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ANALYSIS AND DESIGN OF HIGHWAY CUTS IN ROCK: A SLOPE STABILITY STUDY ON INTERSTATE ROUTES 279 AND 79 NEAR PITTSBURGH, PENNSYLVANIA

Report Prepared by the University of Pittsburgh

Principal Investigator J. V. Hamel

Co-Investigator N. K. Flint

This work was performed in cooperation with the Pennsylvania Department of Highways and the U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads.

Interstate Highway Nos. were changed after this report was done. I-279 in this report is now I-79 & I-79 in this report is now I-279.

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Department of Highways
Bureau of Materials, Testing and Research
Leo D. Sandvig--Director
Wade L. Gramling--Research Engineer
William G. Weber, Jr.--Research Coordinator

ABSTRACT

Analysis and Design of Highway Cuts in Rock: A Slope Stability Study on Interstate Routes 279 and 79 near Pittsburgh, Pennsylvania

by

J. V. Hamel and N. K. Flint

Slope stability analyses and related engineering geology studies were made along Interstate Routes 279 (L.R. 1016, Section 12) and 79 (L.R. 1021, Section 6) a few miles northwest of Pittsburgh, Pennsylvania. Both routes are largely on steep valley slopes. They traverse essentially flat-lying, coal-measure-type strata containing a weak claystone zone 55 feet thick that is composed primarily of the Pittsburgh Redbeds. Ancient landslides, recognizable topographically as hillside benches developed at the head of the slides, are common along the two highways where this claystone occurs.

Several landslides encountered during construction of I-279 resulted from undercutting and reactivation of unrecognized ancient slide masses (colluvium) in the Pittsburgh Redbeds. Two of these are described and analyzed in detail. Sites along I-79 where the geologic setting is similar and where landsliding may develop during construction are also described. These predicted problem sites on I-79 can be used to test the usefullness of the geologic mapping that is recommended as a basis for landslide prediction.

Repeated direct shear tests on 2 by 2 in. specimens of clay from several I-279 failure surfaces yielded residual shear strength parameters $\bar{C}=0$, $\bar{\phi}=11^{\circ}$ to 16°. Friction angles of 12.5° to 15.5° were calculated for limiting equilibrium of the failure masses using the Morgernstern-Price method of stability analysis. The average shear strength mobilized along the failure surfaces of these slides was very close to the residual strength of failure-surface materials. This residual strength condition is attributed to shearing displacements of the ancient landslides.

X-ray diffraction analysis of clays from I-279 shows a concentration of expandable-lattice clay minerals in shear-zone materials but not in adjacent materials. A suspected genetic relation between water seepage along ancient failure zones and the occurrence of such clay minerals should be investigated further.

Key Words

slope stability analysis
weak claystone
predicted problem sites
shear strength
X-ray diffraction
geologic mapping
engineering geology
reactivation of ancient slides
direct shear tests
residual strength
expandable-lattice clays

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Some portions of this report are not included here.

1. CONCLUSIONS

Each rock slope stability study is unique because of differing geologic details from one site to another.

Valid slope analyses cannot be made unless the site geology is adequately known.

Weak beds, bedding plane contacts, fault planes, and ancient landslide surfaces are all contributory factors in slope failure in the coal-bearing rocks of Western Pennsylvania. Ancient landslide surfaces are especially important because previous movements along them have reduced their shear strengths to residual values.

The Pittsburgh Redbeds, a zone of weak claystone (indurated clay), are particularly susceptible to landsliding, both ancient and recent. Many ancient landslides at this horizon are recognizable topographically as hillside benches.

Expandable-lattice clay minerals in shear-zone materials from I-279 near Glenfield appear to be a significant factor in landsliding. They may form secondarily as a result of water seepage along ancient shear zones.

Test samples of materials from ancient landslide shear zones are difficult to obtain by NX core drilling.

Repeated direct shear tests of ancient landslide shear-zone materials yield residual-strength parameters of the magnitude likely to be mobilized in field situations. Tests on intact rock from which the failure-surface materials are derived generally yield about the same residual-strength parameters as the failure-surface materials themselves.

The sliding block method of stability analysis gives results adequate for most engineering purposes. These results are in agreement with those obtained by the Morgernstern-Price method.

2. RECOMMENDATIONS

2.1 A detailed engineering geology survey should be made prior to the design of every large cut slope. This survey should include geologic field mapping and logging of test borings by an experienced geologist familiar with the geology of the site. Outcrop areas of weak strata should be plotted on topographic maps showing the location of a proposed highway so that this information can be used in designing cuts. Field mapping and reconnaissance should, whenever possible, be scheduled for the spring or fall months when neither foliage nor snow obscure important topographic and surficial features.

2.2 If ancient landslide surfaces exist in a slope to be excavated, and the excavation geometry is such that new movements may be initiated along these surfaces, samples of materials from the ancient landslide surfaces should be obtained for laboratory testing. Samples adequate for testing could possibly be obtained with large diameter (e.g. 6-inch) core drills or with sophisticated sampling devices such as the Denison sampler. It is recommended, however, that samples be obtained by hand excavation in test shafts or test pits. This latter sampling method permits a man direct access to a failure surface for purposes of inspection and mapping as well as sampling. Test shafts of approximately 36-inch diameter can be drilled with standard caisson drilling rigs, and test pits to about 16-feet depth can be excavated with tractor-mounted backhoes.

2.3 Residual strengths of materials from the failure surfaces of ancient landslides should be determined wherever there exists the possibility of reactivating the ancient landslide surfaces. Repeated direct shear tests in conventional direct shear devices are recommended for residual strength determinations. These tests can be performed on undisturbed or remolded specimens of shear-zone materials if such specimens are available. In the absence of such specimens, tests can be performed on intact specimens of the rock from which the shear-zone materials are derived. Samples of these intact rock materials are easier to obtain and they yield about the same residual strength parameters as the shear-zone materials.

2.4 If budgetary constraints absolutely preclude sampling and testing, the strength parameters given in Appendix A can be used to estimate design values of strength parameters for similar geologic materials. Any such estimates of strength parameters should be made by someone knowledgeable in engineering geology and the basic principles of soil/rock shear strength behavior.

2.5 Stability analyses should be performed for all slope sections of questionable stability. The sliding block method of stability analysis with interblock forces inclined at an angle of 20° to 30° to the horizontal is recommended for routine engineering analyses of slopes in the coal-measure rocks of Western Pennsylvania.

The two major unknowns in any stability analysis are available shear strength values and groundwater levels in the slope. Residual shear strength values should be used where the failure surface consists of an ancient landslide zone or a fault zone, or where progressive failure is expected to reduce the strength along the failure surface to a residual value. In all other cases, the choice of design values for shear strength parameters is largely a matter of engineering judgment. No general rules can be given.

Groundwater levels in the slope should be determined in the course of the engineering geology survey of the slope site. If they have not been determined in the field, it can be conservatively assumed that the maximum seasonal groundwater level in the slope is at the ground surface.

2.6 It is recommended that further work be done to determine the distribution of expandable clay minerals like vermiculite in and adjacent to shear zones, and to determine the origin of such minerals.

2.7 At least one cut section of I-79 (L.R. 1021, Section 6) should be instrumented in order to monitor its behavior before, during, and after highway construction. The recommended site is the cut between Stations 337 and 344. Other cuts from Station 378 to 384 and from Station 350 to 355 might be considered for instrumentation if funds are available.

Additional subsurface exploration will be required at all these sites to determine the feasibility of instrumentation and to design the systems that might be used. No specific recommendations can be made until such subsurface exploration is done, but it is anticipated that 1) piezometers for water-level observations, 2) inclinometers for lateral (and possibly vertical) movement observations, and 3) surface survey monuments for lateral and vertical movement observations would be included in the instrumentation system.

3. INTRODUCTION

The studies on which this report is based were undertaken with the objective of developing improved methods of rock slope analysis and design of highway cuts. Originally several highway cuts (existing and proposed) in western, north-central, and northeastern Pennsylvania were to be studied. Early in the project, however, the study area was restricted to two sites. One site contains a section of Interstate Route 279 (L.R. 1016, Section 12) which was under construction at the time and along which unstable slope conditions were being encountered. The second site includes a section of Interstate Route 79 (L.R. 1021, Section 6) where geologic conditions are similar and where experience gained at the I-279 site can be used in identifying potential problem areas. The locations of the two sites are shown in Fig. 3 - 1, an index map of the study area.

The project described in this report was carried out in several stages from September, 1968 through November, 1969. At the beginning, a review of pertinent literature on rock slope stability was made. This resulted in an "Annotated Bibliography on Rock Slope Stability" (Hamel et al., 1969) submitted to the Pennsylvania Department of Highways as an interim report. The bibliography was published by the Department as a research report of the Bureau of Materials, Testing and Research. Concurrently, an inspection of existing and proposed highway cuts in western, north-central, and northeastern Pennsylvania was made prior to narrowing down the study area to that described above.

Soil survey reports of the I-279 and I-79 localities were studied. The I-279 construction site was visited on a continuing basis through a period of 15 months to monitor the landsliding that was taking place there, and to collect samples for laboratory shear tests and X-ray diffraction analysis.

A detailed study of the geology of both the I-279 and I-79 areas was made to establish the stratigraphic section, to provide an understanding of the cause of landsliding on I-279, to plot the outcrop distribution of the weak Pittsburgh Redbeds layer on topographic maps, and to provide comparisons between the geology of the two areas.

Direct shear tests of specimens from failure surfaces at the I-279 site near Glenfield were made. X-ray analyses of the clay mineralogy of selected specimens from landslide masses were also made.

Stability analyses of two of the several landslides that occurred at the I-279 site were performed. Strength parameters required for the limiting equilibrium of the two slide masses were calculated with the Morgenstern-Price method of slope stability analysis. The sliding block method of stability analysis was also used to calculate factors of safety of these slide masses.

This report consists of two complementary parts. One part deals primarily with the geology of the entire study area; the other part deals primarily with two case studies of specific landslides on I-279 near Glenfield, Pennsylvania. The geologic studies were conducted by N. K. Flint; the case studies of landslides by J. V. Hamel. Conclusions and recommendations pertaining specifically to the geologic and slope analysis studies are presented at the end of each of those two parts of the report. General conclusions and recommendations are presented in parts 1.0 and 2.0 respectively, of the report.

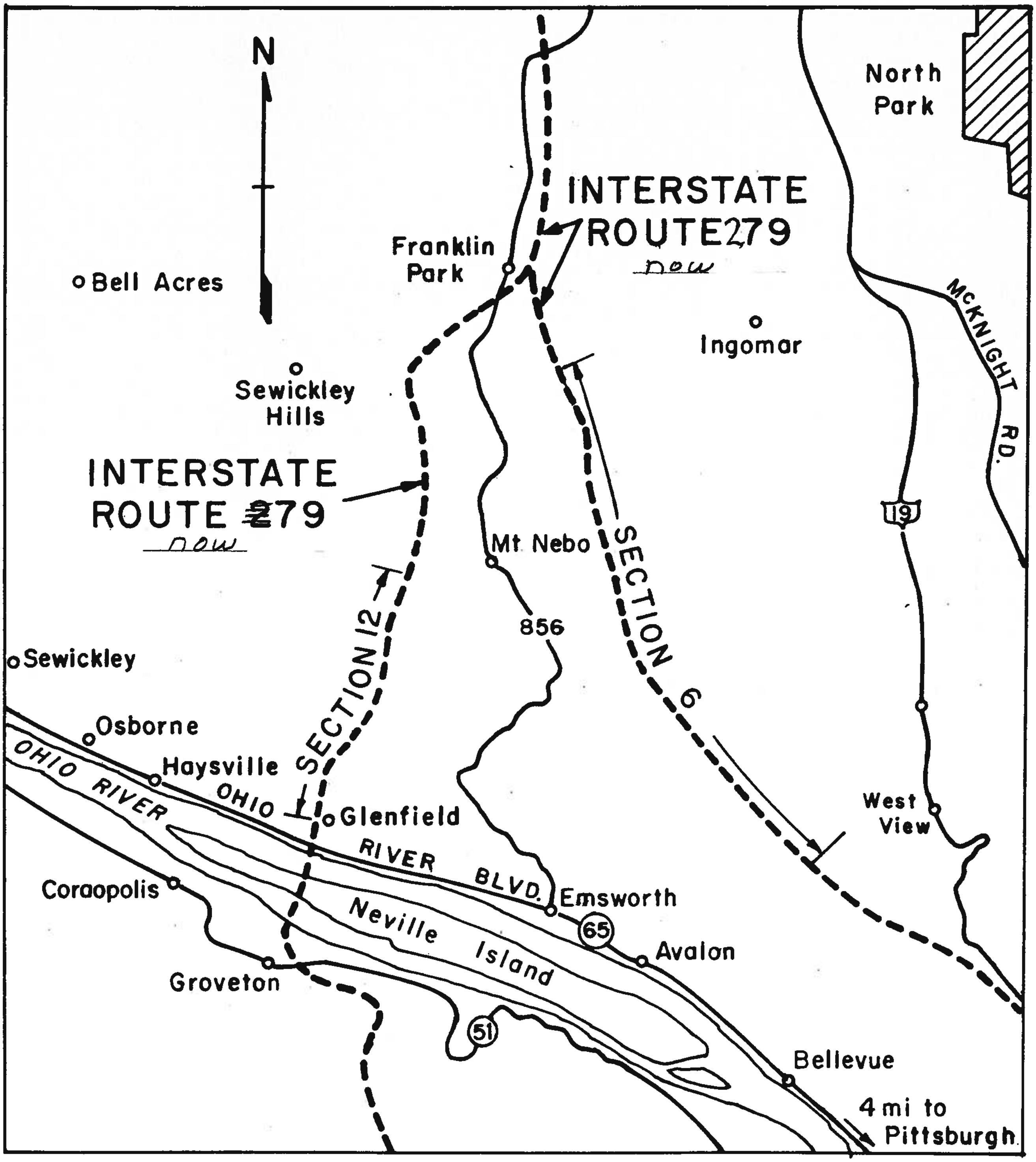
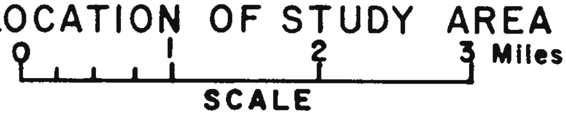


Figure 3-1
INDEX MAP SHOWING
LOCATION OF STUDY AREA



4. REGIONAL GEOLOGY AND ITS RELATION TO LANDSLIDING

4.1 Physiography

The study area lies within the Allegheny Plateau portion of the Appalachian Highlands. It is an area of maturely dissected terrain having an overall relief of about 600 ft. The higher hilltops rise to elevations between 1,200 and 1,300 ft.; the deeper valleys lie at elevations between 700 and 800 ft. Local relief is on the order of 300 to 400 ft.

The area is virtually all in slope except for certain floodplains a few hundred feet wide along the more prominent valleys and a few flattish hilltop areas. Streams form a dendritic to subparallel drainage pattern. They drain to the Ohio River, the principal stream of the region.

A soil cover of three to five ft. is common but bedrock outcrops do occur rather abundantly along the larger stream valleys. More than 50 percent of the area is forested. The populated portions of the area are largely confined to valley bottoms and ridge tops, the valley slopes being generally too steep for easy access.

Very few specific data are available on groundwater levels in the area. It is probable, however, that the general groundwater level (the water table) fluctuates through a range of a few feet between the wetter winter and spring seasons and the drier summer and fall seasons. There is ample evidence of contact-type hillside springs resulting from perched water tables. These develop where overlying permeable beds such as sandstones or shaly sandstones are in contact with fairly impermeable beds such as claystone. Surface water infiltrates downward through the more permeable strata, then flows along the contact between these permeable beds and less permeable ones, and emerges at the surface as a spring.

An important aspect of the terrain of the study area as it relates to highway construction is the presence of hillside benches or terraces that have resulted from ancient landslides or slumps in weak strata. Such ancient landslide topography is of prime interest because of a direct relationship between it and the landsliding encountered during construction of I-279. Similar landslide problems are anticipated in at least one place along I-79 when construction begins. This ancient landslide topography is discussed in part 4.4 of the report.

4.2 Stratigraphy

4.2.1 General Description

The strata exposed in the study area lie within the Conemaugh Group of the Pennsylvanian System. The position of these Conemaugh rocks with respect to strata of other groups of the Pennsylvanian and Permian Systems is shown in Fig. 4-1. Details of the stratigraphic section in the study area are shown in columnar form in Fig. 4-2 and are described in the following portion of this report.

Generally the Conemaugh strata are composed of sandstone, claystone (indurated clay)*, and shale with minor amounts of limestone and coal. It is important to note that there are gradations from one rock type to another among these types, so that shaly sandstones, silty shales, calcareous claystones, clayey limestones, etc. occur. It is also noteworthy that these strata are relatively thin (a few inches to a few tens of feet), meaning that the lithology or rock type changes in short distances in the vertical

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* Indurated clay is a term used by the Ohio River Division, U.S. Army Corps of Engineers, and a few other organizations for massive, slickensided claystone.

direction. Although the stratigraphic units do have a certain degree of lateral continuity, most of them also show lateral rock-type changes as well as thickness changes.

Vertical changes in rock type are characteristic of coal-bearing strata. Not only do they change---there is also repetition of similar rock types in the vertical sequence. Sandstones are repeated, claystones are repeated, limestones are repeated, coal beds are repeated, etc. Bundles of strata containing a variety of rock types that are repeated in such multiple fashion are called "cyclothems." All cyclothem bundles have certain similarities. They also have certain differences which make detailed geologic mapping of them in a given local area of study mandatory for obtaining good engineering geology data. From the engineering standpoint, the presence in these sequences of weak beds only a few feet thick or even a few inches thick is of prime importance. Generalization of rock types, both vertically and laterally, may prove inaccurate and misleading. Slopes designed on the basis of such generalized information can be so steep that they fail or so flat that they are not economical.

Conemaugh strata in the I-79 and I-279 areas include several zones of weak claystone beds as shown in Fig. 4-2. The most troublesome of these is the unit known as the Pittsburgh Redbeds, a zone of weak claystone (indurated clay) in which abundant ancient and recent landsliding has occurred. These redbeds are discussed below, as are other stratigraphic units.

4.2.2 Description of the Stratigraphic Section

As Fig. 4-2 shows, the stratigraphic section in the I-79 and I-279 area of Kilbuck, Aleppo, and Ohio Townships extends from the Buffalo

Sandstone at the base through the Morgantown Sandstone and a redbed-type claystone sequence whose stratigraphic name is uncertain, at the top. This section has a total thickness of roughly 420 ft. The lower strata in the section occur near the bottom of Kilbuck Run and Bear Run Valleys; the intermediate ones occur generally on the intermediate valley slopes but are present toward the valley bottoms in the northern part of the area; and the higher strata of the section occur on the upper slopes.

Shale, silty shale, sandstone, and claystone (indurated clay) comprise the major part of the section. Also present are three thin marine limestone beds that are very useful as stratigraphic marker beds. They are the Pine Creek, Woods Run, and Ames Limestones. In addition, there are calcareous claystone zones at several horizons. Coal is virtually absent in the sequence, but the Duquesne Coal does occur sporadically as a bed a few inches thick. There are two prominent sandstone units, the Buffalo and the Morgantown Sandstones. Morgantown Sandstone occurs in more cuts than the Buffalo owing to the higher stratigraphic position of the former.

Rock types that have considerable strength and resistance to weathering, and therefore do not present any particular problems in highway construction in the study area are sandstone, shale, silt shale, and hard limestone. Clay shales and claystones (indurated clays), however, are weak rocks and therefore deserve special attention, particularly the claystone sequence known as the Pittsburgh Redbeds.

In the following discussion, the total stratigraphic section is subdivided for convenience into four intervals which are described in order from the base upward.

<u>Buffalo Sandstone through Pine Creek Limestone</u>

The <u>Buffalo Sandstone</u>, being the lowest unit in the stratigraphic section, occurs near the bottom of Kilbuck Run Valley at Glenfield, and near the bottom of Lowries Run Valley where Interstate Route 79 crosses that valley. It is a thick-bedded to massive, prominent sandstone of medium to coarse grain size. Its bedding is generally irregular and lenticular. Cross-bedding is common. It contains a minor amount of shaly sandstone beds a few inches thick that are not laterally persistent. They are common in the upper five or ten ft. The thickness of the sandstone is not accurately known, but it is at least 65 ft. thick at the mouth of Kilbuck Run where the sandstone is exposed along Ohio River Boulevard. It is probable that its average thickness is somewhat less.

Immediately above the Buffalo Sandstone lies a gray claystone (indurated clay) unit four or five ft. in thickness. Like all claystone, this unit is not bedded, that is, there is no distinct layering. It fractures randomly, and upon weathering breaks down to innumerable small, irregularly shaped fragments.

This claystone bed is overlain by the Pine Creek Limestone, an abundantly fossiliferous marine limestone that contains shells of brachiopods, pelecypods, and gastropods as well as crinoid stems and bryozoans. It is a good stratigraphic guide. The limestone is dark bluish gray, and dense to finely crystalline. It commonly forms a single ledge. In the fresh condition it is a hard, resistant unit, but at some outcrops it has weathered to a soft, rusty, fossiliferous unit. The Pine Creek Limestone crops out at several places in the vicinity of the junction of Lowries Run and Bear Run at elevations ranging from 860 ft. to 880 ft. The most accessible of these outcrops is one at the intersection of Lowries Run Road and Mt. Nebo Road (at road level), and another at the intersection of Lowries Run Road and Joseph's Lane (just above road level). The limestone is also exposed in a highway cut on Ohio River Boulevard half way between Glenfield and Hays at elevation 782 ft. The average interval from the Pine Creek Limestone to the Ames Limestone in the study area is from 160 to 165 ft.

Pine Creek Limestone through the Woods Run Limestone

A shale sequence that grades upward into shale containing thin interbedded layers of fine- to medium-grained sandstone lies immediately above the Pine Creek Limestone. This unit totals about 40 ft. in thickness. In the lower part, the shale is dark gray to black, well bedded, and is rather brittle and platy. Upward it becomes silty and sandy and contains interbeds of sandstone on the order of a few inches to one foot in thickness. The color changes to pale green and greenish gray upward.

Above the interbedded shale and sandstone unit, there is a 15- to 20-foot claystone (indurated clay) unit. Its coloration is mixed pale green, greenish gray, and dull red. The unit shows no bedding or fissility throughout most of its extent, but in the upper part it does have poorly developed, irregular bedding. It weathers to small irregularly shaped pieces. This claystone is another weak bed like that below the Pine Creek Limestone.

The claystone unit is in turn overlain by six or seven ft. of sandy beds, the lower half of which is a hard ledge-forming sandstone; the upper half is shaly sandstone or sandy shale having a calcareous cement. The total unit is a hard, resistant member.

The next unit above in the sequence is the Woods Run Limestone which is generally an inconspicuous member because of its thinness and its rather indistinct nature. It is commonly only about six in. thick but may occur in two parts separated by a foot of shale in which case the total thickness of the unit is two ft. The Woods Run is an impure limestone that has generally weathered to a rusty, clayey, and sandy residual mass in surface outcrops. It is a marine fossiliferous limestone but its fossils are not easily recognized. Fragments of small crinoid stems appear to be the most common fossil. Cup corals are also present. Without careful work in the field, the Woods Run Limestone can easily be overlooked. A useful technique in identifying outcrops that contain it is to look for the redbed zone in which it occurs. Typically this zone weathers back farther into a slope than does the shale above it or the sandy beds below it. Furthermore, there may be a slight seepage of water at the contact between the base of the limestone and the underlying claystone, producing a damp zone that supports more vegetation at that position than above or below.

The interval from the Woods Run Limestone to the Pine Creek Limestone averages 65 ft.; the Woods Run--Ames Limestone interval averages 95 ft.

<u>Woods Run Limestone through Duquesne Coal</u>

The Woods Run Limestone is overlain by <u>silty shale</u> (or silt shale) which contains a minor amount of thin interbeds of fine-grained sandstone. The unit is pale green to greenish gray in color. Plant fossils are fairly common. Its thickness ranges from 30 ft. to about 50 ft., being at a minimum where the overlying Pittsburgh Redbeds are relatively thick and at a maximum where they are relatively thin. This range in thickness is graphically shown in the stratigraphic section (Fig. 4-2).

The silty shale unit is thin-bedded, and rather uniformly and regularly bedded. It is at this contact between the silt shale unit and the overlying Pittsburgh Redbeds that the slide surfaces at Glenfield described in part 5.2 of this report occur.

The <u>Pittsburgh Redbeds</u> overlie the silty shale unit described above. From the top of this silt shale to the Ames Limestone, an interval ranging from 40 to 60 ft. in thickness, claystone (indurated clay) is present (see photographs, Figs. 4-3 and 4-4). This rock unit deserves special attention because it has very low strength, because it has participated in ancient landslides along I-279 and I-79, and because it is anticipated that it will present construction problems at certain places on I-79. A discussion of this is presented in part 4.4 of the report.

The term "redbed" is somewhat misleading in describing this unit. In addition to red coloration, it has a variety of other colors including gray, pale green, greenish gray, and purple. Perhaps dull red is dominant. These colors are irregularly dispersed through the bed both vertically and laterally except that the upper few feet just below the Ames Limestone appear to be predominantly gray. What is more uniform than its color is its texture and lack of bedding. The unit is predominantly very fine-grained, and is highly fractured in random directions. Slickensides along fracture planes are very abundant. Small limestone nodules are scattered through a part of the unit in some places. Because there is no bedding or fissility, the unit cannot be correctly called shale, which by definition has fissility. This distinction between the non-bedded, randomly fractured redbed-type lithology and the bedded-shale-type lithology is very important from the engineering standpoint because redbeds are generally weaker than shales. It is also important to relate the position of such weak strata to proposed highway cut slopes. Because the Pittsburgh Redbeds are so important in engineering geology studies, a plot of their position (i.e. a geologic map) should be made along the route of a proposed highway. Examples of such plots are shown on I-279 (Figs. 4-8 and 4-10) and on I-79 (Figs. 4-12 and 4-14).

The Ames Limestone overlies the Pittsburgh Redbeds and is itself overlain by another redbed-type claystone unit 10 or 15 ft. thick whose coloration in the study area is predominantly gray (see Fig. 4-5). The zone from the top of this 10- to 15-foot claystone unit down to the base of the Pittsburgh Redbeds, totaling 55 to 75 ft. in thickness and including the Ames Limestone, is a zone of weak beds that shows ample evidence of ancient landsliding on the steep slopes of Kilbuck Run Valley (see Fig. 4-6) and on those of the valley tributary to Lowries Run along which I-79 will be constructed.

The Ames Limestone itself is an abundantly fossiliferous marine unit occurring generally as a single hard bed about two ft. thick that protrudes from the more easily weathered claystone above and below it. The limestone is gray on fresh surface, but weathers to pale green, greenish gray and rusty colors. Its small fossil brachiopods, pelecypods, gastropods, bryozoans, and crinoid-stem fragments commonly appear as whitish blebs in a darker matrix. This leads drillers to log it as conglomerate instead of the fossiliferous limestone that it is. The Ames is laterally persistent and is therefore a valuable stratigraphic guide or key bed. In determining the stratigraphic section of Fig. 4-2, the Ames Limestone was used as a reference unit in identifying less distinctive stratigraphic members such as the Birmingham Shale, various unnamed silty shale units, and even massive sandstones like the Morgantown and Buffalo. The Pine Creek Limestone and Woods Run Limestone are also useful as stratigraphic guides, and are particularly helpful where a sequence containing them, and the Ames as well, is present.

The surface trace of the Ames Limestone outcrop is plotted on selected topographic maps of the I-279 and I-79 area (Figs. 4-8, 4-10, 4-12, 4-14). A structure contour map of the top of the Ames Limestone (Fig. 4-7) was used as an aid in plotting the surface trace of the Ames as it appears on the above-mentioned maps. An explanation of how this map was constructed is presented in part 4.3.2 of the report.

The claystone unit immediately above the Ames Limestone averages about 15 ft. in thickness. It is typically gray to light gray in color but may contain patches of dull red and pale green coloration. Though the lower part of it generally lacks bedding, the upper part is fair to poorly bedded and thus borders on clay shale instead of claystone. Because of vertical gradation from one rock type to another, it is difficult to pinpoint the contact between claystone and shale in this interval. This entire unit is weak and participates in landsliding as does similar material lying below the Ames Limestone.

The strata just above the claystone unit that overlies the Ames Limestone are variable in the I-279 and I-79 areas. Generally there is about a 10-foot zone containing shaly sandstone or sandy shale in the lower part, and gray or red claystone with calcareous claystone containing small limestone nodules in the upper part. Very locally a coal bed a few inches thick, the Duquesne Coal, lies at the top of this zone. In some places the Birmingham Shale lies directly on the claystone above the Ames, and in other places, the base of the Morgantown Sandstone extends down to the top of this claystone. It is common to find a spring line at the contact between either the Birmingham Shale or the Morgantown Sandstone and this claystone. Such spring water that emerges from the ground at this contact may move downward over the surface and infiltrate the disrupted Pittsburgh Redbeds that have been involved in ancient landsliding.

Duquesne Coal to Top of Exposed Section

The Birmingham Shale overlies the Duquesne Coal. Where this coal is absent, as it commonly is, the Birmingham lies directly on either a calcareous claystone unit or an interbedded shaly sandstone and sandy shale unit. The name Birmingham Shale is not completely appropriate because the unit contains, in addition to shale, thin interbedded sandstone layers toward the top. Typically, the lower one to two feet of the unit are composed of black, fissile shale containing tiny indistinct fossil remains that are difficult to detect without magnification. Above this the color of the shale changes to gray and pale greenish to greenish gray. It also becomes silty to sandy upward. In roughly the upper third of the average 30-foot total thickness, the Birmingham contains thin interbedded fine- to medium-grained sandstone layers. It is a prominently jointed unit. Its jointing facilitates the down-slope movement of blocks that break loose from their in-place position. In weathered outcrops the Birmingham Shale commonly overhangs weaker beds that underlie it.

The Morgantown Sandstone directly overlies the Birmingham Shale in the I-279 and I-79 areas. In some parts of the Pittsburgh area there is a thin coal bed (Wellersburg) and an underclay zone between the Birmingham Shale and the Morgantown Sandstone but these were not seen in the study area. The base of the Morgantown Sandstone is unconformable, that is, its irregular base truncates the underlying shale. The thickness of the Morgantown differs from place to place as the stratigraphic section (Fig. 4-2) graphically shows. Its thickness ranges from 60 to 100 ft.

The Morgantown Sandstone is generally thick-bedded to massive in its lower two-thirds. In the upper third it is thinner bedded and contains some interbedded sandy shale or shaly sandstone. Cross-bedding is common. It is generally medium- to coarse-grained, and locally may have irregular beds of conglomeratic sandstone, particularly in the basal part. Two sets of well developed high-angle joints roughly perpendicular to each other are present in the Morgantown Sandstone. This jointing aids in producing loose blocks that participate in rock falls and slumps.

The precise top of the Morgantown Sandstone is difficult to identify because of the gradational nature of the rocks in that position. It is generally overlain by 15 to 20 ft. of sandy shale which in turn is overlain by a complex of redbeds, calcareous claystones, and limestones totaling about 60 ft. Because this part of the section lies near the hilltops in the study area, less is known about it than the lower part of the section. The subdivisions of this 60-foot zone and the characteristics of the rocks in it are shown in the stratigraphic section of Fig. 4-2. Though these are generally weak strata, they do not present any special problems in the I-279 and I-79 area because they occur so high topographically.

4.3 Structure

4.3.1 Regional Structure

The stratified rocks of Allegheny County appear to be flat-lying but are actually folded into broad, gentle anticlines and synclines. The axes of these folds trend northeasterly in general but are locally quite sinuous. They are spaced from about one to three or four miles apart. Structural relief (difference in elevation of a given bed between the crest of an anticline and the trough of an adjacent syncline) is on the order of 100 ft. Regional structure in the Sewickley Quadrangle is represented on the "Structure and Economic Geology" map of the Sewickley Folio (Munn, 1911) published by the U.S. Geological Survey. A larger scale map (1:24,000) showing structure contours on the top of the Ames Limestone in the I-279 and I-79 study area is presented in Fig. 4-7 of this report.

4.3.2 Structure in the I-279 and I-79 Area

Fig. 4-7 shows by means of structure contours on the top of the Ames Limestone that the strata of the I-279 and I-79 area are folded into a broad synclinal structure and a broad anticlinal structure having a south-southwesterly trending spur. This map was constructed by obtaining field elevations on the top of the Ames Limestone at 20 different outcrops. Elevations were determined by hand-leveling from an outcrop to an accurately surveyed, identifiable point on a topographic map of 1-in.-to-50-ft. scale and a contour interval of two ft. These elevations are believed to be accurate to within one foot. Eight other control points on the Ames were obtained from outcrops of the Pine Creek Limestone by assuming an average interval of 162 ft. between the Pine Creek and the Ames Limestones. Using these 28 points of known elevation of the Ames Limestone, a structure contour map with the top of the limestone as the datum plane was constructed by interpolating elevations between the known ones and then drawing lines connecting points of equal elevation at 10-foot intervals. This map could be made more accurate by determining the elevation of the Ames Limestone in core borings from I-279 and I-79. Although drillers' logs do not identify this limestone, it can be identified by an experienced geologist.

The structure contour map (Fig. 4-7) shows that I-279 roughly parallels the 980-foot contour on the Ames Limestone from Station 900 to Station 940, then crosses the axis of a syncline at about Station 960. North from there to Station 970, it is located on the north flank of that syncline, meaning that the Ames Limestone as well as other strata are rising to the north at an average rate of a little more than 100 ft. per mile (about 2%).

I-79, from Station 340 to Station 380, cuts obliquely across a south-southwesterly trending spur of an anticlinal structure whose main trend is east-northeast. At about Station 425, it crosses the axis of this anticline. From there to Station 550 the route is essentially perpendicular to the structural trend. The axial area of a syncline occurs at approximately Station 485. From there to Station 580 the route continues perpendicular to the structural trend, on the north flank of the syncline. The dip of the strata steepens considerably in the vicinity of Stations 545 to 555 where it is slightly more than 300 ft. per mile (about 6%) as compared to about 160 ft. per mile (3%) or less in other places along I-79.

4.4 Relations of the Pittsburgh Redbeds to Slump Benches and Ancient Landslide Topography

The stratigraphic section (Fig. 4-2) shows the position of the Pittsburgh Redbeds with relation to other strata of the I-279 and I-79 area. These redbeds (claystone or indurated clay) plus those unnamed ones that overlie the Ames Limestone have participated in ancient landsliding as evidenced by certain topographic features. Recognition of such areas is of great importance where highway construction is planned because excavation through such landslide material or the undercutting of it may renew the landsliding that has occurred earlier. This is what took place on I-279 (L.R. 1016, Section 12) at Station 900 to 903 (Figs. 5-2, 5-6, 5-7, and 5-8), at Stations 907 to 909 (Figs. 5-3 and 5-9) at Stations 926 to 932 (Figs. 4-8 and 4-9), and at Stations 950 to 955 (Figs. 4-10 and 4-11). This is what could occur at certain places on I-79 as described later in this part of the report.

Ancient landslides at the position of the Pittsburgh Redbeds are recognizable on steep valley slopes as benches or terraces that have developed at the heads of the slides. Such benches are conspicuous in the field (Fig. 4-6) and can also be easily recognized on detailed topographic maps at a scale of one in. equals 50 ft., and a contour interval of two ft. (see Fig. 4-8, 4-10, 4-12, and 4-14). The width of a bench formed in this manner appears to depend on the position (vertical distance) of the Pittsburgh Redbeds above a valley floor and the steepness of the slope extending down to the valley floor. Minor benches have formed where this unit lies less than about 100 ft. above the floor, but more conspicuous ones on the order of 150 to 250 ft. wide and 500 to 700 ft. long have formed where the Pittsburgh Redbeds occur on a steep slope from 100 to 200 ft. above a valley bottom.

The surface of these slump benches generally has relief of less than six ft. In places, the benches have minor closed depressions and topographic domes on them. Downslope from the slump benches, it is common to find large trees whose trunks are curved (convex downslope) because they have been involved in downslope movements of the landslide material during growth. Such trees of one- or two-foot trunk diameter indicate that the movement, at least in part, is old. The slide material may also contain large blocks of out-of-place Morgantown Sandstone that have broken free along joint planes and moved as much as several tens of feet downslope. Tabular "float" blocks of Ames Limestone in various attitudes (position with respect to horizontal) except the horizontal one may also occur on a bench surface or downslope from it, indicating that these blocks have moved away from their in-situ positon in a claystone medium. One must be careful not to use elevations obtained from such float blocks in determining accurate elevations of the Ames Limestone as a reference bed.

In Kilbuck Run Valley where I-279 is being constructed, some slump benches are sufficiently extensive to have been selected as farm sites by the early inhabitants of the area. The benches provided flattish land on the valley slopes where buildings could be constructed and small fields could be maintained. Furthermore, a contact-type spring generally occurs just upslope from the benches so that a source of water was readily available. Similar benches occur at several places along the valley walls adjacent to I-79 but are generally not as extensive as those in Kilbuck Run Valley where I-279 is being constructed. The most conspicuous one on I-79 occurs between Station 337 and 343.

The I-279 sites shown in Figs. 4-8 through 4-11 were selected as examples to show the relations between (1) recent landsliding, (2) the position of the Pittsburgh Redbeds (weak claystone), and (3) the presence of topographic benches that developed where ancient landslides occurred in the redbeds. Geologic mapping of a 55-foot-thick zone consisting primarily of weak claystones (the Pittsburgh Redbeds, Ames Limestone, and an unnamed claystone bed above the Ames) on a large-scale topographic base map indicates a correlation between this weak zone and hillside benches formed at the head of ancient slump masses. The fact that slumping during construction of I-279 took place at sites where these benches occurred indicates that old slump masses were reactivated by the construction. Thus it follows that the recognition of such ancient slump masses <u>in advance of highway design</u> is of utmost importance. As Skempton (1966) has pointed out, "the existence of old slip surfaces is likely to be the most significant fact revealed by the entire geologic and geomechanical investigation of the site for a rock construction." If the position of weak beds is mapped along the route of a proposed highway, and the mapping is combined with information from accurately logged core borings, geologic sections can be constructed at any location along the highway to show the position of such significant features as weak stratigraphic zones and topography resulting from ancient slumping. Fookes (1969) has stressed the value of "geotechnical mapping" prior to construction in order to show features of engineering significance and to provide advance information on engineering behavior. He also advocates geologic mapping during construction to produce "an 'as built' map---to enable comparison to be made of the predicted conditions and the 'as found' conditions." The I-279 studies provide ample support for Fookes' recommendations.

In addition to the selected I-279 sites shown in Figs. 4-8 through 4-11 representing localities where construction is already underway, and where unstable slope conditions have been encountered, two other selected sites are presented in this report as examples of the use of geologic mapping in recognizing potential problem sites along I-79 where construction has not yet begun (Figs. 4-12 through 4-15). At one of these sites (Figs. 4-12 and 4-13), the weak Pittsburgh Redbeds zone will be undercut; at the other (Figs. 4-14 and 4-15), a fill will be placed on this same weak zone.

On the following pages, a station-by-station description of localities along I-79, Section 6 (from the limit of work at Station 324 to the limit of work at Station 575) where the Pittsburgh Redbeds are involved either in cuts or in fills, is presented. The purpose of this is to show the utility of detailed geologic mapping in providing advance information of engineering significance; information that might be used advantageously in the design stage of a highway such as I-79. It should be pointed out that the geologic mapping on which the following remarks are based could be refined if field work were supplemented by an inspection of all cores from borings along I-79. That would allow the subsurface recognition of such key beds as the Ames Limestone which is not recorded in drillers' logs, and the more precise plotting on topographic maps of the position of that stratigraphic unit and the underlying Pittsburgh Redbeds. Thus, the remarks presented below can be considered representative of the type of data useful in highway design in the coal-bearing rocks of Western Pennsylvania.

STATION ON I-79	REMARKS
324 (limit of work) to 326 + 60	Base line of N.B. lane on Pittsburgh Redbeds. Between Sta. 324 and Sta. 325, there is a suggestion of a hillside bench at elevation 985, indicating probable ancient slumping in the redbeds (claystones). At Sta. 326 + 60, a 100-ft. cut is proposed. The lower 30 ft. will be a cut at a 1-1/2:1 slope in a weak zone including the Pittsburgh Redbeds, Ames Limestone, and overlying claystone.
331 + 50 to 344	A cut ranging up to 200 ft. high will truncate the Pittsburgh Redbeds, the Ames Limestone, and the overlying claystone (a 55-ft. zone from approximately elevation 985 to 1,040) near the base of the cut. At Sta. 337 + 00, this 55-ft. zone of weak beds will occur in the lower third of the proposed 200-ft. cut. From Sta. 337 + 00 to Sta. 343, a prominent ancient slump bench occurs along the center line of the N.B. lane at an elevation of 977 ft. The grade of the N.B. lane drops from elevation 954 (at Sta. 340) to elevation 948 (at Sta. 343). A cut in the slope at these stations will probably intersect an ancient shear zone along which slumping has occurred.

Sta. 348 to 357

A cut of 55-ft. maximum height will undercut the Pittsburgh Redbeds (see Figs. 4-12 and 4-13). At Sta. 352 + 50, the top of a 55-ft.-high cut will occur approximately at the base of the redbed zone (elevation approximately 980 ft.).

No well-defined ancient slump benches occur on the slope that will be cut, but there is a suggestion that the slope is unstable. Two prominent ancient slump benches do exist in the Pittsburgh Redbed zone on nearby slopes. One of these occurs at an offset distance of 320 ft. right of the centerline at Sta. 356, and 380 ft. offset right of the centerline at Sta. 357 (see Fig. 4-12). The elevation of the most conspicuous part of this bench is 991 ft. It extends along the slope for at least 250 ft.

Another ancient slump bench in the Pittsburgh Redbeds occurs at an offset distance of 400 ft. right of the centerline at Sta. 350. This extends along the slope (nearly perpendicular to the trend of the highway) for a distance of at least 300 ft. Its elevation ranges from 889 to 996 ft. A closed depression having a low point of 991 ft. occurs on this bench. The presence of these two prominent ancient slump benches definitely indicates <u>an unstable slope</u> at the stratigraphic level of the Pittsburgh Redbeds, at least where the benches occur, and it suggests the possibility of unstable conditions in the slope to be cut between Stations 351 and 357, even though a well-defined slump bench has not developed there.

Sta. 377 to 384

A cut ranging up to about 90 ft. in height will be made in the stratigraphic zone just below the weak Pittsburgh Redbeds. The top of this cut at Sta. 381 + 00 will, at elevation 985, be about five ft. below the estimated position of the base of the Pittsburgh Redbeds (990 ft.). A conspicuous ancient slump bench occurs upslope from this at an elevation of 1,008 to 1,010. This bench is 120 ft. wide; it extends along the slope for 400 ft. A smaller slump bench at the same level occurs at an offset distance of 340 ft. left of the highway centerline at Sta. 383. Almost certainly there is <u>an ancient shear zone</u> along which sliding took place at some depth below this bench. At what position this predicted shear zone emerges on the slope to be cut is not known. It is possible that it exists below the level of the top of the proposed cut between Sta. 378 and Sta. 384.

Sta. 481 to 490

The lower 70 ft. of a 160-ft. cut will be in claystones above and below the Ames Limestone (Pittsburgh Redbeds below; unnamed claystones above) and will also cut the thin Ames Ls. Groundwater seepage can be expected at an elevation of approximately 1,060 ft. at the contact between the unnamed claystone and the overlying Morgantown Sandstone.

Sta. 490 to 514

The N.B. lane will be on fill on the weak Pittsburgh Redbeds (claystone) most of the way between Stations 490 and 514 (for example, see Figs. 4-14 and 4-15).

The S.B. lane will be on a fill on the Pittsburgh Redbeds from Sta. 491 to Sta. 494 + 50 and from Sta. 505 to 514 + 50.

Sta. 514 to 524 — Both the N.B. and S.B. lanes will be at approximately the level of the Ames Limestone (and therefore at the top of the Pittsburgh Redbeds). A prominent topographic bench produced by ancient slumping in the redbeds occurs at elevation 1,003 (about 20 ft. below road grade) between Sta. 515 and Sta. 519. The bench is approximately 75 ft. wide. Its upslope edge is offset 50 ft. left of the centerline of the highway at Sta. 515; it is offset 160 ft. left at Sta. 517, and 180 ft. left at Sta. 519.

Sta. 524 to 575 (limit of work) — The base line of the highway is either on a fill on the Pittsburgh Redbeds or in a low cut in the upper part of the redbeds between these stations.

4.5 Conclusions

As a result of (1) a field study of the geology of the area in which I-279 (L.R. 1016, Section 12) is being constructed and in which I-79 (L.R. 1021, Section 6) will be constructed, (2) a review of consultant reports submitted prior to the design stage of these interstate routes, and (3) a study of large-scale topographic maps of the route sites, the following conclusions are made:

Landslides encountered during the construction of I-279, Section 12, at Stations 899 to 932 and Stations 950 to 955 were caused by excavation of ancient landslide masses (colluvium) in a zone of claystone (indurated clay) about 55 ft. thick that comprises the Pittsburgh Redbeds principally but also includes the Ames Limestone and an overlying unnamed claystone bed. This weak zone of strata is the principal cause of slope instability on both I-279 and I-79.

These ancient landslide masses (of above conclusion) are recognizable in the pre-construction topography in the form of hillside benches at the heads of the dislocated material. Trees with curved trunks commonly occur on these benches or just over the lip of them on the downslope side. The benches can be easily recognized in the field. They are also recognizable on topographic maps at a scale of one inch equals 50 ft. and a contour interval of two ft.

The most likely place for encountering slope instability on I-79 is between Sta. 340 and Sta. 344. Unstable slopes may also occur between Sta. 351 and Sta. 354, and at Stations 380, 382, and 489.

Drillers' logs cannot be relied on in the construction of accurate geologic cross-sections. These logs do not distinguish between rocks with such fundamentally different strengths as claystone and silty shale, nor do they recognize key beds like the Ames Limestone, Pine Creek Limestone, and Woods Run Limestone.

The position of any weak beds known to have caused construction problems in the past (such as the Pittsburgh Redbeds) should be plotted on topographic maps at a scale of one in. equals 50 ft., and should also be shown in cross-section at places where proposed highways will cut through them or undercut them, or where a fill will be placed on them. Such maps and sections are extremely useful in selecting highway alignments as well as in designing the slopes of a proposed highway. The mapping requires a minimal amount of time.

To identify weak strata of a specific stratigraphic unit such as the Pittsburgh Redbeds, it is generally necessary to use a key bed (or key beds) as a reference unit since redbeds occur at several positions in the stratigraphic column of Western Pennsylvania. The Ames Limestone is a good reference unit and can ordinarily be recognized by itself, but the identification is made with more assurance if it can be found in a sequence that also contains other key beds like the Pine Creek and Woods Run Limestones. This is another reason for establishing the total stratigraphic sequence of a given area prior to plotting the position of any one stratigraphic unit on maps and sections.

4.6 Recommendations

Detailed geologic mapping at a scale of 1 in. = 50 ft., complemented by information from available core borings, should be done all along I-79, Section 6, to show the position of the weak Pittsburgh Redbeds zone with respect to the highway.

Additional core borings and perhaps test pits which a man can enter should be made on I-79 at the locations of ancient landslides which may be reactivated during construction. (The locations of several of these landslide sites are given above in part 4.4).

Shear-zone materials from test pits should be studied by X-ray analysis to provide more data on the occurrence and origin of expandable clays.

Core borings logged by drillers should not be used in slope design. All core borings should be logged by a geologist with experience in the geology of the local area.

Slopes on I-79 where instability is expected should be instrumented and monitored throughout the construction stage. Particular attention should be given to the role of pore-water pressure in any movement that occurs.

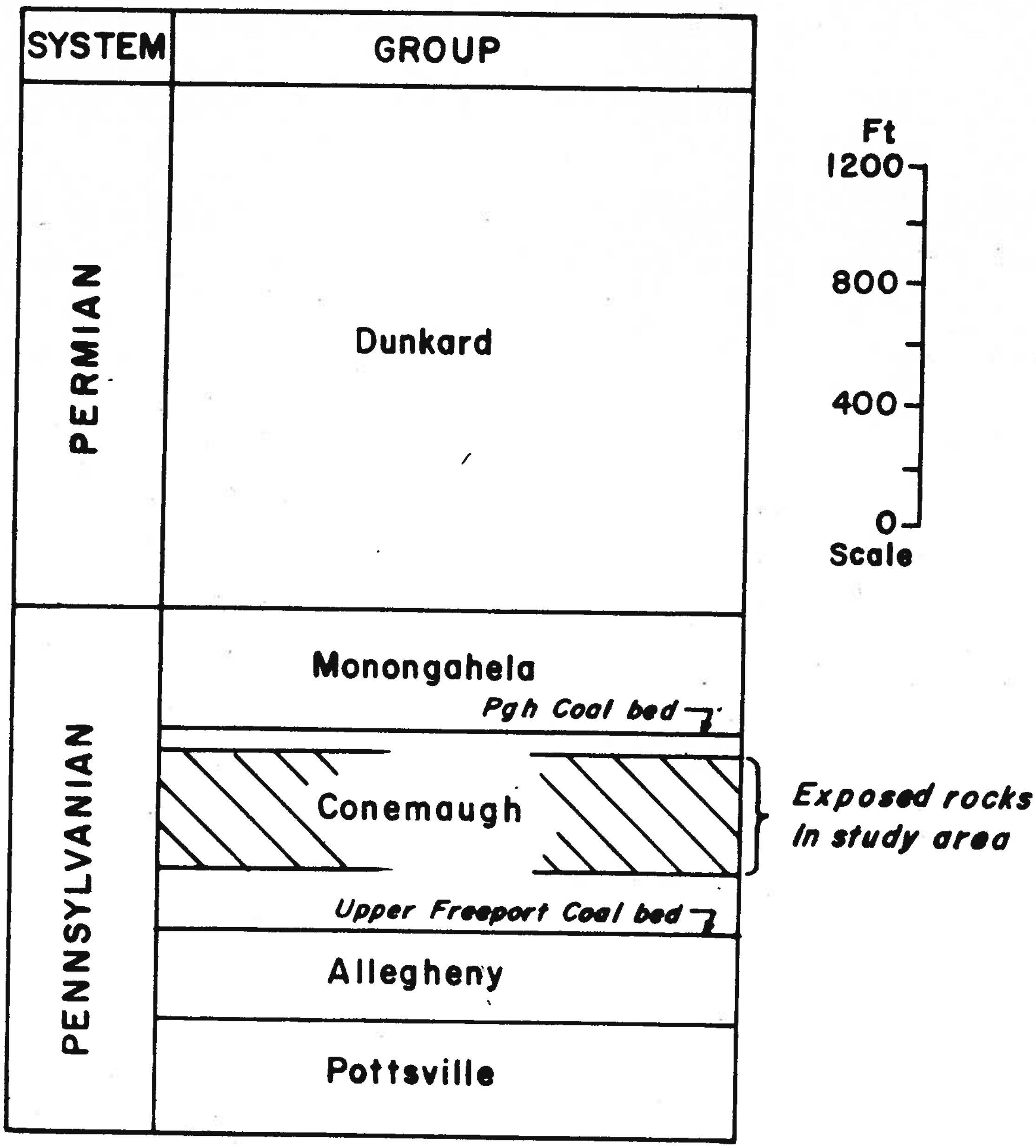
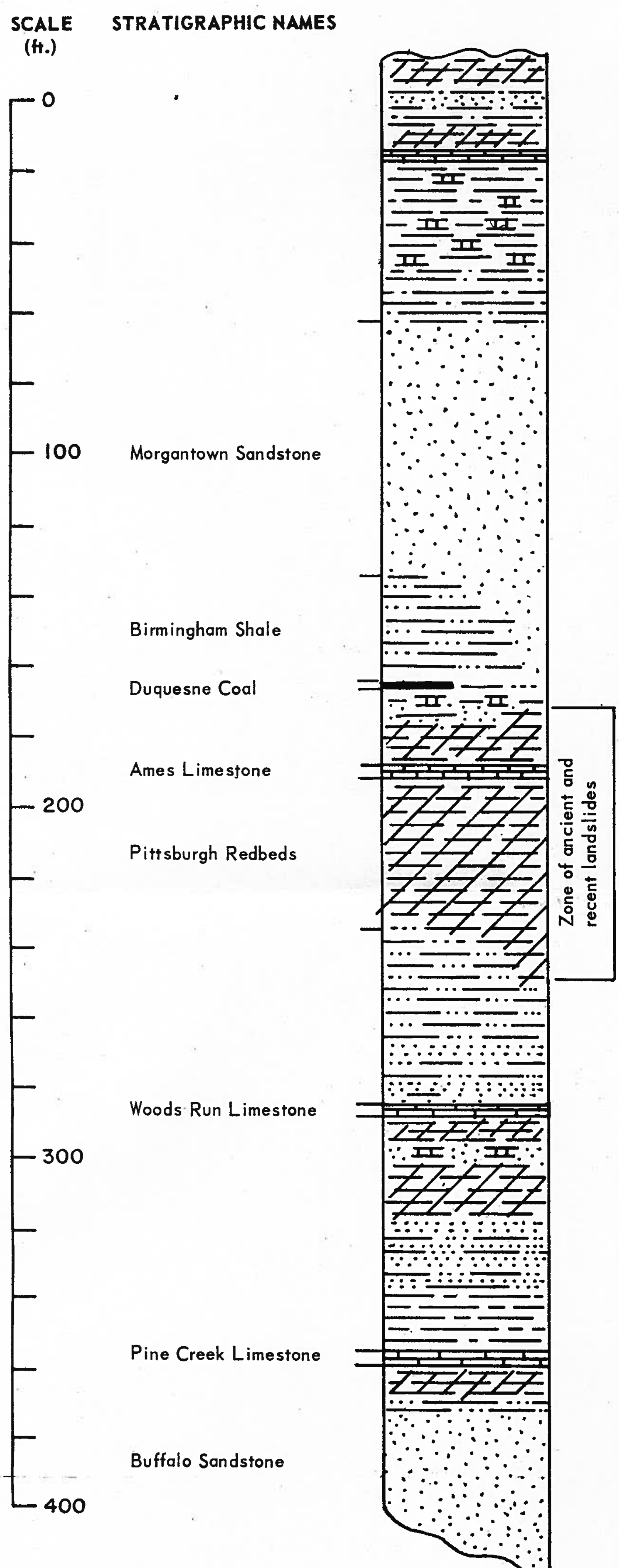


Figure 4-1
STRATIGRAPHIC POSITION OF
EXPOSED ROCKS IN STUDY AREA

STRATIGRAPHIC SECTION IN AREA OF INTERSTATE ROUTES 79 AND 279; KILBUCK, ALEPPO AND OHIO TOWNSHIPS OF ALLEGHENY COUNTY



DESCRIPTIONS OF STRATA

Claystone (indurated clay), mixed red and gray
Sandy shale and shaly sandstone interbedded, pale green
Claystone or indurated clay, mixed gray and red
Limestone, gray, dense
Claystone (indurated clay), with abundant limestone nodules, dull red with minor pale green coloration
Sandy shale, bluish gray, well bedded
Sandstone, light gray to brownish gray, medium- to very coarse-grained, thick-bedded to massive, cross-bedded in part
Sandy shale and shaly sandstone interbedded, gray to brownish gray,
Coal, very patchy
Calcareous claystone, gray, contains limestone nodules
Clay shale and claystone (indurated clay), pale green with patches of dull red
Limestone, gray to pale greenish gray, crystalline, abundantly fossiliferous
Claystone (indurated clay), dull red with patches of gray and pale green, upper few feet generally gray, with limestone nodules in places
Silt shale, gray; with thin interbedded fine-grained sandstone layers in lower part
Limestone, gray to rusty, impure, sparsely fossiliferous
Claystone or indurated clay, pale green with mixed dull red coloration
Calcareous sandstone, pale green to gray
Claystone (indurated clay), pale green and dull red, becomes shaly near top
Shale, pale green and greenish gray, with interbedded fine-grained sandstone in upper part
Limestone, bluish gray, abundantly fossiliferous,
Claystone, gray, generally plastic
Sandstone, gray to light gray, medium- to coarse-grained, medium-bedded to massive, cross-bedded, shaly at top

KEY TO SYMBOLS

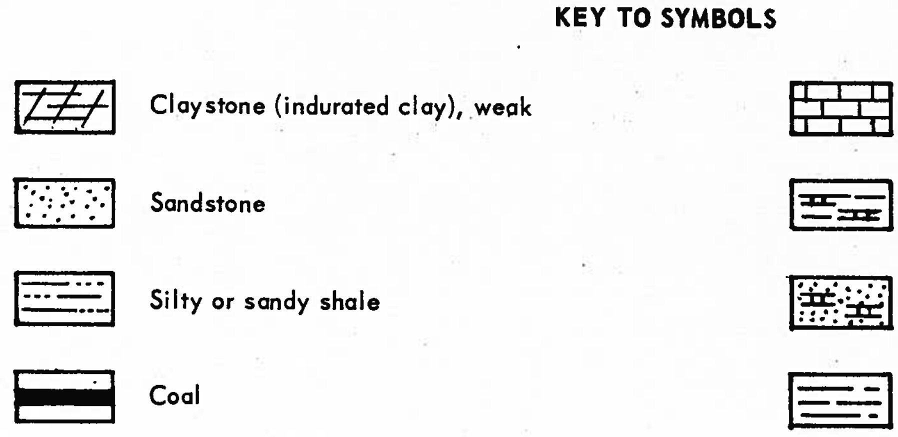


Figure 4-2
STRATIGRAPHIC SECTION

Figure 4-3

The Pittsburgh Redbeds on I-279 between Sta. 980 and 990; arrow points to Ames Limestone outcrop; Nov. 1, 1968

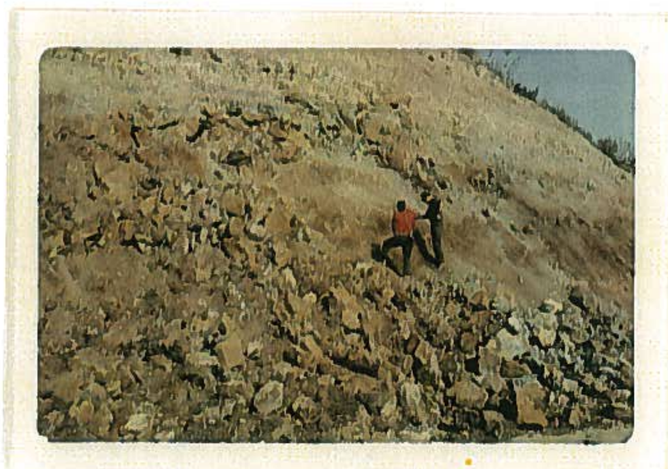


Figure 4-4

Closer view of Pittsburgh Redbeds (Fig. 4-3); Nov. 1, 1968

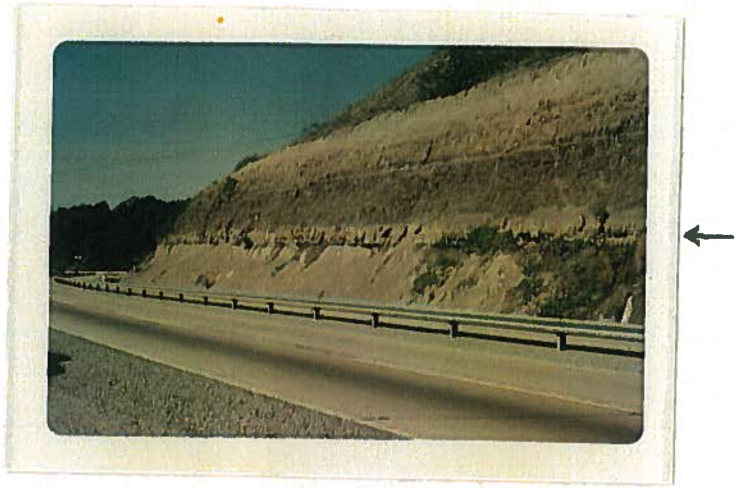


Figure 4-5

The Ames Limestone (shown by arrow) on Pennsylvania Turnpike between the Pittsburgh and Allegheny Valley interchanges; about 10 ft. of claystone (unnamed unit) overlies the Ames; the Pittsburgh Redbeds (mostly talus-covered) underlie it; Sept., 1961

Figure 4-6

West side of Kilbuck Run Valley; looking south on I-279 from about Sta. 900; note slump bench (at level of arrow) and tilted trees March 7, 1969

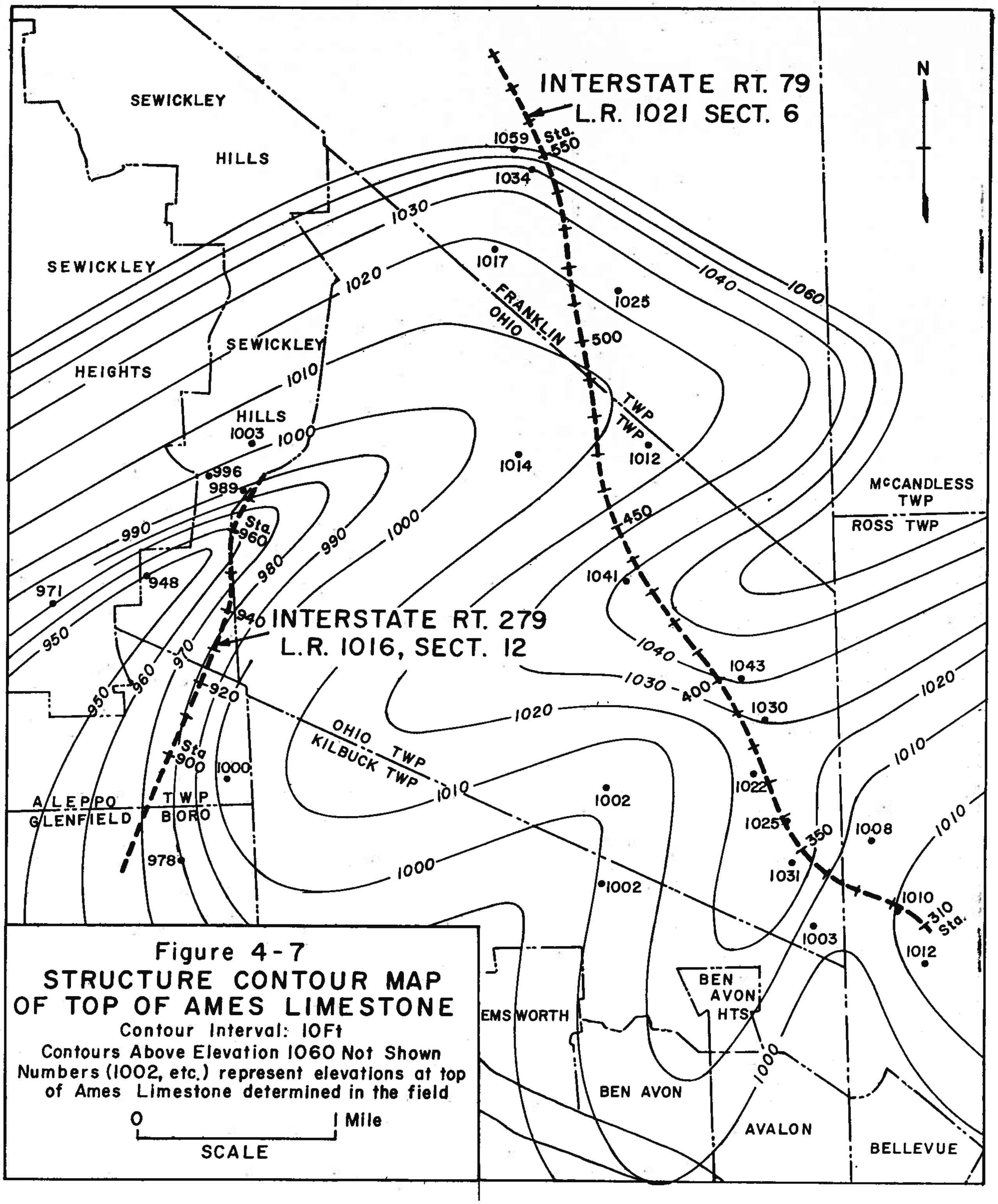


Figure 4-7
STRUCTURE CONTOUR MAP
OF TOP OF AMES LIMESTONE
Contour Interval: 10Ft
Contours Above Elevation 1060 Not Shown
Numbers (1002, etc.) represent elevations at top of Ames Limestone determined in the field

0 — 1 Mile
SCALE

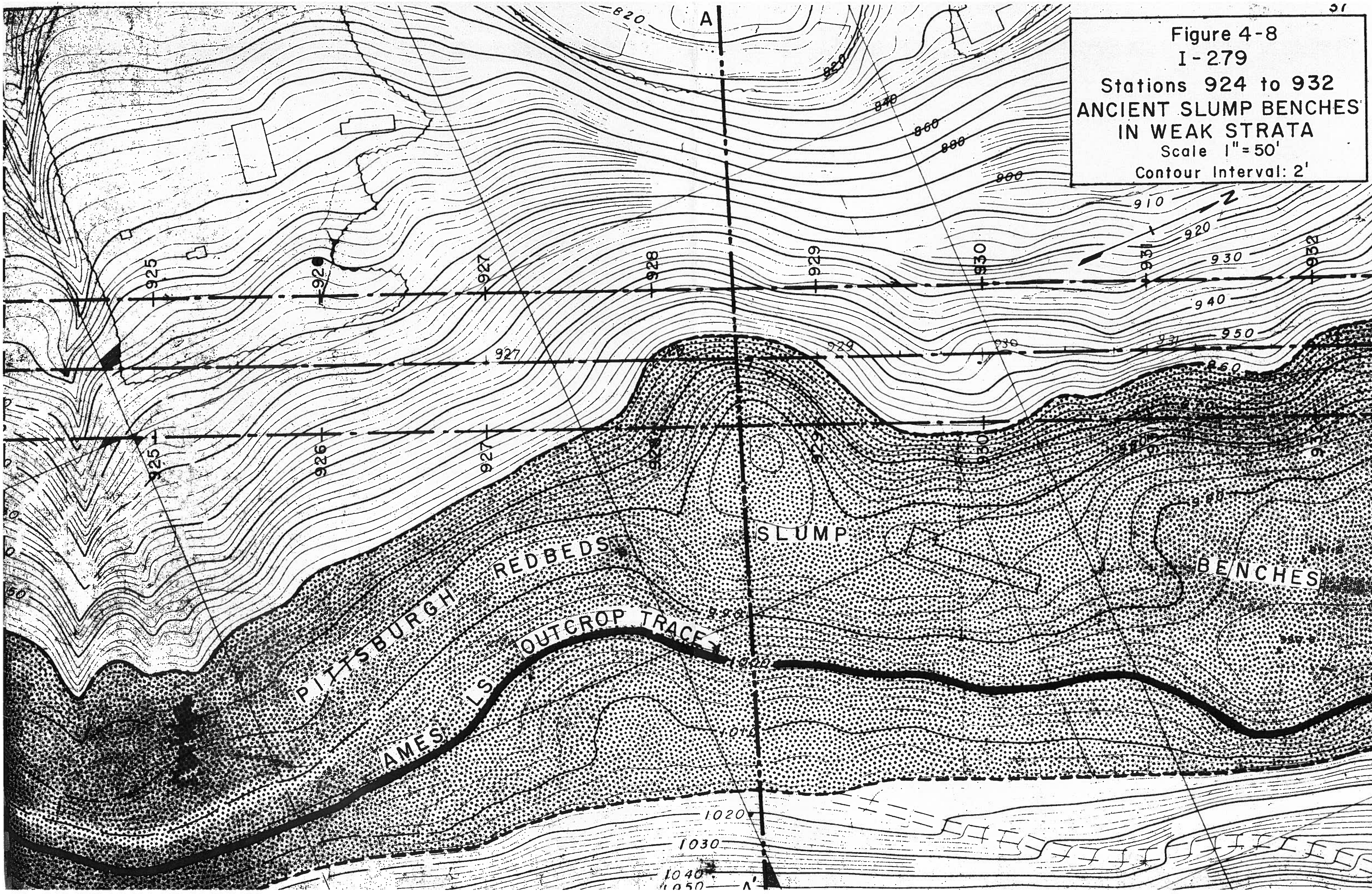
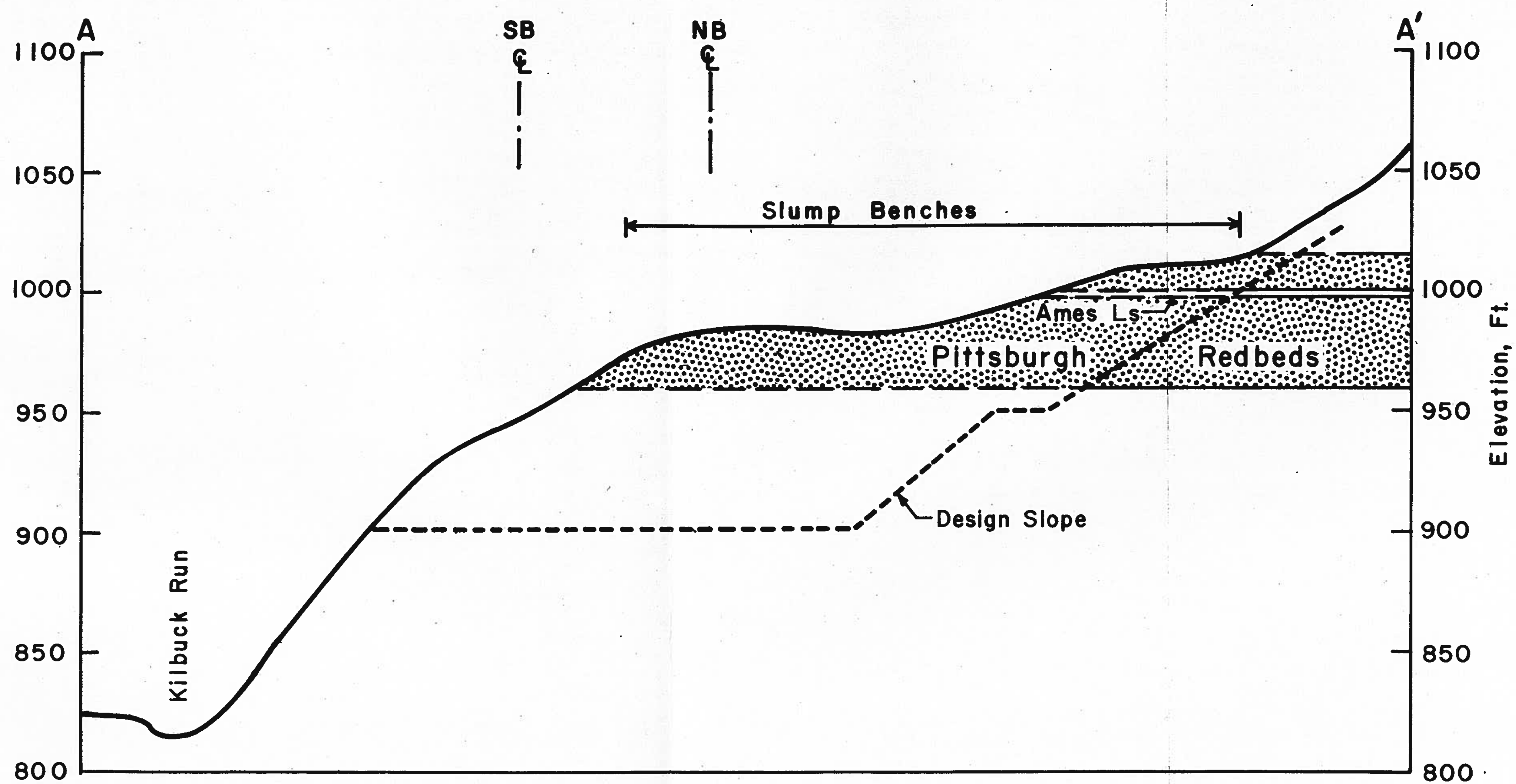


Figure 4-8
I-279
Stations 924 to 932
ANCIENT SLUMP BENCHES
IN WEAK STRATA
Scale 1" = 50'
Contour Interval: 2'



ANCIENT SLUMP BENCHES IN WEAK STRATA THAT WERE UNDERCUT

Figure 4-9
I-279
Section at Sta. 928+50
Horiz. & Vert. Scale 1" = 50'

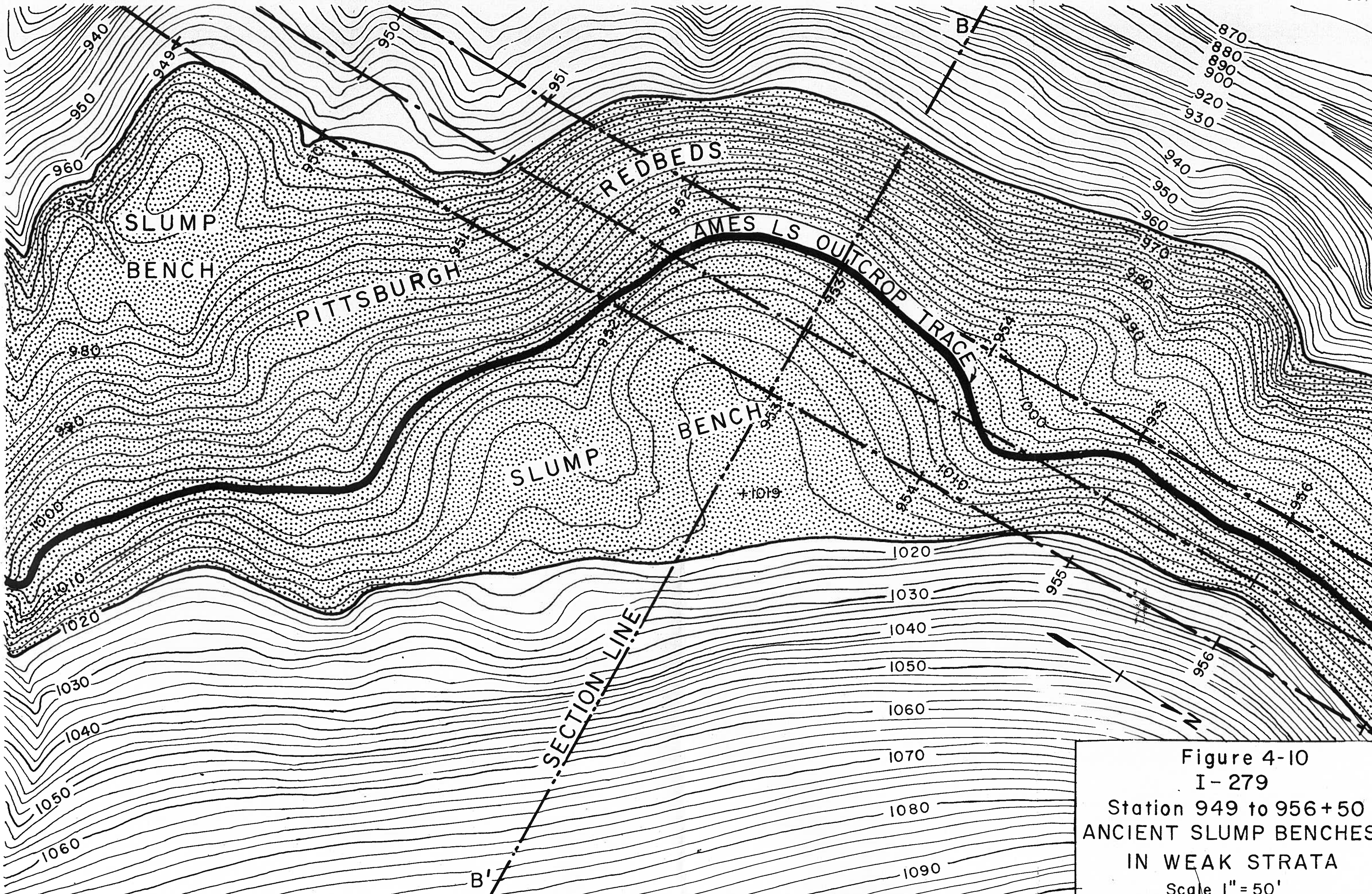
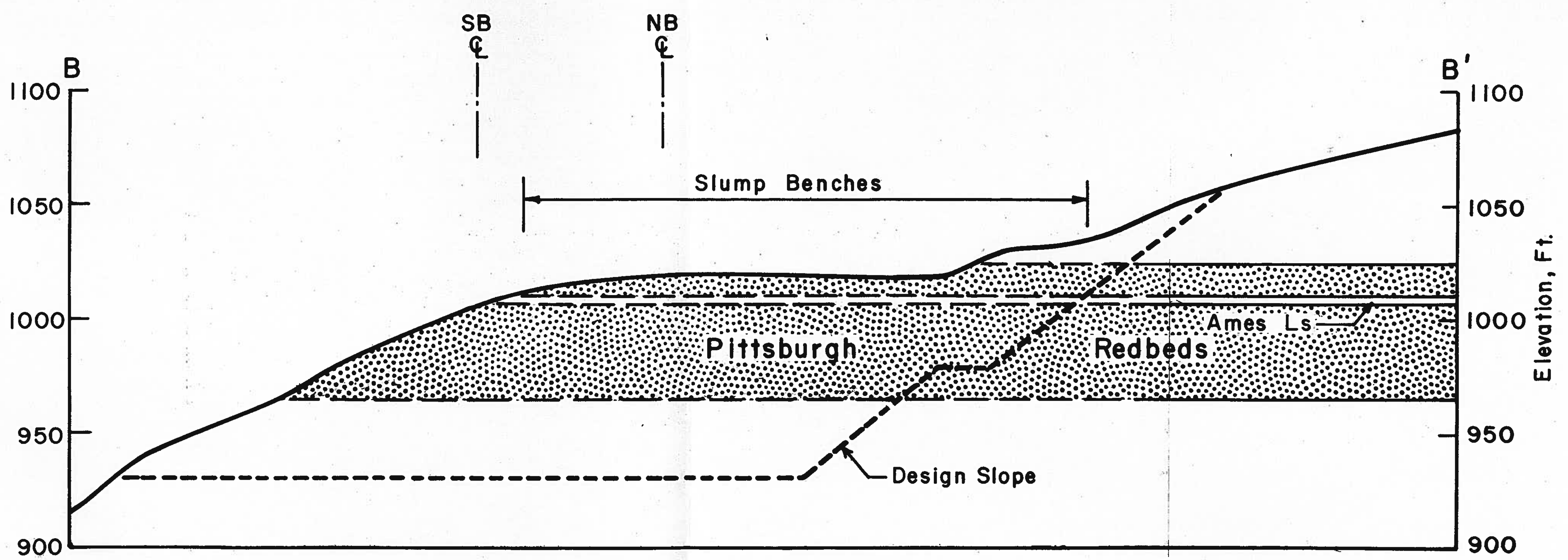


Figure 4-10
I - 279
Station 949 to 956+50
ANCIENT SLUMP BENCHES
IN WEAK STRATA
Scale 1" = 50'



ANCIENT SLUMP BENCHES IN WEAK STRATA THAT WERE UNDERCUT

Figure 4-11
I-279
Section at Sta 953+00
Horiz.&Vert. Scale 1"=50'

5. GLENFIELD SLIDES

5.1 Introduction

Interstate Route 279 crosses the Ohio River nine miles northwest of Pittsburgh, Pennsylvania and then continues northward through a tributary valley of the Ohio. The alignment and grade of one section of this highway are such that it passes through a zone of colluvium in the wall of the tributary valley. When construction began on this section in the autumn of 1968, several slides were initiated along ancient landslide surfaces in the colluvium. These slides were investigated in the present research project. Two of them, investigated in detail, are described in this section.

The geology and geometry of the slide areas are described and the nature of the movements is discussed. Strength parameters required for limiting equilibrium of the slide masses were calculated with the Morgenstern-Price method of stability analysis. The calculated strength parameters are compared with those determined from laboratory tests on failure-surface materials and found to agree with measured residual strength parameters. The sliding block method of stability analysis was used to calculate the factors of safety of the slide masses using these residual strength parameters. Conclusions are drawn concerning the mechanisms of failure, the roles of peak and residual shear strengths, and the applicability of the various methods of stability analysis.

5.2 Description of Slides

5.2.1 Location

Interstate Route 279 (I-279) crosses the Ohio River nine miles northwest of Pittsburgh on a bridge at Neville Island (see Fig. 5-1). This bridge carries the highway into the west wall of the valley of Kilbuck Run, a small stream which flows south into the Ohio River. The highway alignment runs along the west wall of the valley above the village of Glenfield to a point approximately 0.9 mi. north of the Ohio River where it crosses to the east wall of the valley on a second bridge. It continues north along the east wall of the valley for approximately 1.6 mi. and then crosses back to the west side of the valley on a third bridge. This highway alignment was chosen to avoid, as much as possible, the existing houses, roads, and stream on the valley floor.

The slides described herein are located on the east side of the valley just north of the first cross-over from the west side. That is, they are located about 1.0 mi. north of the Ohio River and 0.7 mi. north of the village of Glenfield. Topographic maps and plans of the two slides analyzed in detail are shown in Figs. 5-2 and 5-3. Both slides occurred in sidehill cuts into colluvial masses. Slide B (Fig. 5-2) extends from about Station 899+40 to Station 903+60. A cross valley is located between Stations 904 and 906. Slide A (Fig. 5-3) extends from Station 906+50 to Station 909+50. Another cross valley begins at about Station 910. Figs. 5-4 and 5-5 are photographs of the slide area.

These two slides are typical of those which have occurred on the project to date. Other major slides occurred from Stations 916+50 to 918+50, 920 to 922, 926 to 932, and 950 to 955. It was impossible to study all of these other slides in detail because of time limitations. They were inspected periodically, however, and their significant features were noted. These features are discussed in connection with the significant features of Slides A and B. Samples of failure-surface material were also obtained from several of these other slides. Laboratory tests performed on these samples and on those from Slides A and B are described in Appendix A.

5.2.2 Geology

The site geology is described in detail in Section 4 of this report. Only the more important engineering geology aspects are discussed here. The rocks of the slide areas are essentially flat-lying cyclic sediments of the Conemaugh Group, Pennsylvanian age. Ancient landslides high in the valley walls have displaced the Pittsburgh Redbeds and the rocks immediately overlying them. These displaced rock units form a zone of colluvium along the valley walls. The highway's alignment and grade are such that it passes through this zone of colluvium.

Four geologic sections through the slide area are shown in Figs. 5-6 through 5-9. These sections were prepared from geologic data obtained from test borings and from mapping of the cut faces after excavation had begun. The test-boring locations are shown in Figs. 5-2 and 5-3. Two series of test borings were made. The series prefixed by "3" was made during the soil survey for the highway design in the summer of 1963. The series prefixed by "PDH" was made by the Pennsylvania Department of Highways in February and March, 1969, after the slides had begun.

The material above about elevation 940 is colluvium. The upper part of this colluvium consists of a heterogeneous mixture of angular gravel- to boulder-size sandstone fragments with varying amounts of sand, silt, and clay. Some localized zones of this material consist almost exclusively of highly interlocked sandstone boulders; other localized zones are predominantly clayey. The lower part of the colluvium consists of clay and claystone (indurated clay) derived from the Pittsburgh Redbeds. The approximate boundary between the sand-sandstone and clay-claystone phases of the colluvium are shown in Figs. 5-7 through 5-9. Index and engineering properties of several samples of the colluvium taken at various locations in the project area are given in Appendix A.

The failure surfaces of both the ancient and the recent slides are located at or near the base of the clay-claystone phase of the colluvium. Stratigraphically, this is the level of the base of the Pittsburgh Redbeds. The failure surfaces of Slides A and B and of the slides from Stations 916+50 to 922, 926 to 932, and 950 to 955 are believed to coincide with the failure surfaces of ancient landslides. The outcrops of the failure surfaces of Slides A and B were studied in test pits excavated in the slope face between Stations 906+50 and 909+35 and between Stations 899+70 and 903+60, respectively. The failure surface of the slide from Station 926 to 932 was studied in exposures at the edge of the cut slope between Stations 927 and 929. Shear zones which are believed to correspond to ancient landslide surfaces were also studied in test pits in the slope face at Stations 915, 916, and 946. There has been no perceptible recent movement at these latter stations.

Each of the ancient and recent failure surfaces is characterized by a shear zone of from one to 12 in. thickness. Most shear zones are located at the top of in-place gray silty clay shale or silt shale and are generally overlain by claystone colluvium. The nature of the shear zone varies from location to location. At Station 928, for example, the shear zone as a 1/4 to 1/2 in. thick seam of damp, medium-stiff, slickensided gray clay (see Fig. 5-10). The clay, which was underlain by weathered fissile silt shale, graded upward into relatively intact gray and red claystone. The shear zone at Station 909 (the north end of Slide A) consisted of a two-in.-thick seam of wet, soft, gray silty clay (see Fig. 5-11). It was underlain by a three-in. zone of silt shale and claystone fragments in a silty clay matrix and then by in-place silt shale. This shear zone was overlain by about six in. of claystone fragments and silty clay and then by fractured claystone colluvium.

Most of the shear zones studied along this section of I-279 were similar to the one at Station 909. They had three definite parts. The actual surface of sliding was generally a 1/4 to 1/2 in. thick seam of damp to wet, soft to medium-stiff, gray silty clay with a trace of sand. This seam was usually located near mid-height of the shear zone. The parts of the shear zone above and below the clay seam consisted a mixture of silty clay and angular, sand- to gravel-size claystone and shale fragments. The thicknesses of these upper and lower parts varied considerably but they were typically two to three in. The claystone and shale fragments, especially the shale fragments in the lower parts of the shear zones, were usually aligned parallel to the direction of movement. This is considered a macroscopic manifestation of the parallel particle arrangement hypothesized by Skempton (1964) for residual strength behavior. The shear zones were usually damp to wet, and there were frequently appreciable quantities of seepage from them.

The rocks below the shear zones are in place. They consist of gray shales with some silty and sandy beds, red and gray claystones, and gray sandstones. There is considerable vertical and lateral variation in these rocks as in all the cyclic sediments of Western Pennsylvania. A sandy shale bed, for example, may grade vertically downward into sandstone and laterally into silt shale or silty clay shale.

There was a thin zone of colluvium and weathered rock along the valley wall below about elevation 940 before slope excavation began. The boundaries between colluvium and weathered rock and between weathered rock and sound rock in this zone were difficult to determine from the available test-boring information. The colluvium-rock boundaries shown along the valley wall below elevation 940 in Figs. 5-6, 5-8, and 5-9 are therefore only approximate.

It should be noted that ground profiles along both walls of the valley of Kilbuck Run have the characteristic shape of landslide terrain. This is illustrated by the ground surface profiles in Fig. 5-6 through 5-9. The surface of the colluvium near the top of the cut slope is quite level. This colluvial bench or terrace is quite consistent on both sides of the valley. It occurs at approximately the level of the Ames Limestone (top of the Pittsburgh Redbeds) and probably marks the upper part of the ancient landslide masses.

The surface of the colluvium along the lower parts of the valley wall is hummocky and frequently convex upward. It had a mean inclination of about 23^{o} in the area of Slides A and B and inclinations of 15^{o} to 25^{o} at other locations along the valley walls. This hummocky ground profile along with numerous tilted trees on the valley walls indicates that surface creep is active in the colluvium.

5.2.3 Groundwater

The slides occurred during the winter of 1968-69. Movement has continued through the spring and summer of 1969. This winter was rather mild with relatively light snowfall and frequent warm periods. Spring rains were also lighter than usual.

It was impossible, due to personnel and economic limitations and construction schedule uncertainties, to install piezometers in the slide areas to monitor groundwater conditions. Groundwater levels were, however, measured periodically in the Pennsylvania Department of Highways test borings. The locations of all springs and other seepage evidence in the slide areas were also noted and their elevations were determined by hand-leveling. Additional groundwater observations were made during excavation of the test pits in the slope face. All these groundwater data are given in Table 5-1.

During periods of snow melt and rain in late January and in February, the water table was at or very close to the ground surface in the slide zones. The long-term, steady-state seepage groundwater table in the slide zones is, however, a few feet below the ground surface. The groundwater zone is perched on the relatively impervious clay-claystone colluvium occurring at about elevation 940. The outcrop of steady-state seepage in the slope face was relatively consistent at about elevation 950 at Slides A and B from January to May. Groundwater measurements taken at holes PDH - 2, 3, 4, and 6 during May and June define the steady-state groundwater level further back in the slope.*

There are additional minor groundwater zones perched on the claystone beds in the lower part of the slope. The shales and sandstones in this lower part of the slope are relatively free-draining due to their joints and bedding planes. Water flows horizontally through the sandstones and shales and out the slope face above the claystone beds. The quantity of water in these lower aquifers appears to be much less than that in the groundwater zone perched above elevation 940. This is to be expected since most infiltration from the ground surface would go into the upper aquifer.

5.2.4 History

Construction began in the autumn of 1968 and slides began soon after slope excavation commenced. It was impossible because of personnel

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*Test borings PDH - 2 and 4 pinched off at the elevation of the failure surface of Slide A so water levels observed in them correspond to the perched water table above the failure surface. Hole PDH - 3 did not penetrate the full depth of claystone, so water levels observed in it, too, correspond to this perched water table. Hole PDH - 6 penetrates some 20 ft. into the shale below the claystone. Most of its depth, however, is in the colluvium and water levels measured in it are believed to reflect the perched water table.

and budget limitations to instrument or survey any of the slides in order to observe time rates of movement. Visual observations of slide movement were made, however, during all site visits.

Excavation began in the vicinity of Slide A on November 26, 1968. The colluvium was excavated at an inclination of 1-1/4:1 or 39^{o}. Tension cracks were reported above the cut slope between Stations 906+50 and 908+ 50 on December 4. This slide was first inspected by the writer on December 17 when excavation was down to about elevation 920. On December 17, the slide extended from Station 906+50 to Station 908+50. There was a 3-ft. scarp about 100 ft. back from the edge of the cut slope between Stations 907 and 908 and horizontal movement of three to four ft. had occurred. A relatively planar sliding surface was exposed at the base of the scarp at Station 908 (see Fig. 5-12). This surface consisted of red clayey colluvium and was slickensided from the slide movement. The surface dipped 35^{o} to 45^{o} west (in the direction of the slide movement). There were also many open fissures in the top of the slide mass parallel to the edge of the cut slope.

Movement in Slide A has continued to the present time (July, 1969). Surface-creep indications and new cracks have been observed at the rear of the slide mass since a warm period at the end of January. The slide outline shown in Fig. 5-3 corresponds to late February, 1969. Slide material began falling to the bench at elevation 900 in appreciable quantities at that time. Fig. 5-13 is a photograph of the face of the cut slope at Slide A; it was taken on March 22, 1969.

Excavation began in the vicinity of Slide B on December 10, 1968. The colluvium was again excavated at an inclination of 1-1/4:1 or 39^{o}. Cracks were reported above the cut slope between Stations 901+50 and 903+25 on December 23. This slide was first inspected by the writer on January 7, 1969, when excavation was down to about elevation 920. On January 7, this slide extended from Station 900 to Station 903+50. The rear of the slide was about 80 ft. back from the edge of the cut slope at Station 902; there was a six-ft. scarp and about four ft. of horizontal displacement. A fissure at the base of the scarp was open to a depth of about 15 ft. There were many additional fissures in the top of the slide mass parallel to the edge of the cut slope.

Slide B has also continued to move up to the present time. The south end of this slide extended to about Station 899 by the end of January. Slide material begain to fall to the bench at that time. The slide outline in Fig. 5-2 corresponds to late February, 1969. Fig. 5-14 is a photograph of the slope face at Slide B; it was taken on May 3, 1969.

The other slides which were not studied in detail have histories similar to Slides A and B. Slope excavation began in the vicinity of Stations 917 to 928 on October 23-25, 1968. The first cracks were reported at Station 922 on November 4 and additional cracks were reported at Station 918 on November 20. Excavation began at Station 950 to 955 on February 14, 1969, and cracks were noticed by the writer on March 7. Significant excavation began in the vicinity of Station 930 on February 21. Cracks were noticed by the writer on February 28 but the slide mass from 926 to 932 did not really begin to move until May 17; on May 24, there was a two-ft. scarp in the haul road at Station 931.

All of these slides involved progressive failure to a certain extent. They were preceded by the development of tension cracks at some distance back from the edge of the cut slope and, if the slide masses were not excavated, they continued to move for periods of up to seven months. Most of the slides have also grown laterally from their initial configurations. It should be noted that this behavior is consistent with the mechanism postulated by Bjerrum (1967) for the progressive failure of slopes in overconsolidated clays and clay shales.

5.2.5 Geometric Details of the Slides

All slides observed along this section of I-279 were of the sliding block type. Each failure surface consists of three parts as shown in Fig. 5-15. These parts are a basal surface of sliding, a rear surface of sliding and a tension crack at the ground surface. The basal surfaces of sliding typically dipped two or three degrees and, as mentioned previously, they usually coincided with ancient landslide surfaces near the top of in-place rock.

The rear surfaces of sliding dipped 30° to 60° along their upper portions where they crossed clay-claystone colluvium. Most of these rear-sliding-surface dips were on the order of 45° to 55°; a 50° dip is considered typical. There was usually a 1/4 to 1/2 in. thick layer of soft to medium-stiff, slickensided, plastic clay along these rear surfaces of sliding as shown in Fig. 5-16. It is not certain whether the rear sliding surfaces coincide with segments of ancient landslide surfaces, though it is suspected that they do in some cases.

The details of failure-surface geometry at the intersections of the rear sliding surfaces with the basal sliding surfaces are not well known. It is considered likely that the rear sliding surfaces flatten somewhat or become curved above these intersections but this cannot be verified until failure surfaces are exposed during excavation.

Tension cracks have occurred at the rear of each failure mass observed to date. These cracks begin to open in the early stage of sliding. The depth of tension cracks depends primarily on the nature of the colluvium near the ground surface. In sand-sandstone colluvium, tension cracks form that are generally deeper than those in claystone colluvium. Tension-crack depths of two to ten ft. were measured at the rear of Slide A in clayey colluvium and tension-crack depths of 6 to 23 ft. were measured at the rear of Slide B in sandstone colluvium.

In cases like Slides A and B where the failure mass moved out into the highway cut, the toe of the mass was sometimes cantilevered up to one ft. over the edge of in-place rock. If the toe material was relatively clayey and plastic it sometimes bent under its own weight and "flowed" several inches down over the face of the cut slope before breaking off and falling to the bench below. Definite "streamlines" were observed in this "flowing" clay; rock particles in it were generally aligned parallel to these "streamlines". Non-plastic toe material simply tumbled over the edge of in-place rock and fell to the bench below. This sliding over the edge of the cut induced flexural-type tensile stresses in the upper parts of the failure masses. Additional tension cracks then formed near the edge of the original cut slope as shown in Fig. 5-15.

5.3 Calculation of Strength Parameters

5.3.1 Idealized Slope Cross-Sections

Sets of Mohr-Coulomb shear strength parameters corresponding to the limiting equilibrium condition of factor of safety of unity were calculated for Slides A and B. Before performing these calculations, it was necessary to define idealized cross-sections through the slides. These idealized cross-sections are shown in Figs. 5-17 and 5-18.

Fig. 5-17 shows a section through the center of Slide A at Station 908+00. Two basal surfaces of sliding were used. Both intersect the slope face at elevation 940. The lower basal sliding surface has an inclination of 2.7°. It follows what is believed to be an ancient landslide surface at the base of the clay-claystone colluvium. The upper basal sliding surface has an inclination of 5.5°. It is located near the top of the clay-claystone colluvium along what may also be an ancient landslide surface.

The surface of the colluvial bench at Slide A actually dips slightly away from the highway cut but it was assumed to be level at elevation 988 for purposes of the stability analyses. A five-ft. tension crack was used at the rear of the slide mass. Rear sliding surfaces of three different inclinations- 30°, 40°, and 50° - were drawn from the bottom of this tension crack to the basal surfaces of sliding.

The colluvium in the failure masses analyzed for Slide A was assumed to have an average total unit weight of 145 pcf. Sandstone boulders and the relatively intact claystone in the colluvium typically have total unit weights of 155 to 160 pcf. and the more soil-like parts of the colluvium typically have total unit weights of 120 to 130 pcf. The average unit weight of 145 pcf. is therefore considered reasonable for the total failure masses.

The colluvium along each failure surface was also assumed to have the same average shear-strength parameters. Inspection of samples from test borings PDH - 2 and 4 and of exposures in open fissures indicated that the colluvium at the rear of Slide A was predominantly clayey. This clayey colluvium at the rear of the failure mass was, of course, derived from the same claystone material as the clayey colluvium along the base of the failure mass.

The groundwater level (GWL) shown in Fig. 5-17 is that corresponding to steady-state seepage. It is based on the observed seepage outcrops at elevation 950 in the slope face and the observed average water elevation of 982 in test boring PDH - 2. This groundwater level is considered a good estimate of groundwater conditions at the time of failure.

Fig. 5-18 shows a section through the center of Slide B at Station 901+50. The basal surface of sliding crops out in the slope face at elevation 939; it has an inclination of about 2.3°. This basal surface of sliding is also believed to coincide with an ancient landslide surface. A 24-ft. tension crack was used at the rear of the failure mass and three inclinations were again used for the rear sliding surface. It was considered likely, from the geometry of the slide, that the rear sliding surface in Slide B might be steeper than that at Slide A. Rear sliding surfaces with inclinations of 40°, 50°, and 60° were therefore drawn from the bottom of the tension crack.

The subsurface conditions at Slide B are more complex than those at Slide A. The upper part of the colluvium at the rear of Slide B consists of interlocked angular sandstone boulders and cobbles with some sand and silt and minor amounts of clay. The lower part of the colluvium consists of fissured and jointed sandy to silty claystone. The basal zone of sliding contains the clay and claystone fragments typical of most of the failure surfaces at the site. The boundaries between these zones of colluvium are not well defined in the field. The boundaries shown in Fig. 5-18 are rather approximate. They were determined from the logs of test borings PDH - 6 and 7 and from a study of exposures in the face of the cut slope between Stations 900 and 903.

Because of these differences in the colluvium, it was considered unrealistic to assume that the same average shear-strength parameters existed everywhere on the failure surfaces used for Slide B. Shear-strength parameters for the sandstone and claystone colluvium were therefore estimated and the average shear strength parameters required in the basal zone of sliding were calculated. The sandstone colluvium was assumed to have $\bar{c} = 0.5$ ksf., $\bar{\phi} = 35^{\circ}$ and the claystone colluvium was assumed to have $\bar{c} = 0$, $\bar{\phi} = 28^{\circ}$. These strength parameters were estimated on the basis of field inspection of the materials. They are believed to represent shear strengths approximately half way between peak and residual values for each material. An average total unit weight of 145 pcf. was again used for all materials.

Like the boundaries between different phases of colluvium, groundwater conditions at Slide B are not known as well as those at Slide A. The groundwater level shown in Fig. 5-18 is defined by a straight line connecting the average observed seepage outcrop at elevation 950 in the slope face with the average observed water elevation of 1020 in test boring PDH - 6. This groundwater level is considered a fair estimate of groundwater conditions in the slide mass at the time of failure.

5.3.2 Calculated Strength Parameters

The Morgenstern-Price method of slope stability analysis was used to calculate strength parameters at limiting equilibrium for each of the failure surfaces described in the previous section. The Morgenstern-Price method is a two-dimensional limiting equilibrium method of stability analysis which treats a failure surface of general shape and requires the entire failure mass to be in complete static equilibrium. It has been described by Morgenstern and Price (1965, 1967), Bailey (1966), Whitman and Bailey (1967), and Hamel (1968). Calculations were performed at the University of Pittsburgh Computer Center with the MGSTRN computer program described by Hamel (1968).

The values of $\bar{\phi}$ required at $\bar{c} = 0$ and the values of $\bar{c}$ required at $\bar{\phi} = 0$ were calculated along with the values of $\bar{c}$ required at $\bar{\phi} = 10^{\circ}$. These calculated values are given in Table 5-2. Values of tan $\bar{\phi}$ vs. $\bar{c}$ for 2.7° basal sliding surface of Slide A are plotted in Fig. 5-19. The tan $\bar{\phi}$ vs. $\bar{c}$ values for the 5.5° basal sliding surface of Slide A were not plotted since they fall within the band of tan $\bar{\phi}$ vs. $\bar{c}$ values plotted for the 2.7° basal sliding surface. The tan $\bar{\phi}$ vs. $\bar{c}$ values for Slide B are plotted in Fig. 5-20.

The calculated tan $\bar{\phi}$ vs. $\bar{c}$ values for Slide A define a relatively narrow band. The required $\bar{\phi}$ value at $\bar{c}$ = 0 falls between 12.5° and 15.5° and the required $\bar{c}$ value at $\bar{\phi}$ = 0 falls between 0.72 and 0.85 ksf. The higher values of $\bar{\phi}$ and $\bar{c}$ correspond to the rear sliding surface inclined at 30° and the lower ones correspond to the rear sliding surface inclined at 50°.

The calculated tan $\bar{\phi}$ vs. $\bar{c}$ values for Slide B also define a fairly narrow band and the $\bar{c}$ = 0 end of this band is identical with that of the band for Slide A. The strength parameters required for Slide B with the 40° and 50° rear surfaces of sliding are almost identical. They range from $\bar{\phi}$ = 15.0 to 15.5° at $\bar{c}$ = 0 to $\bar{c}$ = 1.10 to 1.20 ksf. at $\bar{\phi}$ = 0. The required $\bar{\phi}$ value of 12.5° for the 60° rear surface of sliding is somewhat below the $\bar{\phi}$ values for the flatter rear sliding surfaces. The $\bar{c}$ value of 1.15 ksf. at $\bar{\phi}$ = 0 for the 60° rear sliding surface is, however, within the range of $\bar{c}$ values calculated for the flatter rear sliding surfaces.

Some of the scatter in the strength parameters calculated for the different inclinations of rear sliding surfaces for Slide B is attributed to the fact that slightly different percentages of each rear sliding surface pass through the sandstone and the claystone colluvium. As noted previously, significantly different strength parameters were used for these two materials in the stability calculations.

5.4 Comparison of Calculated and Measured Strength Parameters

Laboratory tests were performed on samples of materials from the failure surfaces of Slides A and B and on samples of materials from the failure surfaces of other slides along this section of I-279. These samples were obtained from test borings, test pits, and surface exposures. Sample locations and descriptions are given in Appendix A along with the test results.

The materials from basal sliding surfaces were found to have peak effective cohesion intercepts of 0.20 to 0.80 ksf. and peak effective friction angles of 19° to 25.5°. The peak strength parameters $\bar{c}$ = 0.50 ksf. and $\bar{\phi} = 22^{\circ}$ to 23° are considered typical. The residual strengths of these materials depend upon the amount of sand- and gravel-size claystone and shale fragments present. Specimens containing appreciable amounts of these fragments have residual cohesion intercepts of 0.05 to 0.10 ksf. and residual friction angles of 20° to 25°. Specimens containing few sand- and gravel-size fragments, e.g., specimens from the thin clay seams along the actual failure surfaces, have residual cohesion intercepts of essentially zero and residual friction angles of 8° to 18°. Typical residual strength parameters for the clay along the basal sliding surfaces of Slides A and B are $\bar{c} = 0$ and $\bar{\phi} = 13^{\circ}$ to 16°.

Tests performed on specimens of the clayey colluvium from the vicinity of the rear sliding surface of Slide A yielded strength parameters similar to those of the materials along the basal sliding surfaces. Peak strength parameters $\bar{c}$ = 0.50 ksf., $\bar{\phi} = 19^{\circ}$ and residual strength parameters $\bar{c}$ = 0.10 ksf., $\bar{\phi} = 13^{\circ}$ were measured for this clayey colluvium.

The friction angles of 12.5° to 15.5° calculated for Slides A and B with $\bar{c} = 0$ are in excellent agreement with the measured residual friction angles of the failure-surface materials. This agreement may be somewhat fortuitous, especially in the case of Slide B where there were more material-zone-boundary and groundwater uncertainties in the stability analyses. The fact that the calculated strength parameters for both slides agree with the measured residual-strength parameters is, however, consistent with the history of the slides. The movements in the ancient landslides were of sufficient magnitudes to reduce the strengths of the failure-surface materials to residual values. It is believed that the failure surfaces did not "heal" following the ancient slides and that residual strengths existed along the failure surfaces when slope excavation began.

5.6 Conclusions

The most significant factor in the development of the Glenfield slides was the presence of ancient landslide surfaces in the cut slopes. Movements in the ancient slides reduced the shear strength along these failure surfaces to residual values. It is believed that strengths of the failure-surface materials have remained at or very close to residual values until the present time.

Cut slopes in the colluvium were excavated at an inclination of 1-1/4:1 or 39^{o}. This was too steep for the combination of low shear strength and high groundwater level existing in the slopes. Failure was probably initiated at stress concentrations near the edges of the cut slopes and then worked back into the slopes in the manner suggested by Bjerrum (1967). The movements associated with this progressive failure would have reduced the shear strengths of failure-surface materials to true residual values even if they were not previously at residual values.

The average residual shear strength parameters of the entire failure surface of Slide A and of the basal sliding surface of Slide B are on the order of $\bar{c} = 0$ and $\bar{\phi} = 13^{o}$ to 16^{o}. These strength parameters were calculated from the slides using the Morgenstern-Price method of slope stability analysis, and they were obtained from laboratory tests on failure-surface materials.

The failure surfaces of Slides A and B are highly non-circular. Fellenius' and Bishop's methods of stability analysis which employ circular failure surfaces are therefore inapplicable. The sliding block method of stability analysis is well suited to these slides because the failure surfaces can be well-approximated with two straight-line segments. The results of sliding block analyses employing inter-block force inclinations of 15^{o} to 30^{o} are in good agreement with the results of Morgenstern-Price analyses. The sliding block method of stability analysis can therefore be used with confidence in analyzing slides of this sort.

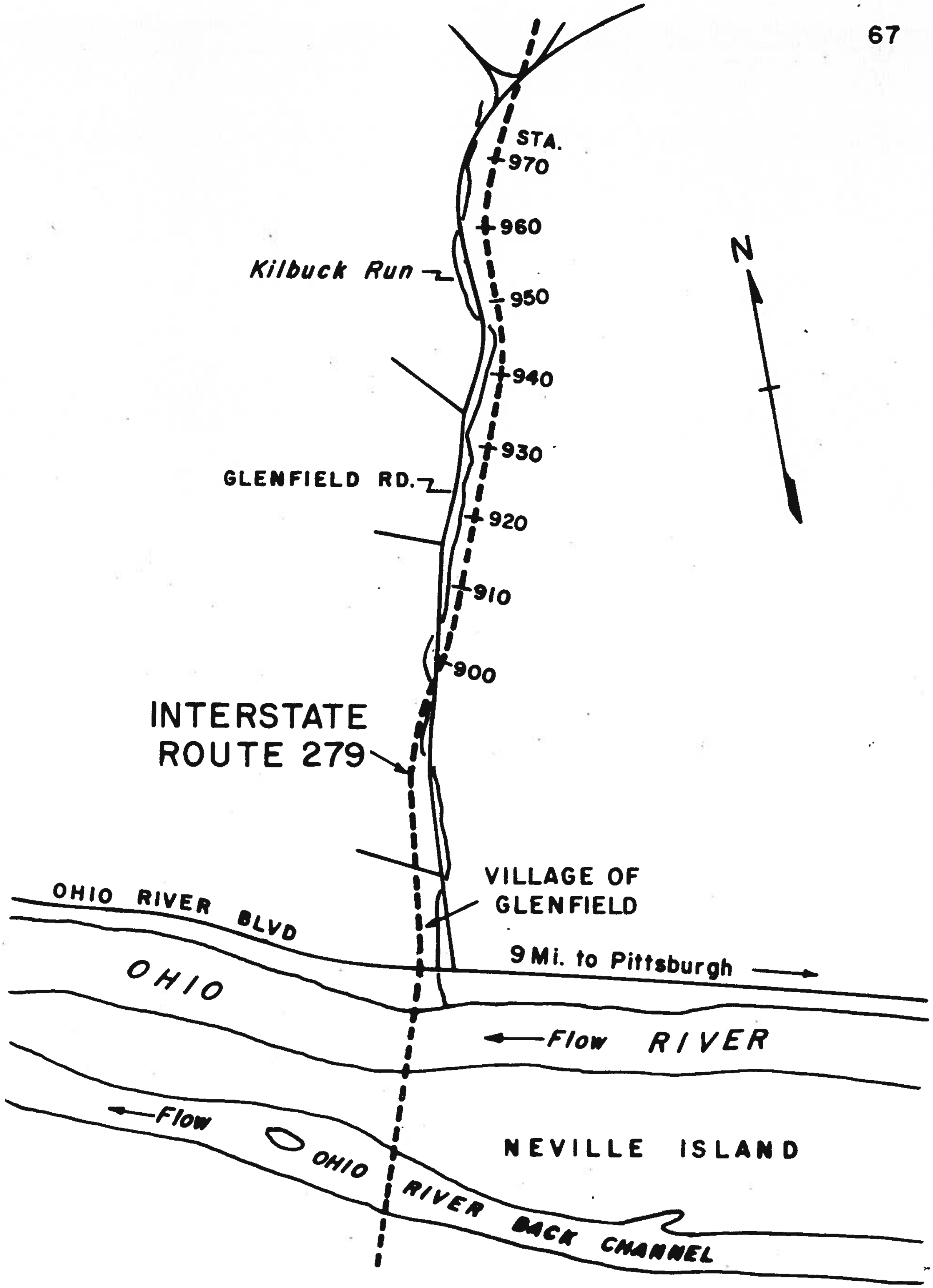

Figure 5-1
LOCATION MAP
GLENFIELD SLIDES
Scale 1" = 2000'

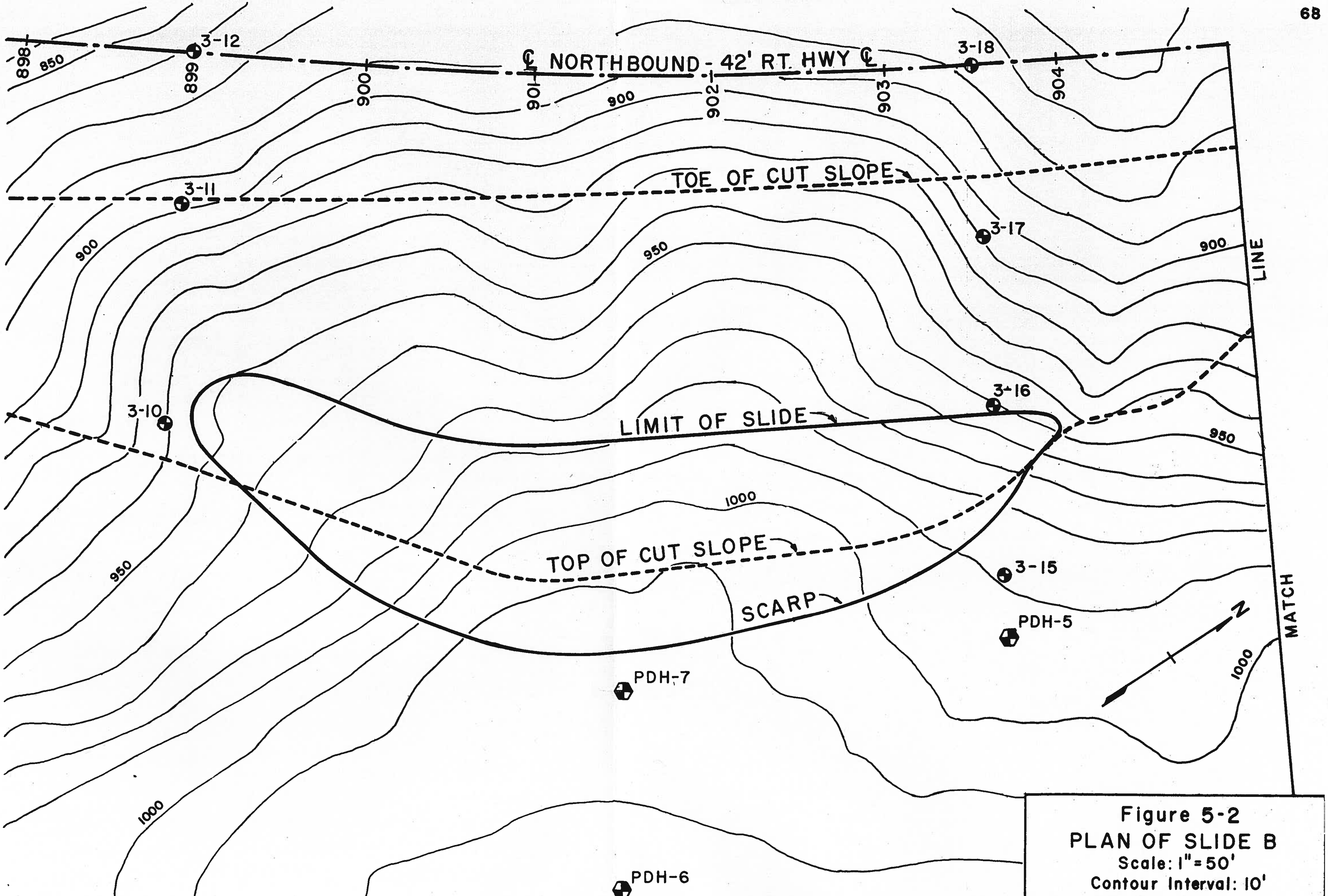

Figure 5-2
PLAN OF SLIDE B
Scale: 1" = 50'
Contour Interval: 10'

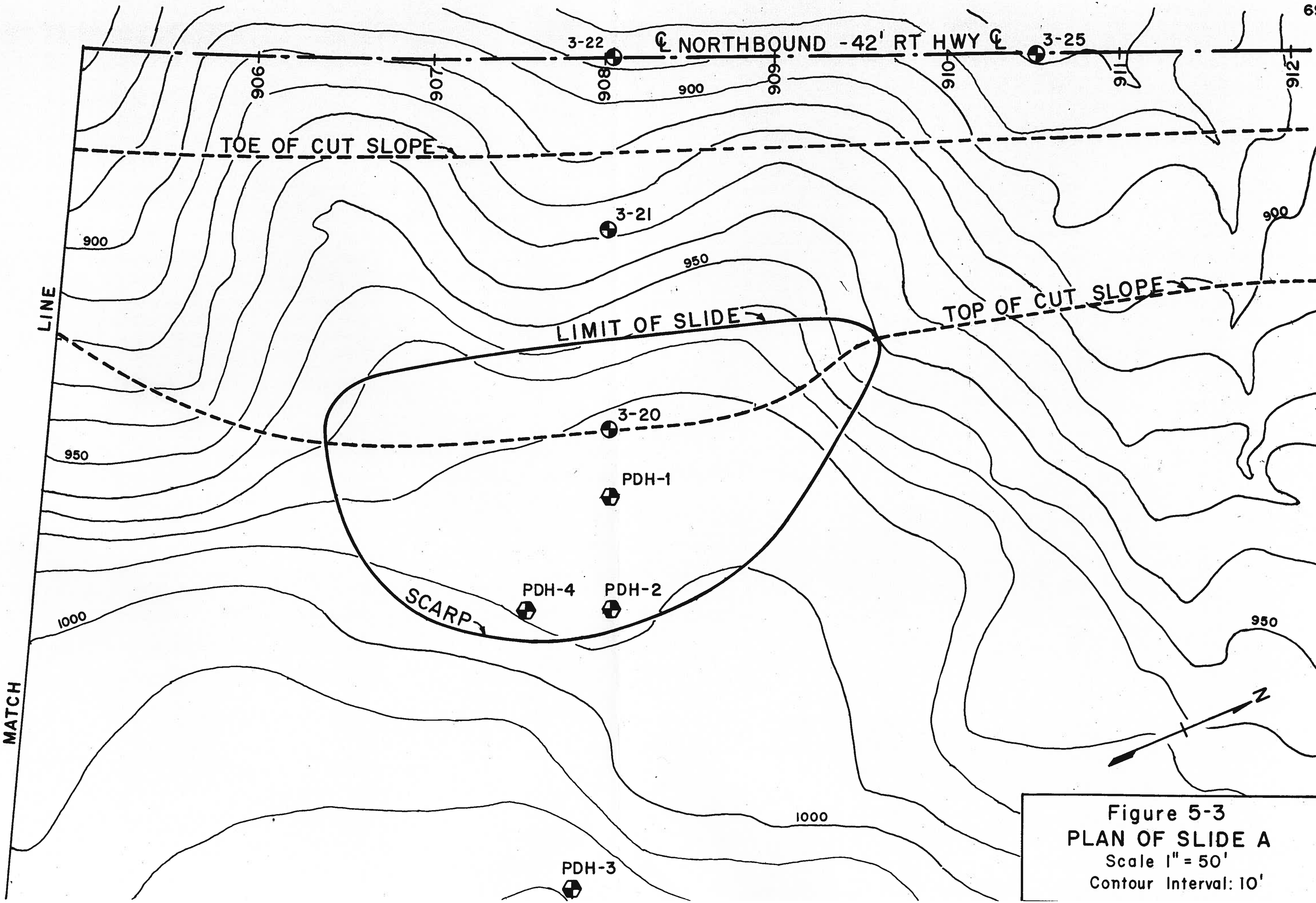

Figure 5-3
PLAN OF SLIDE A
Scale 1" = 50'
Contour Interval: 10'

Figure 5-4

Panoramic view looking north from Sta. 900 on I-279; Slide B in right foreground; Slide A in left center; Jan. 8, 1968

Figure 5-5

Panoramic view looking south from Sta. 915 on I-279; Slide A in left center; Slide B in right center; Feb. 28, 1969

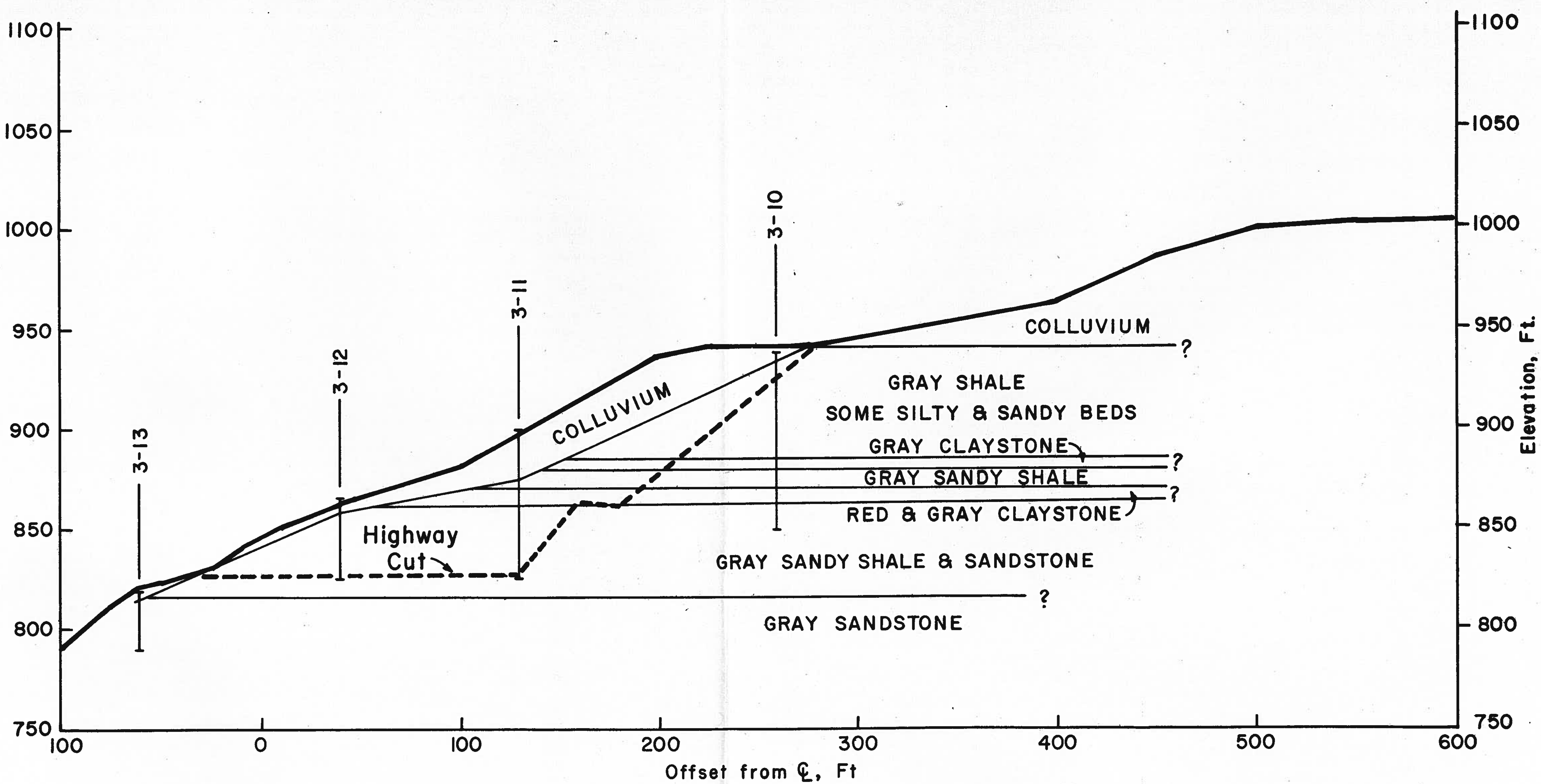

Figure 5-6
GEOLOGIC SECTION
Sta. 899+00

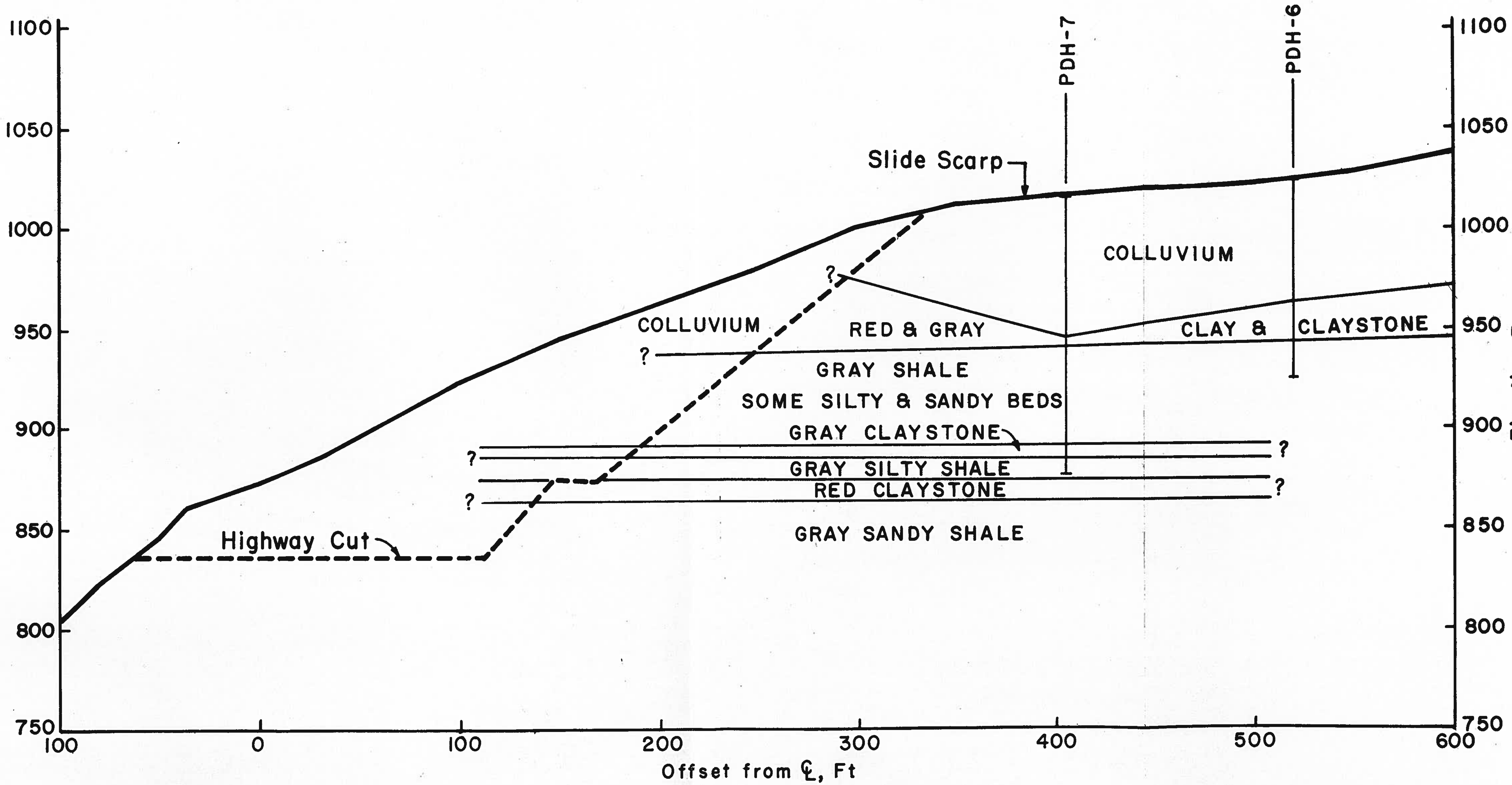

Figure 5-7
GEOLOGIC SECTION
Sta. 901+50

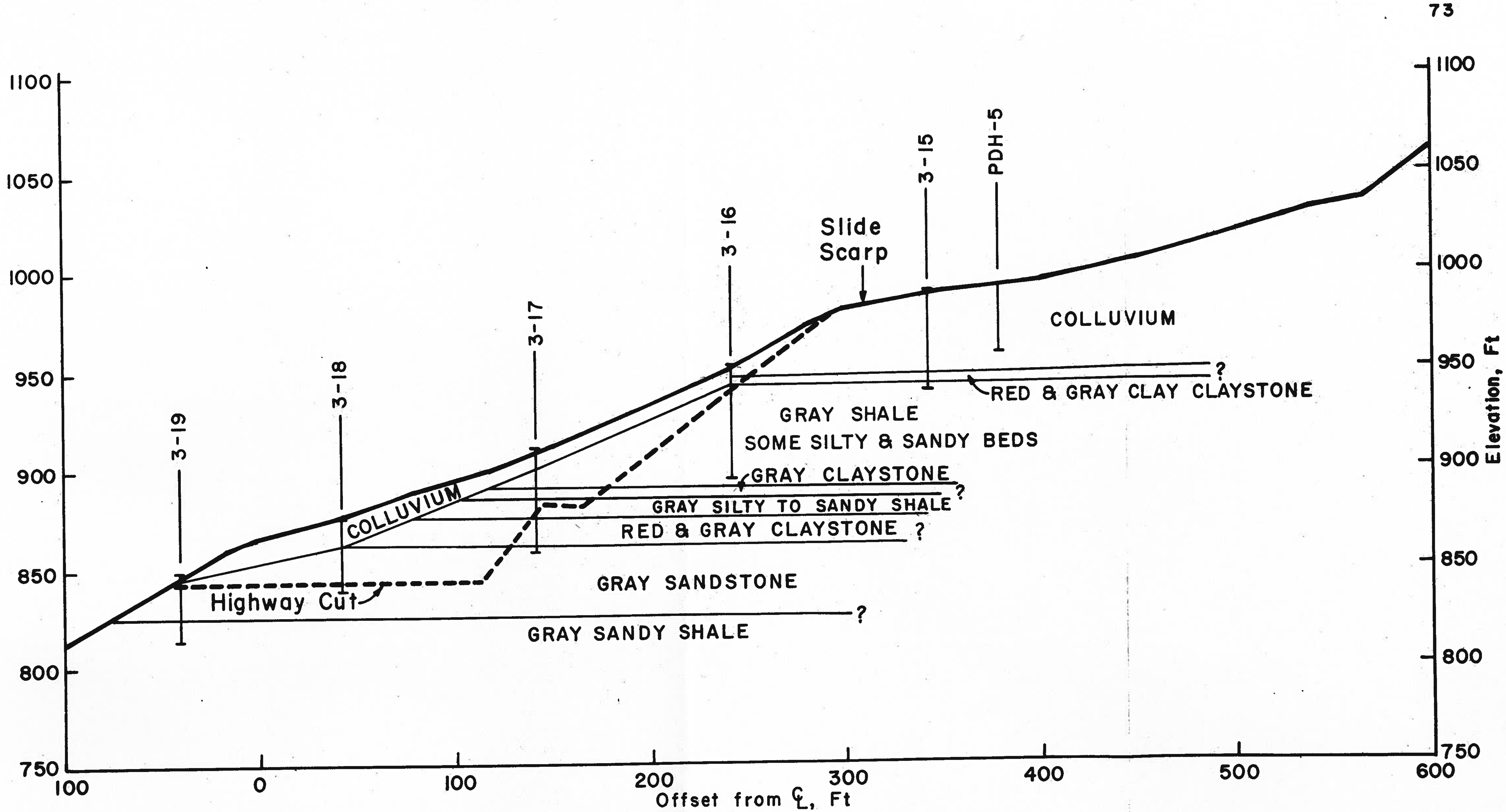

Figure 5-8
GEOLOGIC SECTION
Sta. 903+50

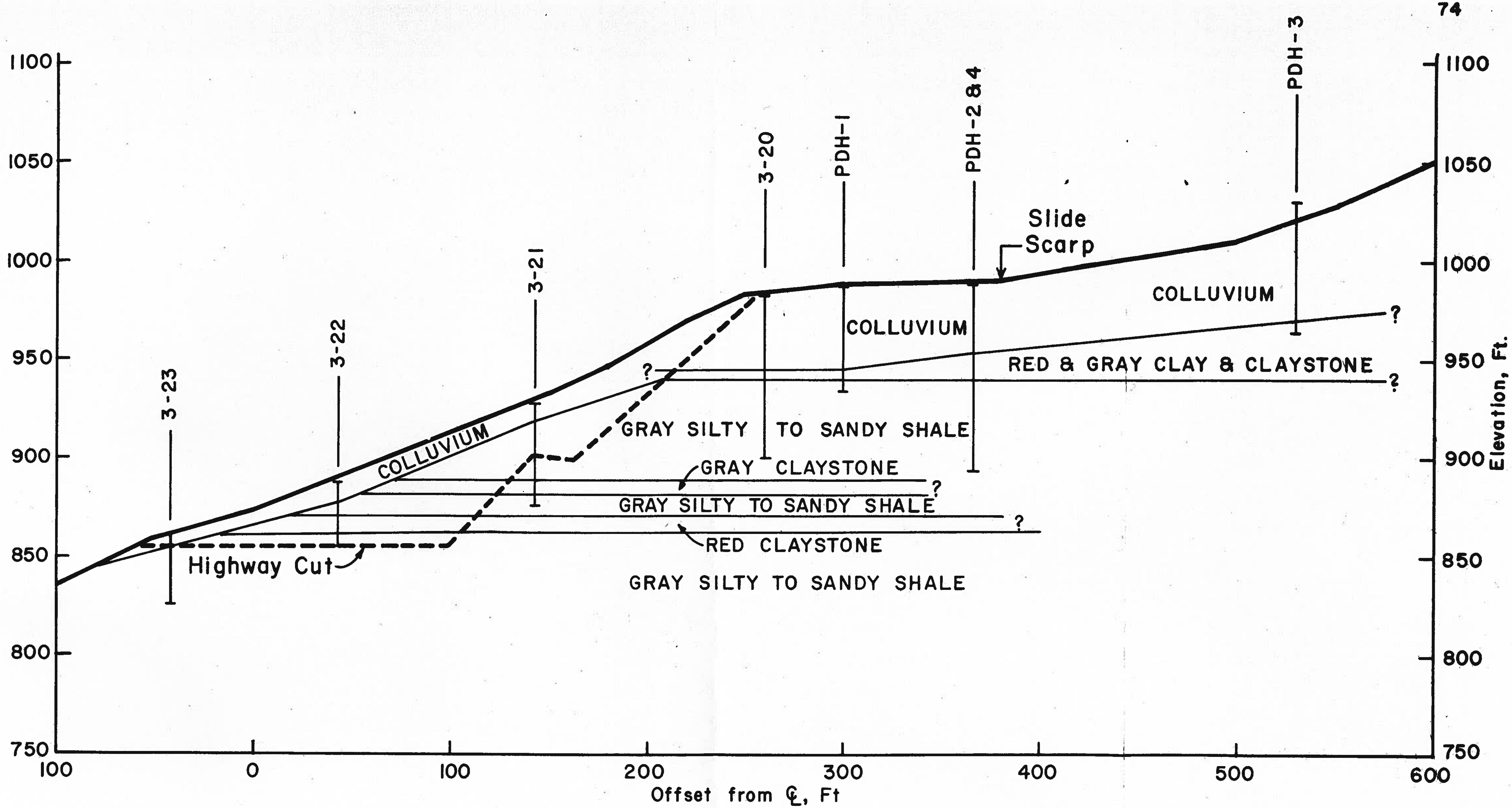

Figure 5-9
GEOLOGIC SECTION
Sta. 908+00

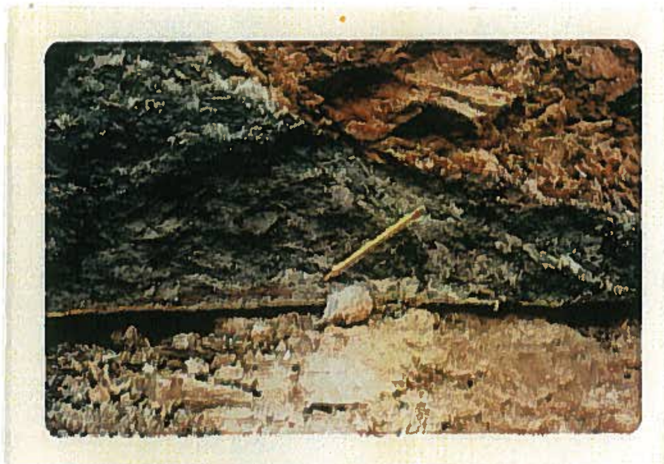

Figure 5-10

Closeup view of shear zone at Sta. 928 on I-279; May 20, 1969

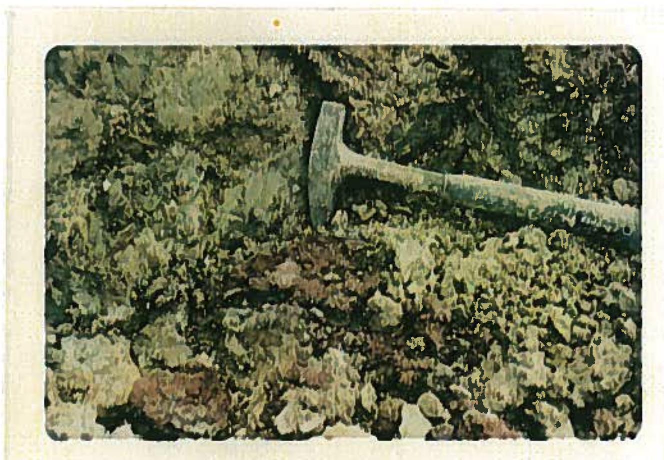

Figure 5-11

Closeup view of shear zone at Sta. 909 on I-279; May 13, 1969

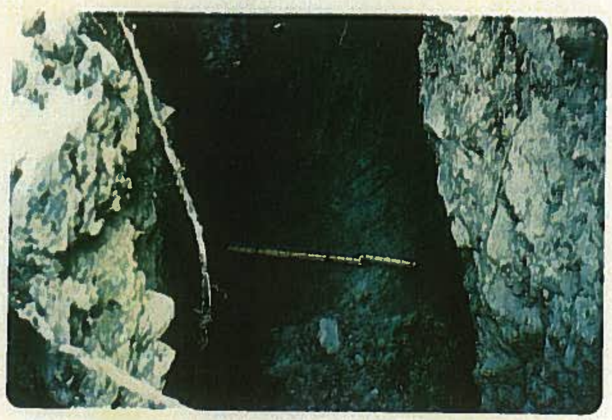

Figure 5-12

Slickensided red clayey colluvium at base of scarp; rear of Slide A; I-279, Sta. 908
Dec. 17, 1968

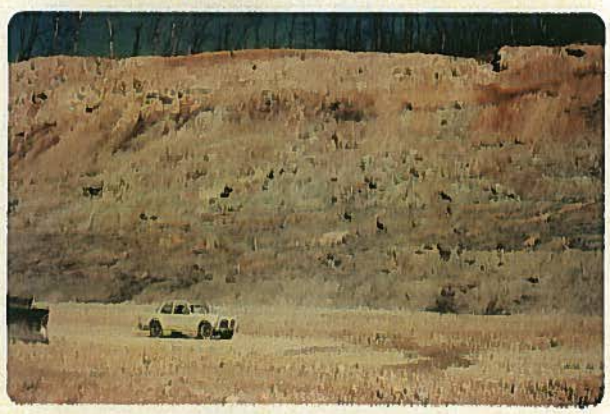

Figure 5-13

Face of cut slope; Slide A; I-279, Sta. 908
March 22, 1969

Figure 5-14

Face of cut slope; Slide B; I-279, Sta. 901
May 3, 1969

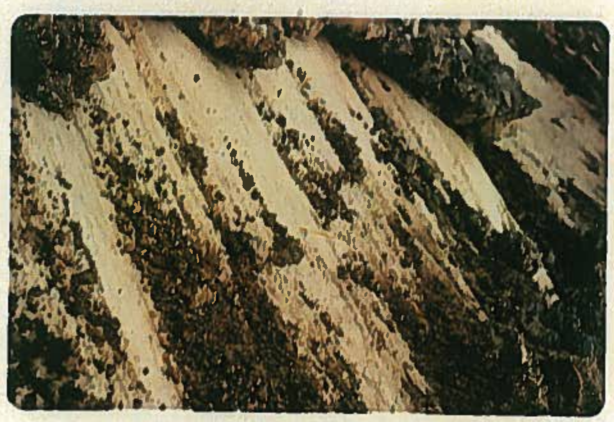

Figure 5-16

Closeup view of typical rear sliding surface;
slickensided gray clay; 50° average dip;
I-279, Sta. 932; May 20, 1969

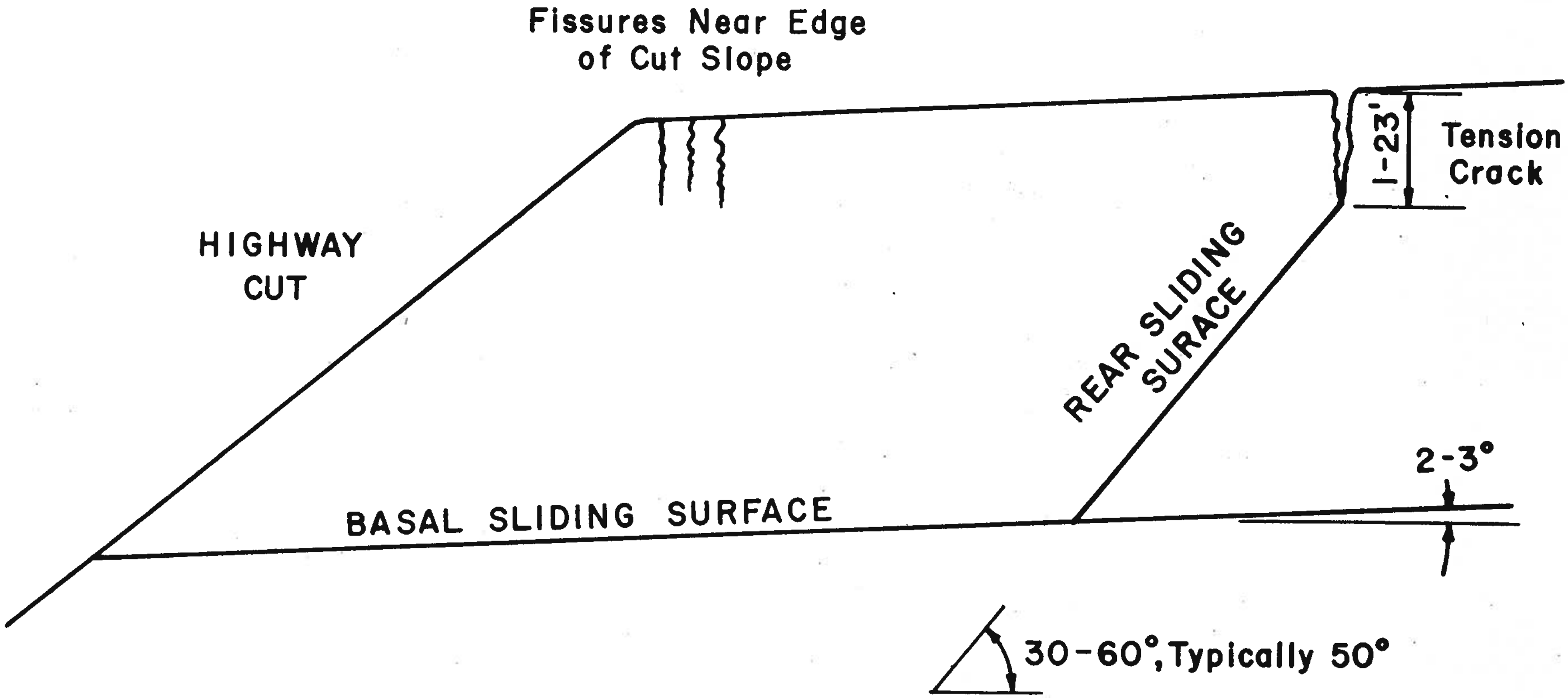

Figure 5-15
SCHEMATIC CROSS-SECTION
OF TYPICAL FAILURE MASS

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