Trestle and Craney survey areas.

These areas are singled out from specific seismic reflection recordings made along the closely spaced seismic survey track lines. The seismic pattern is that of a "mud reflector" which on most records occurs within a foot or two of the water bottom. This "mud reflector", like those characteristic of the buried channel, obscure any deeper seismic reflectors. However, these "mud reflectors" are generally of short length duration, in some places continuous, in other places spotty. The outlines illustrated on the Structure Plan Map are envelopes of linear sigments of seismic profiles where these "mud reflectors" appear on the seismic records.

The outlined areas are labelled "shallow gaseous mud patches" because they apparently do not represent a drastic change in sediment material, but rather are interpreted as normal sediment layers having a gaseous content sufficient to absorb and obscure deeper seismic reflections. Core BC-4 located on the edge of a mud patch, shows zero blow counts to a depth of 33 feet in clayey silt.

Furthermore, as can be seen from numerous seismic profiles, the subbottom geologic strata, although interrupted by the "mud reflectors", appear to be continuous through and under the segments where the "mud reflectors" appear on the seismic records. There is no way of determining from the seismic records how deep a series of sediment layers may be gasified.

The "shallow gaseous mud patches" are interpreted as areas where the sediments probably contain a certain quantity of gaseous substance or volume without necessarily signifying a change in sedimentation characteristic of a buried channel.

The large, shallow, gaseous mud patch located adjacent to and southeast of the Trestle area, contained several seismic profiles where an erosional surface appeared to slant slightly downwards, suggesting a subsidiary buried river channel. All of the seismic records were further checked to establish whether the "mud patch" might be better interpreted as a tributary buried channel leading into the buried channel. No definite case could be established for this interpretation. Rather a more logical interpretation, based on the recognizable seismic patterns, is that these patches probably represent old shallow creeks which were periodically covered with captured, stagnant channel waters.

CENTER LINE PROFILE - DEVELOPMENT

Extent

The Composite Profile extends along the Center Line of the proposed Trestle Route - South Island. Its boundaries are from near to the Suffolk shore line on the south to the South Island proposed for the southern exit of the tunnel section on the north. Its horizontal scale is 200 feet to the inch; vertical scale 20 feet to the inch. Figure 5.

The Center Line profile is referenced to Bore Hole locations between BH-171 on the south to BH-121 (of 1977) on the north, all of which are plotted along the top of the profile sheets.

Profile Development

The Profile represents an interpretation of subbottom geologic bedding and structures along the Center Line; an interpretation derived not from one source of information, but from the combined input of seismic reflections, seismic velocities, blow counts and geologic core logs - an interpretation which appears to best fit the combined input data.

Expanded Scale

The first stage of Profile Development consisted in re-plotting the Center Line on an expanded scale of 200 feet to the inch. The principal horizontal control is from the bore holes whose coordinates were accurately established and whose intervals were held close to 500 feet. Bore Hole No. 171 appears at the southern end of the Profile, while

Bore Hole No. 121, taken in the Fall of 177, appears at the northernmost end of the Profile Sheet.

The top horizontal line is referneced to Mean Low Water and labelled "zero" on the vertical depth scale.

The water bottom line is plotted from the bathymetric records of track lines run directly along the Center Line or closely parallel to it. Water depths are measured and corrected to Mean Low Water.

Geologic core logs were positioned along the profile and blow counts plotted in terms of total blow count per 18 inches. Total count of blows over the 18-inch penetration was used to eliminate an excess of numbers appearing on the sheet, as well as to avoid a preference for using the blow count of the upper 12 inches rather than the blow count of the lower 12 inches.

Sediment descriptions of core contents were plotted with emphasis given to the major sediment material and further, but briefly, clarified by its significant subordinate characteristics. Descriptions serving only as duplications were not repeated.

Seismic Blow Count Correlations

The next major step in the development of a profile interpretation, which would be both geologically and seismically sound, was to correlate significant seismic reflections - and seismic velocities - with geologic bedding and blow count characteristics.

Seismic reflections generally represent changes in geologic strata and densities between and within sediment layers. In one case a seismic reflection will represent a bedding plane between two sediments; in another case the reflection may represent an erosional surface which may be conformable or unconformable with the underlying sediments. Each seismic reflection generally has several characteristic patterns which make it possible to interpret one reflection as bedding, another as a structure or fault, or for example, as an erosion surface which appears at the top of a series of truncated layers.

Attempts were made to correlate identifiable seismic reflections first with blow count changes, particularly large changes, then with geologic bedding changes based on the drillers' core descriptions. In some cases, relatively large blow counts in one core could be correlated with particular well-recorded strong seismic reflections, whose depths were calculated from observed velocities. In other cases where a seismic reflection is obviously continuous over a long distance, the observed blow counts to either side of the correlatable blow count would be either much different or lack relative contrast.

The velocity was re-evaluated, depth to the seismic reflector re-computed, and further attempts made to correlate this seismic reflection occurring at the specific depth with significant blow count changes at various bore holes along specific seismic reflections. Blow count changes from some cores correlated with some seismic reflections, while other blow counts lacked significant changes. Hence, only approximate correlations could be established between strong signalled seismic reflections and blow counts which at one hole were of high value and large contrast and at adjacent cores were of low value and minimal contrast.

Seismic - Geologic Correlation

The next attempt at correlation was between prominent seismic reflections and outstanding geologic bedding changes. Selected strong seismic reflections were computed in terms of depth below the bay bottom based on the velocities measured in near-by holes. Additional depths of the same reflectors were made on the basis of other velocity values observed in other holes. Comparisons were made of seismic reflectors computed for various observed velocities to determine the best and most extensive fit between depth of seismic reflectors and changes in geologic bedding. Tedious inspections were made between many prominent seismic reflectors and geologic beddings within each core and from one core to the next.

A Reasonable Interpretation

The Profile as presented, showing a series of seismic reflections, each one plotted at closely spaced horizontal intervals, is an interpretation which very reasonably correlates observed seismic reflections with geologic bedding and - to a limited extent - with blow counts.

Observed velocities used for the computation of seismic reflectors were drawn from all the bore holes where vertical velocities were evaluated. Controlling velocities were based on the values measured in Bore Hole No. BC-5 where observed velocities were higher than at BH-170, 158 and 150. However, prominent, relatively deep seismic reflections observed at BC-5 could be traced continuously both northward, eastward and westward to the Trestle area, and showed the same characteristic seismic signal pattern over most of the areas where the deeper reflectors were recorded. Furthermore, these seismic reflectors could also be closely correlated with long sections of geologic bedding. Various values of observed averaged velocities were used to correlate observed seismic reflections with the geologic bedding at various depths.

For the relatively deeper seismic reflections south of the buried channel and labelled B-S, C-S, D-S and E-S, an average velocity of 5300 feet per second was used in both the Craney area and the Trestle area. An observed velocity value of 5000 feet per second was used for seismic reflections labelled A-S of intermediate depth and found to correlate closely with the sand-silt interface over much of the southern portion of the Trestle Center Line..

For the shallow seismic reflections in the areas south of the buried channel, a velocity of 4200 feet per second was used to plot sediments from the water-sediment interface down to the prominent erosional surface No. 3. This value was determined on the basis of observed velocities and close correlation over large segments of seismic reflectors with geologic changes.

In the areas north of the buried channel, seismic velocities were drawn from measurements taken in BH-121. Deeper layers were computed from a velocity of 5100 feet per second, intermediate layers from a value of 4950 feet per second. The shallow sediments, particularly above and within the depth zone of the prominent erosional surface No. 1, were computed from a value of 3600 feet per second.

Velocity measurements in BH-170 appeared to be unexpectedly low by comparison with values obtained in BC-5. Closer examination of seismic records and geologic logs suggests a localized change in sedimentation. The core log of BH-170 contains a peat which suggests the presence of a former bog in a localized depression. Correlation of seismic reflections near the area of BH-170 was generally consistent with seismic reflections outside the area computed at the faster velocities observed in other holes.

COMPOSITE PROFILE - SEISMIC-GEOLOGIC FEATURES

General Features

The Composite Profile portrays several prominent features, (1) a sequence of geologic bedding extending from the proposed South Island on the north across Hampton Roads to the near-shore of Suffolk; (2) a buried river channel; (3) several erosion surfaces separating significant geologic changes; (4) a prominent parabolic-shaped erosional river bank with filled in sediments; (5) a thick deposit of cross-bedded sands and silts and (6) a small trench-like depression near the Suffolk shoreline. These features are best observed in the profile sheets themselves.

The general sequence of geologic bedding consists of relatively shallow, five to twenty feet of soft sediments, such as clays, silts, then fine sands to the southern portion of the Profile. Underlying these softer sediments are shallow erosional surfaces which separate the softer sediments from the more compact sediments. The deeper layers range from flat-lying to gently dipping inclinations and are extensive over long distances and wide lateral extents.

North Erosional Surface

The Composite Profile illustrates a seismic reflection north of the buried channel, labelled Erosional Surface One, which extends from the channel bank zone northward and slopes upward, becoming quite shallow in the vicinity of BH-121. This reflector has signal patterns characteristic of erosional surfaces and is very prominent on one of the north-south 3.5 Khz records made during the survey of the Fall of 1977, as well as on many of the 3.5 records made during the Fall 1978 survey. It ranges in depth from 23 feet to 5 feet below the bay bottom.

A second erosional surface, E-S 2, is labelled on some seismic lines and appears below erosional surface No. 1, but it is not clearly defined on all seismic records.

Both erosional surfaces, interpreted as such from the seismic reflections have geological significance. The upper erosional surface No. 1 correlates closely with the push line of the drill bits and separates the soft silts and sandy silts from the somewhat more firm silts and fine sands below. The slightly deeper erosional surface in turn occurs above the brownish gravelly sands and the older greenish gray more dense sands and silts.

Both erosional surfaces also show a relatively sharp downward plunge in the border area of the north bank of the buried channel brought about by a gradual wearing down of the bank area of the buried channel.

Seismic reflections dip uniformly and gently to the southeast indicating that the regional dip of both subbottom bedding and erosional surfaces is to the southeast. A prominent sequence of three deep seismic reflectors, labelled as CN-3, occurs in the vicinity of the proposed South Island. Their seismic pattern is one of three closely spaced parallel reflectors which plot at depths around 140 feet below Mean Low Water. The triple sequence of bedding dips to the southeast and is not evident on the seismic records south of the buried channel, nor amongst the deeper horizons of the Craney Island area. The deep pattern of seismic reflectors to the south of the buried channel suggests a sequence of sediments different from the sequence found to the north of the buried channel. The deep sediment sequence to the north of the channel is probably of older age than those to the south.

The deep sequence of geologic bedding consists of predominantly greenish gray sands and silts, combinations of both and some occasional interbedded compact clays.

Buried Channel Area

The structural and plan features of the James River Buried Channel have been discussed above under the section Geophysical Structures. The buried channel is identified first by an interruption of deep seismic reflections; second by an ever-present "mud reflector" occuring 10 to 20 feet below the water bottom; third, by the characteristic pattern of inclined erosional surfaces on the north side of the buried channel; fourth, on the southern boundary by a steeply inclined reflector interpreted as an erosional bank, labelled erosional surface No. 4; and fifth, an infilling of the bank area with subsequently deposited sediments of varied bedding inclinations. These features are recognizable on all the north-south seismic profiles, and on numerous transverse profiles.

Within the buried channel area, seismic reflections appear across most of the cross section, nearly horizontal or with gentle saucer-like orientations, and in some places gently pinch out.

Approximately twenty feet below the water-sediment interface, the "mud reflector" appears on the seismic records as a dull, low contrast broad signal signifying the depth below which no further seismic reflections are recorded.

The Push Line is deep across the buried river channel extending down to about 125 feet below Mean Low Water.

The sediments below the Push Line are generally compact, have reasonably high blow counts and consist predominantly of sands, silty sands and silty clays.

Erosional Bank Area

A prominent seismic feature appears on the Composite Profile in the vicinity of Bore Holes 156 and 157, and is interpreted as a parabolic-shaped erosional surface, labelled Erosional Surface No. 4.

Its depth is computed on the basis of deeper seismic reflectors, Nos. B-S and C-S which are traceable continuously from Bore Hole BC-5 northward and westward to the Center Line. Seismic reflectors B-S and C-S are clearly recorded and are truncated by the Erosional Surface No. 4 which cuts down and across these layers. The most consistent interpretation of seismic reflections, seismic velocities and geologic core logs favors computing the depth to Erosional Surface No. 4 to this locus.

Seismic reflections appearing above the Erosional Surface No. 4 indicate in-filling of the channel subsequent to its erosion process. The in-filling reflectors have seismic patterns typically characteristic of bedding which have been deposited in a depression.

The seismic signal, interpreted as Erosional Surface No. 4, has been interrupted by the seismic absorption signal just north of its trace on the profile, and is nowhere identifiable on the seismic records.

The relatively large blow counts of sediments appearing at elevations above the depth of the Erosional Surface No. 4 are explained by the theory that the sediments deposited in a deep channel depression would be compacted during the depositional process. Furthermore, they were probably deposited at a time when land elevation still remained well above the then existing shore line.

Erosional Surface No. 3

Erosional Surface No. 3 is a relatively shallow erosional surface. Approximately flat-lying and extends from the south mud boundary of the buried channel southward almost to the present-day shore line at Suffolk.

Seismically it occurs at or close to either the Push Line or areas of low to relatively higher blow counts. It is a very prominent seismic reflector which appears on most 3.5 Khz records.

An important feature of the Erosional Surface No. 3 is its extensiveness, being clearly traceable on seismic records not only along the Center Line, but laterally to both sides of the Center Line as is evident on the transverse seismic profiles. Furthermore, Erosional Surface No. 3 is traceable eastward into and through most of the Craney Island survey area up to the south boundary of the buried channel.

Depth to the Erosional Surface No. 3 is uniformly flat from the south mud boundary of the buried channel southward to the vicinity of BH-165 where a shallow depression exists. The uniform depth of the erosional surface ranges between 32 and 36 feet below water bottom to to a point southward in the vicinity of BH-168 where it grades upward to a depth of 17 feet below Mean Low Water. Seismic recordings of Erosional Surface No. 3 are consistently strong and contrasty from the south channel mud boundary to the vicinity of BH-169; south of which the seismic signal representing Erosional Surface No. 3 becomes faint or unidentifiable.

Cross-Bedding

Seismic reflections between Bore Holes 165 and 169 have patterns characteristic of cross-bedded sediments; reflections occur with saucer-like recordings and traces similar to the mathematical integral sign, i.e. an elongated "S". Such cross-bedded seismic patterns occur in the sediments between Erosional Surface No. 5, which occasionally truncates some of the cross-bedding. They also occur between BH-166 and 169 and upward to the water-sediment interface.

Sediment content of the cross-bedded areas include sands and silts and range through fine to coarse sand sizes; colors vary between gray and tan. The cross-bedded sands and silts range in thickness from a few feet to almost 30 feet between BH-164 and 169.

Cross-bedding is equally pronounced in the Craney Island survey area and extends continuously from the Trestle area eastward almost to the buried channel southern boundary line. Within the Craney survey area the cross-bedding continues in thickness and in some places it is extremely well-definied on the 3.5 Khz seismic records.

Depression at BH-170

Based on the core log contents of BH-170, an anomalous depression exists at an around the area of the bore hole. Except for a top thin layer of fine sand, the upper 50 feet contains mostly a gray clay, relatively soft with a low blow count of 0 to 6. It is anomalous in the fact that bore holes to the north and south do not contain this same soft clay deposit. The clays could extend laterally to the east and west. The depression is possibly a small channel which has subsequently been filled with the soft clay. Seismic indications of the Erosional Surface No. 3 are lacking at this location. The geologic process that created the depression probably also eroded through the Erosional Surface No. 3. It is because of this soft clay and localized geologic condition that seismic velocities observed in BH-170 were not used for sediment depths in areas adjacent to the depression.

Deep Strata

The patterns of the deeper seismic reflections appearing on the records south of the buried channel are unlike the patterns of deeper seismic reflections to the north of the buried channel. Four of the deeper seismic reflections, A-S, B-S, C-S and D-S slope very gradually upward from the south-southwest to the north-northeast along the Center Line. Seismic reflector, A-S correlates closely with portions of the geologic interface between the fine gray sand and gray silt. The deeper seismic reflectors, B-S, C-S and D-S probably represent density changes and in places sediment changes. These seismic reflectors occasionally correlate with blow count changes.

The group of seismic reflectors A-S through D-S are traceable from one seismic line to the next, are continuous and interpreted as representing substantial geologic layering of compact sediments. Horizontal A-S slopes upward to the north coming to within a few feet of the Erosional Surface No. 3. All three deeper reflectors continue northward to the bank Erosional Surface No. 4, where they are truncated. Extrapolation of seismic layers A, B, C and D-S to the north of the buried channel indicates that they become very thin or pinch out entirely. The triple sequence layers, CN-3 to the north, when projected southward, would probably underlie the reflectors observed to the south of the buried channel.

CRANEY ISLAND AREA - GEOPHYSICAL STRUCTURES

Significant seismic patterns within the Craney Island survey include, (1) Erosional Surface No. 3 sloping from its average horizontal locus sharply downward into the channel area and being terminated because of the seismic absorption signal (mud reflector); (2) the seismic absorption signal which occurs over the entire span of the area interpreted as the buried channel and which is generally observed on the seismic records at a depth of 10 to 20 feet below the water-sediment interface; (3) a second Erosional Surface No. 5, shallower than Erosional Surface No. 3 and appearing generally as the truncating surface of the cross-bedded sediments, slopes gradually downward to the level of intersection with Erosional Surface No. 3, where it changes to a sharper downward slope into the channel area, cf Fix 423.1.

Two Sediment Structures

The group of seismic reflections show two distinct areas of geological sequence. One is the area of the buried river channel, interpreted as such from the seismic reflections and further confirmed by selected bore holes, BC-1 and BC-2, both of which are located within the buried channel area. Bore hole, BC-2, positioned approximately in the middle of the channel area, contains the same soft silt with zero blow count down to a depth of 130 feet below Mean Low Water before encountering the firmer gray medium and fine sands. Bore hole BC-1, positioned about 2000 feet north of the buried channel's center line, is an almost exact duplicate of BC-2, showing soft silts with zero blow count to a depth of 105 feet. The firmer older green fine sands lie below the channel bottom gray sands.

Second Geologic Sequence

The second geologic sequence is the entire subbottom area south of the buried channel's sourthern bank erosional boundary extending both eastward to the southwest tip of the Craney West Dike and westward to the Trestle's center line. The geologic sequence observed within the Trestle area south of the buried channel is similar and identical in many respects to the second geologic sequence found in the Craney survey area.

The seismic reflection patterns of the Craney second sequence are essentially the same. The Erosional Surface No. 3 is traceable everywhere from the Trestle area to the erosional bank boundary of the buried channel. It is the locus of the break line of the Erosional Surface No. 3 that defines the channel's bank erosional line. Deep seismic reflectors below the Erosional Surface No. 3 are generally traceable over much of the second sequence area and are reflections within or corresponding to firm geologic bedding.

Sediments above the Erosional Surface No. 3 are of particular interest in that there is a vast expanse of seismic reflections that reveal the presence of thick and extensive cross-bedding. The cross-bedded seismic patterns are illustrated on many of the Craney seismic lines, eg. between fixes 422.2 and 423.2 At this particular location, the cross-bedded sediments rest upon the Erosional Surface No. 3 and are generally truncated at the top by Erosional Surface No. 5. Thickness of cross-bedded sediments bounded by Erosional Surfaces Nos. 3 and 5 range up to about 20 feet.

Cross-bedding is not confined to the zone between Erosional Surfaces Nos. 3 and 5. Elsewhere there are seismic patterns indicating cross-bedding such as at Fix 436.3 where it appears within 10 to 15 feet below the water-sediment interface.

Another prominent area of cross-bedding appears close to the southern shore in very shallow water, i.e. 5 feet, and probably indicates a massive supply of sand at and just below the bay bottom, cf Fix 366.2.

Bore hole BC-3 shows loose sediments, gray clayey silt down to a depth of 30 feet below bay bottom before changing to a more compact green fine sand. It is located near Fix 428.4 and apparently cuts through loose shallow sediments making up the marsh area bordering the channel embankment.

Bore hole, BC-4, located at Fix 198.2, near and south of the erosional bank line, shows soft gray clayey silt to a subbottom depth of about 20 feet, at which depth the core changes to gray fine sand with a blow count of 16 per 18 inches. The seismic profile shows the Erosional Surface No. 5 occuring at the same depth as the gray fine sand. The cross-bedding seismic pattern appears just below Erosional Surface No. 5

The soft shallow sediments, classified according to two core logs as soft gray clayey silts, extend shoreward and southward for about 1500 feet before pinching out upslope on Erosional Surface No. 5.

Craney Island Effect

If the Craney Island land disposal mass is to exert a pushing effect upon the sediments along the Trestle Route west of Craney Island, it is logical to assume that an immediate effect would be discernible close to the west dike of Craney Island. This immediate effect might be recognizable in two forms; first, a downward change in bottom topography immediately adjacent to the west dike, and second, an upward warping or bulging of sediments just west of the downward topographic trough. The cause of such a change would be due to a downward force of the Craney Island mass dragging the adjacent floor downward, thus creating a trough. Coupled with the vertically dragging force would be a horizontal westerly thrust causing the sediments adjacent to the trough to bulge upwards.

The seismic reflection records were closely examined to determine if there were any indications of a topographic trough adjacent to the west dike and further, of sediment bulges. None appeared on the seismic reflection records.

One minor bottom topographic trough appears at the northwest corner of Craney Island, but it is confined to the northwest corner and appears to be caused by fast flowing water currents around the northwest edge of the island.

The east-west trending seismic profiles show no prominent troughs or ridges that might be caused by downward dragging or lateral compression.

A second possible effect of the Craney Island mass upon the Trestle would be along an east-west line. The seismic and geologic evidence along such an east-west line clearly shows that the greater mass of subbottom sediments between the west dike and the Trestle contain firm compact sands, silts, some clays, some gravel. These beddings extend almost from the water-sediment interface downward through more than 100 feet of firm sediments. Of particular significance is the mass of cross-bedded sands and compact silts that cover a large portion of the shallow sediment areas say between -10 and -30 feet below water level and between 0 and -20 feet below bay bottom. The only relatively loose unstable material occurs along the shoulder of the south erosional bank and forms a tapering wedge of material from near to the west dike westward to the Trestle area, covering an expanse of about 1500 feet and ranging in thickness from about 30 feet to a pinch out.

A third possible effect of the Craney Island mass upon the Trestle would be along a northwest line parallel to the axis of the buried channel, where the buried channel sediments are predominantly soft clays and silts and extend downward to depths of 100 and 130 feet below Mean Low Water. These sediments, by virtue of their soft texture, probably have low shear values which are further lowered by the presence of gas; probably methane gas.

The soft sediments, if exerted upon by a massive lateral pushing force, would probably not respond as a solid irresistible mass, but rather would probably give and bulge up locally and close to the area of the applied lateral forces. It appears that lateral forces could not be transmitted any great distance through soft sediments with reduced shear characteristics because of the presence of gas.

CONCLUSIONS

(1) Any proposed route from Newport News south to Suffolk County must clearly cross the deep (120-130 ft.) buried river channel of some 4800 feet in width.

(2) Sediments within the buried river channel are soft and unable to support the face of a push rod to a depth of 120 to 130 feet.

(3) North and south of the buried river channel the shallow subbottom sediments ie. approximately 20 feet thick are also soft, but become firmer with depth.

(4) Craney Island Dump Site apparently lies almost entirely within the buried river channel. Increased deposition of dump materials on the island will likely create an equilibrium effect.

(5) The combined efforts of closely-spaced seismic reflection profiles and selective bore holes provides important data from which a geological picture of layering and structures can be pieced together to portray the nature of the subbottom.

ACKNOWLEDGEMENTS

This paper has been presented and revised for publication under the approval of Mr. George Meadors, Chief Geologist for the Virginia Department of Highways and Transportation.

The geophysical surveys 1977 and 1978 were conducted by the staff of Ocean/Seismic/Survey, Inc., formerly the Ocean Services Division of Alpine Geophysical Associates, Inc. and the 1972 survey by the staff of Alpine. The cores were taken by Girdler Foundation and Exploration Co. of St. Petersburg, Florida, and the core logs presented by Mr. Meadors for inclusion in this presentation.

FIGURE CAPTIONS

Figure 1	Location Map Routes/Seismic Surveys shows the area of route investigation, the selected proposed route, the boundaries of geophysical surveys of 1972, 1977 and 1978.
Figure 2	Trestle Route - Seismic Survey Plan Map Shows the tracks of all geophysical survey lines run within the swath of the proposed route of I-664. Numbers along track lines signify fix numbers in sequences of six decimals per fix number eg. Fix 119.1 - 119.6, 120.1 etc.
Figure 3	Craney Island - Seismic Survey Plan Map Track lines of geophysical survey lines run between the west dike of Craney Island westward to the proposed route.
Figure 4	Structure Plan Map Shows selected seismic lines for reference and outlines of significant geological structures including the James River Buried Channel, Channel Mud Boundaries, Old Channel Erosional Bank and Shallow Gaseous Mud Patches.
Figure 5	Center Line Composite Profile Showing prominent seismic reflections, core logs, and interpreted geophysical features. Presented in two sections extending from the Suffolk land mass across the Hampton Roads to Newport News.
Figure 5A	Legend for Soils Identification
Figure 6	Sample Seismic Reflection Profile Showing interpreted seismic reflections.

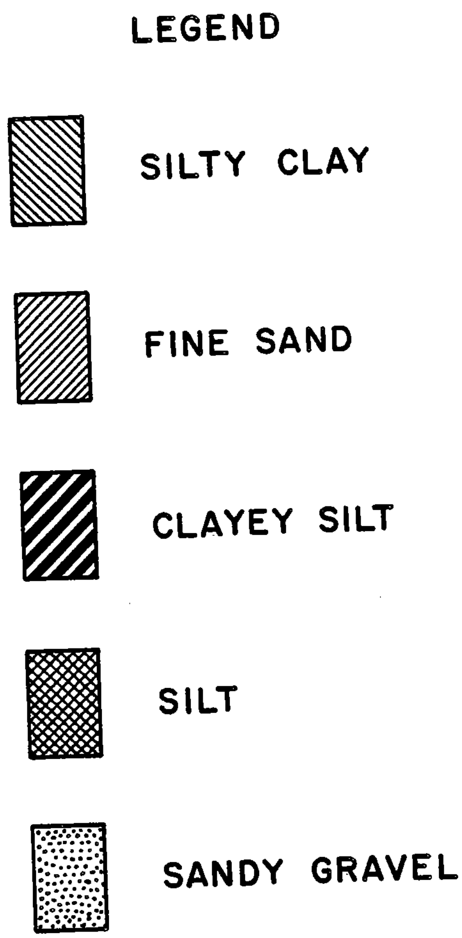
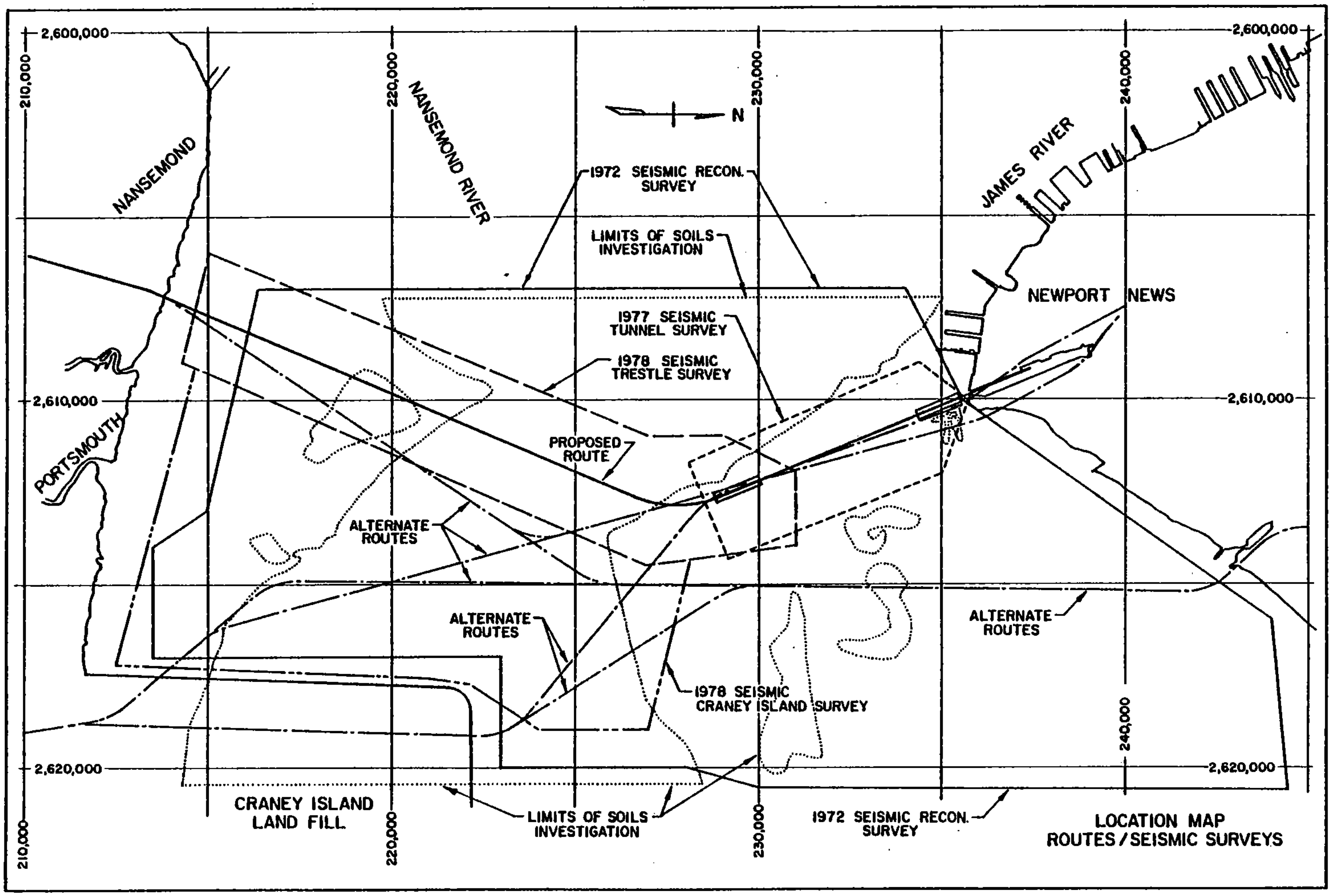


Figure 5A



2,600,000
210,000
220,000
230,000
240,000
2,610,000
2,620,000
N
NANSEMOND
NANSEMOND RIVER
JAMES RIVER
NEWPORT NEWS
PORTSMOUTH
1972 SEISMIC RECON. SURVEY
LIMITS OF SOILS INVESTIGATION
1977 SEISMIC TUNNEL SURVEY
1978 SEISMIC TRESTLE SURVEY
PROPOSED ROUTE
ALTERNATE ROUTES
ALTERNATE ROUTES
ALTERNATE ROUTES
1978 SEISMIC CRANEY ISLAND SURVEY
CRANEY ISLAND LAND FILL
LIMITS OF SOILS INVESTIGATION
1972 SEISMIC RECON. SURVEY
LOCATION MAP
ROUTES / SEISMIC SURVEYS

Figure 1

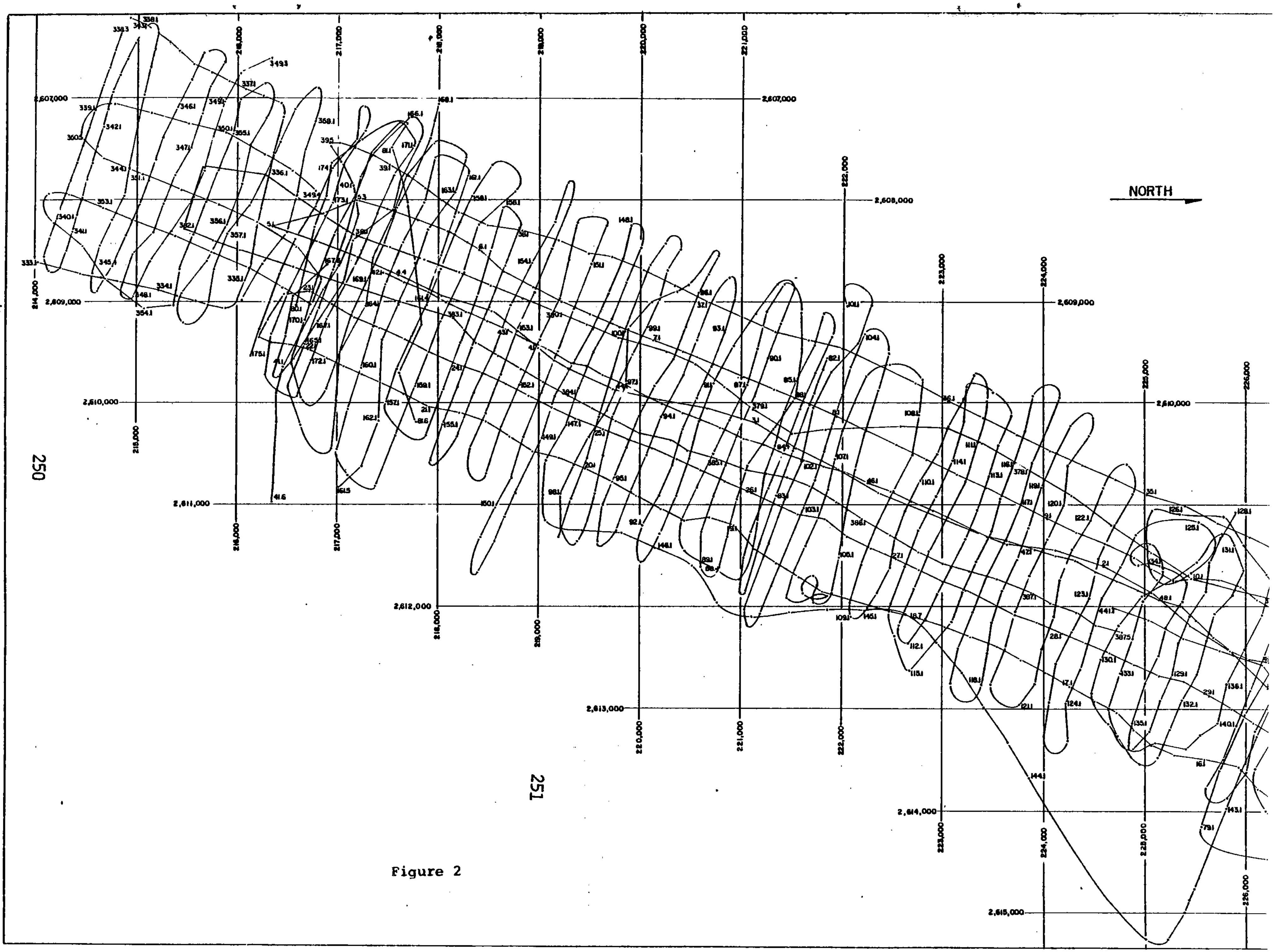


Figure 2

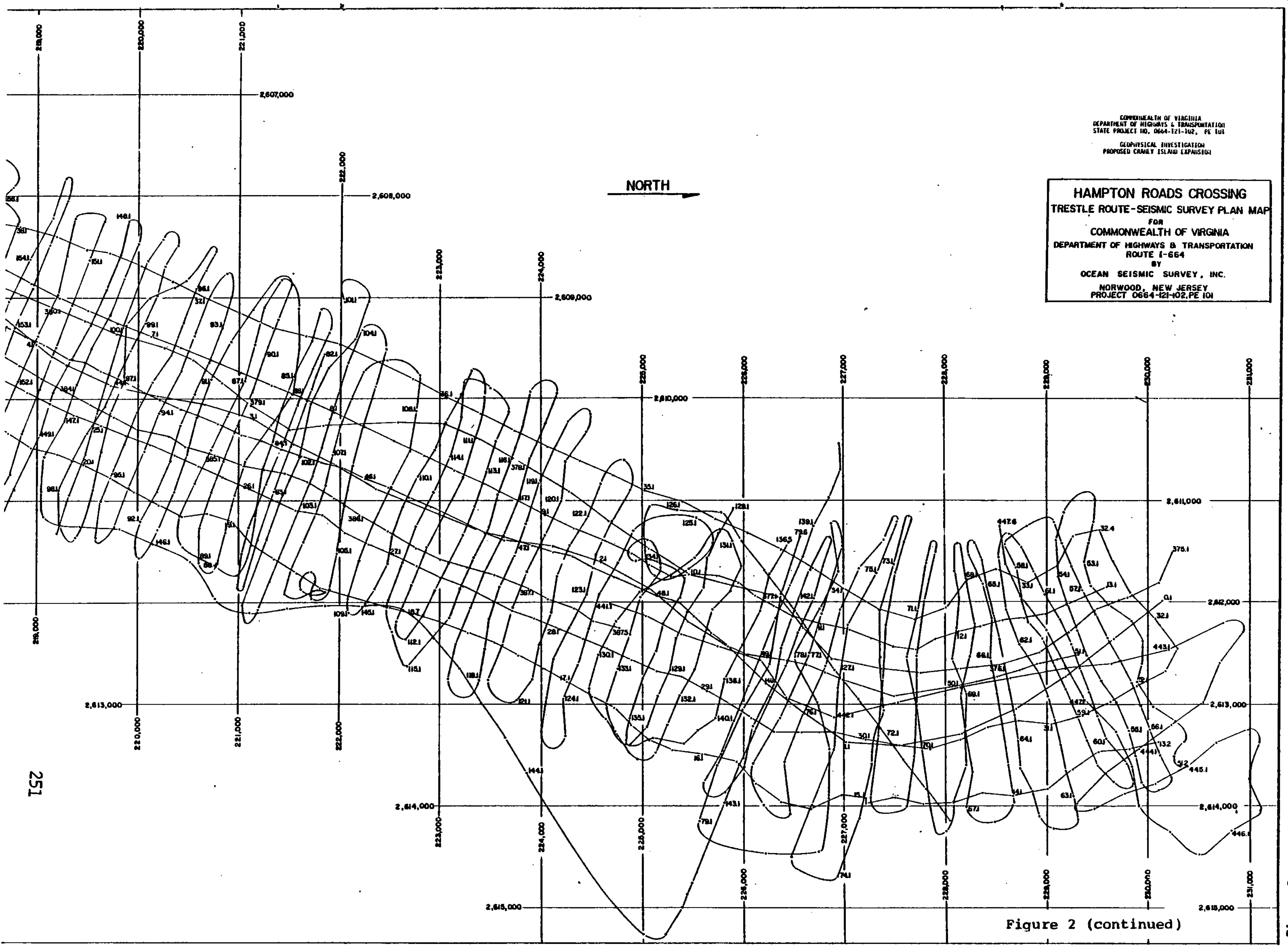


Figure 2 (continued)

Figure 2

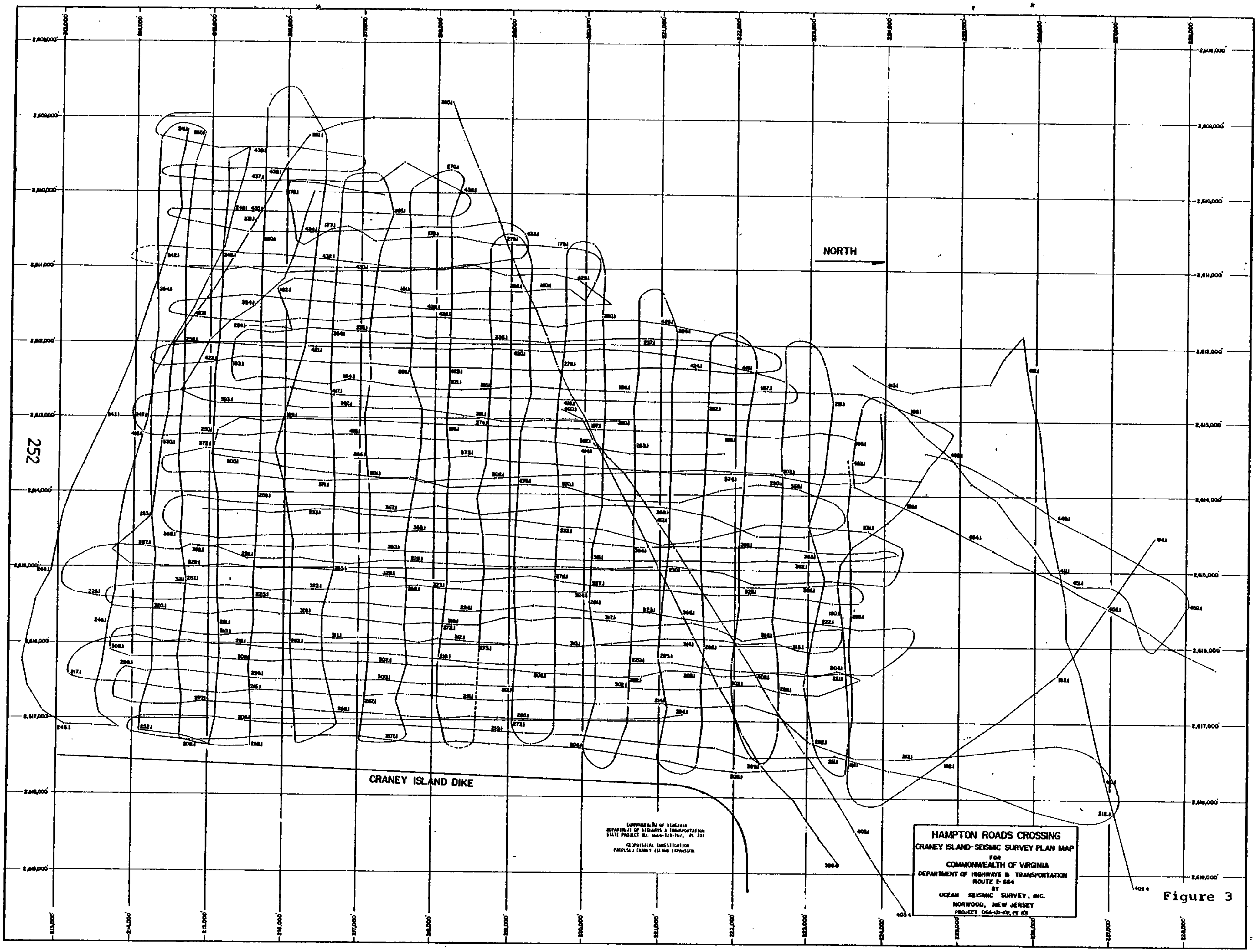
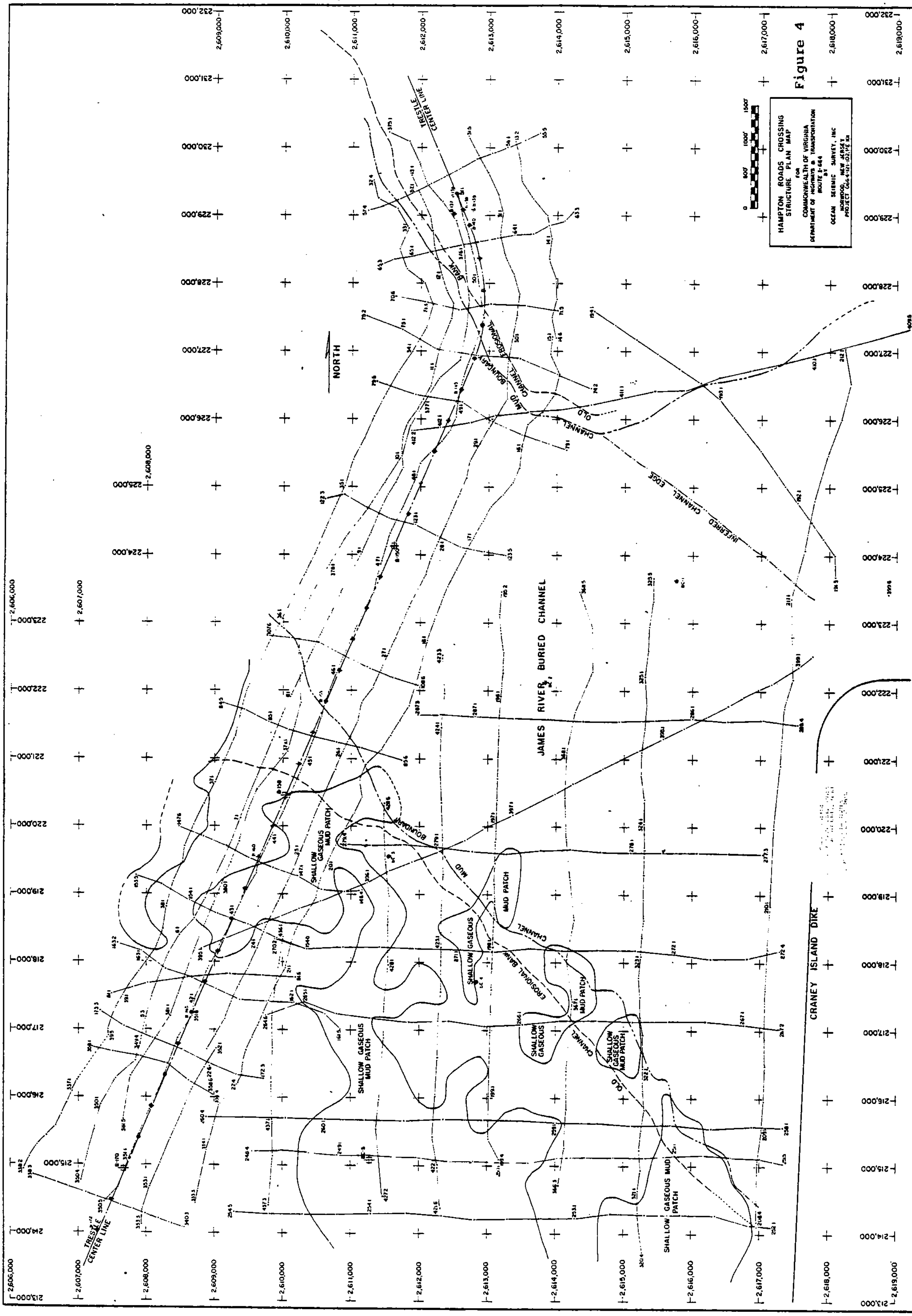

HAMPTON ROADS CROSSING
CRANEY ISLAND-SEISMIC SURVEY PLAN MAP
FOR
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS & TRANSPORTATION
ROUTE I-664
BY
OCEAN SEISMIC SURVEY, INC.
NORWOOD, NEW JERSEY
PROJECT 064-121-102, PE 101
NORTH
CRANEY ISLAND DIKE

Figure 3


HAMPTON ROADS CROSSING
STRUCTURE PLAN MAP
FOR
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS & TRANSPORTATION
ROUTE I-664
BY
OCEAN SEISMIC SURVEY, INC
NORWOOD, NEW JERSEY
PROJECT C664-121-102,PE 101
0 500' 1000' 1500'
NORTH
TRESTLE CENTER LINE
JAMES RIVER BURIED CHANNEL
INFERRED CHANNEL EDGE
OLD CHANNEL
MUD BOUNDARY
EROSIONAL CHANNEL BANK
SHALLOW GASEOUS MUD PATCH
MUD PATCH
CRANEY ISLAND DIKE
Figure 4

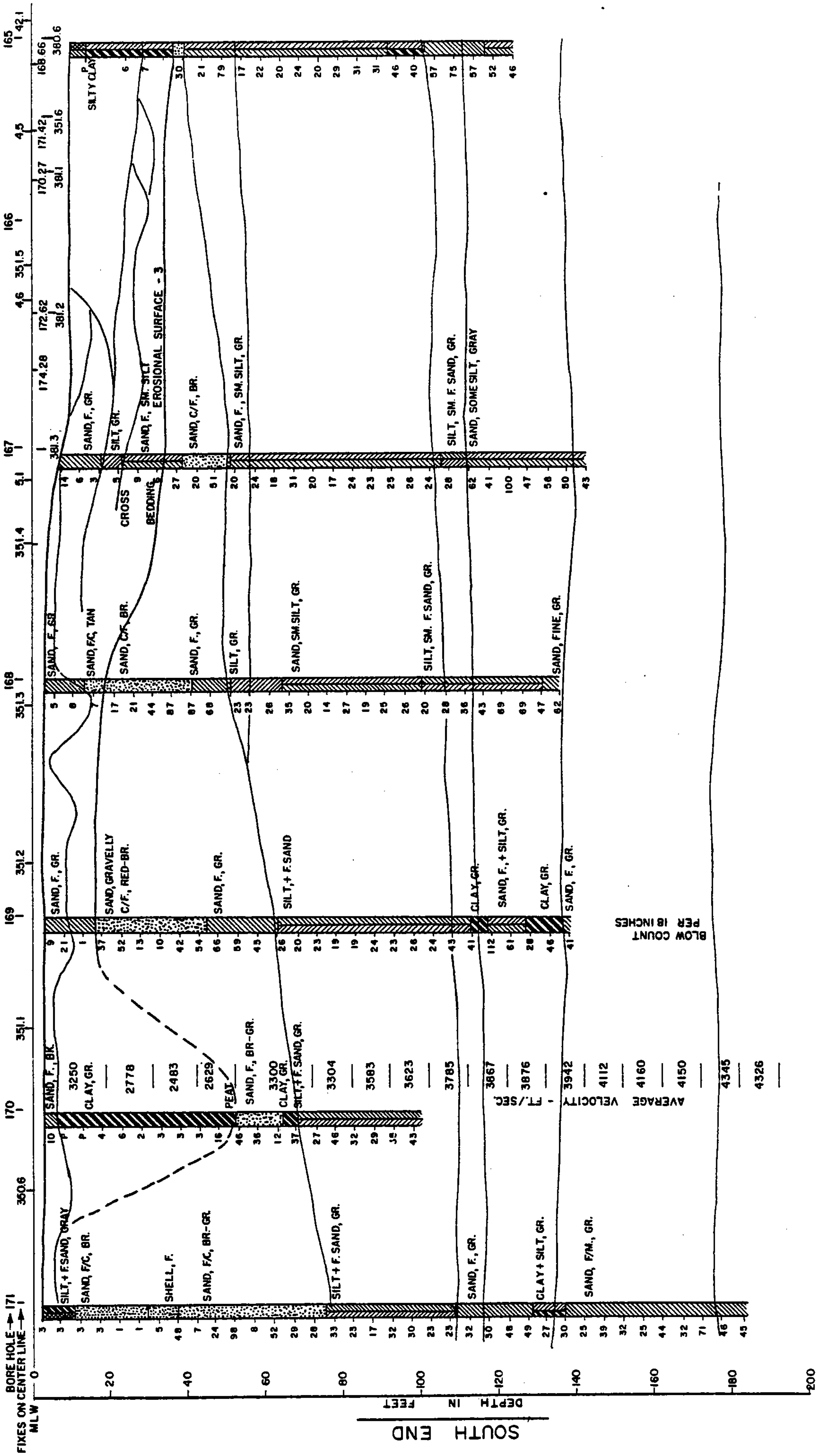
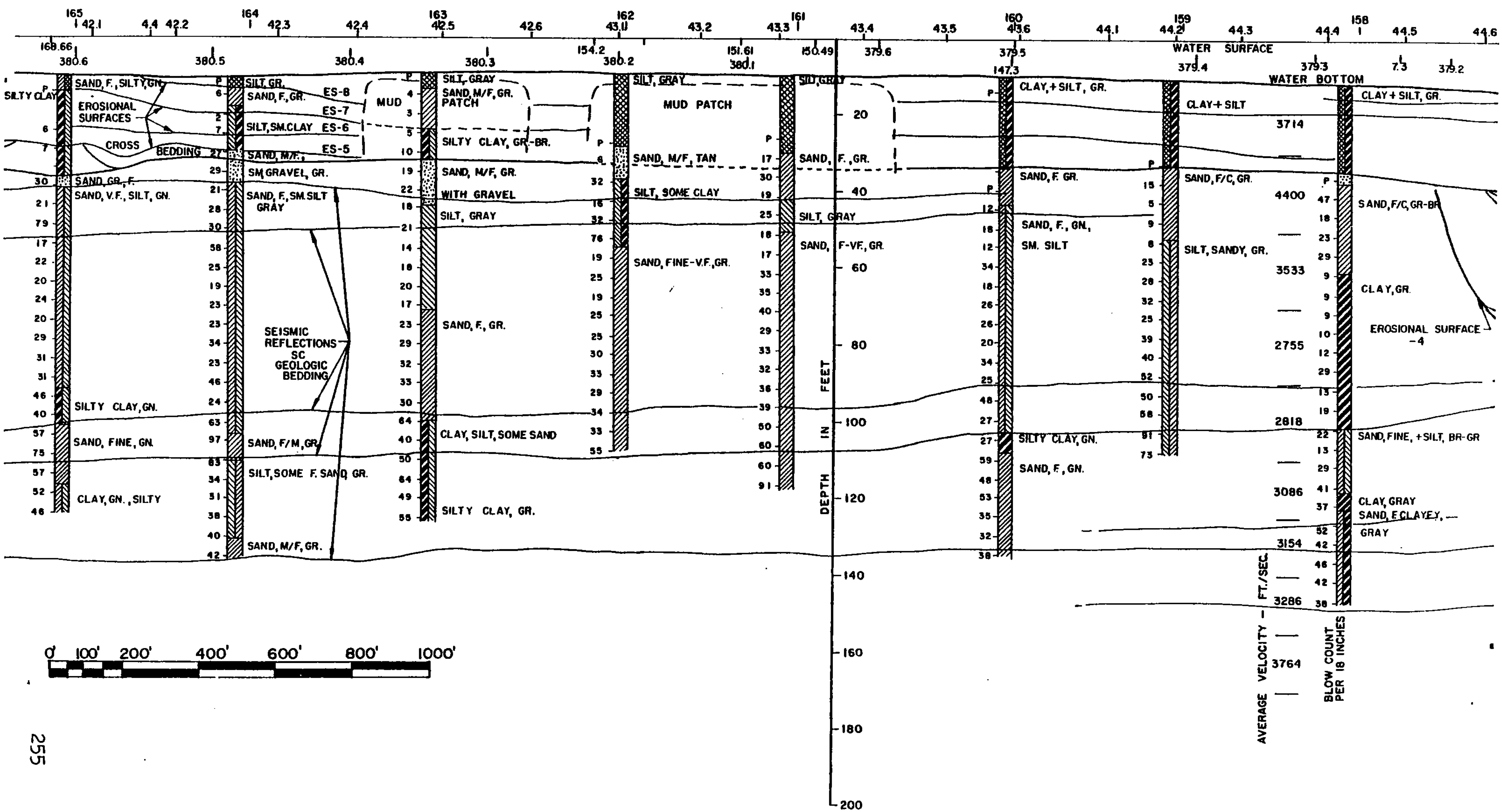

SOUTH END
FIXES ON CENTER LINE
BORE HOLE
MLW
DEPTH IN FEET
AVERAGE VELOCITY - FT./SEC.
BLOW COUNT PER 18 INCHES
EROSIONAL SURFACE - 3
CROSS
BEDDING
SILTY CLAY
SAND, F., GR.
SILT, GR.
SAND, C/F., BR.
SAND, F., SM. SILT, GR.
SILT, SM. F. SAND, GR.
SAND, SOME SILT, GRAY
SAND, F/C, TAN
SAND, SM. SILT, GR.
SAND, FINE, GR.
SAND, GRAVELLY C/F., RED-BR.
SILT, + F. SAND
CLAY, GR.
SAND, F., + SILT, GR.
SAND, F., BR.
PEAT
SAND, F., BR-GR.
SILT, + F. SAND, GR.
SILT, + F. SAND, GRAY
SAND, F/C, BR.
SHELL, F.
SAND, F/C, BR-GR.
SILT + F. SAND, GR.
CLAY + SILT, GR.
SAND, F/M., GR.
3250
2778
2483
2629
3300
3304
3583
3623
3785
3867
3876
3942
4112
4160
4150
4345
4326

Figure 5



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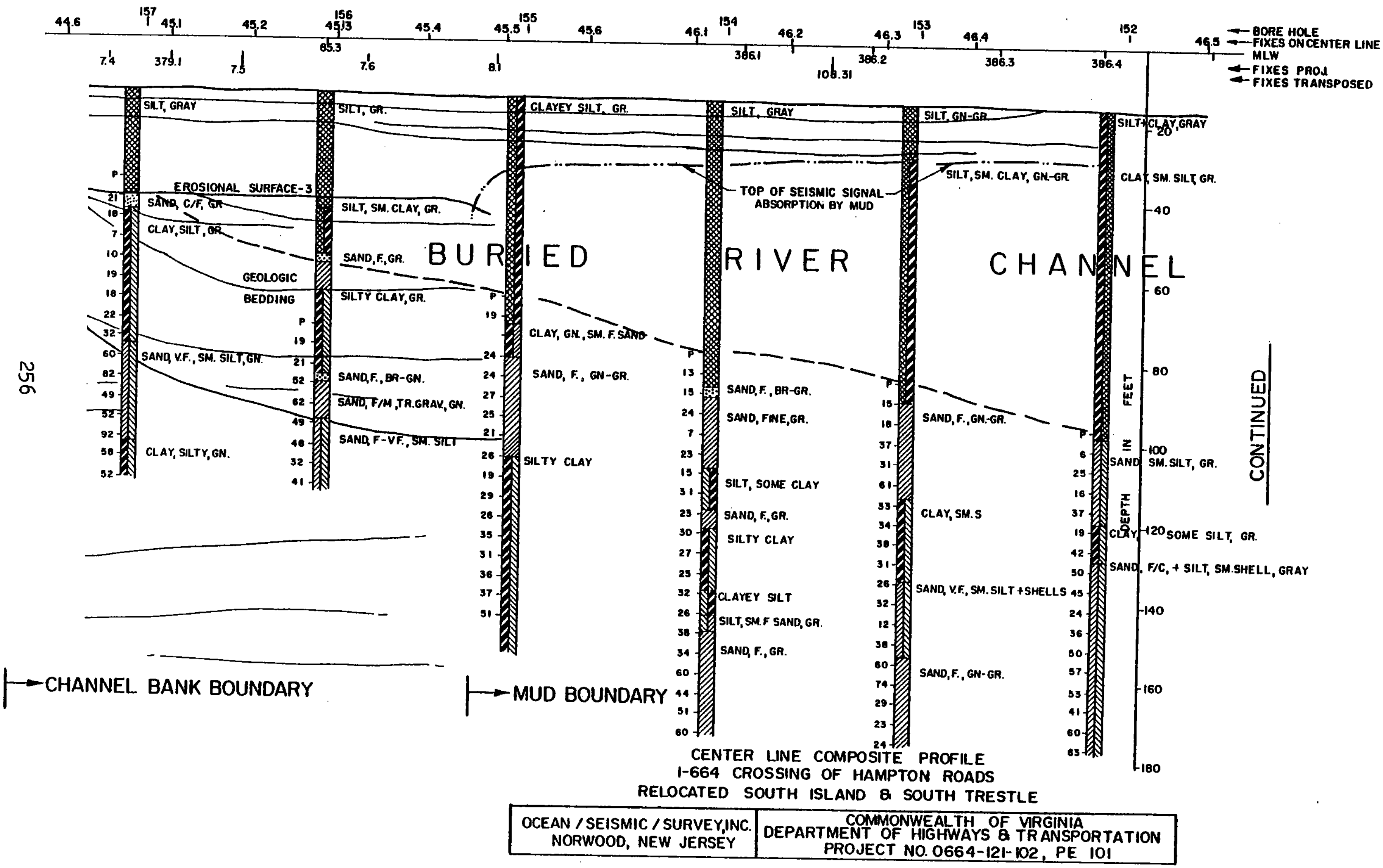

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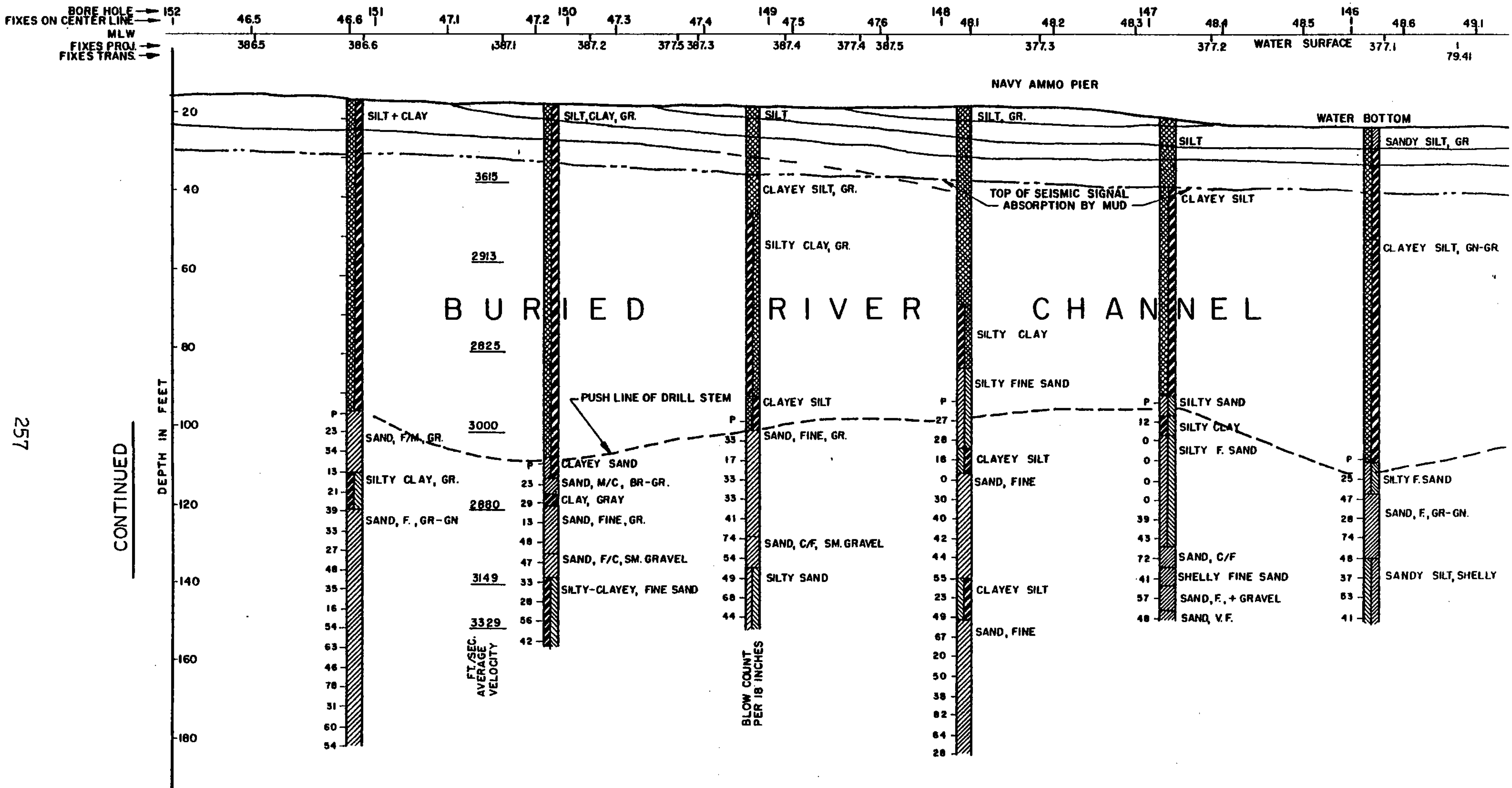

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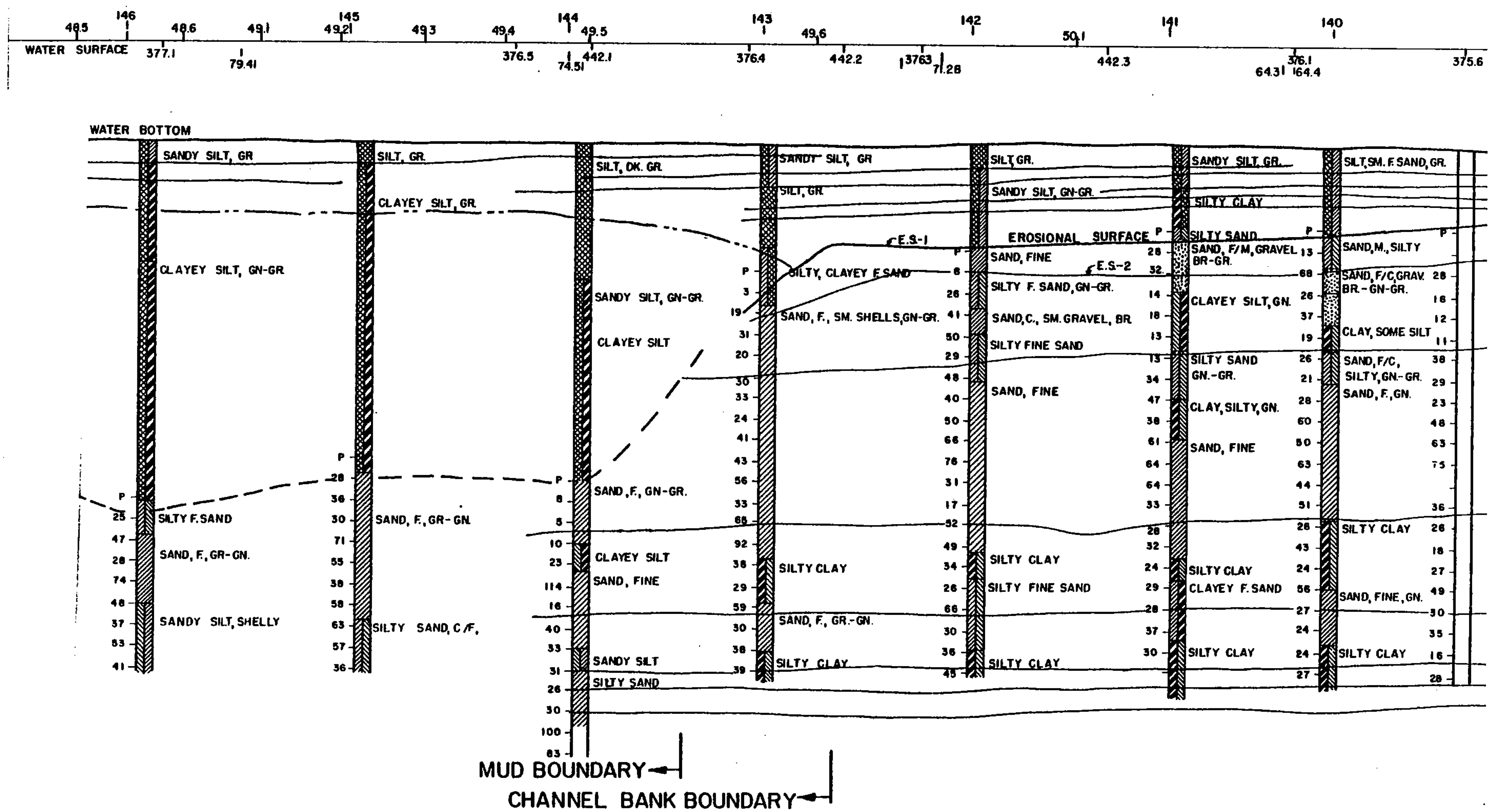

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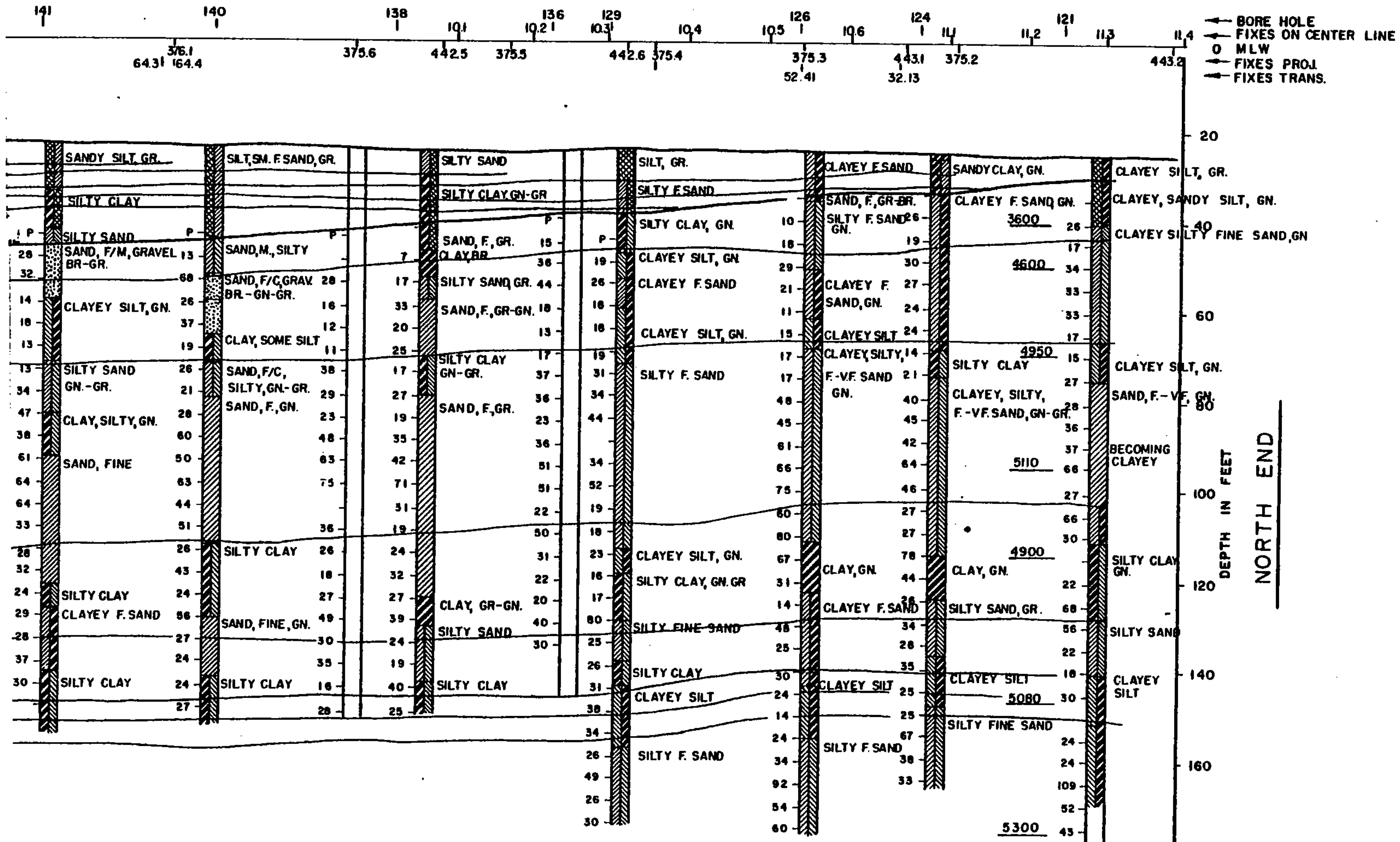
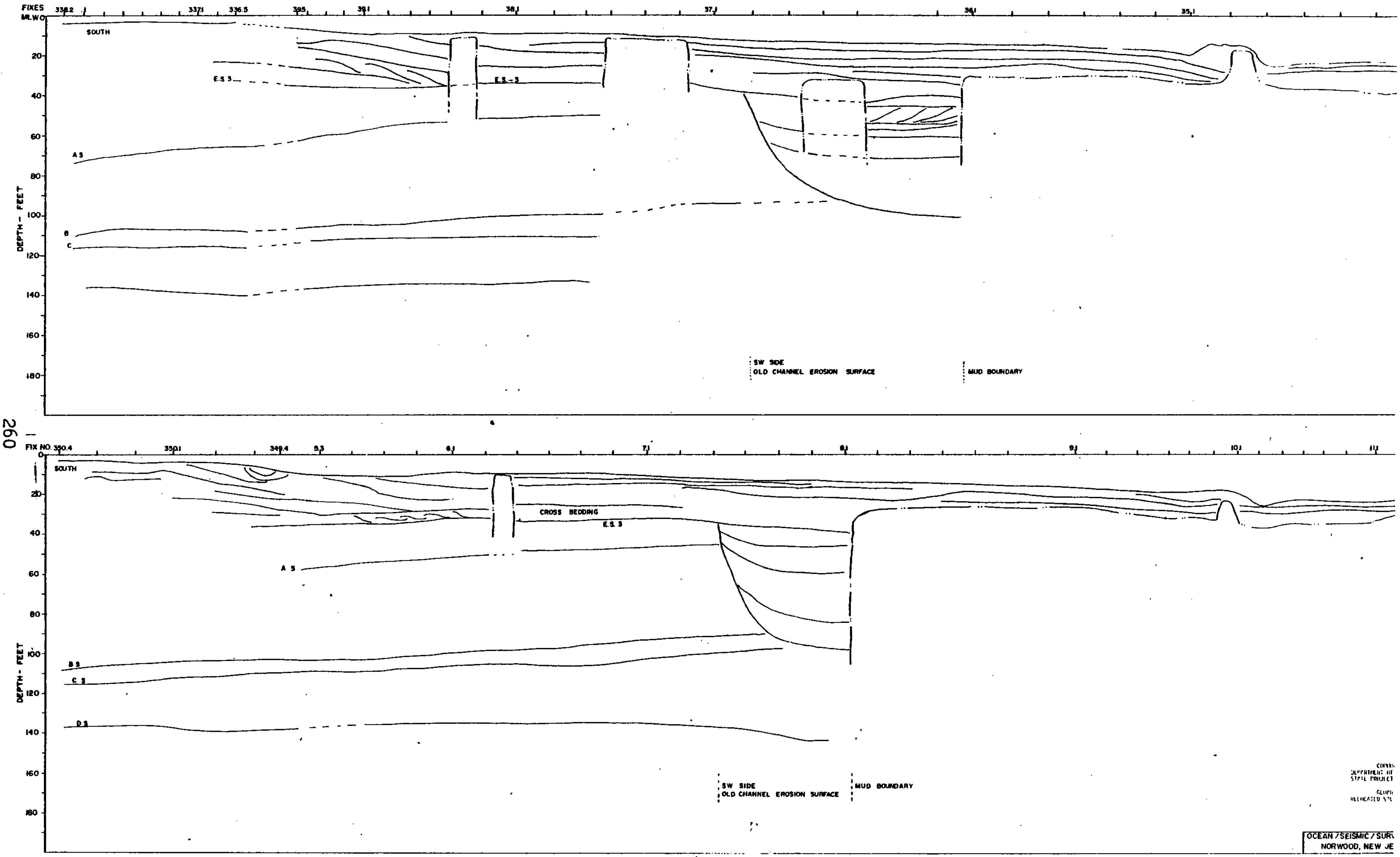


CENTER LINE COMPOSITE PROFILE
I-664 CROSSING OF HAMPTON ROADS
RELOCATED SOUTH ISLAND & SOUTH TRESTLE

OCEAN/SEISMIC/SURVEY, INC. NORWOOD, NEW JERSEY	COMMONWEALTH OF VIRGINIA DEPARTMENT OF HIGHWAYS & TRANSPORTATION PROJECT NO. 0664-121-102, PE 101

Figure 5 (continued)


FIXES
MLWO
338.2
337.1
336.5
39.5
39.1
38.1
37.1
36.1
35.1
SOUTH
E.S. 3
E.S. - 3
A S
B
C
DEPTH - FEET
SW SIDE
OLD CHANNEL EROSION SURFACE
MUD BOUNDARY
FIX NO. 350.4
350.1
349.4
5.3
6.1
7.1
8.1
9.1
10.1
11.1
CROSS BEDDING
B S
C S
D S
OCEAN / SEISMIC / SURV
NORWOOD, NEW JE

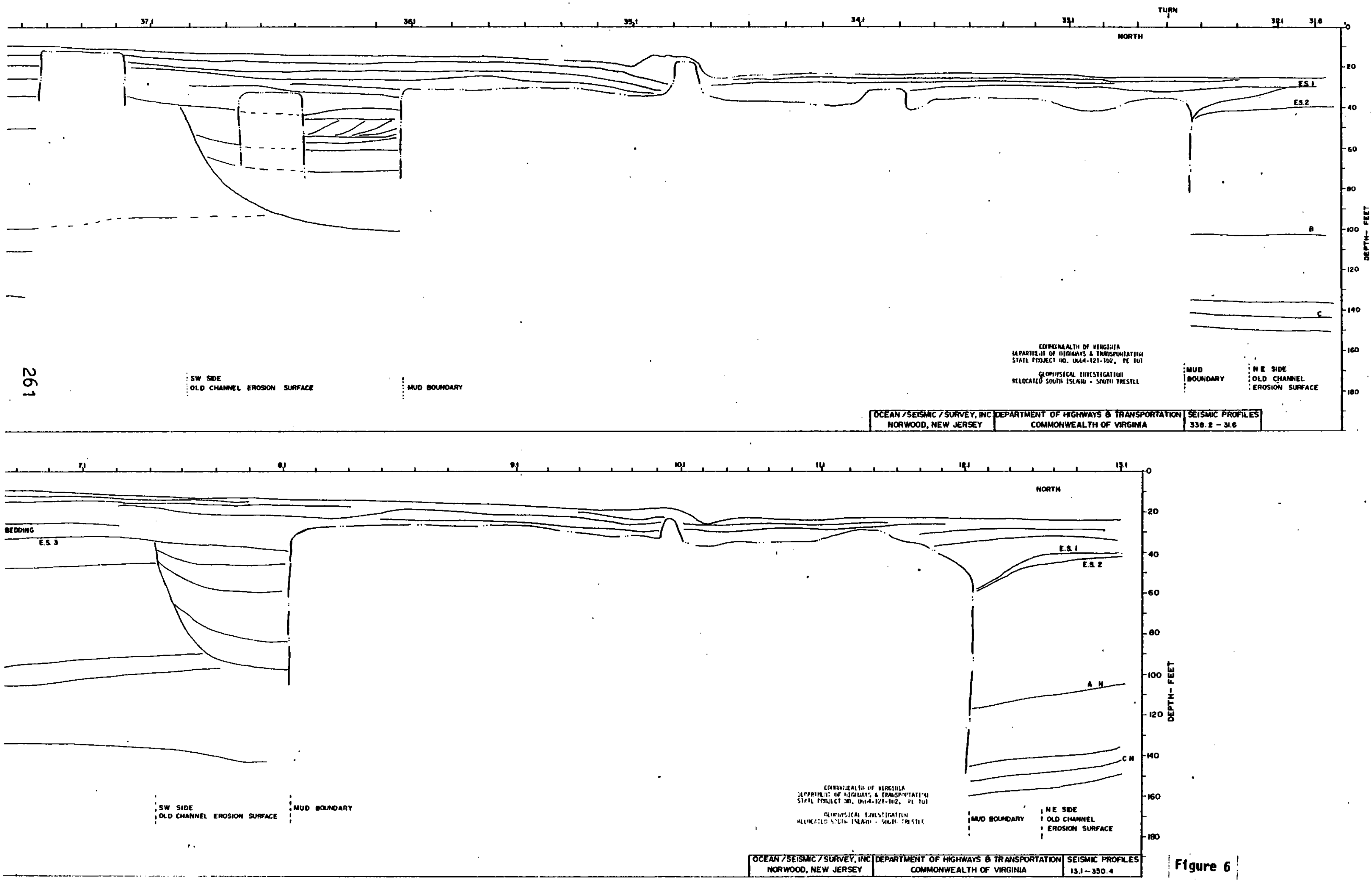

TURN
NORTH
E.S.1
E.S.2
B
C
DEPTH - FEET
SW SIDE
OLD CHANNEL EROSION SURFACE
MUD BOUNDARY
NE SIDE
OLD CHANNEL
EROSION SURFACE
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF HIGHWAYS & TRANSPORTATION
STATE PROJECT NO. 0664-121-102, PE 101
GEOPHYSICAL INVESTIGATION
RELOCATED SOUTH ISLAND - SOUTH TRESTLE
OCEAN/SEISMIC/SURVEY, INC
NORWOOD, NEW JERSEY
DEPARTMENT OF HIGHWAYS & TRANSPORTATION
COMMONWEALTH OF VIRGINIA
SEISMIC PROFILES
338.2 – 31.6
BEDDING
E.S.3
A N
C N
SEISMIC PROFILES
13.1 – 350.4

Figure 6

A DEMONSTRATION PROJECT FOR DEICING OF BRIDGE DECKS

by

Charles H. Wilson, David H. Pope, Assistant State Bridge Engineer, Wyoming Highway Department

Vic A. Cundy, John E. Nydahl, and Kynric M. Pell, University of Wyoming

An experimental facility to study the use of gravity operated heat pipes to couple earth heat to a bridge deck for snow and ice control has been developed at a site in southeastern Wyoming. Fifteen heat pipes of three different designs were incorporated in the design and construction of a composite brige deck. Nine standard heat pipes 24.4 m (80 ft.) long and 2.5 cm (1 in.) outside diameter were installed transverse to the direction of traffic flow 5 cm (2 in.) below the deck surface on 15 cm (6 in.) centers. These pipes extend from depth of 15 m (50 ft.) in the earth up through the earth surface and through the edge of the deck to the bridge centerline. The performance of the heat pipe system has been monitored and recorded continuously at one minute intervals for over one year using a variety of instrumentation transducers and a digital data acquisition system. In addition the surface conditions on the deck and the adjacent roadway were recorded photographically at five minute intervals during daylight hours. The results obtained demonstrate that heat pipes can be an effective means of snow and ice control on bridge decks.

A DEMONSTRATION PROJECT FOR DEICING OF BRIDGE DECKS

by
David H. Pope, Assistant Bridge Engineer;
Charles H. Wilson, State Bridge Engineer,
Wyoming Highway Department

and

Vic A. Cundy, John E. Nydahl, and Kynric M. Pell
Department of Mechanical Engineering
University of Wyoming

INTRODUCTION

State, county and municipal governments currently are spending over $500 million annually for snow and ice control on the nation's roads in response to public demand for clear highways. In the case of some extremely hazardous bridges and ramps, designers have actually resorted to heating the structures in order to maintain ice free surfaces. The current energy situation has spurred the development of renewable energy sources, in particular, the use of ground energy for controlling ice. This paper will address the power and energy requirements for a snow melting system; briefly review the historical use of the earth as a low grade energy source; and describe an experimental project conducted jointly between the Wyoming Highway Department and the University of Wyoming, to investigate the coupling of a bridge deck to the ground with heat pipes for ice control.

ENERGY REQUIRED TO MELT SNOW

The American Society of Heating, Refrigerating and Air-conditioning Engineering (ASHRAE) Guide and Data Book summarizes a decade of work by W. P. Chapman and his colleagues on specification of energy and power requirements for snow melting. In spite of this work, determination of the power and energy requirements for snow melting systems embedded in pavements remains one of the most problematical aspects of snow melting system design. In an attempt to refine the specification of power and energy requirements for snow melting systems a detailed study of approximately ten years of hourly weather data for the cities of New York, New York; Madison, Wisconsin; and Dodge City, Kansas, was undertaken. The study was based on a rather simple heat transfer code which performed an energy balance on the idealized surface shown in Figure 1.

Two different surface heating requirements were included in this study. The first model imposed the boundary condition that the surface must be maintained 1°C above freezing throughout the winter and that precipitation is melted as rapidly as it falls (Model A). The second model required that the surface be held at 1°C above freezing only during precipitation events and that all of the snow is melted as it falls, (Model B). This study indicated that, given the requirement that snow be melted as rapidly as it falls, the ASHRAE Guide seems to provide a satisfactory estimate of the energy and power requirements. A general overview of the ASHRAE Guide Standards for the contiguous 48 states is summarized in Figures 2 and 3. Figure 2 corresponds to Model A and

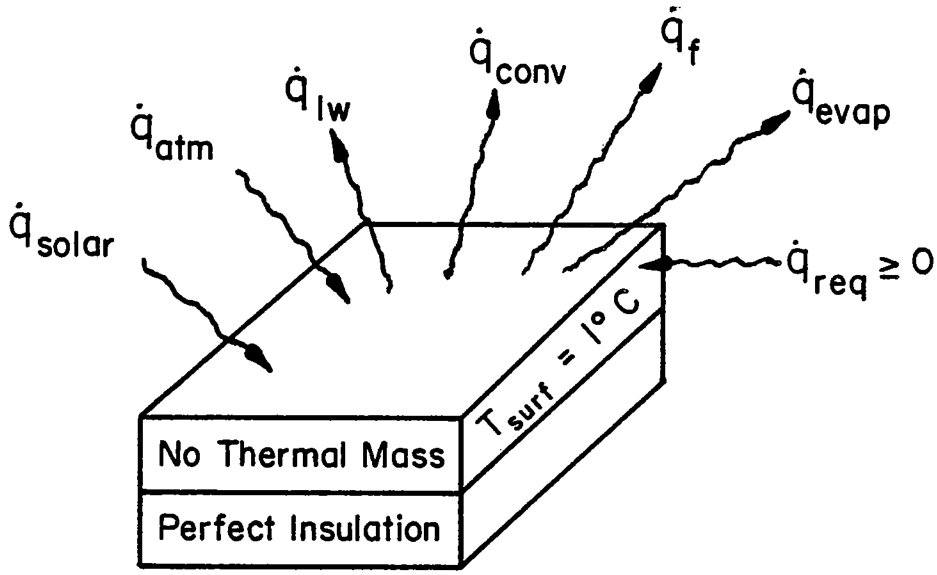


Figure 1. Energy balance on the surface

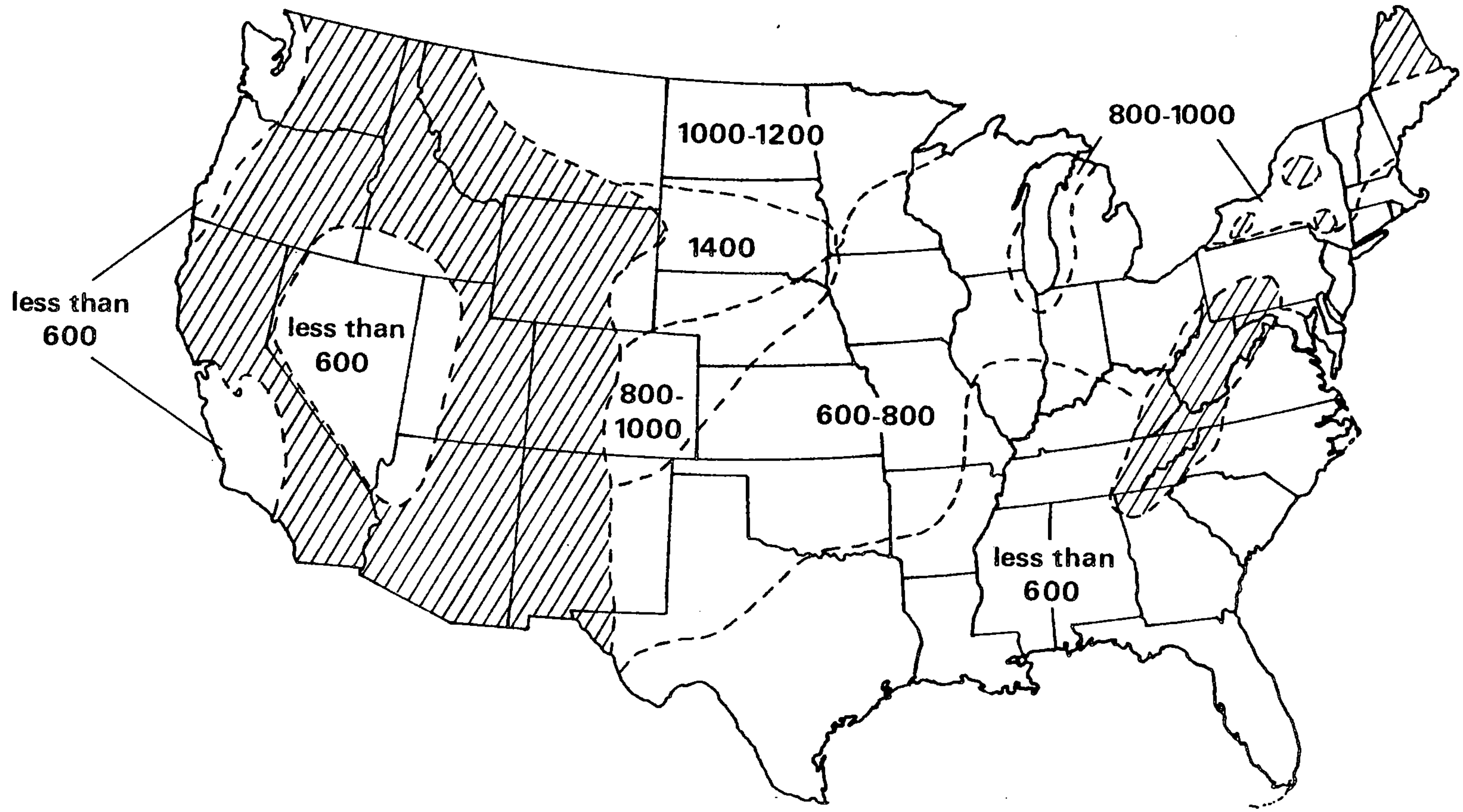


Figure 2. Peak power required for snow melting. (W/m^2)

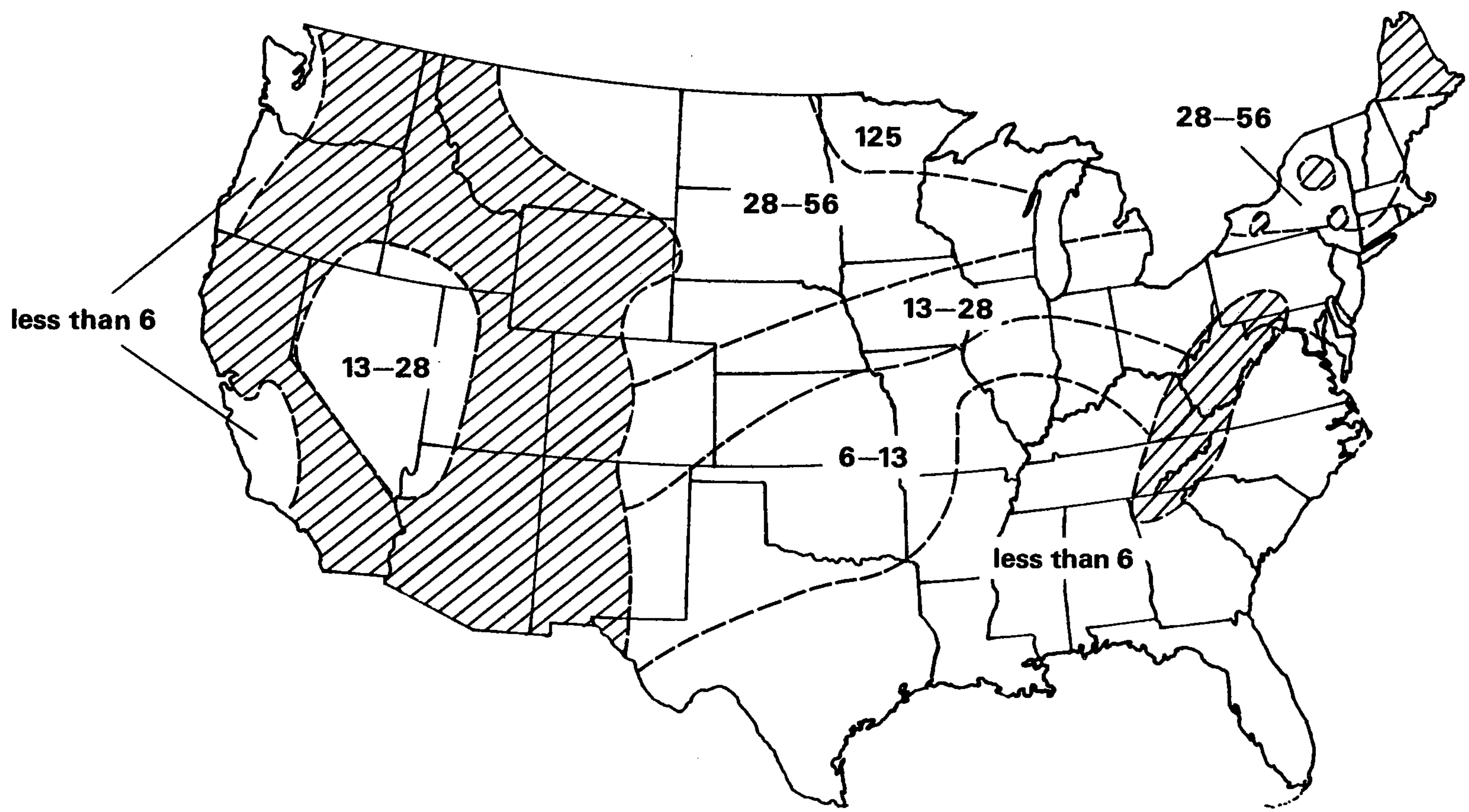


Figure 3. Annual energy required for snow melting. (KWh/m^2)

shows the peak power required to maintain a surface 1°C above freezing temperature through the winter and melt precipitation as rapidly as it falls. Figure 3 corresponds to Model B and shows the energy required to melt snow as rapidly as it falls and hold the surface 1°C above freezing temperature only during precipitation events.

The study also provided the instantaneous power required to melt snow and hold the surface at 1°C. This allowed calculation of the power spectra shown in Figure 4. It may be seen that the power spectra for many of the locations are very similar and that there appears to be a good agreement between the ASHRAE Guide data and the corresponding data obtained from Model B. It is quite evident from this figure that only a small percentage of snow events require the large design powers cited in the Guide. If the design criteria is relaxed to being able just to melt the snow as it falls for 90% of the events (rather than the ASHRAE standard of 98%) the design power would drop by approximately 40%.

Figure 5 indicates the power delivered to the surface, plotted as a function of the temperature difference between embedded pipes and the surface of a concrete deck. In order to deliver 400 w/m^2 to a 1°C surface from 2.54 cm (1 in) diameter pipes located on 15.2 cm (6 in) centers and covered with 3.81 cm (1.5 in) of concrete, the pipe temperatures would have to be around 8°C (46°F) if a thermal conductivity (K) of 2.42 w/m°C (1.4 BTU/hr ft°F) is assumed for the concrete, which was the measured thermal conductivity for the bridge deck used in this study. Figure 5 also indicates the dependence of the

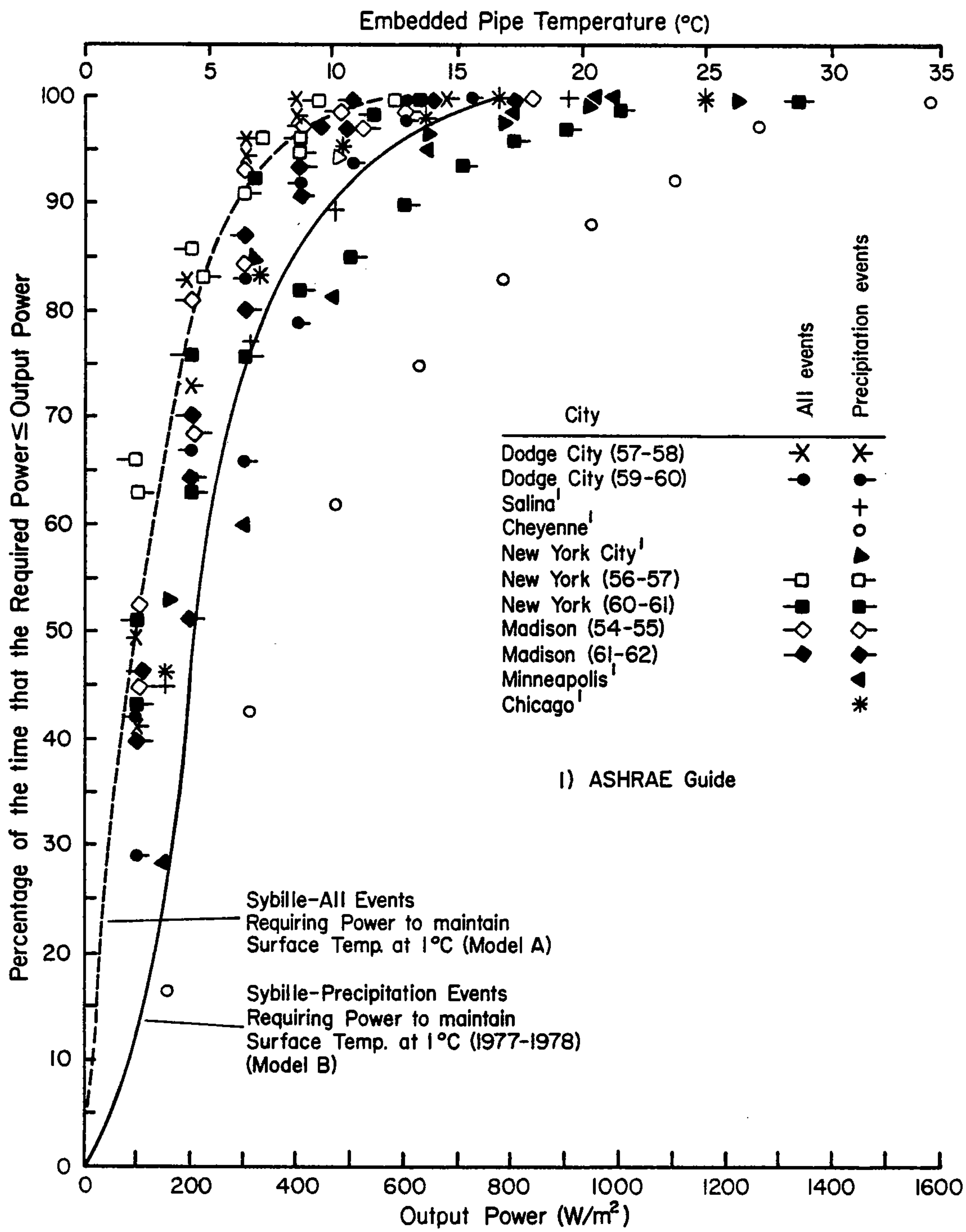


Figure 4. Snow melting capability for fixed power systems

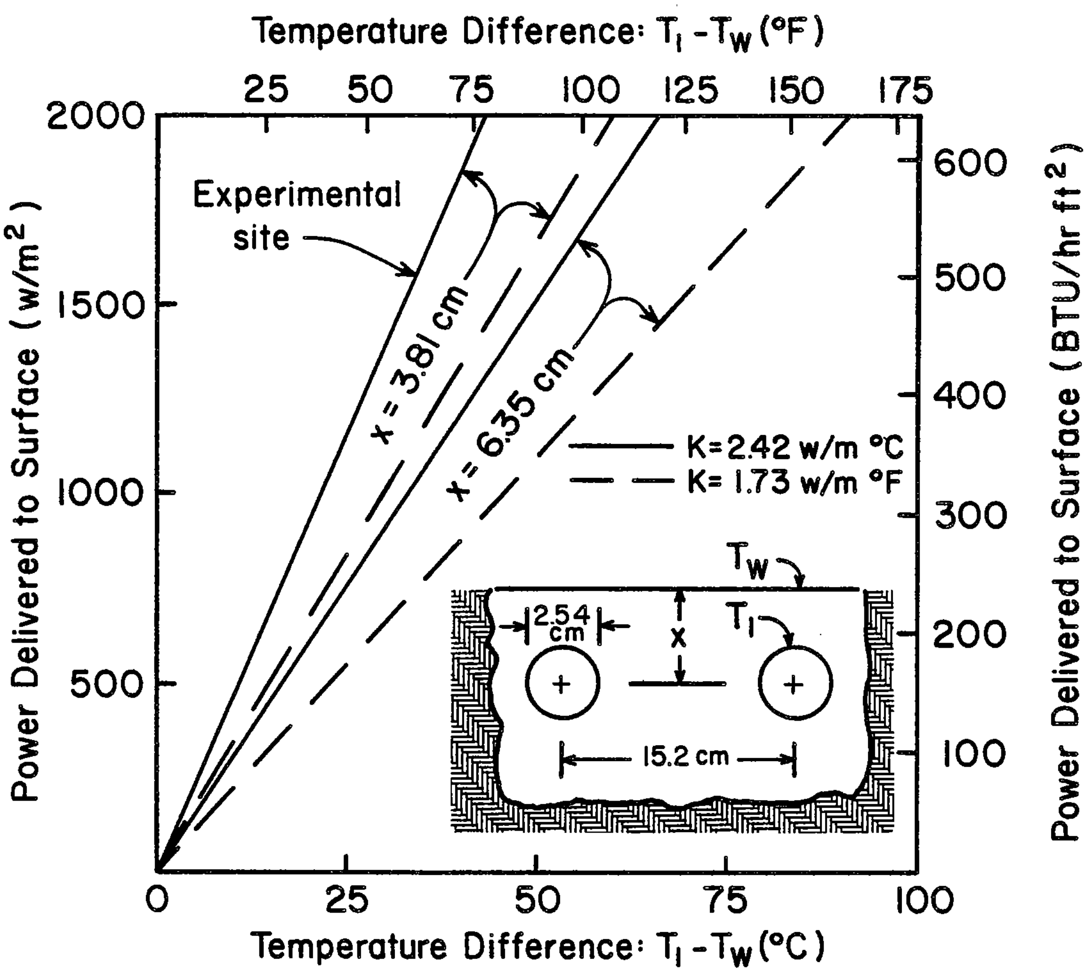


Figure 5. Power delivered to a deck surface as a function of embedded pipe temperature

power on the depth of cover over the embedded pipes. For powers of this magnitude, the earth is an excellent low quality energy source, and the ground temperature below the upper 3 to 5 meters (10 to 16 ft) does not vary significantly from the mean ambient air temperature of the region.

PREVIOUS USE OF THE EARTH AS AN ENERGY SOURCE

The use of the ground as a low temperature heat source is an idea that was developed in the 1950's in conjunction with the development of the heat pump. The typical domestic heat pump uses the ambient air for its energy source, which limits its application to moderate climates, since its performance would be decreasing as the ambient air temperature fell while the required heating load would be increasing. The use of shallow horizontal ground heat exchangers in place of the ambient air heat exchanger was investigated with the hope of increasing the performance of the heat pump by decreasing the range of temperatures it would have to operate over. The ground-coupled heat pump was not a commercial success at the time, mainly due to its high installation cost versus cheap energy, but there also were technical problems involved.

During the period of 1969 through 1975, the New Jersey Department of Transportation conducted a study of the use of buried heat exchangers to take energy from the earth to a pavement using a pumped ethylene glycol/water system. The results of this study indicated that systems of this type apparently delivered between 20 w/ft^2 and 40 w/ft^2 when 1 inch pipes in the pavement were buried 2 inches below the surface on 6 inch centers.

The use of gravity operated heat pipes for roadway heating was first proposed and demonstrated by Dynatherm Corporation between 1971 and 1974, under the sponsorship of the Federal Highway Administration (FHWA). As depicted in Figure 6, a gravity operated heat pipe is a closed chamber, generally constructed of metal, containing a volatile working fluid. The pipes are evacuated and filled with the fluid, generally ammonia. Over the temperature range that a heat pipe is exposed to, part of the ammonia resides as a liquid in a pool at the bottom of the pipe, while the remaining ammonia is in the vapor phase filling the rest of the pipe. Any time the deck temperature falls below the temperature of the ground in contact with a heat pipe, the vapor condenses in the deck section and flows towards the bottom of the pipe. At the same time, energy is conducted from the ground to the colder heat pipe where it evaporates part of the liquid ammonia. The spring in the evaporator section is used to increase the area that is wetted by returning condensate. The heat pipe is essentially an isothermal device, since the energy is transferred in the form of latent heat of evaporation instead of sensible heat. It is therefore ideally suited for this application where low grade energy is extracted from the ground. Besides its apparent constructional simplicity, this device is self-regulating and should have a long lifetime.

The Dynatherm project included the construction, instrumentation, and evaluation of heat pipes imbedded in concrete slabs installed at the Fairbanks Research Station of FHWA. In spite of the fact that some heat pipes developed condenser blockage due to inadequate cleaning during

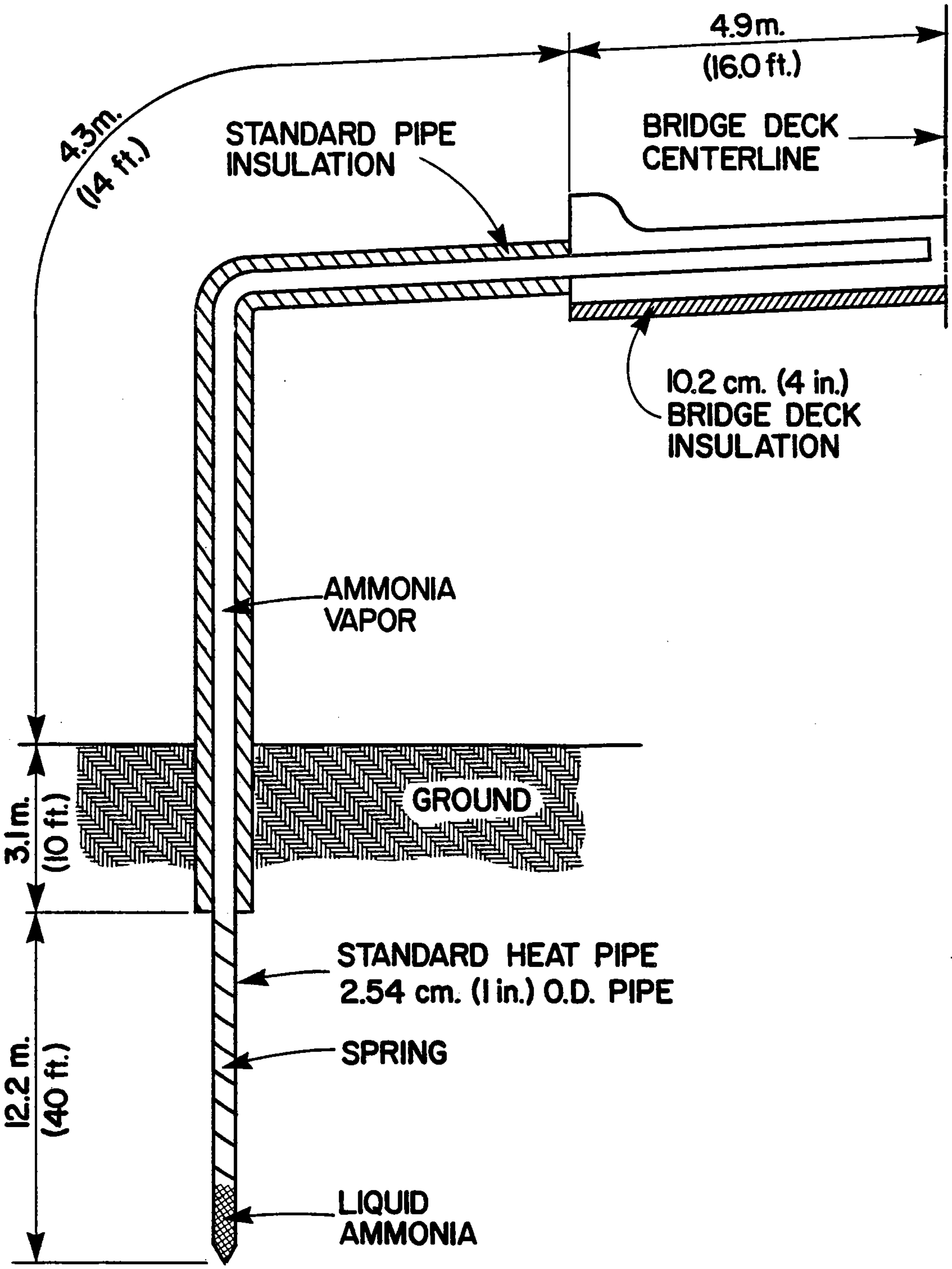


Figure 6. Conventional heat pipe

fabrication, and that there were repeated failures of the temperature measurement instrumentation, the project indicated that heat pipes using energy from the earth can be an effective means of snow and ice control on pavements. However, practical aspects of the system design and installation required further study. In 1975 this idea was tested successfully again in a 366 m (1200 ft) long interchange ramp in Oak Hill, West Virginia. Heat pipes were also installed in some of the foundations that support the Alaskan Pipeline in order to cool the ground surrounding the foundations to such an extent as to maintain the permafrost.

These types of studies have generated an increased international interest in various applications of gravity operated heat pipes. Although insufficient data was available to mathematically characterize the previous applications, the favorable results which were obtained prompted an investigation of gravity operated heat pipes for thermal control of a bridge deck.

THE DEMONSTRATION PROJECT

In the Spring of 1976, the Wyoming Highway Department and the University of Wyoming entered into an agreement to conduct a rather extensive research program on the thermal control of bridge decks through the use of heat pipes. A small experimental facility was installed at a bridge site in southeastern Wyoming at Sybille Canyon. A 6' x 16' portion of this bridge was heated, using twelve 80-foot long heat pipes and three short, electrically heated pipes, which were used to measure the output power of the ground pipes. Specific goals of the program included:

1. Measurement of the power and energy delivered to the deck by a heat pipe
2. Characterization of the thermal coupling between the earth, the heat pipe, and the bridge deck
3. Investigation of the thermal recovery of the earth surrounding the heat pipe during periods when the heat pipes were not functioning
4. Quantitative comparision of the surface condition of the unheated portion of the bridge deck, the heated section of the deck, and the adjacent roadway during an entire winter season.

Installation

After all of the footings had been placed at the bridge site, but prior to construction of any above-ground formwork, a drilling crew from the Geology Division of the State Highway Department drilled 12 holes 15 cm (6in) in diameter and 18.3 m (60 ft) deep. The location of the holes with respect to the structure is shown in Figure 7. The holes were cased with 10 cm (4 in) polyvinyl chloride (PVC). All 12 pipes were installed in the holes using a crane in approximately two hours. Subsequent to forming the deck and placing the rebar, all of the heat pipes were bent into place and tied to the rebar at a depth of 5 cm (2 in) below the surface on 15 cm (6 in) centers. The 2% slope from the centerline of the bridge to the curb was adequate to insure that the condensate would flow back to the evaporator. The heat pipes were then insulated from a point 10 ft below ground surface to the curb of the deck as

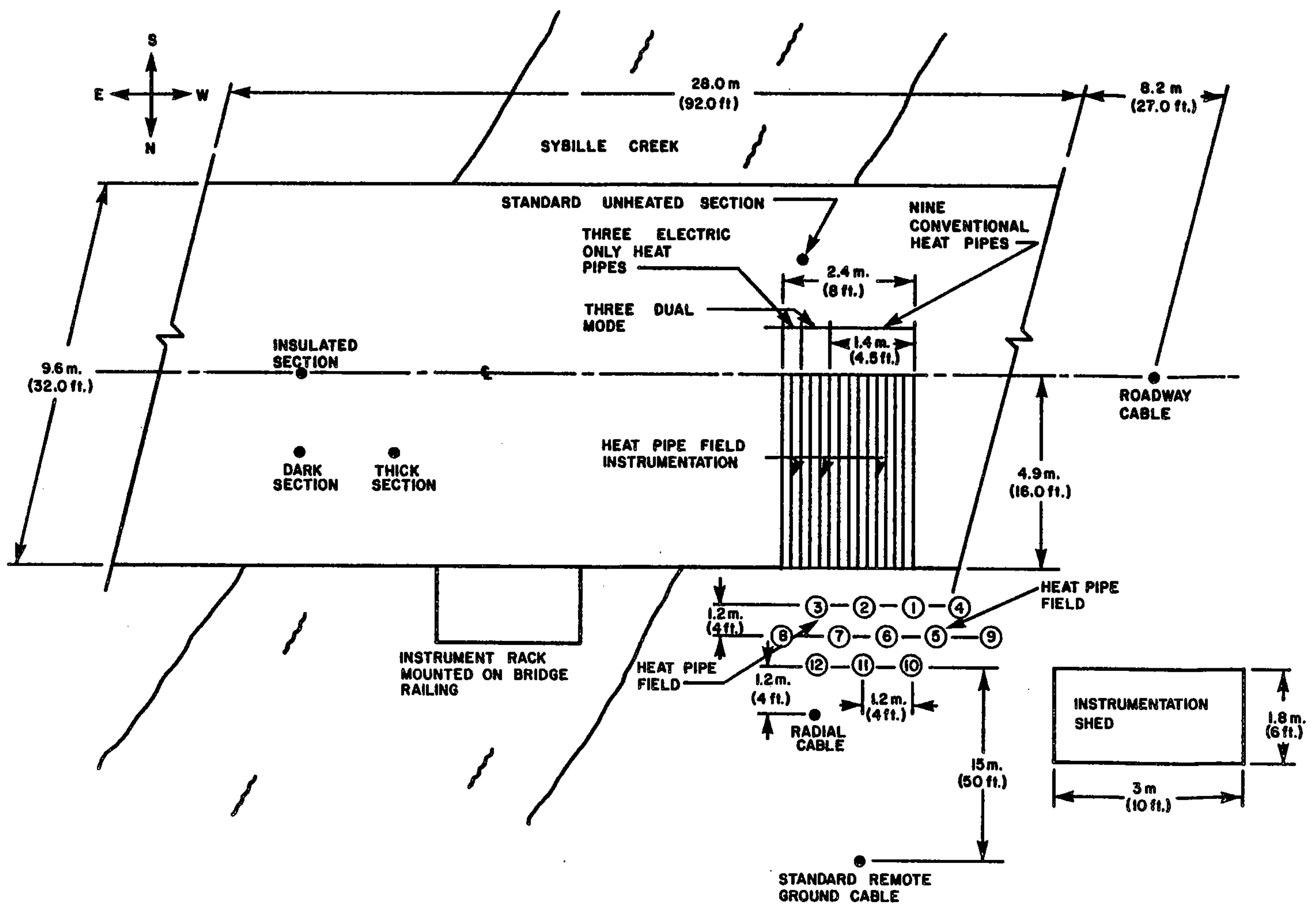


Figure 7. Sybille Creek experimental facility

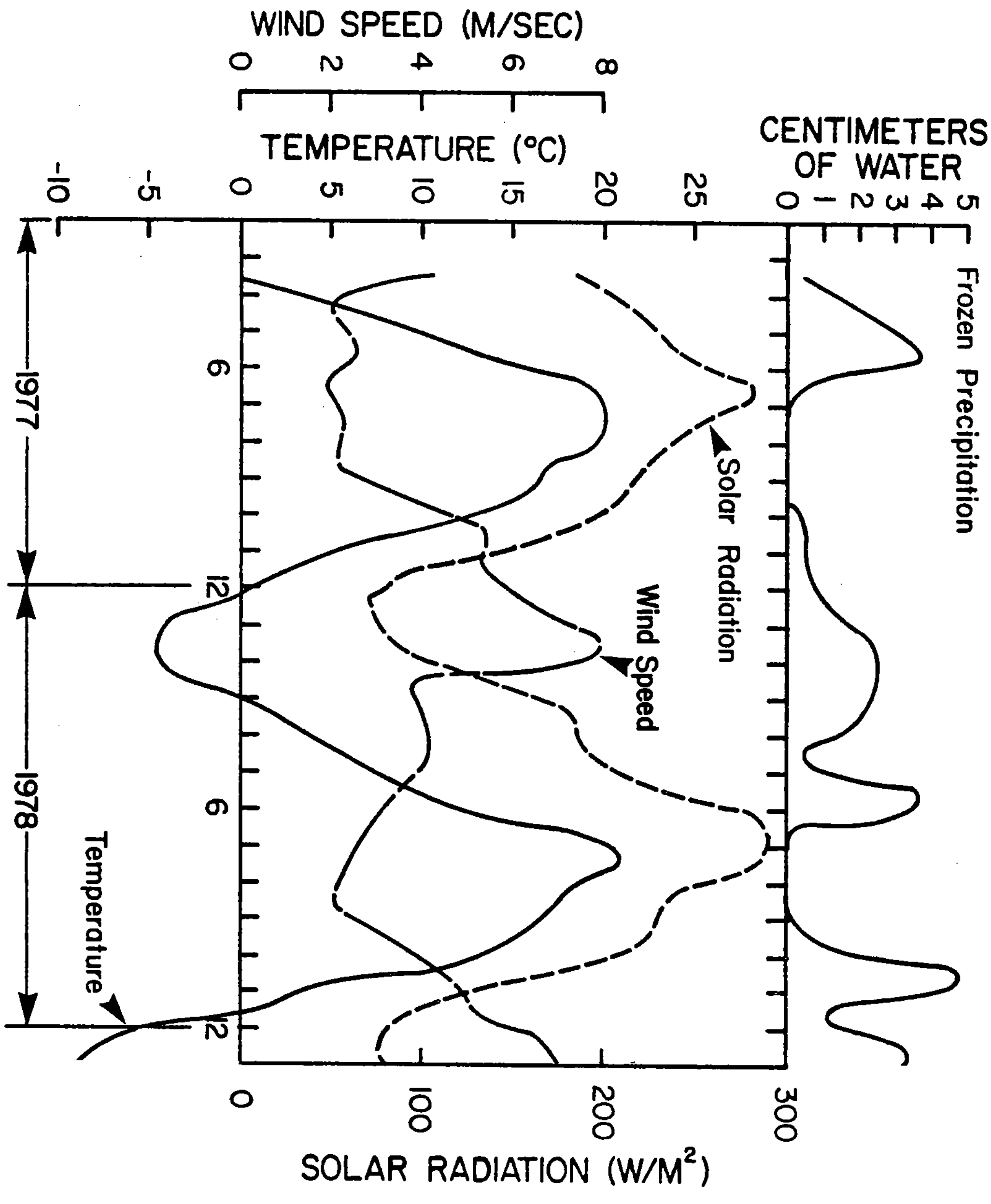


Figure 8. Monthly averaged environmental parameters

indicated in Figure 6.

The location of the experimental facility at an elevation of 1.82 km (5960.0 ft) above sea level provided a quite severe environment to test the feasability of the heat pipe system. Figure 8 presents the monthly averaged values of the main environmental paramenters and indicates that some of the monthly average air temperatures were below freezing. As may be seen, rather harsh winter weather is possible over the eight-month period from October through May, which could severely tax any snow melting system.

Experimental Results

Perhaps the most important aspect of the performance of the heat pipe system in the bridge deck is presented graphically in Figure 9. This figure plots the average upper surface temperature of the heated section of the bridge deck, the unheated section of the bridge deck, and the approaching roadway surface. As expected, the temperature of the unheated and heated sections of the deck coincide during the summer months when the heat pipes do not operate. Around September the pipes begin to function, and the surface temperatures start to depart. During the 1977-78 winter, the heated section of the deck remained above freezing on the average, whereas the standard unheated section of the deck was frozen from December 1977 to February 1978. Although the upper surface of the heated portion of the deck did not remain unfrozen throughout the entire observed period, this surface experienced less than half of the total frozen time of the approach roadway surface, and exhibited one half the severity of freezes in terms of degree days frozen.

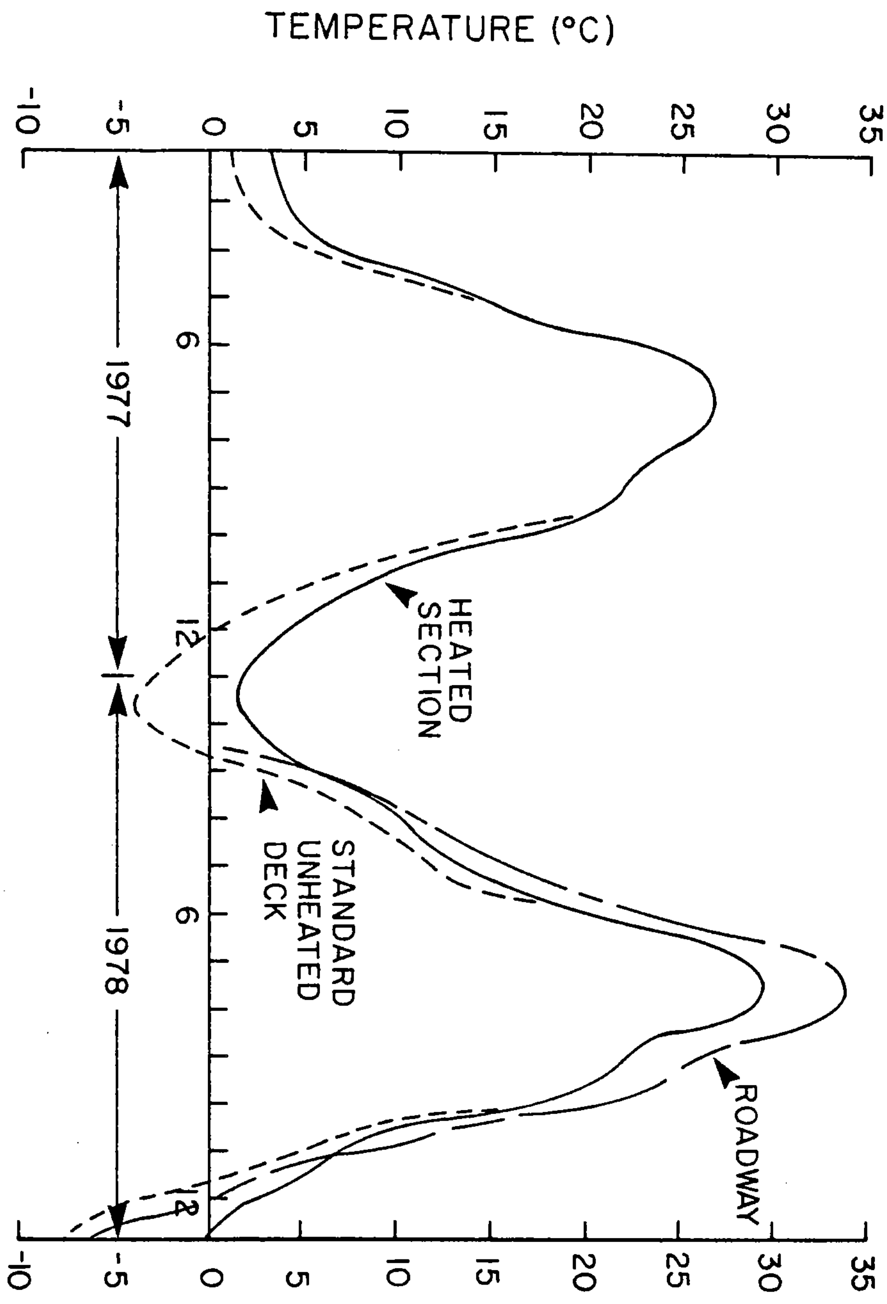


Figure 9. Monthly average surface temperature of the unheated (standard) deck, heated deck and adjacent roadway

The effect that the heat pipe system had on the ground at depth is shown in Figure 10. This plot presents weekly average, undisturbed ground temperatures, located at a 10.1 m (33.0 ft) depth below the earth's surface. The ground temperature at this depth oscillates sinusoidally with a period of one year and an amplitude of 1 1/2° C (5.4°F).

The lower plot presented in Figure 10 represents the percentage of time each month that the heat pipes were operational. A very favorable phase relationship between maximum ground temperatures and periods of peak energy requirements exists at this depth. Figure 10 indicates that the monthly average ground temperature at the 10.1 m (33.0ft) depth within the heat pipe field can be depressed by as much as 7.0°C (12.6° F). This is evident in March 1977 when the remote ground temperature was 6.9°C (44.2° F) while the ground in the vicinity of the heat pipe was cooled to 2.5°C (36.5° F) and again in January 1978 when the remote ground temperature was 10.4° C (50.7° F), and the ground near the heat pipes was at a temperature of 3.4°C (38.1° F). Peak operational periods for the heat pipe system occur in December and January with the system functioning up to 90% of the time during these months. Even though the system can continue to operate into May, as evidenced in 1978, the ground in the vicinity of the heat pipes begins to recover in February. By June 1977 and July 1978, the ground in the vicinity of the heat pipes completely recovered and followed the remote ground temperature until September of each year when the heat pipes again become operational. The delayed recovery in 1978 is due to the fact that the heat pipe system did remain operational throughout May, with the pipes functioning approximately 17% of May 1978.

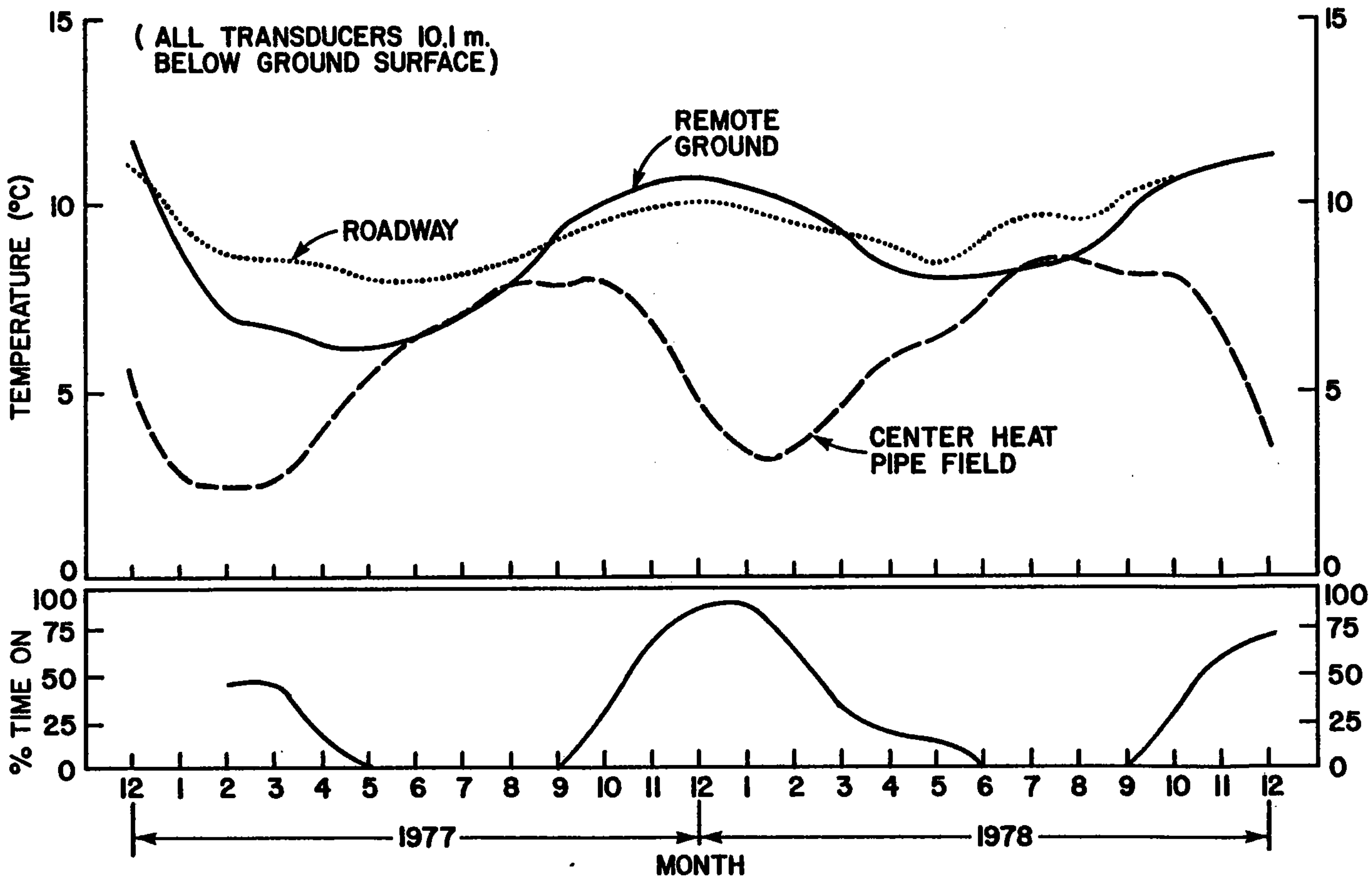


Figure 10. Monthly average values of various transducers located 10.1 m (33.0 ft) below the earth's surface with an indication of percent time each month that the heat pipes are operational

A plot of the total energy delivered to the deck surface calculated on a monthly basis is presented in Figure 11. It should be noted that each heat pipe was thermally coupled to a 0.75 m^2 (8.0^2 ft) area of the bridge deck surface. As shown, the heat pipes were operational periodically over a 17-month interval from March 1977 through December 1978. Throughout this 17-month period, each heat pipe delivered in excess of 1.0×10^9 J (9.5×10^5 BTU). The average rate of power delivered when the heat pipes were functional was 89 w/m^2 (28.2 BTU/hr-ft^2). Over the four-month period from November 1977 through February 1978, which corresponds to the harshest climatic environment during the 1977-78 winter, the energy delivered by each heat pipe per month reached a fairly constant rate of 124 megajoules (1.2×10^5 BTU). Throughout this period, the average power delivered to the bridge deck from the earth was 82 w/m^2 (26.0 BTU/hr-ft^2). Over a three-month period from December 1978 through February 1979, the peak power delivered was 208 w (710 BTU/hr) per pipe.

CONCLUSION

The Sybille Canyon experiment demonstrated that a gravity operated heat pipe system can eliminate preferential freezing between a bridge deck and the adjacent roadway even in a severe climate. Although the heat pipes were not capable of melting all of the snow as fast as it fell, they were completely successful in eliminating all instances of preferential icing.

One of the dominant questions which persists, with respect to design of large scale heat pipe systems, is that of whether the earth temperature within the heat pipe field becomes permanently depressed.

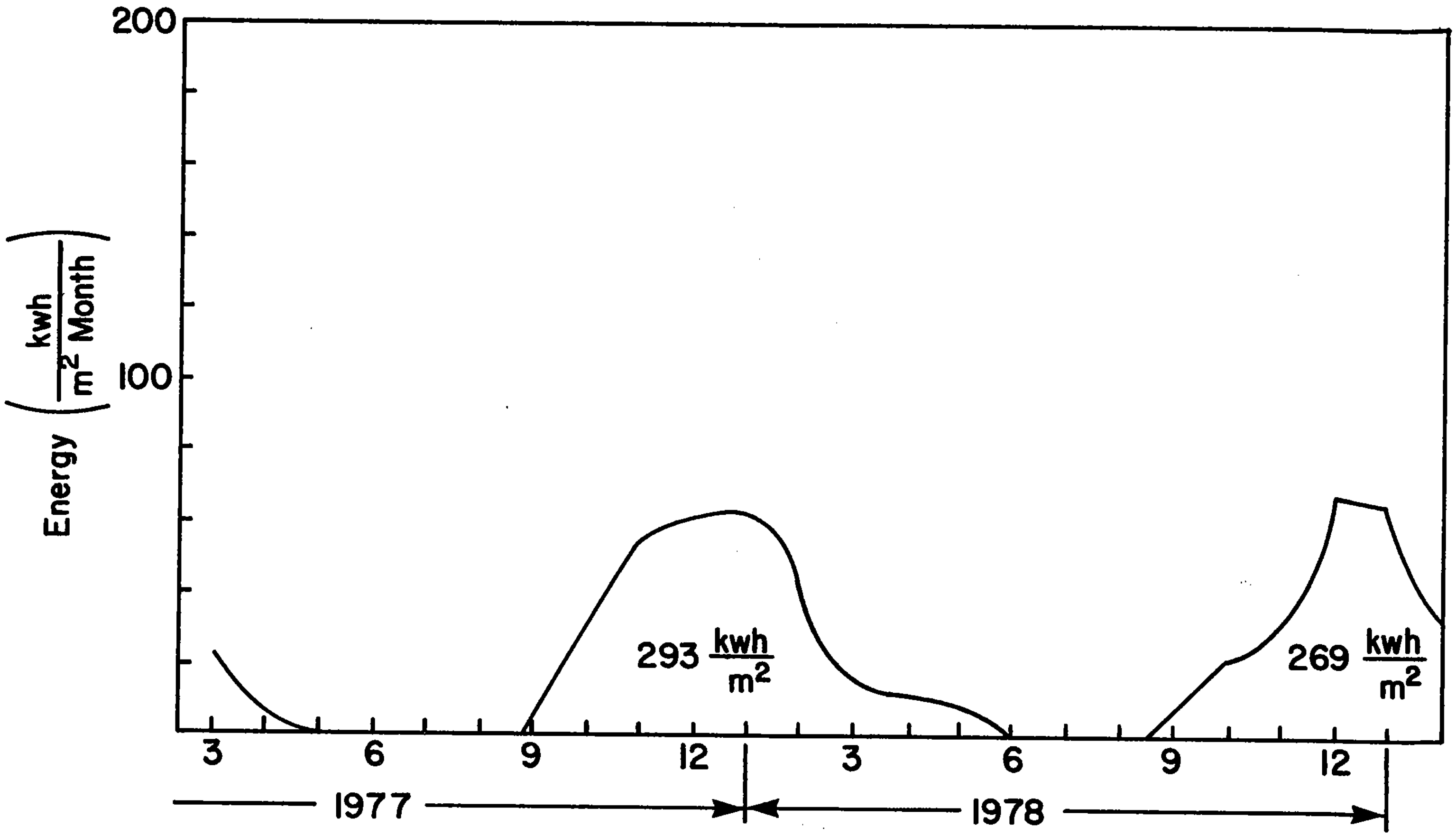


Figure 11. Energy delivered to the heated deck

The Sybille Creek project has demonstrated that the temperature of the earth within the heat pipe field is reduced by as much as 7.0°C (12.6°F) when compared to earth temperatures outside the heat pipe field. However, this depression was recovered during both summer periods which were observed. In spite of the fact that the temperature of the earth within the heat pipe field recovered very rapidly during the spring and summer months and eventually did assume the temperature of the undistrubed earth, the relatively small number of heat pipes which were installed in this project precludes any general conclusions which might be offered in regard to the recovery of the earth in a large scale project.

COMPACTION PRESTRESS

MAKES A DIFFERENCE

by

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Contribution to the Proceedings

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COMPACTION PRESTRESS
MAKES A DIFFERENCE

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Abstract

Interpretation of the strength and compressibility characteristics of natural clays relies heavily upon the determination of a maximum past pressure (prestress) and the relationship of the present effective stress to that maximum past value. Excavation and fragmentation of natural soils, preliminary to their compaction in fills, substantially removes the effect of the geologic prestress, but the compaction process produces a new prestressing effect. The behavior of the compacted soils is then as strongly influenced by the compaction prestress as the natural soils were by the geologic value.

The importance of the compactive prestress is illustrated by experimental evidence on the compressibility and undrained shearing behavior of laboratory compacted shales and clays. The value of the prestress may be interpretated from the conventional consolidation test on the as-compacted material. It varies not only with nominal magnitude and time-rate of applying the compaction pressure but also with the material and its water content. When highly prestressed material are saturated and sheared undrained, the pore pressures decrease with strain and the effective stress paths curve up and to the right. Slightly prestressed samples show essentially constant pore pressures with strain and stress paths which are essentially upright.

In an embankment of significant height, the material at lesser depths can be expected to demonstrate the highly prestressed behavior, while at greater depths the body forces of the embankment will remove (exceed) the prestress. With a better understanding of the compactive prestress, it may be possible to control it through the compaction specification, and to produce embankments which settle (or heave) less and which have higher shear strengths.

Prestress

Compaction produces a transient loading which is borne, at least in part, by the solid matrix or skeleton. When this loading produces residual changes in the matrix, subsequent loadings of an equal or lesser magnitude have little or no effect on the matrix. Heavier loads would be expected to produce additional matrix changes.

The irreversible changes produced by a compactive loading amount to a prestressing of the matrix by that load. Since larger loads produce more irreversible changes, the skeleton is prestressed by the maximum load which it has carried.

Compaction prestress is both similar to and different from geologic preconsolidation in natural sediments. In both cases, we identify a maximum pressure which the solid matrix has experienced. In the short term, the loading produces excess fluid pressures in the pores, and disruptions, dislocations, and degradations in parts of the solid matrix which are not favorable to prestressing. In the long term, the loading survives the transient effects to produce the plastic material deformations and the interlocking of grains and aggregates which are the physical reasons for a prestress.

The sustained nature of the geologic loadings is in contrast to the highly transient compaction ones. Thus the compactive load is coexistent with the temporary pore pressures and matrix dislocations and degradations, and much of its potential to effect prestress is dissipated in this fashion.

The recognition of compactive prestress is probably important for both conceptual and practical reasons. One would not attempt to explain the range of responses of saturated natural clays without reference to preconsolidation pressure and the overconsolidation ratio. Since the responses of partially saturated compacted soils are more complex, the compactive prestress reference may be even more helpful in organizing and interpreting experimental data.

Practically speaking, current compaction specifications are implicitly controlling prestress values through restrictions on: roller type and rating, roller speed, compactive effort, lift thickness, and water content. The end result primarily controlled is dry density. The evidence offered in the remainder of this paper suggests that it may be desirable to explicitly control the prestress, since by so doing we can probably effectively reduce settlements, control heave, and increase the shear strength of compacted fills.

Compressibility

Compacted clays and shales will show two general levels of compressibility, viz., a relatively low one at pressures below the prestress and a relatively high one above this stress level. This is illustrated for a laboratory compacted clay in Figure 1. The figure also shows that the compaction water content influences the value of the prestress. The compaction pressure should also be an important factor of influence, and this is shown in Figure 2.

While prestress (P_s) increases with compaction pressure (P_c), it does so at a decreasing rate, and the ratio $\frac{P_s}{P_c} \leq 1$ is an interesting one. Table 1 shows a collection of these ratios for several materials. The values vary from unity to almost zero. The higher ratios correspond to: (a) a longer dwell time of the compressive loading, (b) a higher permeability (which allows more dissipation of excess pore pressures), and (c) a load level which does not excessively dislocate or degrade the solid matrix.

Working with a highly plastic clay in kneading type laboratory compaction, DiBernardo (1979) developed the following statistically valid equation for prestress.

$$\hat{P}_s = -343.13 - 0.0020\, w^2 P_c + 48.91\, P_c^{\frac{1}{2}}$$

where $\hat{P}_s$ = value of estimated prestress in kN/m^2

$w^2 P_c$ = interaction term between compaction water content (%) squared and compaction pressure (kN/m^2)

$P_c^{\frac{1}{2}}$ = square root of compaction pressure in kN/m^2.

Figure 3 is the graphed version of the equation for the relevant data field. This equation form was just as valid for a physically contrasting material, the New Providence shale. Witsman (1979) developed the constants as,

$$\hat{P}_s = 33.96\ P_c^{\frac{1}{2}} - 0.0032\ w^2 P_c.$$

The plot of the model in the data field is shown in Figure 4.

An increase in the compaction pressure will increase the prestress and decrease the compressibility of the as-compacted soil. A more important part of the embankment settlement would occur when the compacted soil became saturated in service. Figure 5 shows that increasing the compaction pressure (P_c) would make the St. Croix clay less susceptible to settlement (collapse) on saturation. The data are for a highly plastic material (DiBernardo, 1979), and an increase in P_c would also make the fill more likely to swell and heave. The statistically valid regression equation states that,

$$\frac{\Delta \hat{V}}{V_o} = 25.47 - 0.872\ w - 0.0048\ P_c$$

where $\frac{\Delta \hat{V}}{V_o}$ is the estimated value of one dimensional volume change (%) on saturation

w is the compaction water content (%) and

P_c is the compaction pressure (kN/m^2).

The possible use of compaction pressure to produce prestress which in turn limits volume changes in compacted embankments is intriguing. Keep in mind that the data fields showing the above trends are limited. Also, recall that compaction is a process of producing local (not general) bearing capacity failures under the foot pressure. Thus the allowable compaction pressure is a function of the soil strength, and has a definite upper limit.

Strength

While compactive prestress values have not been extracted from strength testing, evidence of their existance has been recognized by Abeyesekera (1978), Johnson (1979) and Weitzel (1979). The most obvious effects of the prestressing have been: decreases in pore pressures during undrained shear, lowered values of the Skempton pore pressure parameter (A), and volume increases during shear.

Figures 6 and 7 from Abeyesekera show selected results from $\overline{\text{CIU}}$ testing of a compacted and saturated Indiana shale. For the tests depicted in Figure 6, the compaction pressure was relatively high, while the preshear consolidating pressures were relatively low. This produced a highly prestressed shale matrix, as emphasized by: the decreases in pore pressure during shear, the relatively low A values, and the effective stress paths curving to the right. The samples of Figure 7 had a relatively low compaction pressure and consolidating stresses which were closer to that pressure. Hence these samples were only slightly prestressed, as evidenced by: pore pressures which do not decrease with strain, A factors which are higher, and stress paths which are more or less upright.

The highly prestressed compacted shale is stronger than the slightly prestressed version. Such strength may be expressed as a function of effective stress in the very familiar Mohr-Coulomb equation.

$$s = c' + (\sigma_{ff} - u_f)\ \text{TAN}\phi'$$

$$= c' + \sigma'_{ff}\text{TAN}\phi'$$

For this shale, $c' = 1$ to 2 psi and $\phi' = 28°$ to $30°$, regardless of the compaction history. However, the pore pressure parameter at failure, A_f, decreased with increasing compaction pressure. This meant that u_f was smaller, and σ'_{ff} was larger, for the samples with higher prestress.

Johnson (1979) working with a highly plastic Indiana clay reached similar conclusions from $\overline{CIU}$ testing on compacted samples which were saturated and consolidated. The c' and ϕ' values varied little with the compaction history, viz., $c' = 15 \pm 8$ kN/m^2 and $\phi' = 20 \pm 1°$. However the A_f value decreased with an increase in the probable prestress. Thus for both of these materials, there was little effect of prestress on the effective stress strength parameters, but the effective stress at failure was increased by the prestress. Accordingly the prestress increased the long term undrained shear strength.

Weitzel (1979) found that prestress probably increased the as-compacted undrained shear strength of the St. Croix, Indiana clay. Higher prestress was evidenced by a greater tendency to volume increase during UU testing and a larger stress difference required to cause failure. Figure 8 shows the proposed relationship among undrained shear strength, prestress, and compaction variables for the St. Croix clay.

Summary

Both compacted shales and clays have been shown to demonstrate the influence of a prestress established during the compaction. This value is strongly related to the compaction pressure and is influenced by the other compaction variables, as well. The prestress is physically established and maintained through plastic deformations, dislocations, and interlockings in the solid matrix.

The prestress has logical and predictible influences on both the short and long term strengths and compressibilities. It may be worth the effort to specify and control the compaction pressure and other compaction variables which strongly influence the prestress. The concept of compaction prestress should help considerably in organizing and understanding experimental data on the performance of compacted materials.

Acknowledgements

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References Cited

Abeyesekera, R. A. (1978). "Stress-Deformation and Strength Characteristics of a Compacted Shale". PhD Thesis, Purdue University. May. 420 pp. Also Joint Highway Research Project Report No. 77-24.

DiBernardo, A. (1979). "The Effect of Laboratory Compaction on the Compressibility of a Compacted Highly Plastic Clay". MSCE Thesis, Purdue University. May. 187 pp. Also Joint Highway Research Project Report No. 79-3.

Johnson, J. M. (1979). "The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation". MSCE Thesis, Purdue University. Aug. 271 pp. Also Joint Highway Research Project Report No. 79-7.

Weitzel, D. W. (1979). "The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay". MSCE Thesis, Purdue University. Aug. 218 pp.

Witsman, G. R. (1979). "The Effect of Compaction Prestress on Compacted Shale Compressibility". MSCE Thesis, Purdue University, Aug. 181 pp.

Table 1. Compaction Pressure and Prestress (after Witsman, 1979)

Reference	Material	Compaction Method	Compaction Pressure P_c (kPa)	Dwell Time (seconds)	Prestress Values P_s (kPa)	Prestress Ratio P_s/P_c
Campbell (1952)	Bituminous Concrete	Static	17238	60	1034 to 1448	0.06 to 0.08
Yoshimi and Osterberg (1963)	Silty Clay	Static 20 psi/min	648	280	648	1.00
Mishu (1963)	Residual Clay	Kneading 30 blow/min	655	1	249	0.38
			862	1	249	0.29
			1172	1	383	0.33
Abeyesekera (1978)	Shale Aggregate	Kneading 30 blow/min	345	1	345	1.00
			690	1	469 to 510	0.68 to 0.74
			1380	1	676 to 869	0.49 to 0.63
DiBernardo (1979)	Residual Clay	Kneading 30 blow/min	460 to 657	1	72 to 315	0.11 to 0.63
			657 to 814	1	55 to 505	0.08 to 0.64
			2466 to 3919	1	85 to 1120	0.03 to 0.40

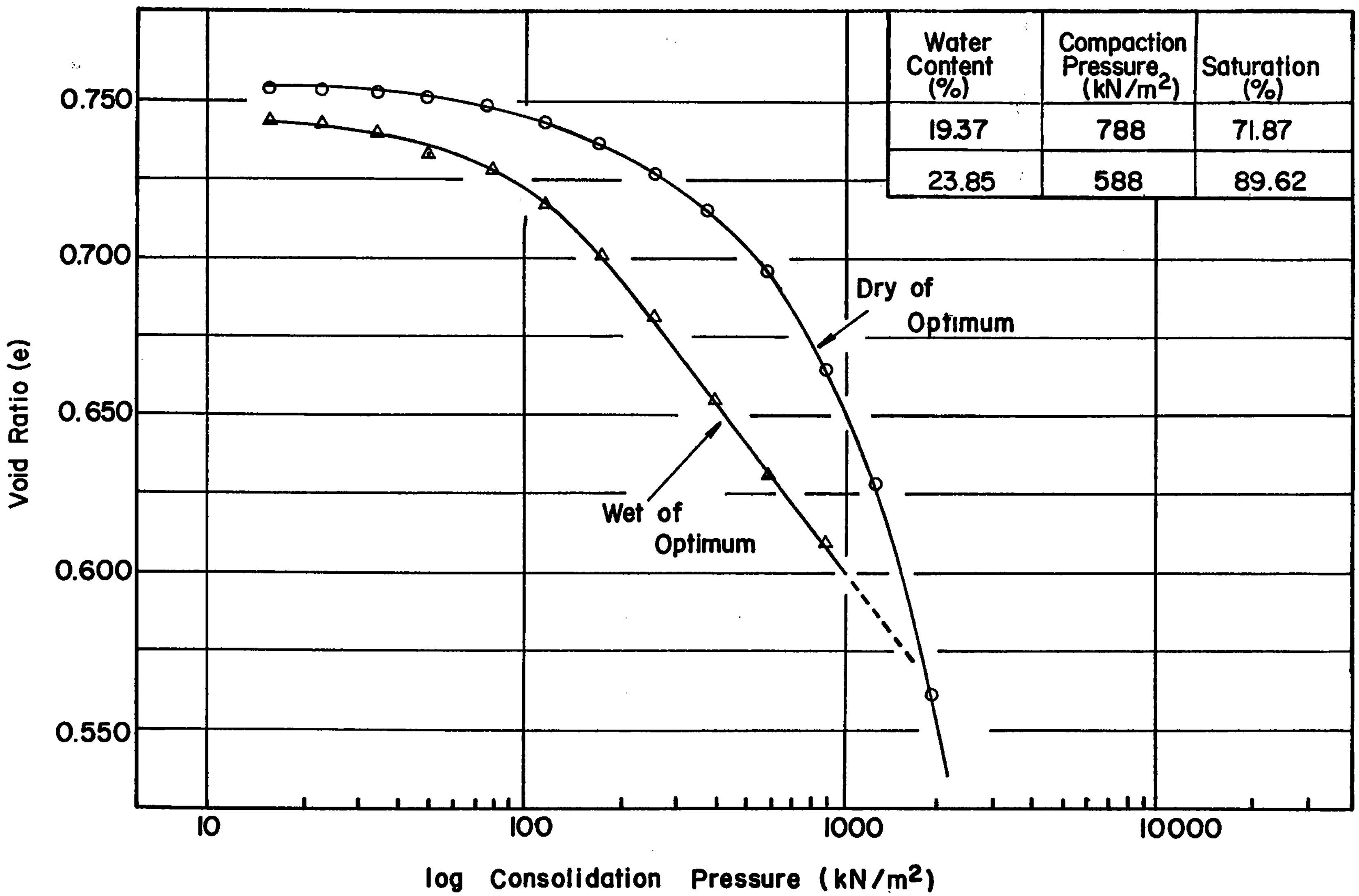


FIGURE I EFFECT OF MOISTURE CONTENT ON COMPRESSIBILITY (STANDARD PROCTOR); ST. CROIX CLAY

After DiBernardo (1979)

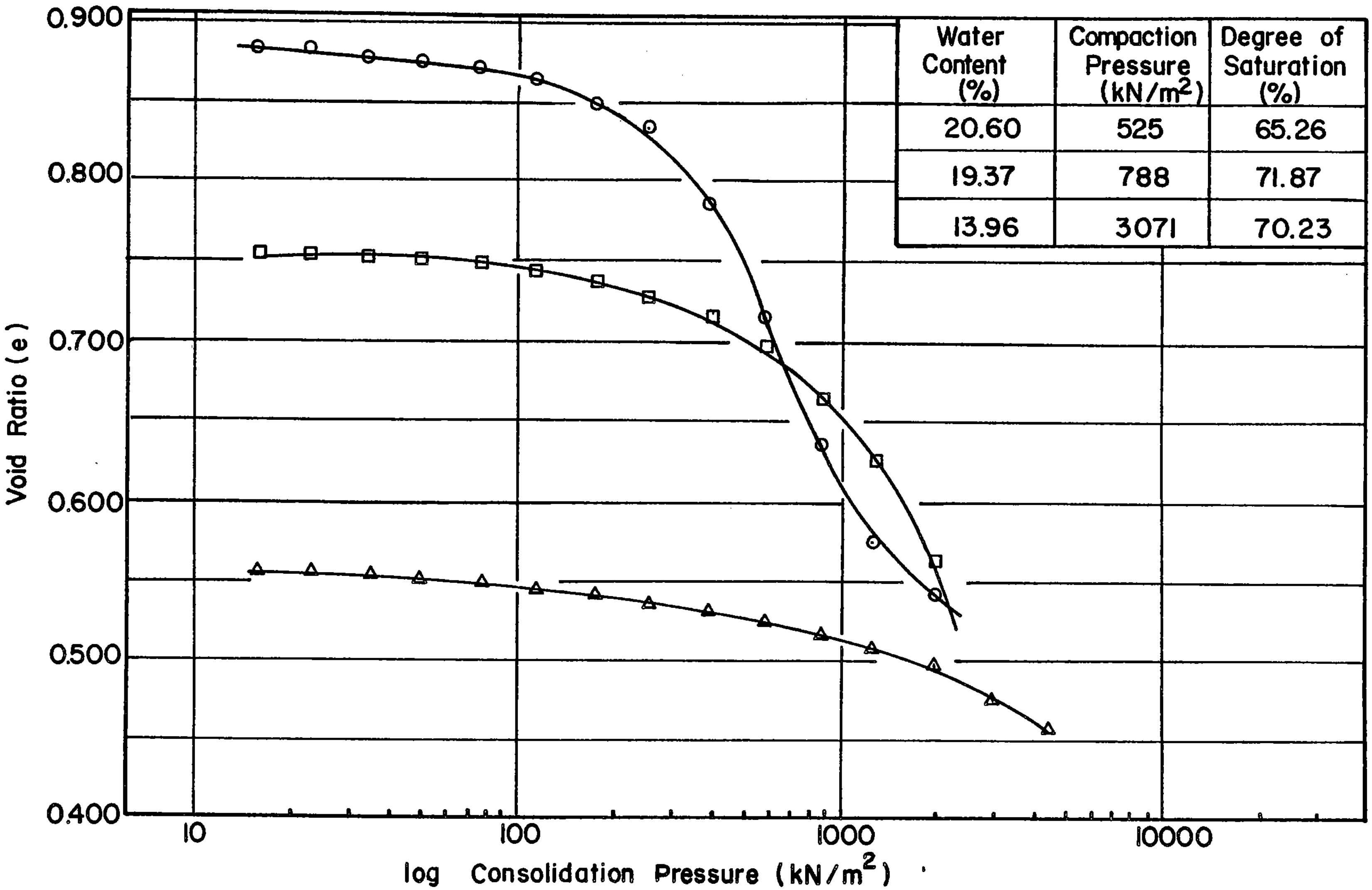


FIGURE 2 EFFECT OF COMPACTIVE EFFORT ON COMPRESSIBILITY (DRY OF OPTIMUM); ST. CROIX CLAY

After DiBernardo (1979)

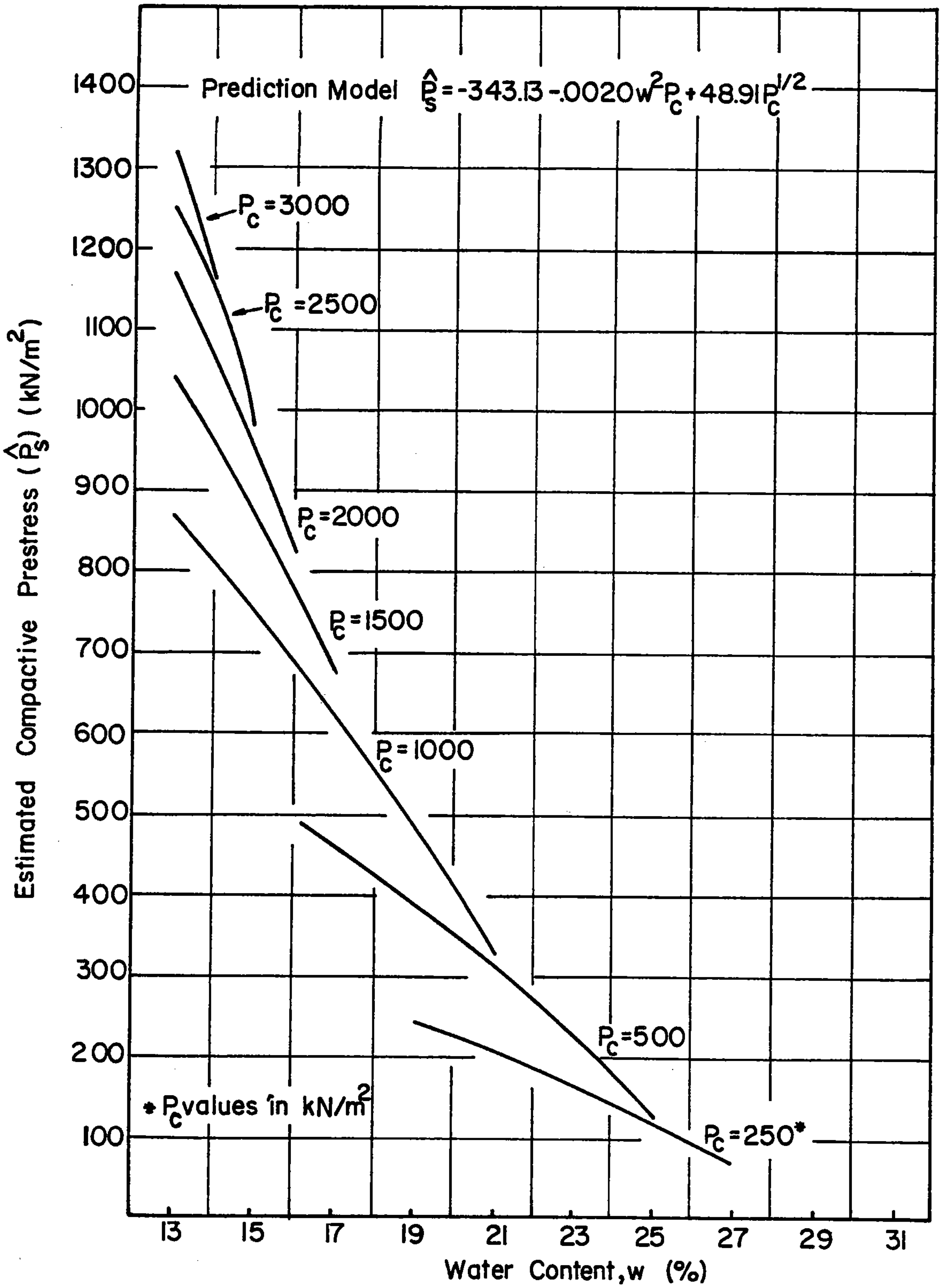


FIGURE 3 PREDICTION OF COMPACTIVE PRESTRESS; ST. CROIX CLAY

After DiBernardo (1979)

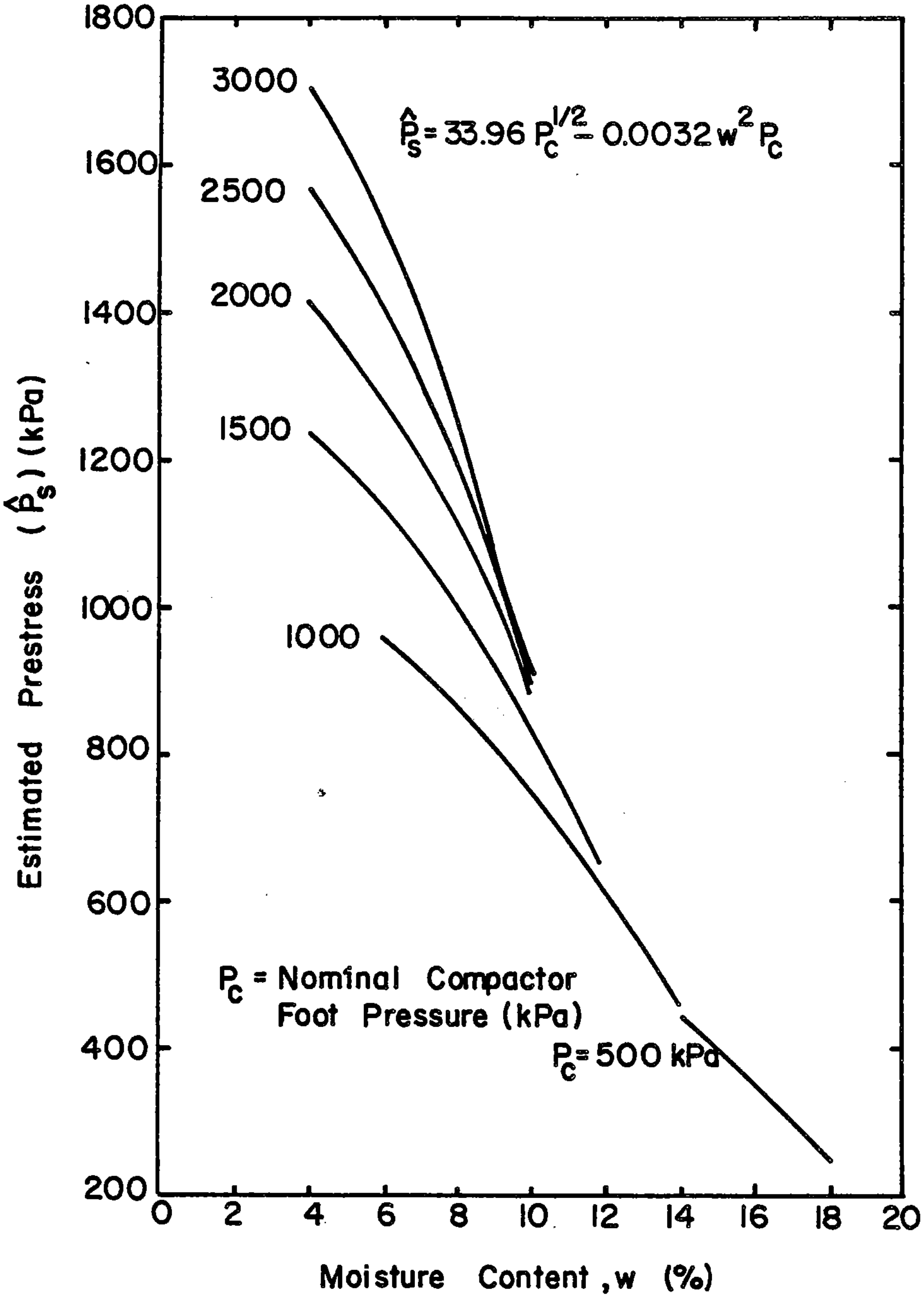


FIGURE 4 COMPACTIVE PRESTRESS MODEL; NEW PROVIDENCE SHALE

After Witsman (1979)

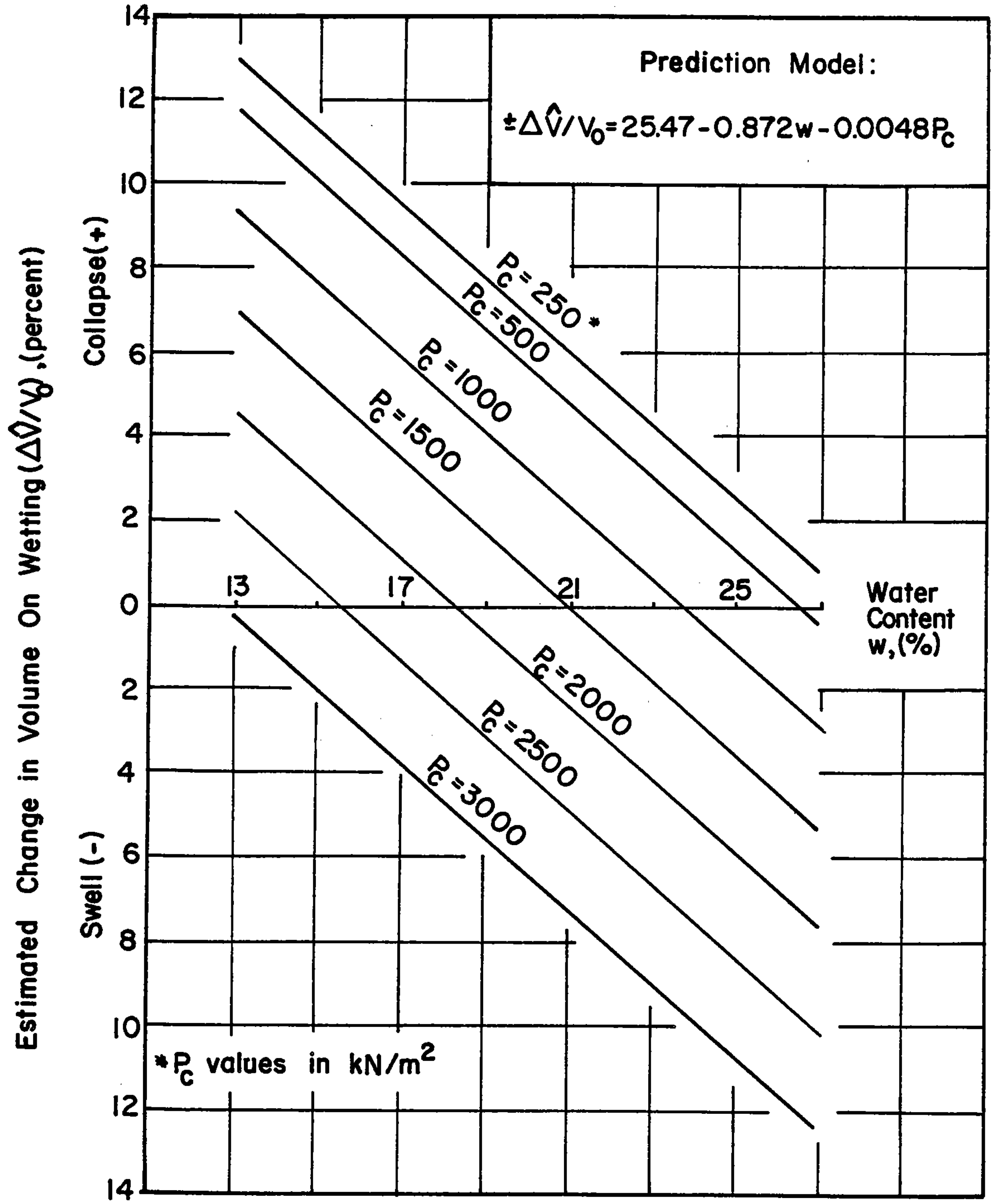
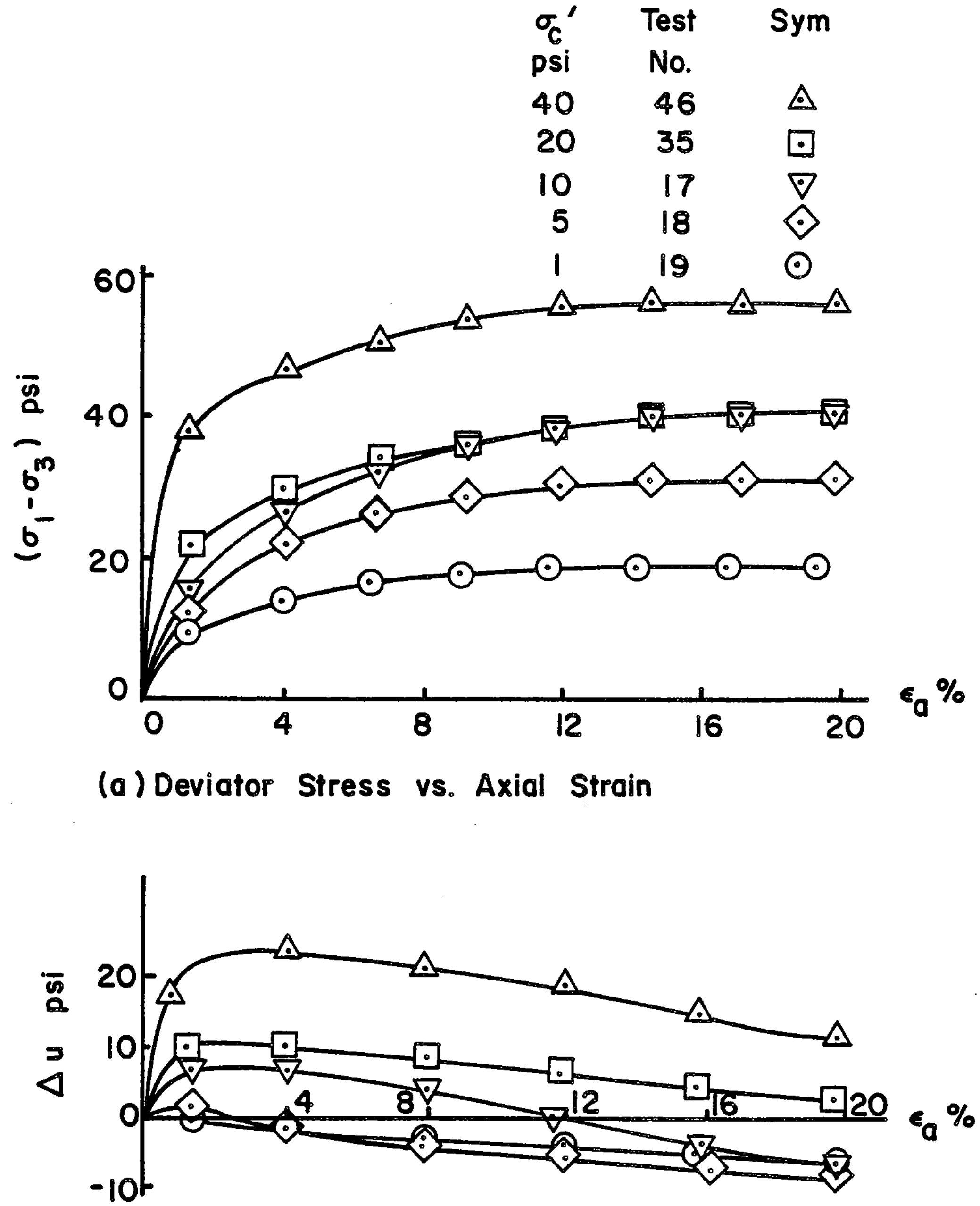


FIGURE 5 PREDICTION OF PERCENT VOLUME CHANGE ON WETTING FOR ST. CROIX CLAY

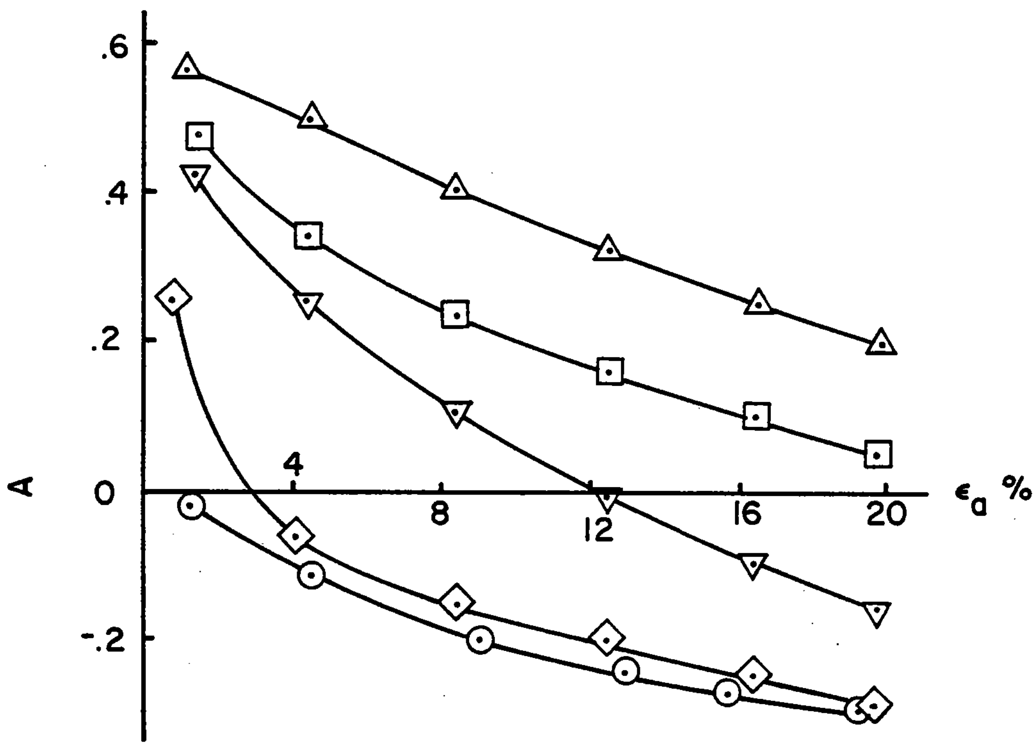
After DiBernardo (1979)



(σ_c' is Consolidating pressure)

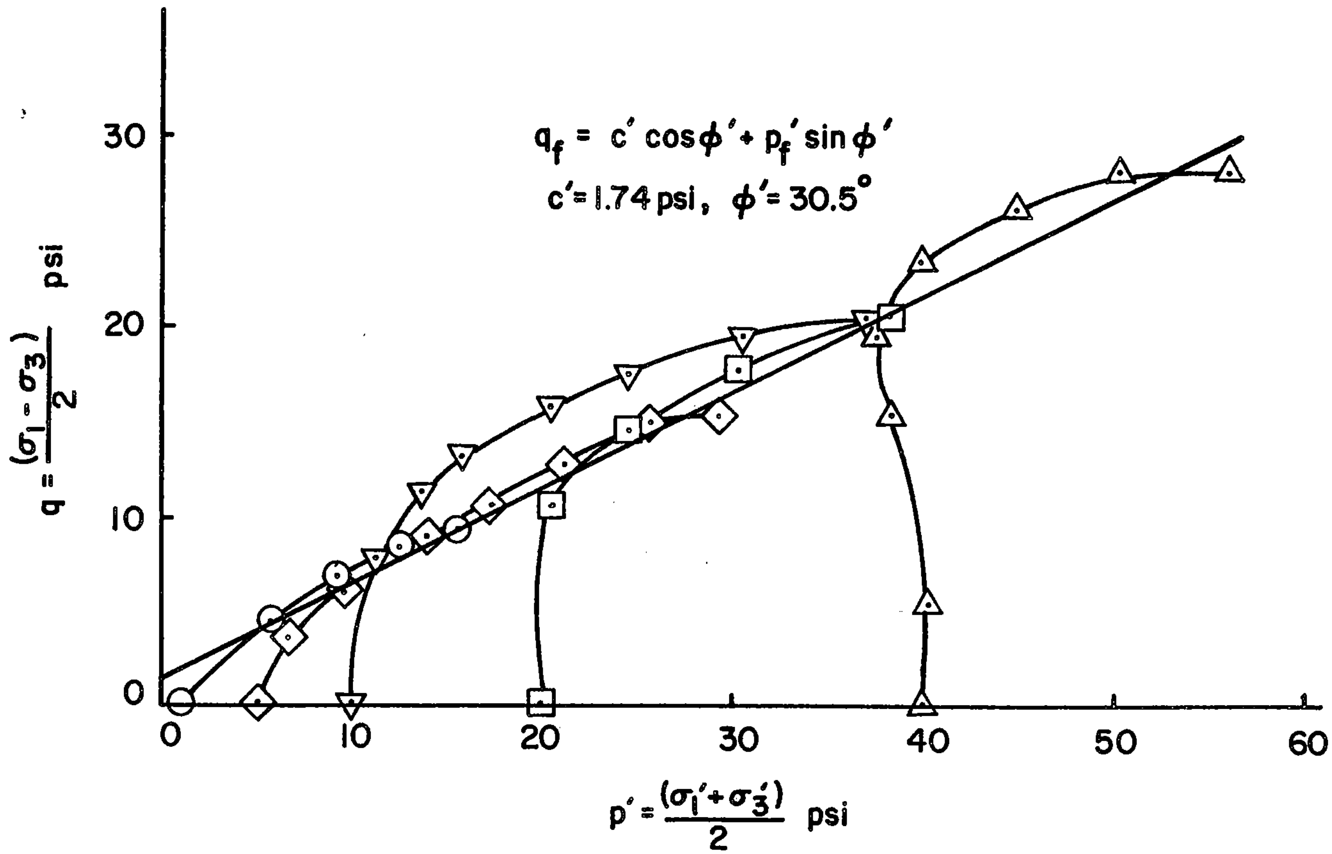
FIGURE 6 $\overline{\text{CIU}}$ RESULTS, SERIES 2-25-200; NEW PROVIDENCE SHALE

After Abeyesekera (1978)



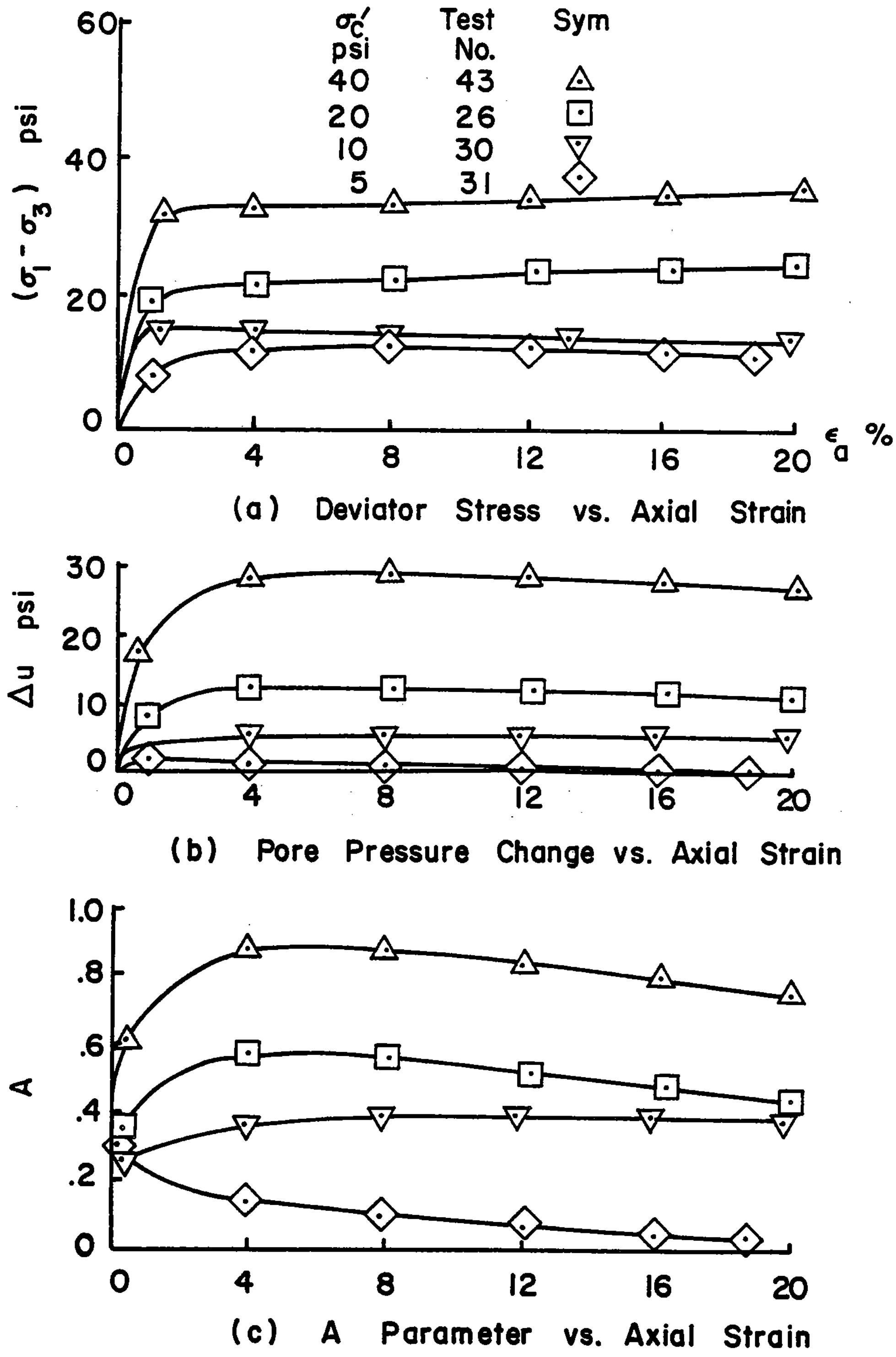
(c) A Parameter vs. Axial Strain

FIGURE 6 CONT'D



(d) Effective Stress Paths

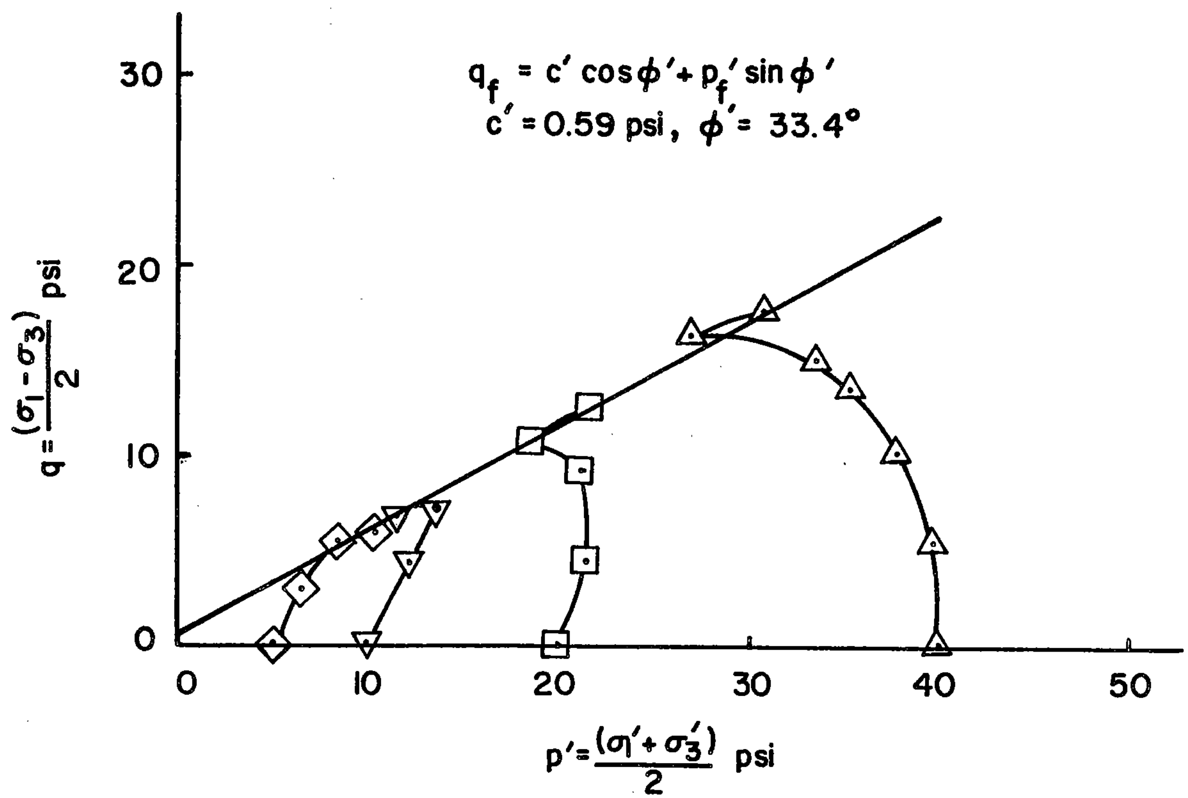
FIGURE 6 CONT'D



(σ_c' is Consolidating pressure)

FIGURE 7 $\overline{\text{CIU}}$ RESULTS, SERIES 5-30-50; NEW PROVIDENCE SHALE

After Abeyesekera (1978)



(d) Effective Stress Paths

FIGURE 7 CONT'D

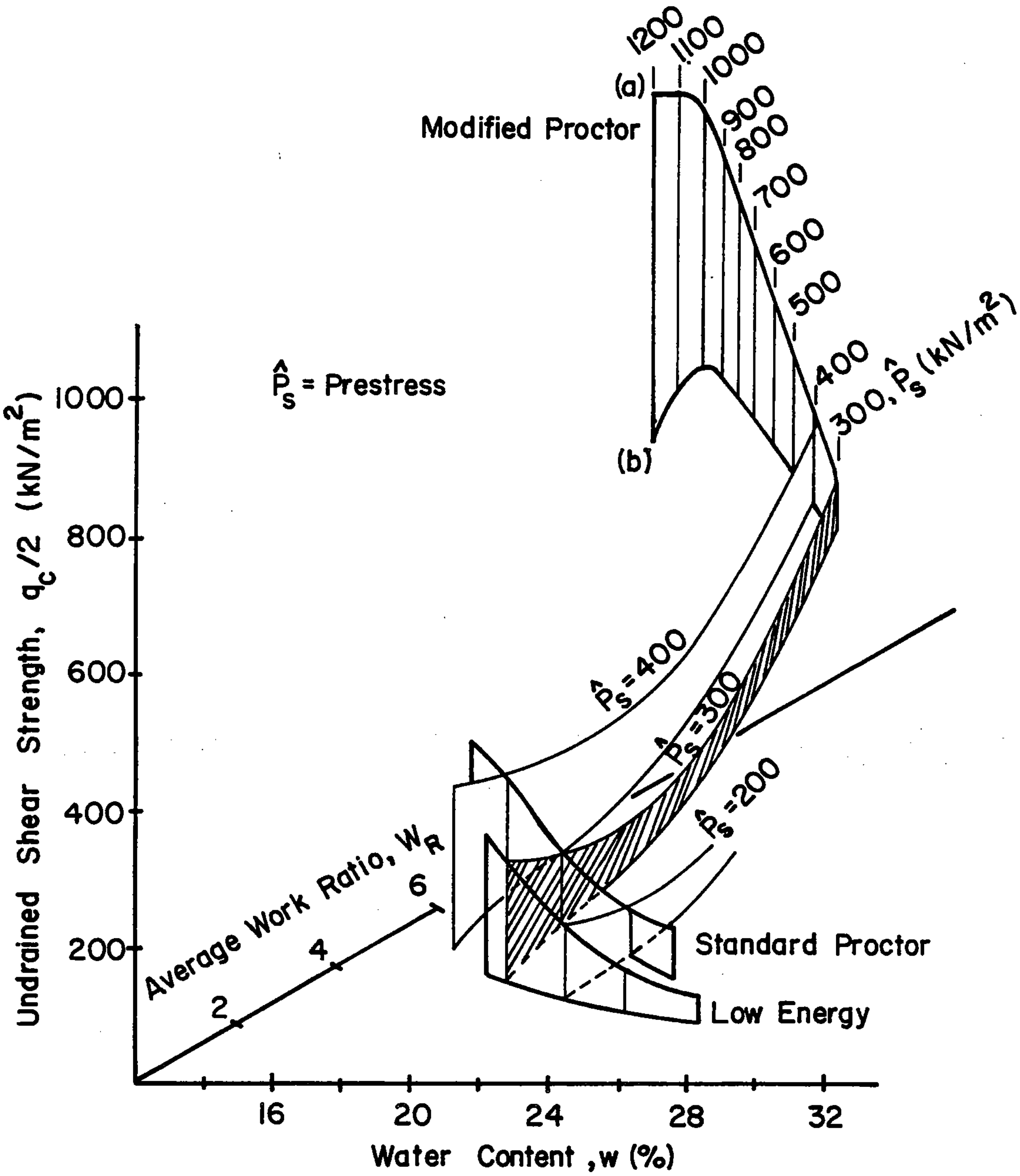


FIGURE 8 RELATIONSHIP OF PREDICTED PRESTRESS TO UNDRAINED SHEAR STRENGTH FOR ST. CROIX CLAY

After Weitzel (1979)

A CASE FOR UTILIZING PEDOGENIC AND GEOLOGIC STRATIGRAPHY TO REDUCE SAMPLING AND TESTING VARIABILITY FOR HIGHWAY SOILS INVESTIGATIONS

AUGUST, 1979

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INTRODUCTION

This paper is a partial summary of a highway soils investigation project conducted by an engineering consulting firm in Iowa. The primary objective of the investigation was to fulfill the requirements as set forth in a contract with the Iowa Department of Transportation for subsurface investigation and design. Specific responsibilities included reconnaissance and planning, test boring and sampling, testing of samples, the report of findings and design recommendations outlining construction methods or controls necessitated by subsurface conditions.

Another facet of the investigation was to try to demonstrate the utility of using existing geologic and detailed agricultural soil survey information for a reconnaissance of engineering conditions, and potentially as a guide to the on-site investigations. This will be the primary topic addressed in this paper.

Project Location

The project involved a nine (9) mile length of Iowa Highway No. 16 from the intersection of Iowa No. 280 east to the intersection of U.S. No. 218. Geographically, the highway is located in the northeast corner of Lee County, which is in the southeast corner of Iowa, Figure 1.

Preliminary Reconnaissance and Planning

Reconnaissance and planning was the first stage of the project and consisted primarily of compilation of existing geologic data, agricultural soil surveys, and other related pertinent information. The major objective was to predict the occurrence of:

1) Unsuitable Soil Materials, such as those soils with:
   a) Low Bearing Capacity
   b) High Organic Matter Content
   c) Expansive Clays

2) Frost Action Susceptibility
3) Water Table Position
4) Non-Uniform Soil Materials
5) Sources for Borrow
6) Excavation Problems, resulting from:
    a) Water Seepage
    b) Cut Slope Stability
7) Erosion Problems
8) Potential Settlement Problems

An important segment of this stage of the study was the construction of a geologic cross section for the length of the highway. Since detailed topographic maps were not available, surface elevation control was obtained by using the elevation control for the existing road which very nearly followed the natural ground contours.

Another important aspect of this study was the use of agricultural soil survey information. In Iowa, modern soils surveys are conducted in great detail, the mapping being done at a scale of 4 inches equals 1 mile, for the entire state. More importantly, however, is that the soil survey program is an integrated cooperative program directed jointly by the USDA, Soil Conservation Service, Iowa State University, the Iowa Department of Soil Conservation, and with considerable research input from the Iowa Geological Survey. This integrated effort results in extensive quantitative research into the geologic-geomorphic relations of the soils, as well as a high degree of quality control on the mapping and correlation of the soil series. The soil series are defined by their physical properties, which of course, control their engineering characteristics.

The detailed modern agricultural soil survey was utilized to interpret soil materials. By transferring the agricultural soil series boundaries to the centerline cross section (see Figure 2), it was then possible to begin to define material layers, since soil series are directly related to soil parent materials. This information, joined with a regional knowledge of loess thickness, the nature of the palesols, and the relations of these materials to the landscapes, resulted in the predicted cross section seen in Figure 2.

From the cross section, three (3) major geologic and two (2) pedologic units are identified. The geologic units are till, alluvium and loess, with the pedologic units being the palesols and the modern surficial soils.

Till

The tills in Iowa are generally medium to moderately fine loam and clay loam textured. This project area is believed to have been glaciated at least twice with the upper till classically considered Kansan in age. The Kansan till in Iowa is generally loam textured. The till is only exposed at the surface where erosion has stripped the overlying deposits, generally on the steeper sideslopes.

Paleosols

There was perhaps as much as several hundred thousand years between the deposition of the Kansan till and the deposition of the overlying loess (Hallberg and Boellstorff, 1978). During this time, extensive weathering took place on the till. On the uppermost tabular divides, a soil developed through Yarmouth and Sangamon time. The soil is called the Yarmouth-Sangamon paleosol. This soil has a grey color, is highly weathered, clay rich and is 8 to 20 feet thick. Erosion also took place which formed extensive erosion surfaces below the Yarmouth-Sangamon upland. This erosion produced a stepped landscape of interfluve ridges which are separated from the uplands by steeper slope gradients (Ruhe, 1969a).

A paleosol also formed on these younger erosion surfaces; it is also very clay rich, but is only 5 to 8 feet thick. A knowledge of these landscape characteristics allows the prediction of where these problematic soils will occur. Also, in the study area, different soil series are mapped on these different paleosols: the Clarinda series on the thick Yarmouth-Sangamon, the Keswick series on the lower, younger paleosols.

Loess

Loess is an eolian deposit consisting dominantly of silt size particles. The landscape is mantled by loess except in steeply sloping areas where erosion has removed the loess, exposing the till and associated paleosols. The loess thickness is eight (8) feet in this area. Basal radiocarbon dates in the loess are approximately 28,000 to 21,000 years before present, placing the age of the loess in late Wisconsinan time.

Alluvium

Erosion has been taking place throughout geologic time creating alluvial deposits having a wide range of compositions (texture). Prior to loess deposition, the alluvial deposits should be expected to be somewhat sandy, as compared with post loess deposits which should have a higher silt content.

"Old alluvium" is also encountered along the route. This old alluvium is encountered in terraces. The terraces are mantled by loess, which obscures the nature of the underlying alluvium. In many cases, the modern surficial soils formed in the loess on these terraces is very similar to the soils developed in the loess in the upland areas, and hence the same soil series may be mapped on the upland and terrace positions. The difference in the substratum for engineering purposes is very significant (i.e., loess over paleosols and till, vs. loess over paleosols and alluvium or just alluvium). Consequently, in the course of soil survey mapping, the soils on the terraces have been annotated with a prefix (such as T130, see Figure 2) to clearly indicate this important difference.

Modern Surficial Soils

Although the surficial soils comprise a relatively thin veneer in the cross section, they are perhaps the most important to recognize. Pedogenic or soil development involves the weathering of a parent material such as loess or till into a soil possessing distinct genetic layers or horizons. The two most important layers are the A and B horizons. Organic matter accumulation and the removal of clay, iron and aluminum are characteristic processes associated with the A horizon. The B horizon is most noted for the addition of illuvial clay size materials from above horizons and insitu formation of clay size minerals. The soils in Iowa are moderately to strongly developed, producing significant differences in the horizons. This horizonation is often overlooked in some engineering investigations. As will be shown in the following sections, these horizons formed by pedogenic processes significantly effect the engineering properties of surficial soils and paleosols.

Summary

A complete stratigraphic cross section for an upland position in the project area should include a strongly developed modern soil derived in loess which mantles a thick paleosol developed in till. In the strongly sloping areas, the surficial soils are less strongly developed and are formed in the various geologic materials exposed on the hill slopes. The valleys and waterways are filled with alluvium which varies in texture depending on the source of the sediment.

ENGINEERING PROPERTIES

With the knowledge of geologic materials and stratigraphy, and the characteristics of the soil profiles, a fair number of qualitative predictions can be made concerning design and construction. In Iowa, quantative data is also available for these geologic and pedologic units. During the last 25 years, highway soils engineering data has been obtained through the cooperation and cooperative effort of the USDA Soil Conservation Service, the Iowa State University Agriculture and Home Economics Experiment Station, and the Iowa Department of Transportation. Soil samples for major soil series (such as Grundy, Haig, Clarinda, etc., in Figure 2) were collected by soil scientists as soil mapping programs were conducted in various counties. These carefully collected samples were then analyzed for certain specific engineering properties by the Iowa DOT.

The results of this work have been informally distributed over the years. Recently, the results to date for 264 soil types were compiled and published in Iowa Geological Survey, Technical Information Series

No. 7 (Miller, Highland, and Hallberg, 1978). This is the source of the quantitative data below. This data is not for design purposes, but is intended to be used for preliminary design and problem evaluation as shown below.

Using this data for the appropriate soil series, in combination with the known stratigraphy, some important engineering properties can be quantified for the project area. For example, Figure 3 shows an annotated stratigraphic column for the upland areas, which can be represented by the data for the Haig, Clarinda, and Shelby soils (see Figure 2). These soil series represent soils developed in loess, the paleosol, and the till, respectively. From this data some specific conclusions may be made, about potential problems to be encountered, prior to any actual investigation:

1) The A horizon of the surficial loess derived soil is thin and has a relatively low maximum proctor density limiting it to restricted placement in the subgrade.

2) The B horizon is about 3.5 feet thick and has a very high clay content resulting in an A-7-6(20) classification. This designates the material as unsuitable, requiring a special type of disposal.

3) The C horizon or loess parent material has better compaction characteristics as reflected in the maximum proctor density, but the high silt content will require some restrictions on its placement in the subgrade.

4) The paleosol also has a very high clay content resulting in an A-7-6(20) classification, which requires special disposal.

5) The till meets the requirements for select material or borrow, and judging from the aforementioned observations, would be needed in large quantities.

Figure 2 shows that several different loess derived soils and three (3) soils derived from paleosols are crossed by the highway. It is important to note here that different soil series developed in the same material can possess differing engineering properties. The detail and control of the agricultural soils mapping in Iowa produces reliable and repeatable information though, and a particular soil series will be represented by a narrow range of engineering properties, regardless of where it is mapped. Also, soil series (developed in the same material) which occur in association will generally have similar properties which are gradational between the associated end members.

All of the loess soils encountered are similar in their properties, and for this evaluation, for practical purposes they have been lumped together in Table 1. Table 1 is a summary of the properties for the major soils involved in the project.

Figure 4 is a plot of the maximum proctor density data from Table 1. This graphically shows the range of properties of the different soil or geologic horizons. Even with a minimum number of observations, one property combined with stratigraphy can be used to separate the layers into distinct groups. A combination of the measured properties is used to determine the use of the material in road construction. Since the range of variation within each test is narrow, this should suggest that the field investigation be conducted in such a manner as to modify or refine preliminary summary properties and to accurately measure the thickness and depth of the horizons.

From Table 1, is becomes apparent that the conclusions drawn from Figure 3 apply for a major portion of the project. Some additional items can also be added to the reconnaissance and planning considerations from other studies:

1) Montmorillonite is the dominant clay mineral in Iowa soils (Hallberg, Lucas, and Goodmen, 1978). Thus, as the clay content increases, so does the risk of shrink-swell induced problems (Hallberg, 1978).

2) The Yarmouth-Sangamon paleosol is nearly impermeable and perches water which infiltrates through the moderately permeable loess. The result is a perched water table (Hallberg, Fenton and Miller, 1978).

3) In the natural landscape where the loess - paleosol contact is exposed, seeps often occur (Ruhe, 1969a).

4) Since the loess is thin and the annual rainfall is relatively high, the water table can be expected to be near the surface resulting in construction problems (Ruhe, 1969b).

5) Also the high water table in combination with the silty soil results in a high frost heave susceptibility.

6) Study of the diagrams indicates a large volume of unsuitable materials overlying the till. Potential borrow areas will be limited to steeply sloping areas where geologic erosion has removed most or all of the unsuitable materials.

7) Other problem areas may be identified relating to slope stability, revegetation, poor subgrade materials, etc.

<u>Summary</u>

The compilation and summary interpretation of the preliminary information would seem to be a prerequisite before any field investigations are conducted. It would seem prudent and economic to design the intensity of the investigations around the problems described above. Also, many important design and construction considerations were anticipated prior to a field investigation. This information allowed design personnel to proceed with a good background of items to take into consideration during the design process, minimizing the risk of the need for comprehensive changes in design relating to problems identified in the field study.

This approach can change the goals of a field investigation from a "go out and find out what is there approach" to a more specific task of modification or refinement of existing information. Rather than "rediscovering" geologic stratigraphy, we should concentrate on accurate measurement of the layers.

Unfortunately, in many similar studies, much time is spent laying out arbitrary test boring patterns with no review of this existing data. The end result may be less accurate than a compilation of existing knowledge of stratigraphy and related engineering properties, because important units go unrecognized or are lumped with other materials.

TESTING

The frequency and depth of the test borings was defined in the IDOT contract. Thus, in reality, only minimal planning was needed for this part of the project. The preliminary planning did indicate a multilayered system of geologic materials, and strongly horizonated soils with some of the layers being significantly different than others with respect to engineering properties. Furthermore, the layers were relatively continuous across the landscape with some of them being thin.

To avoid missing layers or mixing different layers of soil materials into one sample, continuous cores were collected using a Giddings sampling device. The sampler collects an undisturbed core which is easily removed from the sampler and is ready for quick and accurate determination of stratigraphic or pedologic units and for sample testing and collection.

Field density was determined by using an Ely Volumeter. The volumeter sample was placed in a moisture tin and dried in an oven overnight for moisture determination. Bulk samples were returned to the laboratory for particle size analysis, Proctor density and moisture, Atterberg limits and classification, and consolidation tests. Rather than test a sample from a single hole, the bulk sample was a combination of samples from 2 or 3 consecutive holes for the same layer. The idea was to measure the engineering properties of specific soil layers over an areal extent rather than just point source data.

RESULTS

Stratigraphic Cross Section

After completion of the field borings and sampling, another stratigraphic cross section was constructed. Figure 5 shows the final cross-section based on the actual test drilling. Test holes were placed at least every 200 hundred feet. A comparison with Figure 2 shows only minor differences; mainly refinements in the thicknesses of various units.

One revision was the addition of a unit which was called Early Wisconsin Sediment. The origin of this material is not fully understood but its areal extent seems to be greatest in extreme S. E. Iowa. Other changes included more detail in the alluvial areas and in the loess-mantled "old alluvial" stream terraces.

Engineering Properties

Table 2 is a partial summary of the engineering data derived from the test samples for the Highway 16 project. A comparison of this data with the preliminary data in Table 1 show only a few differences. For example, Figure 6 is a plot of the preliminary versus actual measured maximum proctor density data. Shown are means and 95% confidence intervals for the means. The A, B and C horizons of the loess compare very well. Only small differences exist and are related to the number of observations and the amount of control exercised in sample collection. Obviously all the predicted values are in the proper range of the actual values.

The till data shows a little more spread between the preliminary and actual values. With increasing depth, the density of the till increases as the degree of weathering decreases. The preliminary optimum density till data is measured in the C horizon of a soil developed in till, while the project values are measured on till samples collected at various depths. If the till data is summarized by depth, the change in maximum proctor density with depth or for that matter other properties can be predicted.

Comparison of other properties can also be made, but the conclusions will be the same. Most of the differences between the preliminary data and actual project data can be attributed to the number of observations and sampling techniques. More important though, is the fact that the engineering interpretations made for the units in the preliminary data set are the same for the actual project data.

CONCLUSIONS

The main objective of this paper has been to demonstrate the utility of using existing geologic and agricultural soil survey information as a basis for a reconnaissance of engineering conditions and potentially, as a guide to the on-site investigation. Today's economics demand prudent use of our resources. Field investigations should be designed and conducted with the major goal of building on existing knowledge and information. The high degree of correlation between the laboratory data and field observations with the preliminary compilation of stratigraphic and engineering data supports this type of approach for highway soils investigations or other types of preliminary site evaluation. This degree of correlation and the narrow range of test results of the individual materials can be accomplished only if the field engineer is trained to examine and interpret geologic and pedogenic stratigraphy in the field. A result of this kind of sampling can be even better predictive models for design and construction with a reduction in random repetitious sampling and testing.

This method would seem to have particular merit for use in alternative route or site evaluation. Engineering problems and problems with suitable borrow and fill could be predicted accurately, so that alternative costs and designs could be reviewed, without any (or little) expensive preliminary field investigations.

REFERENCES CITED

Hallberg, G. R., 1978, Calibration of Cole-calculated engineering data for montmorillonitic soils, Agron. Abs., 70th Ann. Meeting, Amer. Soc. Agron., p. 169.

Hallberg, G. R., and Boellstorff, J. B., 1978, Stratigraphic "Confusion" in the region of the type areas of Kansan and Nebraskan deposits, Geol. Soc. Am., Abs. with Prog., V. 10, No. 6, p. 255-256.

Hallberg, G. R., Fenton, T. E., and Miller, G. A., 1978, Standard Weathering Zone Terminology for the description of Quaternary sediments in Iowa, Iowa Geol. Surv., Tech. Info. Ser. No. 8, p. 75-109.

Hallberg, G. R., Lucas, J. R. and Goodmen, C. M., 1978, Semi-quantitative analysis of clay mineralogy, Iowa Geol. Surv., Tech. Info. Ser., No. 8, p. 5-22.

Miller, G. A., Highland, J. D., and Hallberg, G. R., 1978, Highway Soil Engineering data for major soils in Iowa, Iowa Geol. Survey, Tech. Info Ser. No. 7, 109p.

Ruhe, R. V., 1969b, Application of Pedology to Quaternary Research, in S. Pawluk (ed.), Pedology and Quaternary Research, Univ. Alberta, Edmonton, p. 1-23.

Ruhe, R. V., 1969a, Quaternary Landscapes in Iowa, Iowa State Univ. Press, Ames, Iowa, 255p.

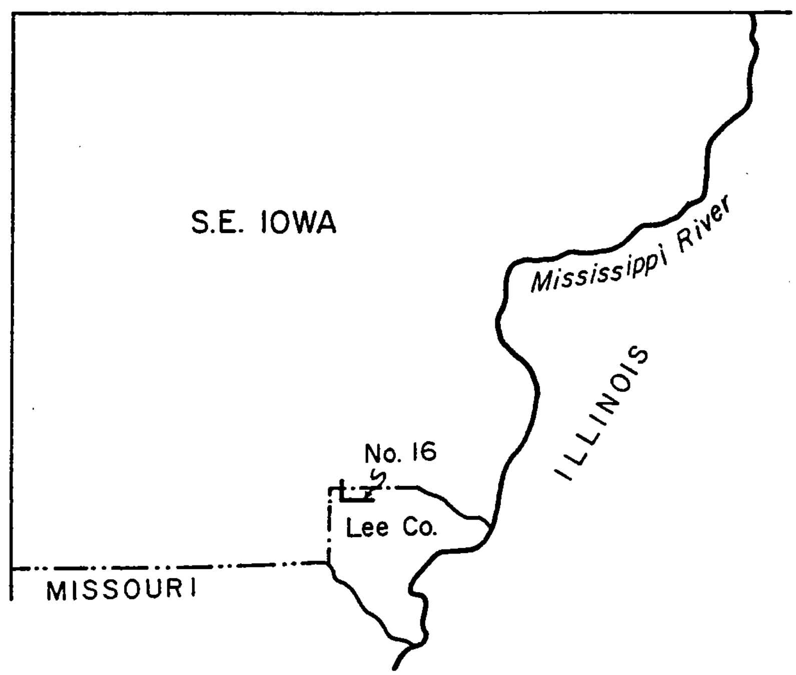


Figure 1. Location map of project area, southeast Iowa.

TABLE 1
SUMMARY OF ENGINEERING DATA FOR PROJECT AREA
SOIL SERIES FROM (MILLER, HIGHLAND & HALLBERG, 1978)

Sample		n	Maximum Proctor Density (pcf)	Optimum Moisture (%)	Clay <2μ	Liquid Limit	Plastic Index	AASHTO Classification
Surficial Soil (Loess)								
A Horizon	Mean	8	96.6	21.3	27.0	39.8	13.5	A-6 (8-10) &A-7-6 (11-12)
	Std. Dev.		3.1	1.7	5.3	3.7	2.5	
	95% CI		± 2.6	± 1.4	± 4.4	± 3.1	± 2.1	
B Horizon	Mean	8	93.8	23.6	47.0	65.0	35.5	A-7-6 (20)
	Std. Dev.		4.7	2.8	5.9	4.5	4.1	
	95% CI		± 3.9	± 2.3	± 4.9	± 3.8	± 3.4	
C Horizon	Mean	6	107.7	17.5	30.8	42.8	21.7	A-6 (11-12) &A-7-6 (13-16)
	Std. Dev.		1.2	0.9	2.0	2.8	3.0	
(Loess Parent Material)	95% CI		± 1.3	± 0.9	± 2.1	± 2.9	± 3.1	
Paleosol								
B Horizon	Mean	4	98.8	21.1	47.5	59.5	35.3	A-7-6 (16-20)
	Std. Dev.		3.8	3.4	3.4	8.3	7.4	
	95% CI		± 6.0	± 5.4	± 5.4	±13.2	±11.7	
Till	Mean	3	112.3	15.7	28.0	38.7	22.3	A-6 (9-12)
	Std. Dev.		0.6	0.6	2.0	1.5	1.2	
	95% CI		± 1.4	± 1.4	± 5.0	± 3.8	± 2.9	
Alluvium	*		Highly Variable - Not Enough Data for Analysis.					

95% CI = Confidence Interval for Mean
Std. Dev. = Standard Deviation

TABLE 2
PARTIAL SUMMARY OF ENGINEERING DATA
FOR THE HIGHWAY #16 PROJECT

Sample		n	Field Density (pcf)	Field Moisture (%)	n	Maximum Proctor Density (pcf)	Optimum Moisture (%)	n	Clay <2u	Liquid Limit	Plastic Index	AASHTO Classification
Surficial Soil (Loess)												
A Horizon		17			22			22				
	Mean		92.4	27.2		97.4	21.1		22.1	42.6	13.2	A-5(8-9), A-6(8-11)
	Std. Dev.		5.6	3.3		2.5	3.4		3.6	3.6	2.7	
	95% CI		± 2.9	± 1.7		± 1.1	± 1.5		± 1.6	± 1.6	± 1.2	A-7-5(10-12), A-7-6(10-14)
B Horizon		28			19			25				
	Mean		92.1	29.0		96.3	23.3		39.6	59.6	32.3	A-7-5(19-20), A-7-6(17-20)
	Std. Dev.		3.4	2.8		1.7	1.5		3.6	5.6	6.5	
	95% CI		± 1.3	± 1.1		± 0.8	± 0.7		± 1.5	± 2.3	± 2.7	
C Horizon		33			27			25				
	Mean		95.5	27.6		104.4	19.1		26.9	42.0	20.3	A-6(10-12), A-7-6(12-16)
	Std. Dev.		2.0	2.0		2.8	1.5		2.2	3.2	4.4	
(Loess Parent Material)	95% CI		± 0.7	± 0.7		± 1.1	± 0.6		± 0.9	± 1.3	± 1.8	
Early Wisconsin Sediment		27			21			22				
	Mean		105.9	21.6		112.0	15.5		23.3	32.7	15.6	A-6(7-12)
	Std. Dev.		4.04	2.5		3.7	1.5		4.7	5.0	3.6	
	95% CI		± 1.6	± 1.0		± 1.7	± 0.7		± 2.1	± 2.2	± 1.6	
Paleosol		26			0			5				
	Mean		98.2	25.3		*	*		50.2	71.6	57.0	A-7-6(20)
	Std. Dev.		6.7	3.5					1.1	5.8	7.7	
	95% CI		± 2.7	± 1.4					± 1.1	±7.2	±9.5	
Till		21			27			27				
	Mean		118.8	15.2		119.1	12.3		23.4	32.2	16.3	A-6(3-12)
	Std. Dev.		4.4	1.5		2.5	0.8		4.0	2.8	6.3	
	95% CI		± 2.0	± 0.7		± 1.0	± 0.3		± 1.6	± 1.1	± 2.5	

n = # of Samples
* = No Observations

95% CI = Confidence Interval for Mean
Std. Dev. = Standard Deviation

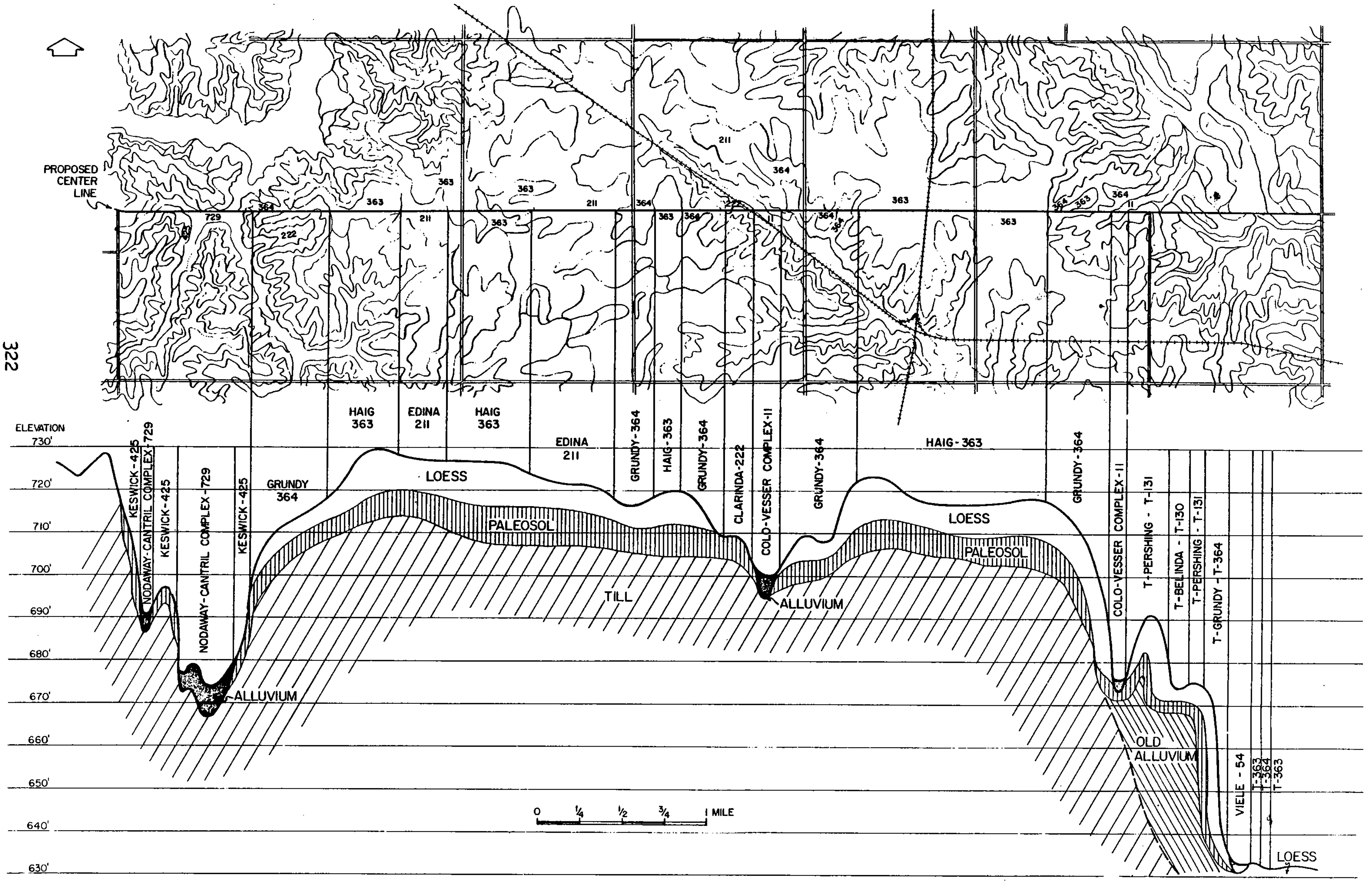


Figure 2a. Schematic cross-section along road center-line, compiled from existing data. Upper map view shows the soils map for the area. For clarity, the soil series (names and/or numbers) are only annotated along the center-line.

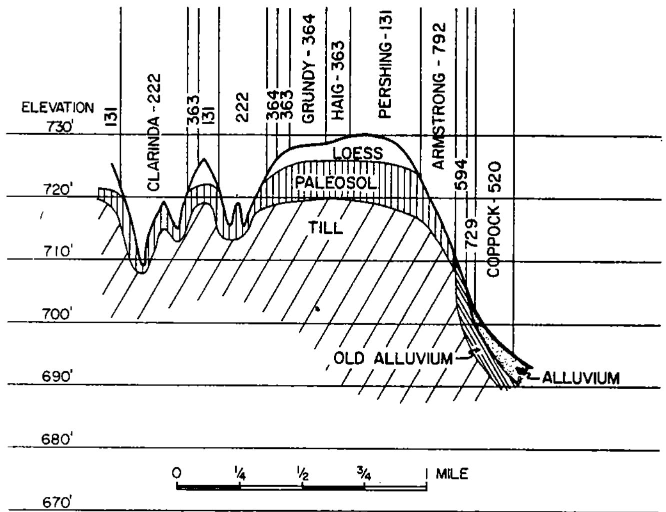


Figure 2b. Schematic cross-section along road center-line compiled from existing data.

Figure 3. Schematic stratigraphic column for upland area, compiled from existing data.

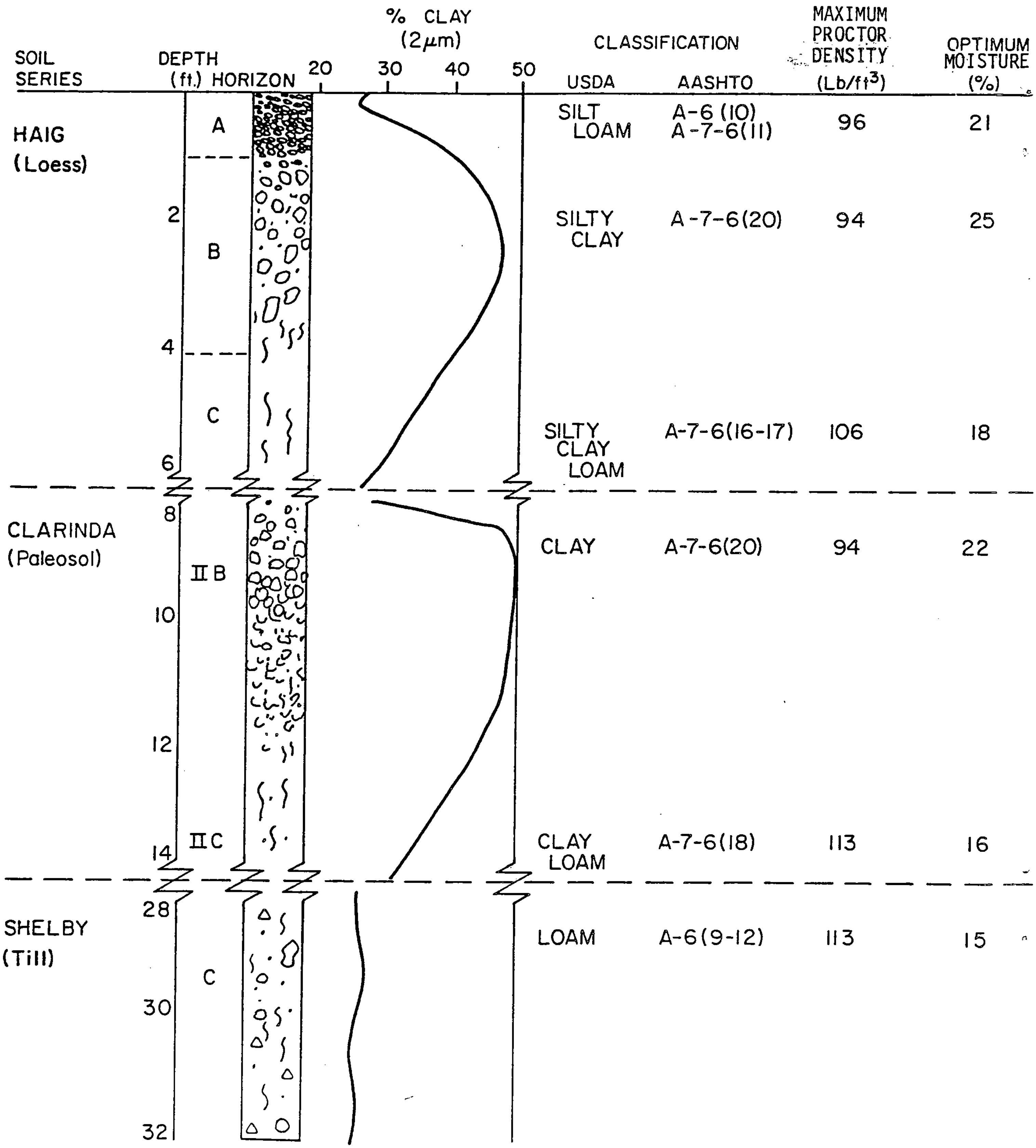
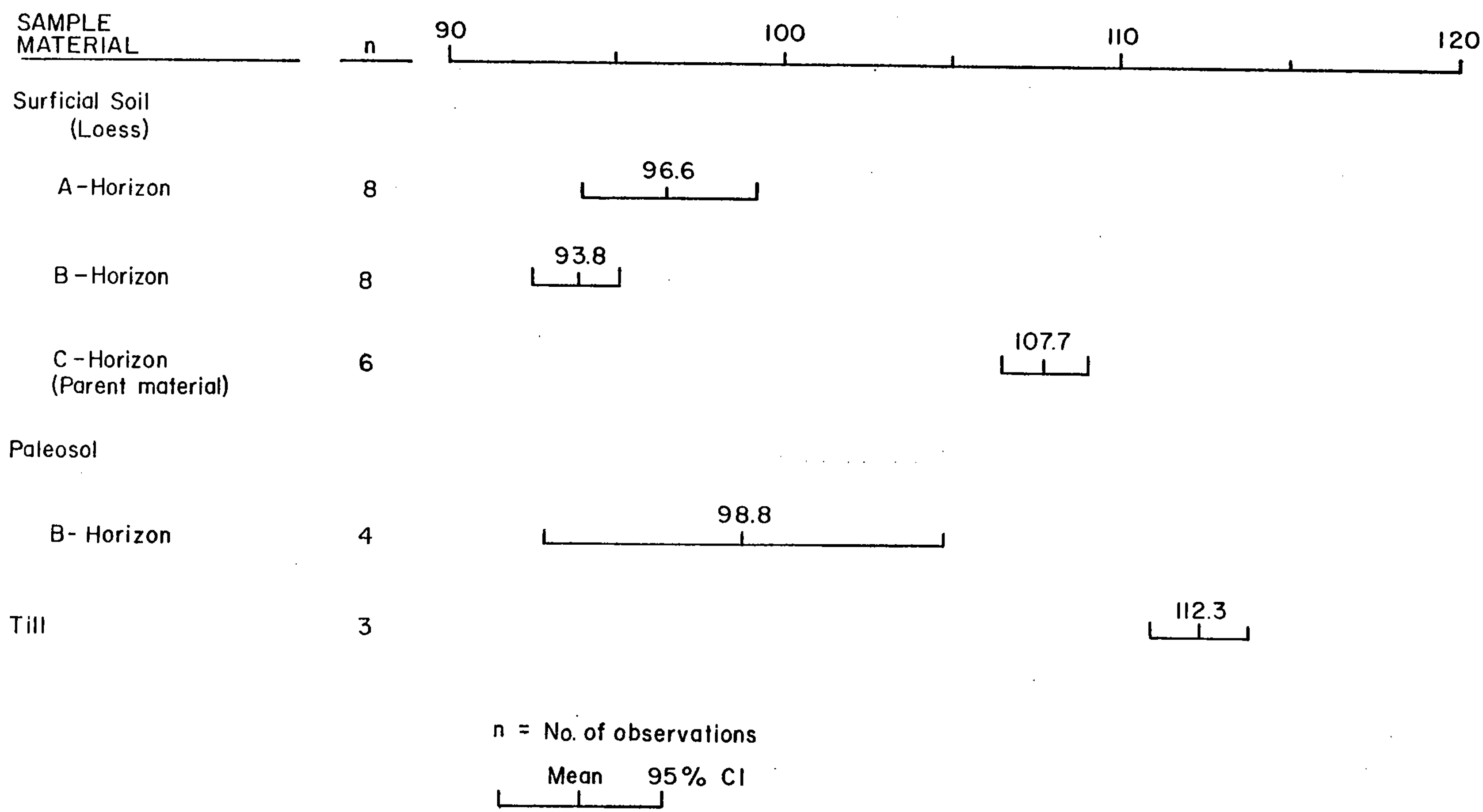
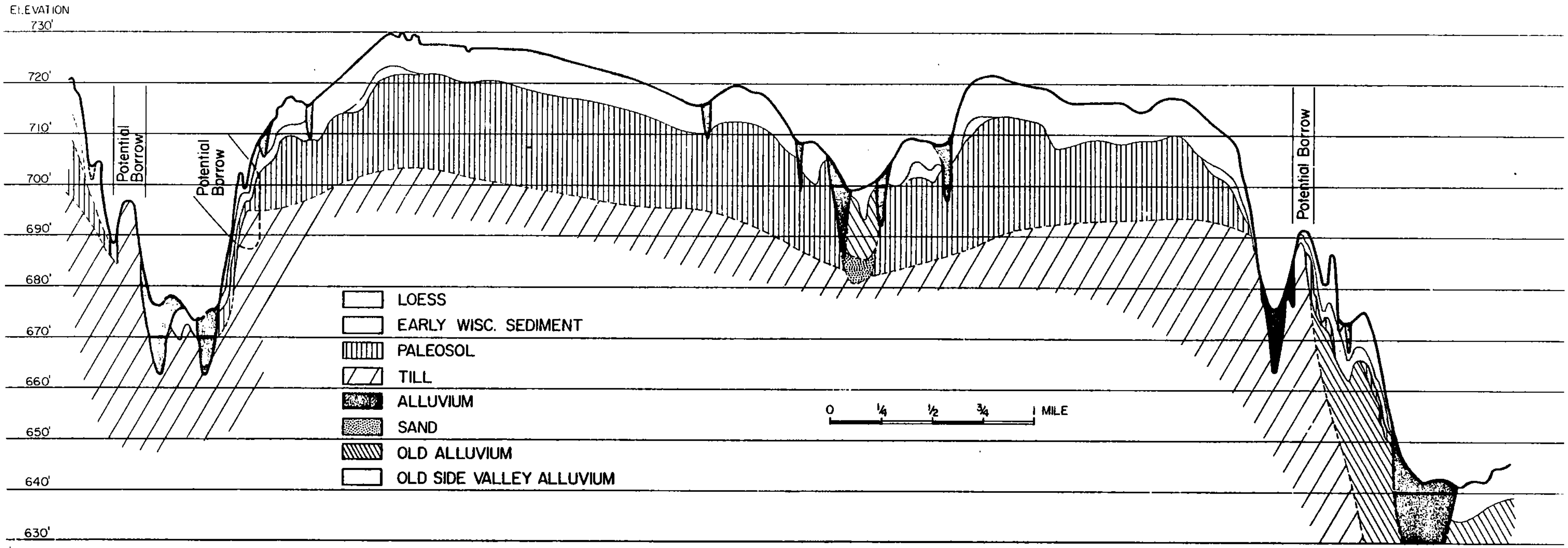


Figure 4. Summary plot of maximum proctor density data in pounds per cubic foot (from Miller, Highland, and Hallberg, 1978).





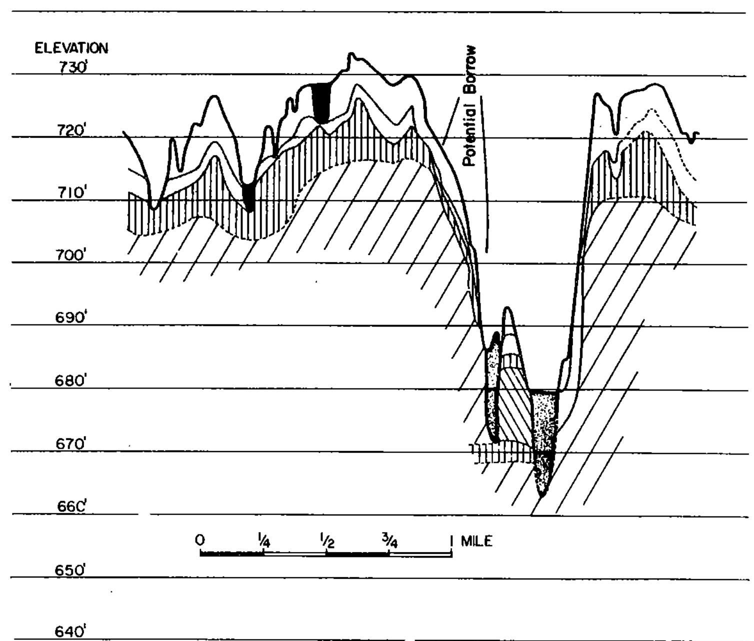
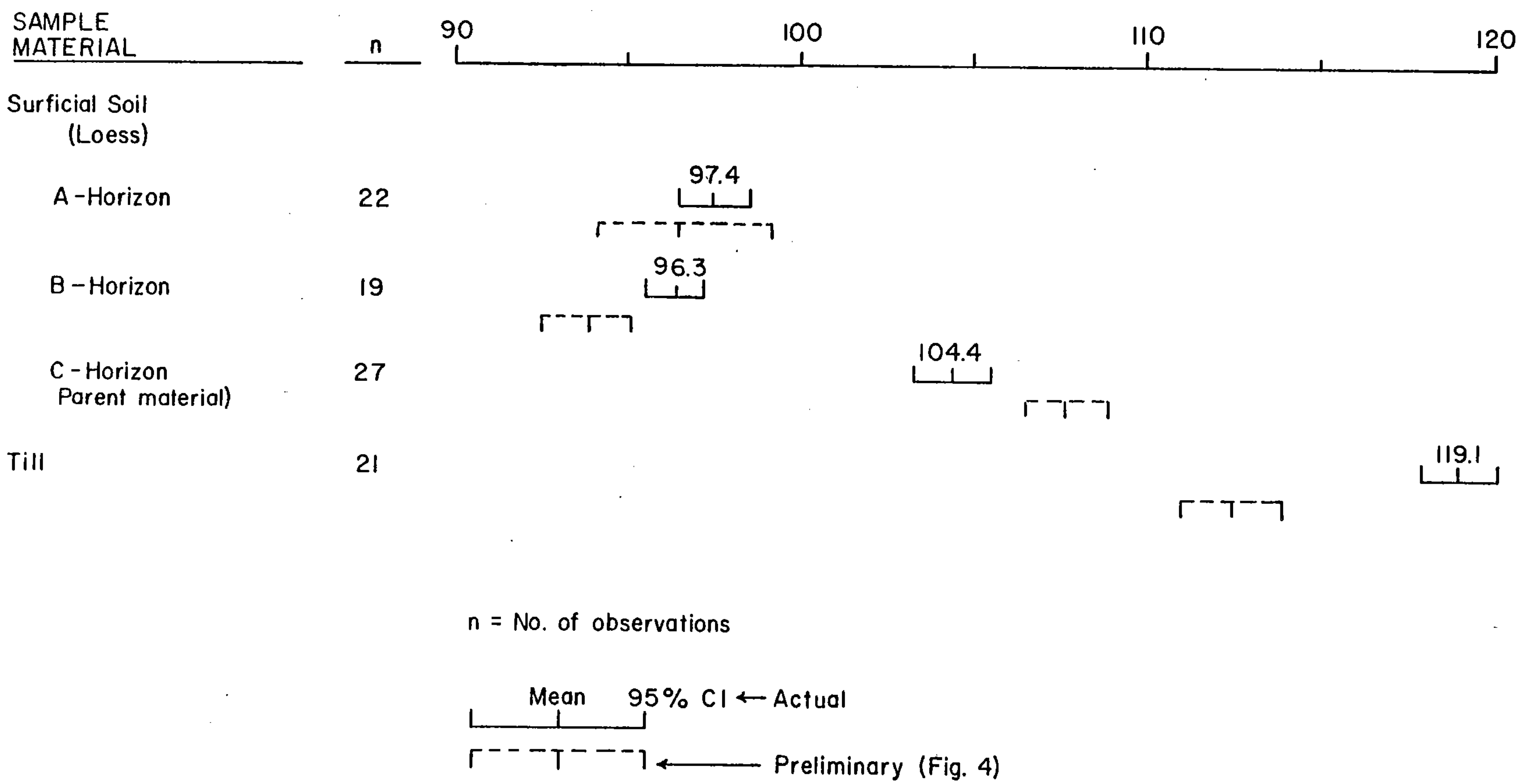


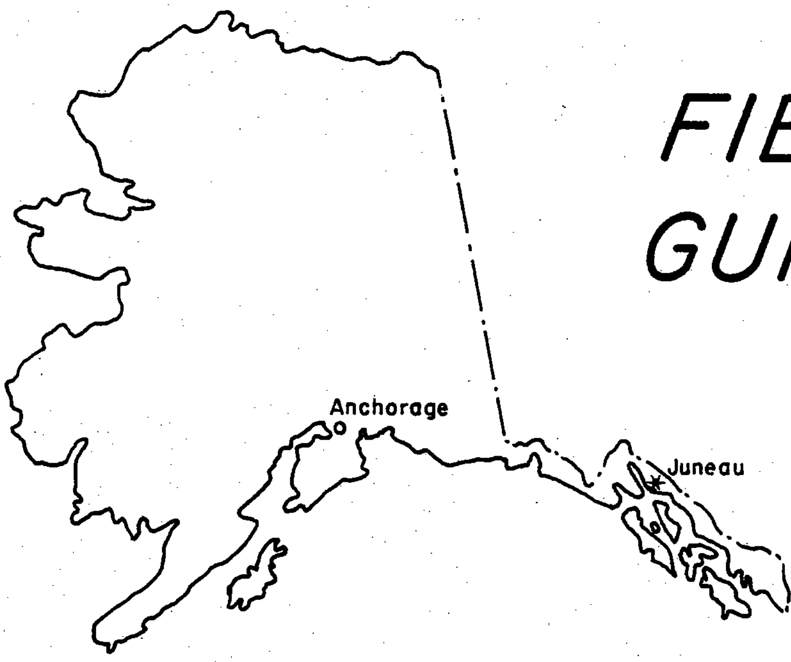
Figure 5. Final cross-section based on actual drilling data; compare with fig. 2.

Figure 6. Comparison plot of actual vs. preliminary estimated (Figure 4) maximum proctor density (in pounds per cubic foot).

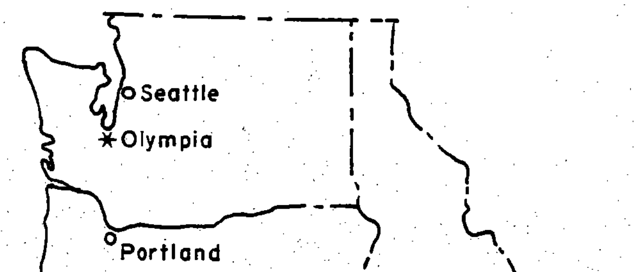


THIRTIETH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 8-10, 1979 PORTLAND, OREGON



FIELD TRIP GUIDE BOOK



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1979 HIGHWAY GEOLOGY SYMPOSIUM FIELD TRIP

GEOLOGY AND SCENERY OF THE LOWER COLUMBIA RIVER GORGE AND NORTHERN CASCADE RANGE, OREGON

The Field Trip selected for the 1979 Highway Geology Symposium traverses the beautiful, often spectacular Lower Columbia River Gorge, carved by the mighty Columbia River, and the northernmost portion of the scenic Oregon Cascade Range, including majestic Mount Hood. The trip, approximately 200 miles in length, combines some of the Northwest's most beautiful scenery with some of the country's most youthful geology, and offers the traveler both scenic majesties and a sense of the forces that created them.

In addition, the trip will make a number of stops of a purely geotechnical nature at sites felt to be pertinent to Highway Geology and Engineering Geology. Discussions will be held at each of the stops, and are also included in the Road Log. Scenic stops for photography, wandering, and just "enjoying" will be made at Crown Point, Multnomah Falls, and Timberline Lodge on Mount Hood, the official "end" of the trip. We hope you all enjoy yourselves!

CONTENTS

SUMMARY OF THE GEOLOGY OF THE LOWER COLUMBIA RIVER GORGE AND NORTHERN CASCADE RANGE

The rocks and structures of the Lower Gorge and the Cascade Range reflect both the youthfulness and the power, sometimes violence, of the geological forces that shaped this part of the Pacific Northwest. The predominant rock-types in the Gorge are volcanic, consisting of lava flows of the Columbia River Basalt, and, at the western end, the younger Boring Lava (see stratigraphic column for relative ages of formations). Some sedimentary and volcanic sedimentary sequences, such as the Eagle Creek, Ohanapecosh, and Troutdale Formations also occur, but no metamorphic rocks are found. The Cascade Range consists predominantly of lava flows and eruptive, pyroclastic volcanic rocks of a generally andesitic composition. At its northern end the Range is formed of upwarped lava flows and sedimentary strata, onto which the younger rocks of the Cascade volcanoes, such as Mount Hood and Mount Jefferson, were deposited to form the High Cascades.

Because the Range is in part the result of folding, the oldest rocks to be seen on the Field Trip, those of the Miocene-age Eagle Creek Formation, are exposed in the deepest part of the Gorge, near the axis of anticlinal folding. The youngest rocks, on the other hand, occur at the western end of the Gorge, and include the fluvial and lacustrine sand, silt, and gravel terrace deposits of the Pleistocene-age Spokane Floods, as well as the more recent sediments of the Columbia and Willamette Rivers upon which the city of Portland is built.

STRATIGRAPHIC COLUMN OF LOWER COLUMBIA RIVER GORGE AND

NORTHERN CASCADES (OREGON)

FROM ALLEN, 1979

Surficial Deposits

Quaternary Alluvium (Holocene) - - - - Qral*

*Rock-unit abbreviations correspond to the symbols used on the Geologic Map of Oregon West of the 121st Meridian, a portion of which is reproduced herein.

Loose deposits of silt, sand and gravel bordering the Columbia River and its tributaries, usually less than 50 feet above water level. Includes some sand dunes, as at Rooster Rock State Park.

Quaternary Landslide Debris (mostly Holocene) - - - - Qls

Surficial slide deposits associated with large landslides, and composed of boulders and finer debris mostly derived from the Eagle Creek Formation and overlying lava flows. Best developed north of the river.

Quaternary Flood Deposits and Lake Beds - - - - Qpun

Unconsolidated gravel, sand, silt and clay deposited during the latest Pleistocene advances of the ice in northern Washington, by the torrential waters of the Missoula (Spokane) floods, especially the last Bretz flood. Contains widespread angular ice-rafted erratic boulders up to 8 feet in diameter. Forms high bars along the river, particularly behind promontories and within reentrants and up tributaries, and mantles much of the Portland and Vancouver area below 400 feet in elevation.

Quaternary Terrace Deposits (mostly Pleistocene) - - - - Qpln

Flat-lying elevated deposits of gravel, sand and silt of fluvial (river) and glacio-fluvial origin, best developed along the Sandy River and in East Portland. Divided by Trimble (1963) on the basis of elevation and degree of weathering into the Springwater (highest and oldest), Gresham (intermediate), and Estacada (lowest and youngest) Formations. Of equivalent age, but not mapped since it mantles much of the topography above 400 feet, is the Portland Hills Silt, a yellowish-brown loess (wind-blown silt) which is chiefly quartzose, with minor sand and clay; greatest thickness is 145 feet, averages less than 25 feet. It is thickest over the Portland Hills and on the Springwater Terrace, and has been divided into three units.

Stratigraphic Units

Pliocene and Quaternary Volcanic Rock - - - - QTba, Qa, Qb, QTw & Tpb

Up to several thousand feet of lava flows and minor amounts of cinders and breccias of dominantly basaltic composition. Usually light gray in color, frequently porous (diktytaxitic). Includes the Cascade Lavas (QTba and QTw) which form the thin flows in the upper cliffs on the south wall of the Gorge beneath the upland surface, and the shield volcanos which rise above that surface; the Boring Lavas (QTba) from numerous cinder cones in the Portland area; the young Intracanyon Lavas (QTba and Tpb) which lie in valleys tributary to the Gorge; and the Cascades Formation (Qa and Qb) which includes light gray andesite and pyroclastic rocks forming the composite volcanic cone of Mount Hood. Flows are generally less than 50 feet thick, except where ponded in narrow canyons.

Pliocene Sedimentary Rock - - - - Tpn (Troutdale Formation) and
Tmma (Rhododendron Formation)

Up to 1,400 feet of fluvial deposits of mudstone, sandstone, siltstone, conglomerate and volcaniclastic materials, occupying structural depressions and erosional valleys in Yakima Basalt. Overlain by Cascade and Boring Lavas west of Hood River. Includes the Troutdale Formation, best exposed along the Sandy River, in the Portland Basin and in the south wall of the Gorge; The Dalles Formation in and around The Dalles and Mosier basins; the Rhododendron Formation in the south wall of the Gorge between Tanner and Herman Creeks; and the Sandy River Mudstone, which underlies the Troutdale conglomerate in the Portland basin, but is not exposed in the Gorge. The Troutdale conglomerate contains high percentages of basalt and andesite pebbles, but also sometimes high concentrations of exotic quartzite pebbles and occasional pebbles of granite, schist and other non-volcanic rocks that could only have been derived from the Precambrian and Paleozoic metamorphic terrain of British Columbia. The Dalles and Rhododendron rocks are dominantly andesitic to dacitic volcaniclastic debris, which also makes up much of the matrix between the pebbles, and may form interbeds in the Troutdale. South of the Gorge, Rhododendron may underlie Troutdale as well as Cascade Lavas.

Yakima (formerly "Columbia River") Basalt of the Columbia River Group
(Middle Miocene) - - - - Tmmb

Up to 2,000 feet of dense, flow-on flow, usually black, glassy basalt in flows less than 100 feet thick, forming the major cliffs in the Gorge. Includes interbeds of soil as at Oneonta Gorge and pillow lavas as at Multnomah Falls and east of Crown Point and The Dalles. Extends many miles south of the Gorge beneath Cascade Lavas and Rhododendron Formation and west of the Gorge beneath Troutdale. On the north side of the Gorge west of Hood River it occurs only in isolated patches less than 10 miles from the river. Erupted from 12 to 16 million years ago from dike swarms far to the east. Has been divided by Swanson (1976) into three formations: a lower Grande Ronde Basalt, to which nearly all the flows west of Mosier belong; a middle Wanapum Basalt east of Mosier and beneath Crown Point; and an upper Saddle Mountain Basalt in the Mosier basin.

Eagle Creek Formation (Lower Miocene) - - - - Tmop

Up to 1,000 feet of poorly-sorted volcanic sediments of torrential and mudflow origin, mostly andesitic conglomerates, breccias and tuffs containing some carbonaceous swamp sediments. Best exposed between McCord and Eagle Creeks, and in the high cliffs of Hamilton and Table Mountains and Greenleaf Peak. North of Stevenson, patches of Eagle Creek rest upon a red-weathered paleosoil developed upon the underlying lavas.

*Lavas of Three-Corner Rock (Lower Miocene) - - - - (not mapped in Oregon)

* Recently described and mapped by Hammond (1979)

Up to 3,000 feet of interbedded andesitic lava flows, breccias and tuffs, best exposed north and east of Stevenson between Rock Creek and Wind River. Also occurs high on the mountain north of Beacon Rock. Formerly considered to be part of the Ohanapecosh Formation, now thought to be part of the Fife's Peak Group of central Washington.

*<u>Stevens Ridge Formation</u> (Mostly Upper Oligocene) - - - (not mapped in Oregon)

* Recently described and mapped by Hammond (1979)

From 50 to 5,000 feet of chiefly varicolored tuffs and a few sandstones and conglomerates. Occurs several miles north of Dog Mountain and northwest of Beacon Rock. Formerly considered to be part of the Ohanapecosh Formation.

<u>Ohanapecosh Formation</u> (Mostly Lower Oligocene) - - - (not mapped in Oregon)

These oldest rocks exposed in the Gorge appear above river level only on the north side, around and north of Camas and Washougal (where it has been called the Skamania Volcanics), and east of Wind River, where they are more than a mile thick. Farther northwest they may total more than two miles in thickness. Like the two overlying formations, the Ohanapecosh lavas, volcaniclastic breccias and tuffs are more andesitic than basaltic, and characteristically exhibit greenish and reddish colors, a result of the pervasive and widespread development of low-grade metamorphic zeolitic and argillic minerals.

<u>Igneous Intrusions</u>

<u>Tertiary Intrusions</u> (Mostly Pliocene) - - - - Tif

Stocks of fine-grained quartz diorite at and north of Wind and Shellrock Mountains and east of Cascade Locks, and the basalt plugs or dike at and north of Beacon Rock. May have been feeders to Pliocene lava flows now worn away in the cutting of the Gorge.

FIELD TRIP ROAD LOG: ADAPTED IN PART FROM

<u>GEOLOGIC FIELD GUIDE TO COLUMBIA RIVER GORGE TRIP</u>

BY JOHN ELLIOT ALLEN, PORTLAND STATE UNIVERSITY, PORTLAND, OREGON, 1958

<u>Mile</u>

0.0 Thunderbird Inn parking lot, Jantzen Beach. Drive across warped parking lot pavement and enter freeway, Interstate 5 (South), at Jantzen Beach on-ramp. Proceed south toward Portland. Route takes us across Fremont Bridge to west (downtown) side of Willamette River, then back to east side of river via Marquam Bridge, proceeding easterly toward Columbia River Gorge.

The Fremont and Marquam Bridges are Portland's newest and most expensive traffic structures. Of the two, the Fremont Bridge, completed in 1974 at a cost of $81,000,000, was both the costliest and the most difficult to construct. The center main span of the bridge, having a length of 902 feet, a width of 81 feet, and weighing 6,000 tons, was assembled 1.7 miles downstream from the bridge site, and floated upstream on barges to be lifted into position. Using 32 jacks, eight at each corner of the span, the bridge was lifted 170 feet at the rate of 4 feet per hour to its final position, making it, at that time, the heaviest lift in the history of construction.

The Marquam Bridge, completed in 1966, was constructed to carry the Interstate 5 freeway over the Willamette River in Portland. Like the Fremont Bridge, the Marquam has two decks, each with four traffic lanes carrying traffic in one direction. The bridge main span has a length of 1,043 feet, and a navigational clearance of 130 feet. It required 4,000 tons of steel, 9,000 cubic yards of concrete, and cost approximately $5,100,000 (excluding interchanges) to construct.

Geologically, West Portland lies upon Spokane-Missoula Flood silts and gravels, Troutdale Gravels, and Columbia River (Yakima) Basalt, here dipping up to 30° to the northeast on the west flank of the Portland Hills anticline. The river flows northwest along the east side of this fold, which is probably also faulted along its east flank, for 10 miles before it joins the Columbia. East Portland is built upon terraces cut by the Willamette and Columbia Rivers in flood-deposited lacustrine silts and gravels (the "Portland Delta") of Pleistocene-age. Well-defined terraces occur at about 100, 200, and 275 foot elevations in the Portland area, and up to 400 feet along the Sandy River south of Troutdale.

15.6 Exit freeway on 82nd Avenue, proceed north to Lombard Street, thence to Marine Drive. The rocky knoll visible to the east of 82nd Avenue is Rocky Butte, a Boring-age volcano. On the east side of the Butte, between it and the Banfield Freeway, is a prominent scour channel left by the Missoula Flood.

19.7 Intersection Lombard Street and Marine Drive; turn right, proceeding east on Marine Drive. Columbia River on left, "Portland Delta," developed on Missoula Flood deposits, underneath.

20.4 I-205 Bridge site. After stop, continue east on Marine Drive to Troutdale. The I-205 Bridge, to be known as the Glenn L. Jackson Bridge upon its completion, will provide the final link in the 36-mile long I-205 by-pass of the cities of Vancouver and Portland. The freeway, which begins at Tualatin, Oregon, on Interstate 5, and connects with Interstate 5 north of Vancouver, Washington, will reduce traffic pressure on the Interstate 5 Bridges, and provide a much-needed facility to move traffic efficiently in and around the cities. Planning for the river crossing was begun in 1950, and construction began in 1978. The bridge was designed with a low enough profile on the Oregon side to provide clearance for the main approaches to Portland International Airport, and with a high enough profile on the Washington side to provide clearance for ocean-going vessels in the North Shore navigation channel. The overall length of the project is 11,750 feet. Twin structures, each 7,460 feet long, will stretch between Government Island and the Washington shore. The south channel, between the Oregon shore and Government Island, will be bridged by two 3,120-foot structures, and the two pairs of structures will be separated by 1,170 feet of fill across Government Island.

28.8 Leave Marine Drive, enter Columbia River Scenic Highway; turn left west side of bridge, into Troutdale, resting on the 100-foot terrace. Chamberlain Hill shield volcano (Boring-age) dead ahead across Sandy River. Bluff above the river is of Troutdale gravels, capped by an upper 500 feet of lava.

29.6 Sandy River Bridge. Turn right, following Sandy River on alluvial terrace.

30.8 Outcrops on left in Troutdale Formation sands and gravels.

32.5 Dabney State Park. Note incised meanders with terraces at several levels along the Sandy, and riprap along bank on opposite side of river.

33.7 Enter Springdale, on the 200-foot terrace.

33.9 Bear left at intersection leaving Springdale, on Bell Road.

35.0 Intersection, Bell Road and Scenic Highway; bear left.

35.7 Entering Corbett. Summit surface here is mostly Troutdale Formation, capped in a few places by Boring Lava.

37.7 Chanticleer Lookout, 850 feet elevation. Crown Point and Vista House ahead at 12:00 o'clock, Beacon Rock volcanic plug directly above; Mount Pleasant and Mount Zion shield volcanoes across river at 10:30 and 11:00 o'clock; Rooster Rock at river level at 10:30 o'clock, and Phoca Rock (named for the harbor seal Phoca vitulina) at 11:45 o'clock; Larch Mountain shield volcano on skyline at 12:30. Beneath Crown Point, contact of south-dipping Columbia River (Yakima) Basalt with the overlying Troutdale Formation is visible. Road cuts between here and Crown Point are in Troutdale, overlain by 50-foot flow of Boring Lava; contact visible from 1:00 to 2:00 o'clock. Trend of gorge here is approximately N 70° E.

38.0 Larch Mountain Road. The western larch does not grow on Larch Mountain. Early lumbermen called the noble fir (Abies nobilis) larch.

38.3 Baked contact of Boring Lava above Troutdale gravels.

38.8 STOP. 15 minutes. Crown Point Vista House. Elevation 720 feet. Restrooms inside. Crown Point is a remnant of a thick intracanyon flow of Columbia River Basalt, which filled a late Miocene canyon of the Columbia River to a thickness of nearly 700 feet. The bluff contains palagonite tuff, pillow lavas, and basalt lavas, and the flow manifests an 80-foot basal colonnade, and a 475-foot thick hackly entablature. Rooster Rock, below, is a landslide block of a portion of the Crown Point intracanyon flow.

39.7 Entering "Figure Eight Loops" of old scenic gorge highway. This highway was finished in 1916 and was then considered a remarkable feat of highway engineering.

41.2 Latourell Falls, 249 feet. From here to Shepherd's Dell, note narrowness of road and geology of lava flows - curved columns of lower, basal colonnades, and "brickbat" basalt of upper entablatures. Extensive soil creep and landsliding visible in draws. Note planar failures in columnar basalt, leading to erosion of basal colonnades, and "mushroom" effect of more resistant entablature.

42.4 Shepherd's Dell Falls.

43.5 Bridal Veil Creek. Falls are below the road.

44.1 Bridal Veil Junction. Turn right.

46.6 Wahkeena Falls, from a Yakima Indian word meaning "most beautiful." Falls are 242 feet high.

47.1 STOP. 30 minutes. Multnomah Falls. Restrooms, coffee shop, gift shop. These are the best known, and highest falls in the Gorge, with a total drop of 620 feet. The main falls drops 541 feet, across three lava flows, and the lower falls drops 69 feet, across a fourth flow. The cliff above rises to approximately 1,600 feet elevation. Multnomah is an Indian tribal name, believed originally to have meant "down river." After stop, continue eastward on Old Highway.

47.7 Note landslide at 3:00 o'clock and debris in river at 9:00 o'clock. Spring rains in 1946 caused a landslide in the talus slope being quarried by the State Highway Department for road aggregate. Approximately 300,000 cubic yards of material slid into the river, temporarily blocking the railroad and the new highway below. The navigation buoy now visible in the river was raised 20 feet out of the water and tilted 30° by the uplifted toe of the slide.

49.4 Oneonta Gorge. The base of the lava flow here contains numerous molds of trees that were buried by the flow.

49.7 Horsetail Falls. Good basalt flow geology.

51.1 Enter freeway, Interstate 80N, proceeding east.

52.5 Beacon Rock, across river at 9:00 o'clock, is the eroded vent-filling, or plug, of a Pliocene volcano, rising 840 feet above the river. The rock was named by Lewis and Clark in 1806.

53.5 McCord Creek. Eagle Creek Formation exposed in road cuts.

55.5 Good exposures of Eagle Creek Formation.

56.7 Bonneville Dam and Fish Hatchery visible on left. Table Mountain and scar of great Bonneville landslide are visible across the river to the north. Highway cuts visible in next half-mile or so are in lava of a thick, diabase dike, upon which south end of dam is founded.

56.9 Tunnel in landslide block of Columbia River basalt. Landslide across river extends 3 miles back from river to base of Table Mountain, and has diverted river nearly 1-1/2 miles here from its originally straight course, to form a large loop. The landslides along the north side of the river, including the large Bonneville and Girl Scout (Wind Mountain) landslides, result from the combined effects of erosional undercutting by the Columbia River and of downslope movement of large masses of Eagle Creek Formation and Columbia River Basalt, along a 30-50 foot-thick saprolite zone that occurs at the contact between the Eagle Creek and the underlying Ohanapecosh Formations (Waters, 1973). The saprolite, which dips gently toward the Columbia River at from 2^{o} to 10^{o}, becomes lubricated by rainwater seeping through joints and fractures in the overlying formations, providing a slippery plane on which movement can occur. It is believed that the Bonneville slide may have completely dammed the Columbia River, approximately 700 years ago, giving rise to the Indian legend of the Bridge of the Gods. The Girl Scout slide is presently the largest active slide in the Gorge, and is moving toward the river at a rate of up to 15 meters per year (Palmer, 1977).

58.8 Bridge of the Gods (modern) on left.

59.7 Bridge entrance and town of Cascade Locks.

60.5 Well-developed radial fan joints are visible in intracanyon lava flow on left throughcut.

64.3 Wind Mountain (north side of river) and Shellrock Mountain (south side) Stocks visible ahead. These stocks, of hornblende-pyroxene quartz diorite, are part of a north-south line of intrusives, ranging in age from 13.4 to 7.8 million years, that runs the full length of the Cascades.

64.6 Begin Fountain Slide area. For the next 3/4 mile, the freeway passes over unstable ground of the Fountain Slide, which extends several thousand feet up the adjacent slope. The landslide has been a known slide area for years, ever since construction of the first roadway across it in the 1920's. The first attempts to stabilize the slide were made in 1968, when construction of Interstate 80 caused renewed and more vigorous activity to occur, with a rate of movement of 2.0 feet per year horizontally, and 15 feet per year vertically (Munoz and Gano, TRR #482, 1974). Initial and repeated efforts to stabilize the slide both by drainage and unloading have been only partially successful, and the highway suffers continual damage, especially during the rainy winter season.

The investigation of the slide has seen the installation of a total (in 1974) of 63 slope indicators, plus numerous piezometers and vertical test wells. As a result of this thorough investigation, a precise knowledge of the cause of the slide has been gained, even though complete stabilization of the slide has not been achieved. The slide mass is primarily a talus-like material containing vesicular, coarse-grained andesite and fine-grained basalt (Munoz and Gano, 1974), in a matrix of silt, sandy silt, and sandy, silty clay. Lenses of claystone and mudstone occur throughout the slide, and, where weathered, have resulted in a zone of weakness along which sliding occurs. The thickness of the clay-mudstone layer is 2 to 4 feet, and movements have occurred at depths as great as 200 feet. Sliding has been most pronounced in areas where the weakness zone is inclined at an angle of 12 degrees or more.

In addition to the clay-mudstone layer, the slide mass also contains a great volume of trapped or perched water distributed randomly throughout the slide mass. Because of its non-uniform distribution, water in the slide is not easily intercepted by drains, and attempts to stabilize the slide by drainage have had only limited success. The slide-warning system, built and operated by the Oregon State Highway Department, was an attempt to warn the motoring public when slide movements reached a certain level of severity, but the system has never performed satisfactorily.

65.0 STOP. Fountain Slide Highway Warning instrumentation shed, operated by Oregon State Highway Department.

65.2 End active landslide area.

67.8 Begin talus slope retaining walls.

76.4 Excellent exposures of palagonite tuff and breccia from nearby Pliocene volcanoes in throughcuts. Palagonite is formed by the hydration of volcanic glass when lava flows into water (in this case, a lava-dam lake) and occurs in these outcrops, both as selvages on basalt pillows and breccia fragments, and as a constituent of the yellowish, tuffaceous matrix of the breccia.

80.0 Hood River exit. Continue eastward about 5-1/2 miles to the Mosier exit. Notice the abrupt change in vegetation over the next few miles from the green, wet, coniferous forest vegetation of Western Oregon and Washington to the brown, dry, arid climate and vegetation of Eastern Oregon and Washington, which lie in the rain shadow of the Cascade Range. Portland's average annual rainfall is approximately 40 inches, while the average rainfall at The Dalles, 90 miles to the east, is only 15 inches. Because of the sparseness of vegetation on this side of the Cascades, geologic features are much more easily seen, and in this section from Hood River to Mosier, striking views of the folded basalt stratigraphy may be had. From west to east, note fold axes of Bingen anticline, Mosier syncline (followed in southwesterly direction here by Columbia River), faulted Ortley anticline, and The Dalles syncline.

85.4 Rowena Loop exit. Leave freeway and cross overpass.

85.5 Return to freeway westbound.

90.9 Hood River/Mount Hood exit. Leave freeway. Begin climbing up Hood River Valley toward Mount Hood. Hood River Valley occupies the Hood River syncline, which is faulted along its east limb, to produce an escarpment of about 1,000 feet of uplifted Columbia River Basalt on the east side of the valley. The valley bottom is occupied by abundant glacial till and outwash deposits, consisting locally of dense, coarse to very coarse boulder conglomerate and sand terraces. Glacial erratics, rocks ice-rafted to their present positions by post-glacial floods, have been found at elevations as high as 750 feet in the Hood River Valley. Farther up the valley, deposits consist of mudflows, vocanic debris flows, and ash falls from Mount Hood, as well as glacial outwash.

95.4 Turn-off to Panorama Point. <u>LUNCH</u>. 45 minutes.

97.9 Return to Loop Highway, continue south toward Mount Hood.

130.9 White River Bridge. STOP. 15 minutes. White River is a glacially fed stream with an aggrading channel that has presented problems to bridge designers for many years. Because the stream channel is aggrading, or rising in elevation with time, the problem of how best to ford the stream is not a simple one. The presently-existing bridge is the third one to be constructed over the White River at this site. The first was a timber structure built in 1924 about 50 feet downstream, which had a vertical clearance to the stream bed of about 14 feet. The channel filled over the years, and the bridge was actually covered over on several occasions. A second bridge, with a length of 156 feet, was completed in 1951. The deck elevation of this bridge was 9 feet higher than the original, and the clearance under the bridge was again about 14 feet. This bridge was also covered on several occasions.

In 1967, the present bridge was constructed. The deck was widened 10 feet and raised 7 feet, primarily to provide sufficient clearance beneath the structure to allow equipment to move underneath the bridge to open the clogged channel. In addition, seven dikes, totaling 2,890 feet in length, were constructed to block off the overflow channels, in order to direct the flow toward the bridge and increase the velocity of the stream underneath. Since this latest construction, the channel has continued to aggrade, filling to within 4 feet of the bottom of the bridge at times. Continuous maintenance is required to maintain clearance beneath the bridge.

In time, a rapidly aggrading stream will build itself high enough above the surrounding terrain that capture by an adjacent stream will occur. Capture of the White River will most likely occur either by Green Alder Creek on its east side, or by Mineral Creek to the west.

Historically, this section of highway follows the approximate location of the Barlow Toll Road. This road was constructed in 1845 by Samuel Barlow to by-pass the final, very difficult miles of the Oregon Trail, which followed the Columbia River into Portland.

137.3 Turn-off to Timberline Lodge.

142.7 Timberline Lodge on Mount Hood, official "end" of the Field Trip. STOP. 1 hour. Mount Hood is the highest mountain in Oregon (11,245 feet) and is typical of the lofty, composite-type High Cascade volcanoes of Oregon and Washington. These volcanoes formed, mostly during Quaternary time, by the alternating eruption of predominantly andesitic lava flows and pyroclastic debris flows, which accumulated to form composite cones, or stratovolcanoes, on the already uplifted Cascade Range. The volcanoes of Oregon and Washington (and Mount Lassen, in Northern California) are part of a chain of dormant-to-active volcanoes that surround the Pacific Ocean Basin (the Circum-Pacific "Ring of Fire"), and include the volcanic regions of the Andes, the Aleutians, Japan, and New Zealand, to name but a few. Eruptions have occurred from the Northwest's volcanoes as recently as the 1840's (Mount Saint Helens) and 1915 (Mount Lassen), and steam has been noted rising from the summit craters of Mount Hood and Mount Baker in recent years, so it seems most appropriate to describe the Cascade volcanoes as "dormant," rather than active or extinct.

Five major skiing areas are located on the slopes of Mount Hood, providing winter recreation for thousands of Oregonians. In addition, the mountain is climbed by hundreds of professional and amateur mountain climbers each year, and is used as one of the summertime training areas for the U.S. Olympic Ski Team. Timberline Lodge, built by the WPA in the 1930's, has a 96-foot high fireplace and peg floors of Oregon Oak. Many of the Ponderosa Pine support beams display beautiful hand carvings, and the Lodge, originally owned by the U.S. Forest Service, was recently designated a National Historic Site.

148.1 Return to Highway 26, turn right, heading down the west side of Mount Hood. Trip from here to Portland (Jantzen Beach) is a through run, with one rest stop in the town of Sandy. Geologically, the route followed by Highway 26 takes us down the southwest flank of the volcano, into the valleys of the Zigzag and Sandy Rivers, then westward through the foothills of the Cascades toward Portland. On the mountain, rock outcrops in highway cuts are of the Cascade Andesite, except for a small, local intrusion of quartz diorite just downhill from Government Camp. After entering Mount Hood National Forest in the Zigzag River Valley, exposures are poor, due to the dense forest vegetation, but consist mainly of mudflows, tuffs, and breccias of the Rhododendron Formation.

Farther west, toward Sandy, the steep slopes of the Range give way to the more gentle, rolling topography of the foothills, interrupted here and there by the steeper cones of eroded Boring Lava volcanoes. The intervening, low-lying countryside is underlain by Troutdale Formation, and by glaciofluvial and lacustrine gravels and silts from the Zigzag and Sandy River Valleys, which were glaciated during the Pleistocene. Closer to Portland, the geology is much the same, and the city itself is underlain by Columbia River Basalt and Troutdale Formation, plus the silts, sands, and gravels of numerous Pleistocene and Holocene depositional periods.

206.1 Arrive Jantzen Beach.

REFERENCES CITED

Allen, J. E., 1958, Columbia River Gorge, Portland to The Dalles, (in) Guidebook for Field Trip Excursions, Cordilleran Section, Geol. Soc. America, Univ. of Oregon, Eugene, Oregon, March 27, 1958, p. 4 - 23.

________________, 1979, The Magnificent Gateway - A Layman's Guide to the Geology of the Columbia River Gorge: Timber Press, 150 p.

Munoz, A., and Gano, D., 1974, The Role of Field Instrumentation in Correction of the Fountain Landslide, (in) Landslide Instrumentation: Transportation Research Record 482.

Oregon Department of Transportation; various undated and untitled fact sheets regarding Portland bridges.

Palmer, L. A., 1977, Large Landslides of the Columbia River Gorge, Oregon and Washington: Geol. Soc. America Reviews in Eng. Geol., v. III, p. 69 - 83.

Waters, A. C., 1973, The Columbia River Gorge: Basalt Stratigraphy, Ancient Lava Dams and Landslide Dams, (in) Beaulieu, J. D., Geologic Field Trips in Northern Oregon and Southern Washington: Oregon Dept. Geology and Mineral Industries Bull. 77, p. 133 - 162.

Wells, F. G., and Peck, D. L., 1961, Geologic Map of Oregon West of the 121st Meridian, 1:500,000: Oregon Dept. Geology and Mineral Industries.

APPENDIX

GEOLOGIC MAP OF FIELD TRIP AREA

GEOLOGIC CROSS-SECTIONS IN NORTHWESTERN OREGON

ABBREVIATED STRATIGRAPHIC COLUMN FOR LOWER COLUMBIA RIVER GORGE AND NORTHERN CASCADES

LOCATION MAP FOR MAJOR LANDSLIDE DEPOSITS IN THE GORGE

GEOLOGIC MAP OF BONNEVILLE LANDSLIDE

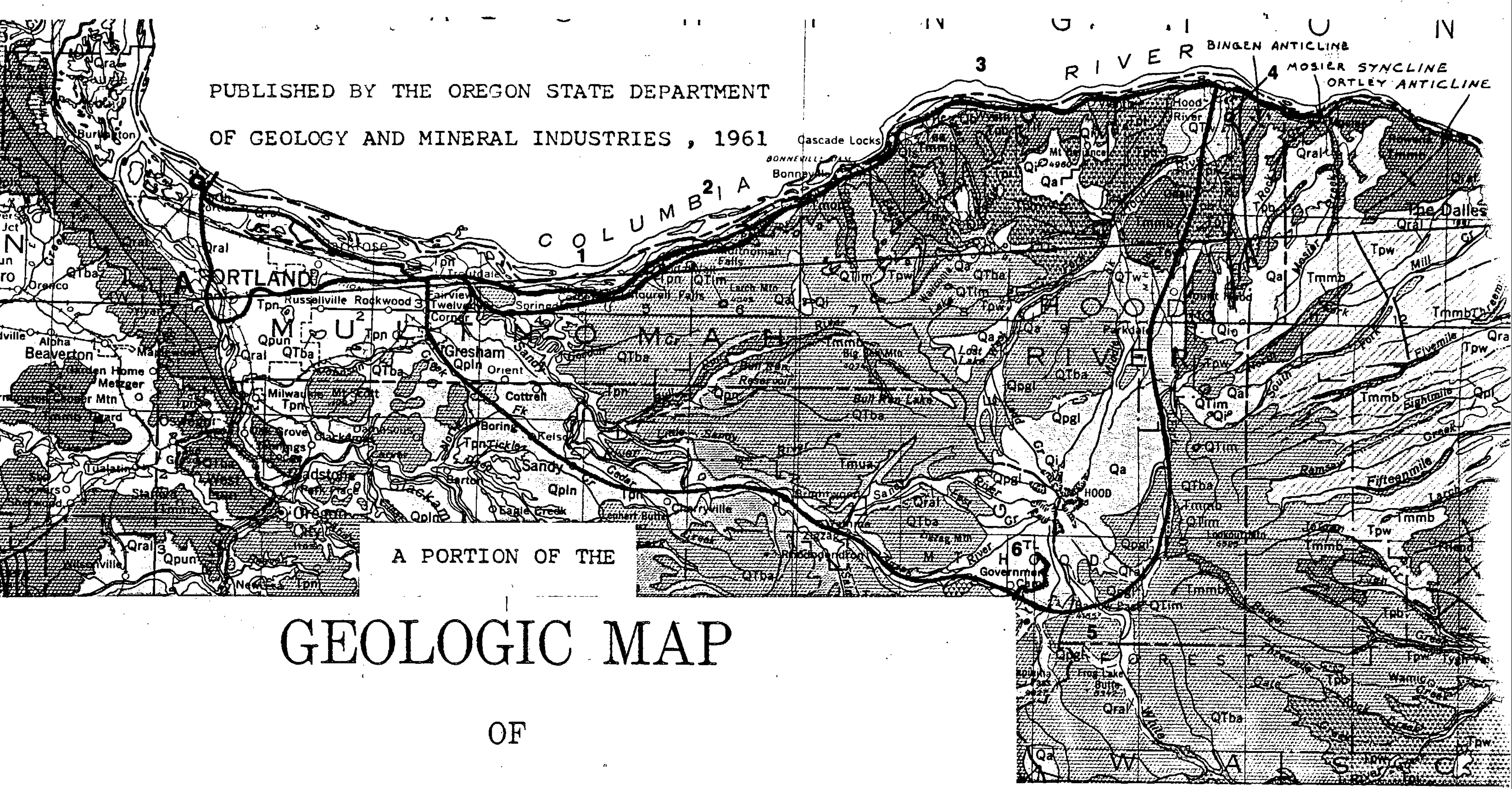
CROSS-SECTION AND STRATIGRAPHIC COLUMN FOR BONNEVILLE SLIDE AREA

BLOCK DIAGRAM OF CROWN POINT INTRACANYON LAVA FLOW

PHOTOGRAPH OF LOWER GORGE AREA, LOOKING UPSTREAM FROM NEAR CROWN POINT

ENGINEERING CHARACTERISTICS OF GEOLOGIC UNITS IN THE PORTLAND AREA

PUBLISHED BY THE OREGON STATE DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, 1961



A PORTION OF THE GEOLOGIC MAP OF OREGON WEST OF THE 121ST MERIDIAN

Prepared under the direction of Francis G. Wells.
Compiled by Dallas L. Peck

Scale 1:500 000

10 5 0 10 20 30 Miles

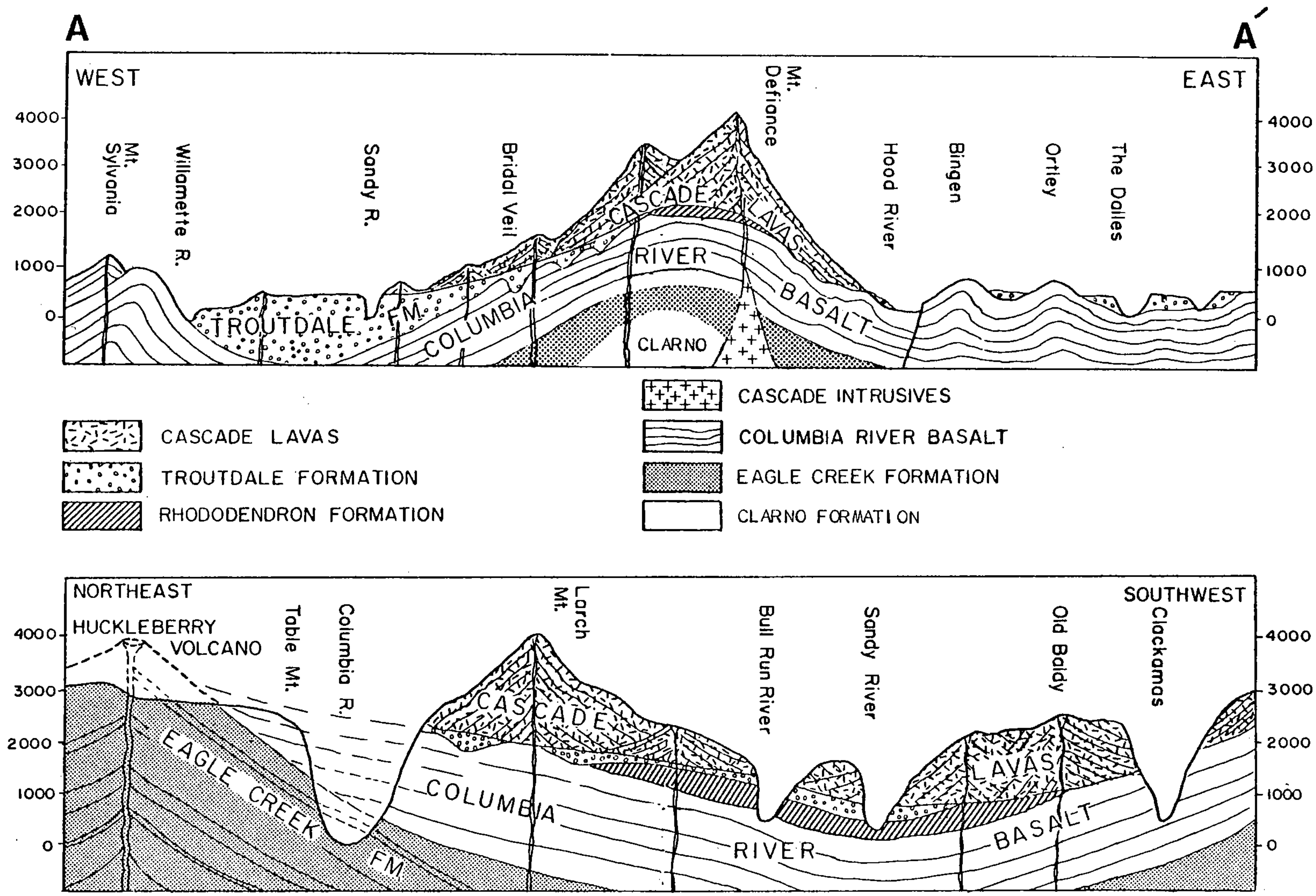
LEGEND

FIELD TRIP ROUTE (IN RED) ———

NUMBERED STOPS 1, 2, 3, ETC.

ROCK UNITS --- QTBA, TMMB, ETC. ---

SEE TEXT FOR DESCRIPTION OF UNITS



DIAGRAMMATIC CROSS SECTIONS ILLUSTRATING STRUCTURAL & STRATIGRAPHIC RELATIONS

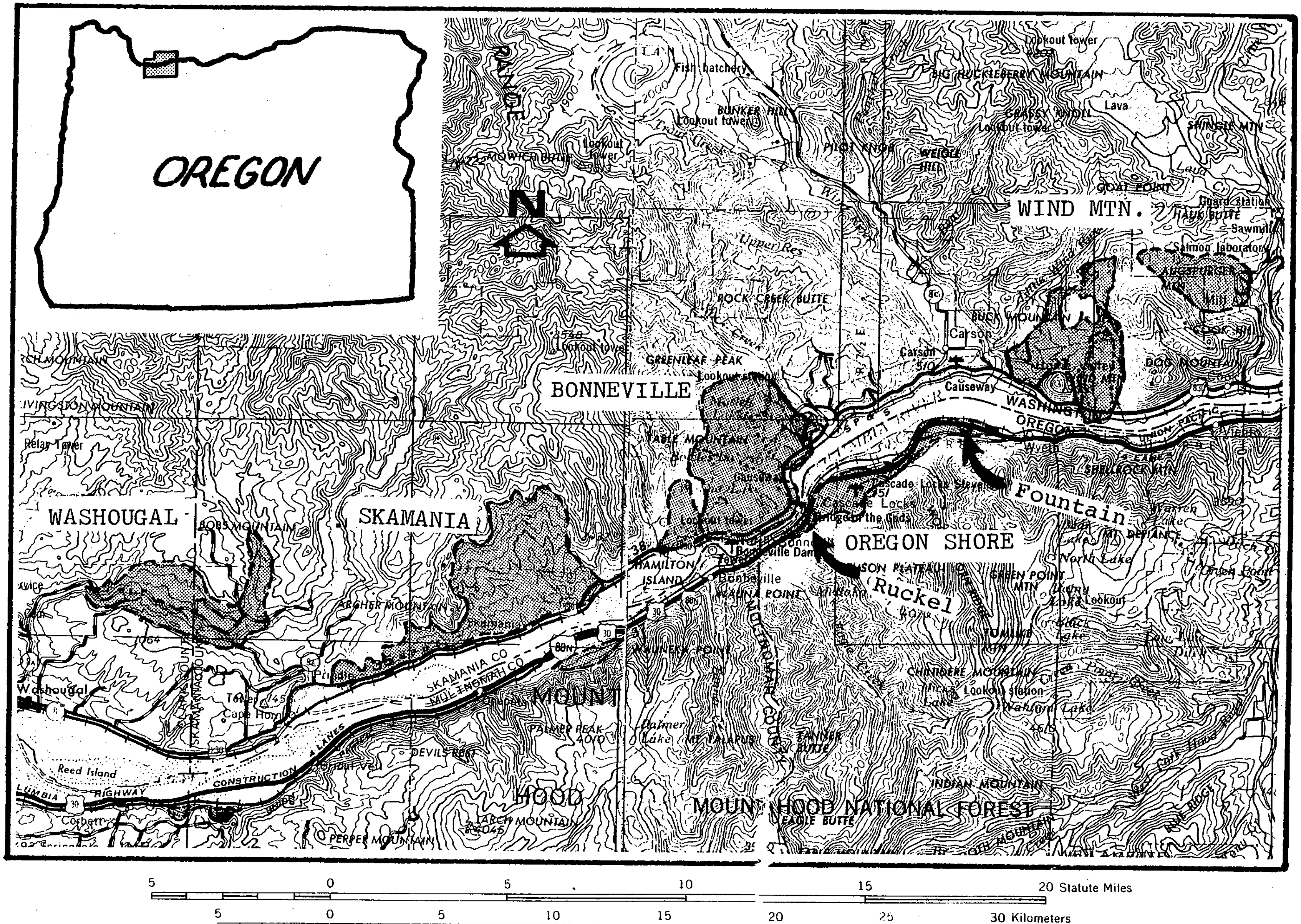
in northwestern Oregon

Adapted from publications by John Eliot Allen.

Table 1. Stratigraphy, Columbia River Gorge

	A. Alluvium, talus, active landslides, and debris flows.	
PLEISTOCENE-HOLOCENE	B. Olivine basalt lavas, cinder cones, and hyaloclastic deposits; examples: Big Lava Bed, cinder and lava cones of Hood River valley, lavas and delta from Wind River valley, Mt. Defiance and Starvation andesites, major strato-volcanoes - Mounts St. Helens, Hood, and Adams.	
PLIO-PLEISTOCENE	Widespread olivine basalt volcanism. Examples: Larch Mountain, Mt. Zion, Underwood and White Salmon shield volcanoes, Boring lava of Willamette valley.	
	Troutdale gravels:	Depositional fan from Columbia River.
	Dalles Formation:	Chiefly stream deposited volcaniclastic rocks; local airfall and nuée deposits.
PLIOCENE		
	Ellensburg Formation:	Stream deposited pumice-rich volcaniclastic rocks; local mudflow and airfall deposits.
MIOCENE	Yakima Basalt:	Flood basalts; thick tholeiitic flows, pillow basalts, and hyaloclastic tuffs.
LOWER MIOCENE ?	Eagle Creek Formation:	Coarse cobble gravels and other volcaniclastic rocks; mostly andesitic.
UPPER EOCENE	Ohanapecosh Formation equivalent:	Zeolitized and argillized lavas and volcaniclastic rocks, chiefly of andesitic composition, but increasing in basalt toward the west.

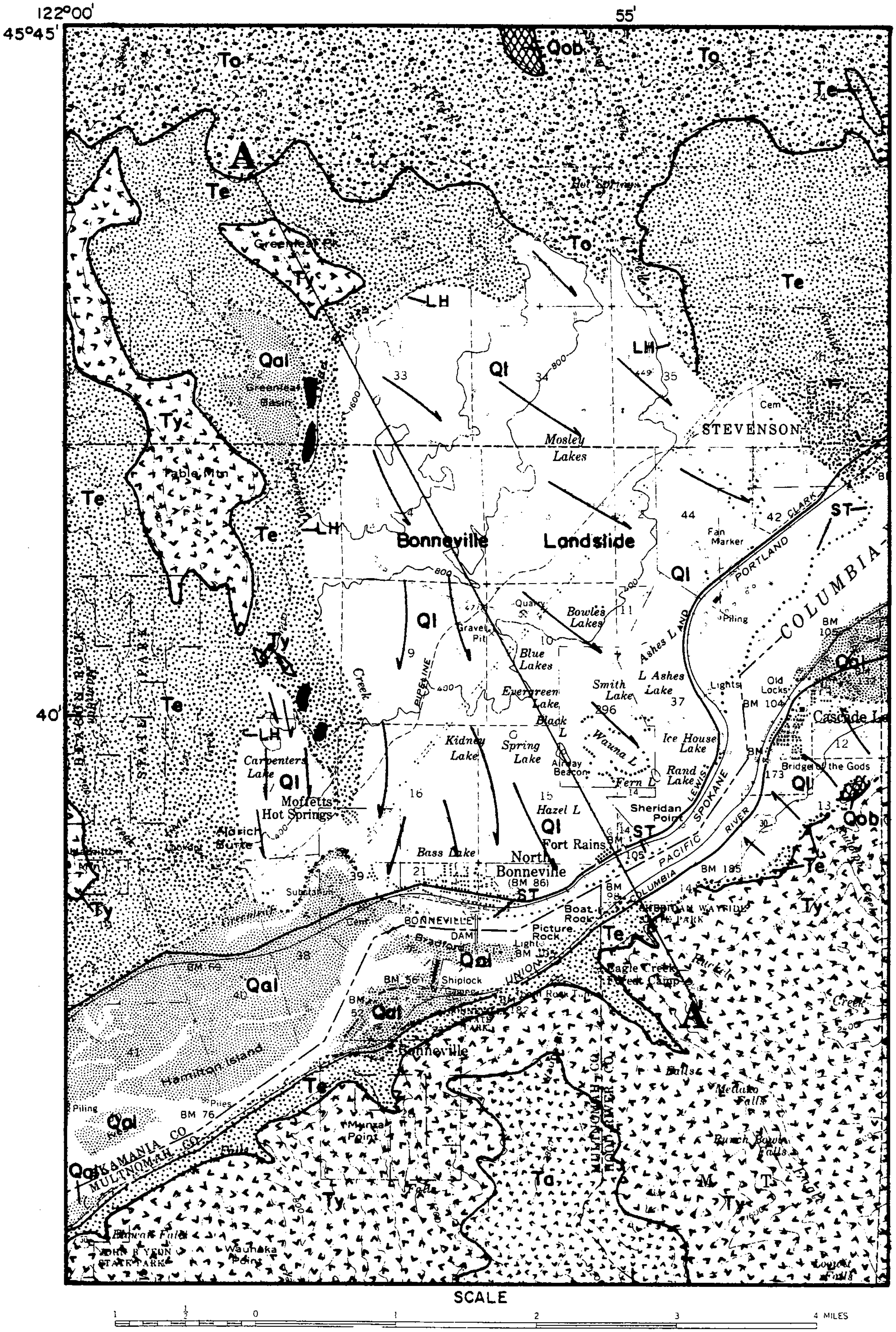
(From Waters, 1973)



Location map showing major landslide deposits (shaded areas outlined by dashes) and some actively sliding areas (shaded areas outlined by dots).

From Palmer, 1977.

(From Waters, 1973)



Geologic Map *of* Bonneville Landslide Area

EXPLANATION

Quaternary

Qal — Alluvium and stream terrace deposits

Ql — Landslides

Unconformity

Qob — Olivine basalt flows
(Intrusive masses shown in black)

Unconformity

Pliocene

Ta — Pyroxene andesite and olivine andesite flows

Unconformity
(A few patches of Troutdale Formation occur here)

Miocene

Ty — Yakima Basalt (thick flows of tholeiitic basalt)

Unconformity

Te — Eagle Creek Formation (andesitic conglomerate, sands and silts)

Unconformity
(Saprolite at this unconformity indicated in black in cross section)

Eocene

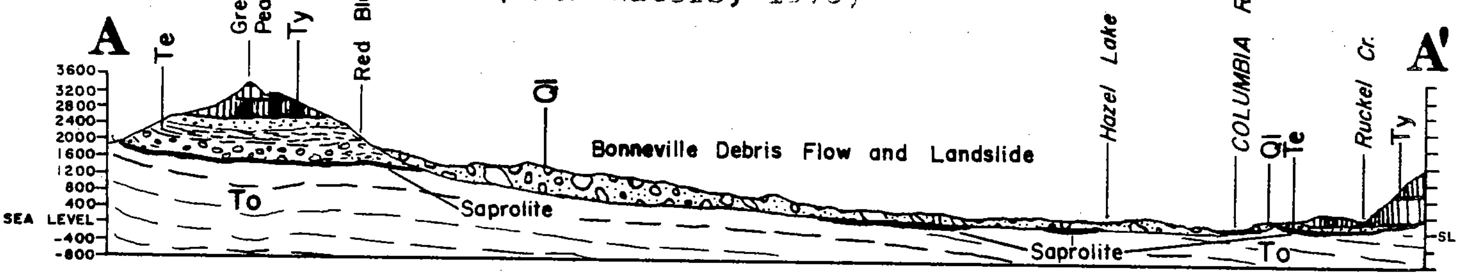
To — Ohanapecosh Formation equivalent
(Zeolitized and argillized volcaniclastic rocks)

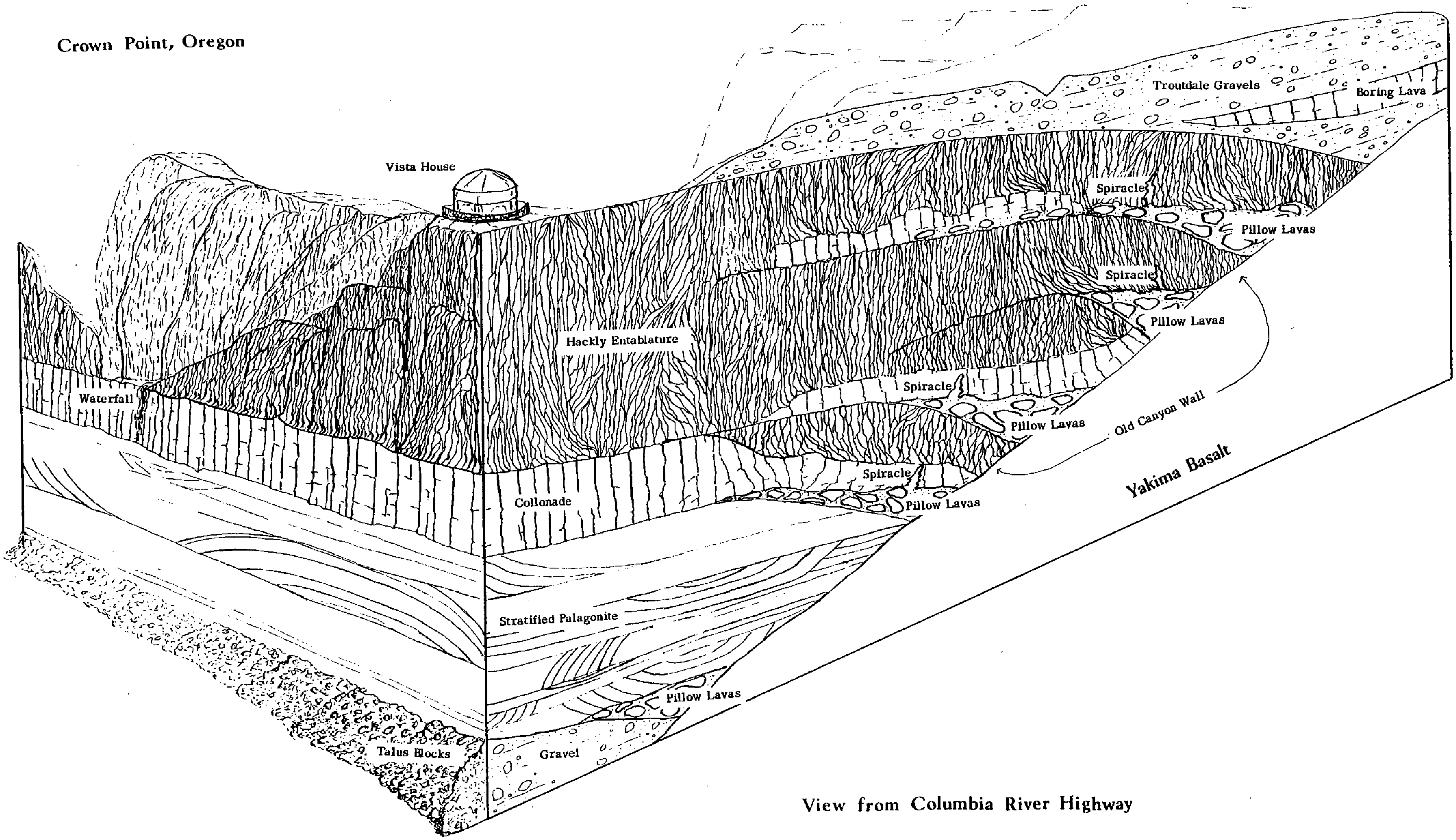
LH — Landslide headscarp

ST — Submerged toe

Geologic Cross Section

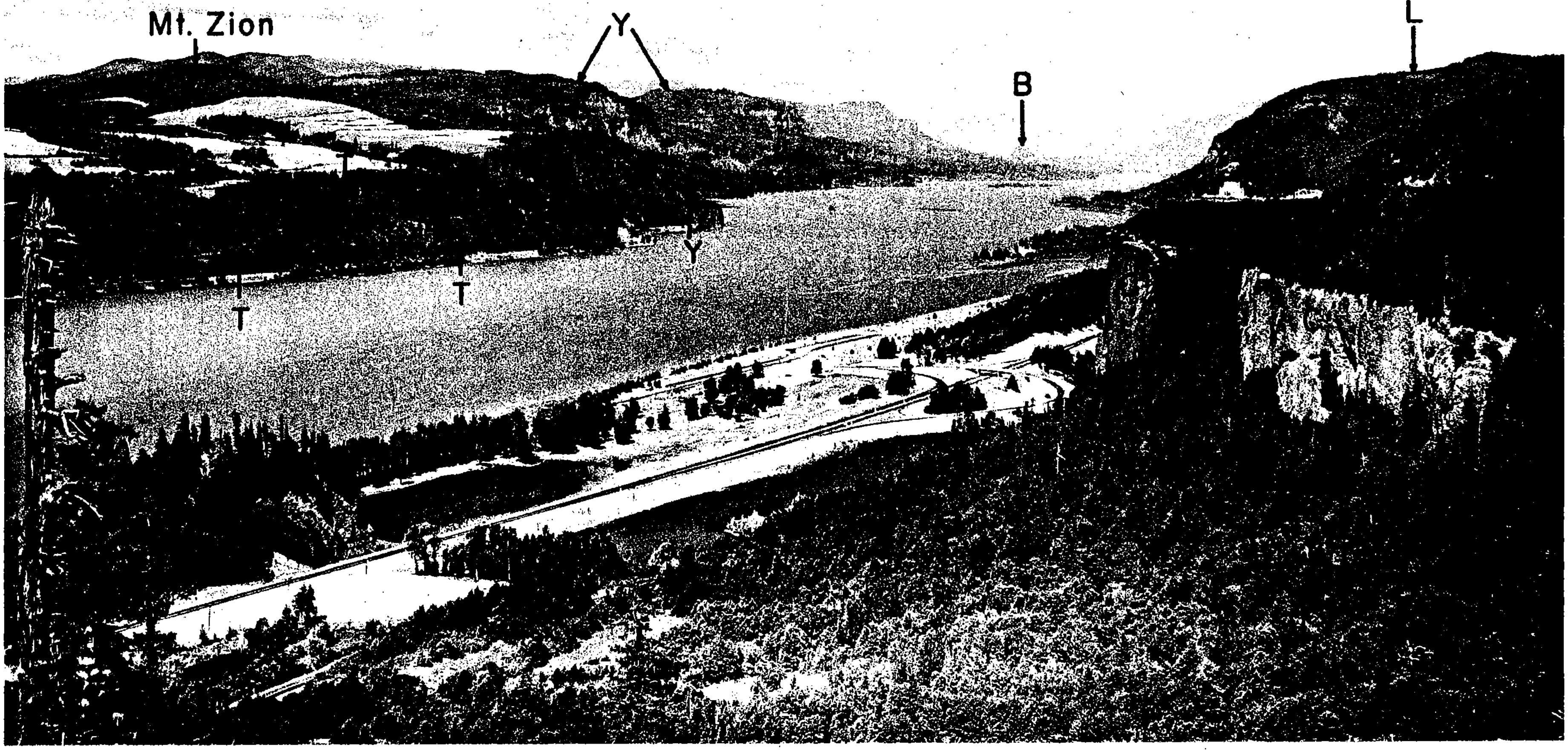
(From Waters, 1973)





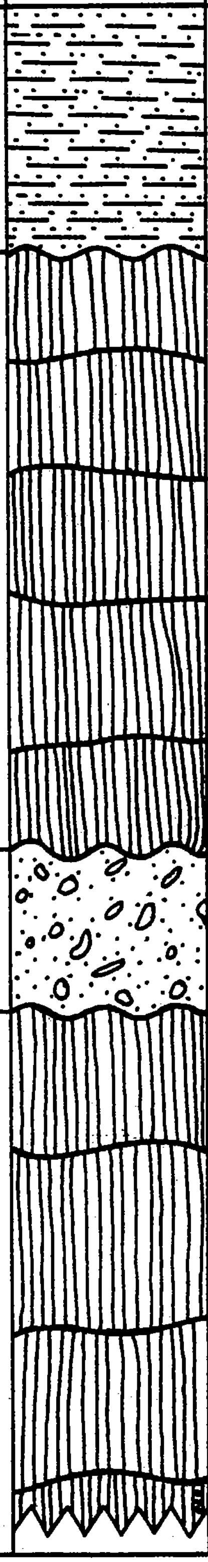
Block diagram of the remnant of an intracanyon lava fill at Crown Point.

(From Waters, 1973)



Looking east up the Columbia River Gorge from above Crown Point. On the left, the fan of Troutdale gravels (T), surmounted by Zion shield volcano, has been incised by Columbia River. Outliers of Yakima Basalt (Y) rest on Ohanapecosh and Eagle Creek Formations which are obscured by landslides. Beacon Rock (B), a plug of olivine basalt, rises in the distance.
South of the river, Crown Point (C) is a remnant of an intracanyon flow, over 650 feet thick, stuck to the south wall. Farther upstream are superposed flows to Yakima Basalt (Y) capped by the north flank (L) of the Larch Mountain shield volcano. (Photo by Oregon State Highway Division.)

(From Waters, 1973)

AGE	FORMATION	LITHOLOGY	DESCRIPTION
Pleistocene	Portland Hills Silt		Portland Hills Silt: 0-42+ feet thick. Clayey silt and silty clay; mudflows and slumps on slopes over 15%; low permeability; makes moderately strong and compressible fills; shallow perched water table in some areas.
Pliocene	Boring Lava		Boring Lava: 0-100+ feet thick. Gray olivine basalt with an expanded texture; generally solid and stable except where highly weathered or where volcanic sediments are interbedded.
	Troutdale Formation		Troutdale Formation (?): 0-25+ feet thick. Pebble conglomerate, moderately indurated; small localized deposits, usually breaks down to constituent particles upon excavation.
Miocene	Columbia River Basalt		Columbia River Basalt: 700+ feet thick. Weathered and unweathered basaltic lava flows possibly with interflow zones of ash, breccia, and/or baked soil; generally solid and stable except in steep exposures where highly weathered and where movement is possible on dipping, clayey interflow sediments.

Engineering characteristics of the geologic units in the Portland area, Oregon.

(From Palmer, 1977)