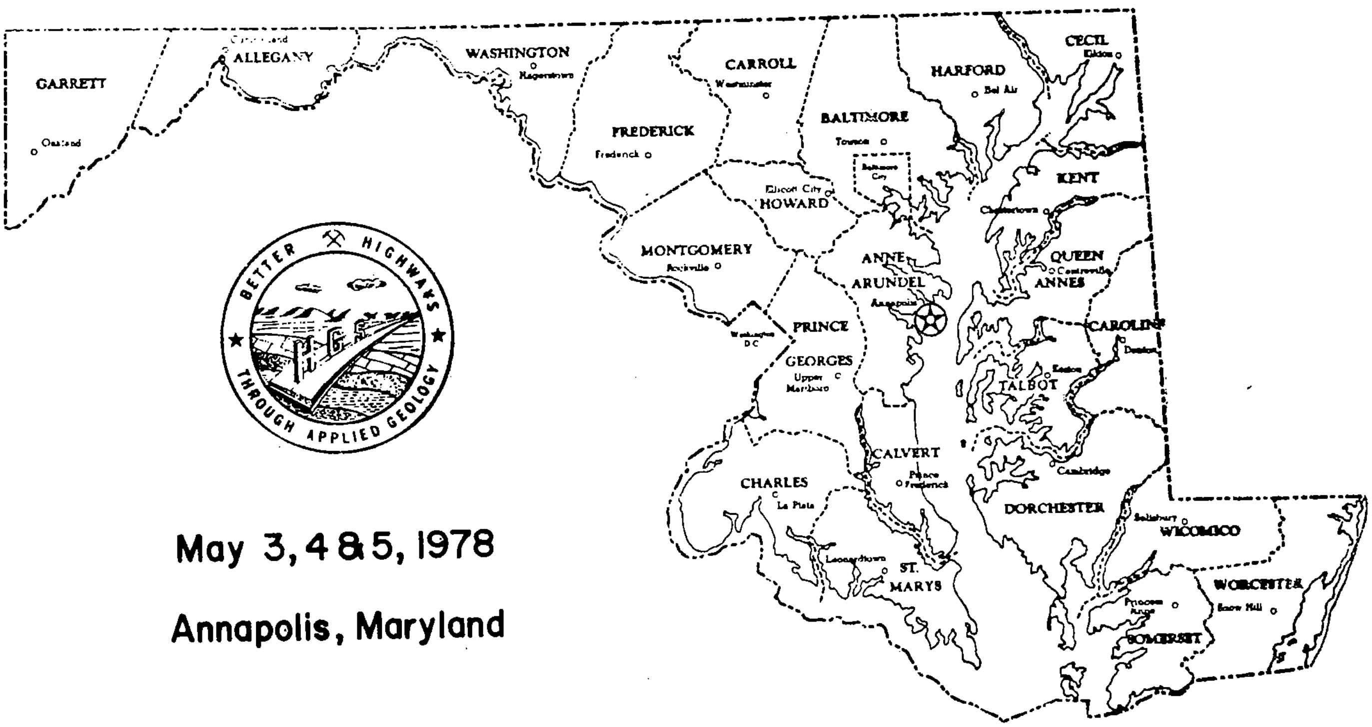
29th Annual

Highway Geology Symposium

PROCEEDINGS



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TWENTY-NINTH ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

Chairmen

Session	
I	Michael A. Ozol and Henry W. Janes
II	Kenneth N. Weaver and Michael A. Ozol
III	Henry W. Janes and Emery T. Cleaves

PREFACE

It has been my pleasure to serve as chairman of the Technical Program and Proceedings Committee for the 29th Annual Highway Geology Symposium and, in collaboration with Dr. Kenneth N. Weaver, to review the manuscripts for these Proceedings.

The theme of the Symposium was Urban Geology on the Fall Line. The papers presented here reflect the range of geological circumstances that affect the construction of highways. These include not only the traditional engineering geological perspectives such as slope and grade stability but also the characteristics and availability of the materials for their construction.

More and more the engineering problems of urban transportation and construction are coming to extend beyond the basic of design and construction. These problems now involve the consideration of environmental effects, availability and deliverability of aggregates and other materials, control of noise and vibration, monitoring of ground water and the avoidance of conditions leading to deterioration of ground-water supply and quality.

The common thread connecting these topics is geological science, whether in its classifical form or in the specialized areas of geotechnics, geophysics, environmental geoscience, hydrology, and other subdisciplines. This group of papers illustrates the range of subjects that now concern the professional engineering geologist involved in highway and related construction in and near large areas of the eastern United States.

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GEOLOGIC CONTRASTS ACROSS THE FALL ZONE

by

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The theme of the 29th Annual Highway Geology Symposium was "Urban Geology on the Fall Line". In the initial paper of the volume I will emphasize the "Geology" of the theme, and attempt to sketch a general picture of the geology that borders the Fall Line, or Fall Zone, as I prefer to call it. The Fall Zone (Figure 1) separates the unconsolidated to semi-consolidated sediments of the Coastal Plain Province, from the metamorphosed crystalline rocks of the Piedmont Province.

The Piedmont Province is characterized by rolling uplands dissected by narrow steep-wall valleys. The gorges cut by the Potomac and Susquehanna rivers are spectacular examples. The terrain is underlain by Precambrian and Paleozoic crystalline rocks. Gneiss domes, cored by Precambrian Baltimore Gneiss are overlain by the Setters Formation, and Cockeysville Marble. Mafic rocks like amphibolite and gabbro are present as well as serpentinites such as those at Soldiers Delight and Hunting Hill (Figure 1). The Hunting Hill Serpentinite has a certain notoriety, due to concern about asbestos and environmental health. Felsic schists, gneisses, and phyllites comprise most of the remaining terrain in the eastern Piedmont.

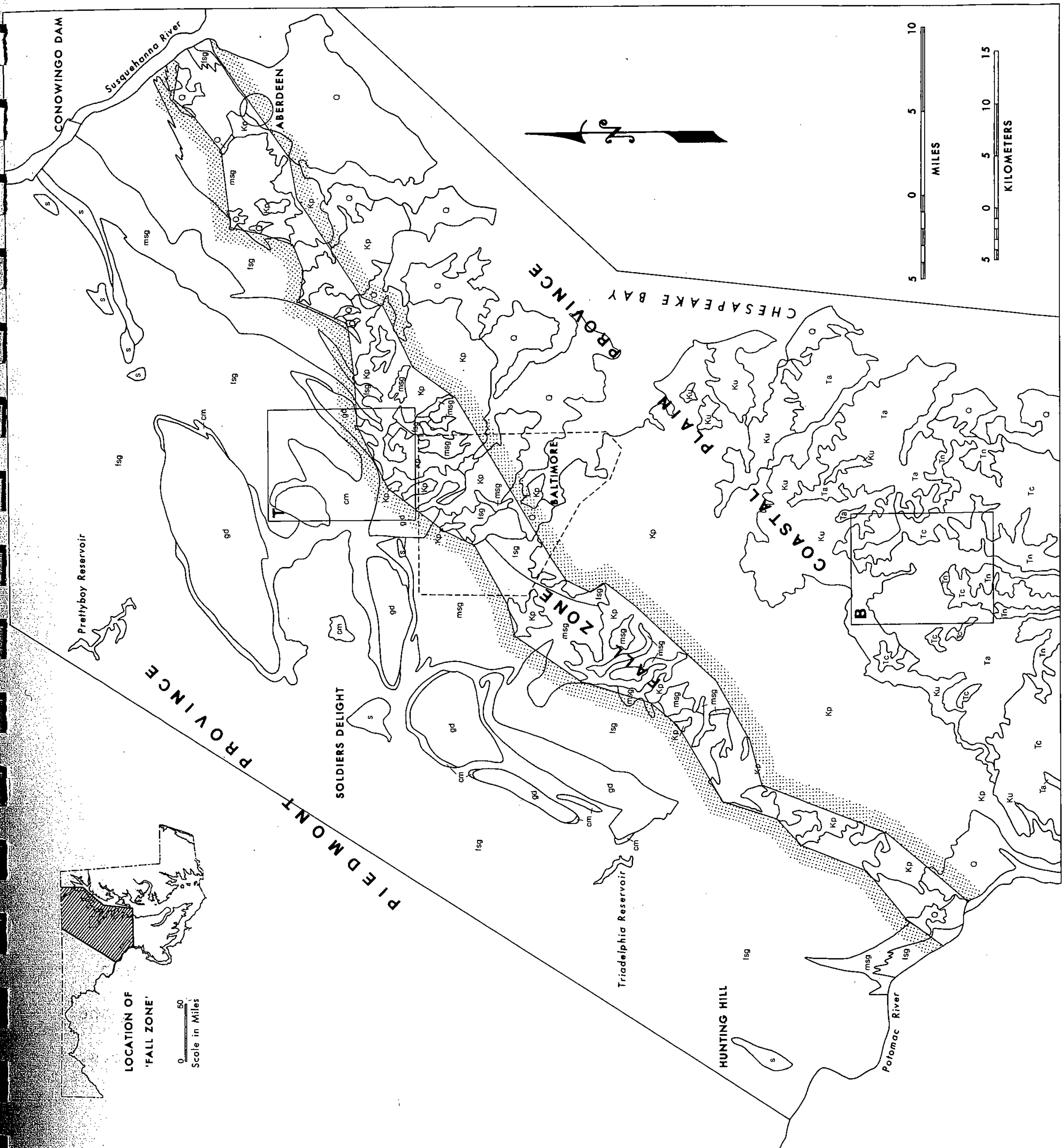
These hard crystalline metamorphosed rocks contrast strikingly with the unconsolidated to semi-consolidated gently dipping Coastal Plain formations composed of gravels, sands, silts, and clays. The Coastal Plain Province south and east of the Fall Zone is characterized by broadly rolling to dissected plains cut by tidal rivers and estuaries which extend back to the Fall Zone. Coastal Plain rocks include those of the Cretaceous Potomac Group which is a fluvial-deltaic complex of gravels, sands, silts, and

Figure 1. Geologic Map of the Fall Zone in Maryland

Geology generalized from "Geologic Map of Maryland" (Cleaves, and others, 1968).

Geologic symbols: Q, Quaternary; Tc, Calvert Formation; Tn, Nanjemoy Formation (includes Marlboro Clay); Ta, Aquia Formation; Ku, Monmouth, Matawan, and Magothy Formations; Kp, Potomac Group; fsg, felsic schists and gneisses; msg, mafic schists, gneisses, and amphibolites; s, serpentinite; cm, Cockeysville Marble; gd, gneiss domes including Setters Formation and Baltimore Gneiss.

Other symbols: T, Towson Quadrangle; B, Bowie Quadrangle.


CONOWINGO DAM
Susquehanna River
ABERDEEN
MILES
5 0 5 10
KILOMETERS
5 0 5 10 15
CHESAPEAKE BAY
COASTAL PLAIN PROVINCE
FALL ZONE
BALTIMORE
PIEDMONT PROVINCE
Prettyboy Reservoir
SOLDIERS DELIGHT
Triadelphia Reservoir
HUNTING HILL
Potomac River
LOCATION OF 'FALL ZONE'
0 50
Scale in Miles

clays. Southeast of the Potomac Group marginal marine and marine formations are exposed such as the sandy Magothy Formation, the glaucontic, sandy Aquia and Nanjemoy Formations, and the famous Miocene, highly fossiliferous silts and clays of the Calvert, Choptank, and St. Marys Formations.

The geologic contrasts carry through to the mineral resources. In the Coastal Plain, sand and gravel for the construction industry constitutes the major resource. Until recently clay was extensively exploited, and bog iron ores were very important in the 1850's. In the Piedmont, crushed stone, derived from marble, gabbro, serpetinite, and gneiss is presently the major product. Formerly, chrome, copper, feldspar, slate, and dimension stone were major resources.

Of all the resources, water is by far the most important. In the Piedmont, surface water reservoirs such as Prettyboy and Triadelphia (Figure 1) are required to provide the major urban areas with adequate water. Ground-water from wells in the crystalline rock provides the water supply for individual homes, small industries, and some smaller communities beyond the reach of the Baltimore and Washington distribution systems. In the Coastal Plain wells tap sedimentary aquifers and provide water for individual homes, industry, and cities such as Annapolis. The contrasting geology of the two Provinces underlies the difference in water-supply sources.

In addition, more detailed examples further illustrate contrasts between the Coastal Plain and Piedmont. Two recently published atlases of the Maryland Geological Survey provide examples — the Bowie Quadrangle (Glaser, and others, 1973) located beside the Patuxent River in the Coastal Plain, and the Towson Quadrangle (Cleaves, and others, 1974) located just north of Baltimore City, in the Piedmont Province (labelled B and T, respectively on Figure 1). Considering the Bowie Quad first, the Quadrangle Geologic Map (Figure 2a) displays a typical Coastal Plain situation, gently dipping, unconsolidated to semi-consolidated sediments consisting of gravel, sand, silt, or clay, with the exact lithologies varying in each geologic unit. The formations range in age from the Aquia Formation of Paleocene Age to the Marlboro Clay of Eocene Age (a thin plastic clay with silt partings) to Pleistocene Age terrace deposits along the Patuxent River, and alluvial deposits of Holocene Age along the rivers and streams.

Potential mineral resources are confined to the Pleistocene Age sand and gravel terrace deposits located along the Patuxent River and the Marlboro Clay (Figure 2b). The sand and gravel has been, and is, being extensively excavated and used by the local construction industry. Sand and gravel reserves are still available. However, their exploitation is influenced not only by the need for the material but also by regulations pertaining to sediment pollution, wetlands, and local zoning. In contrast, the clay deposits have not been exploited. Although suitable for face brick and structural tile, it seems unlikely that the clay will be developed due, in great part, to the value of the land for farming and housing developments.

In the Bowie Quadrangle, some geologic factors which affect general construction conditions are specific to coastal plain sedimentary terrain. One such factor is the occurrence of the Marlboro Clay (Unit 1, Figure 2c). The clay is essentially impermeable. During wet periods water movement along the top of the clay creates conditions favorable for slope failure and landsliding, both in the natural state and when modified by construction activities. On the other hand, flood plains (Unit 2, Figure 2c) are a geologic factor

Figure 2. Bowie Quadrangle Maps (adapted from Glaser, and others, 1973).

(a) Geologic:

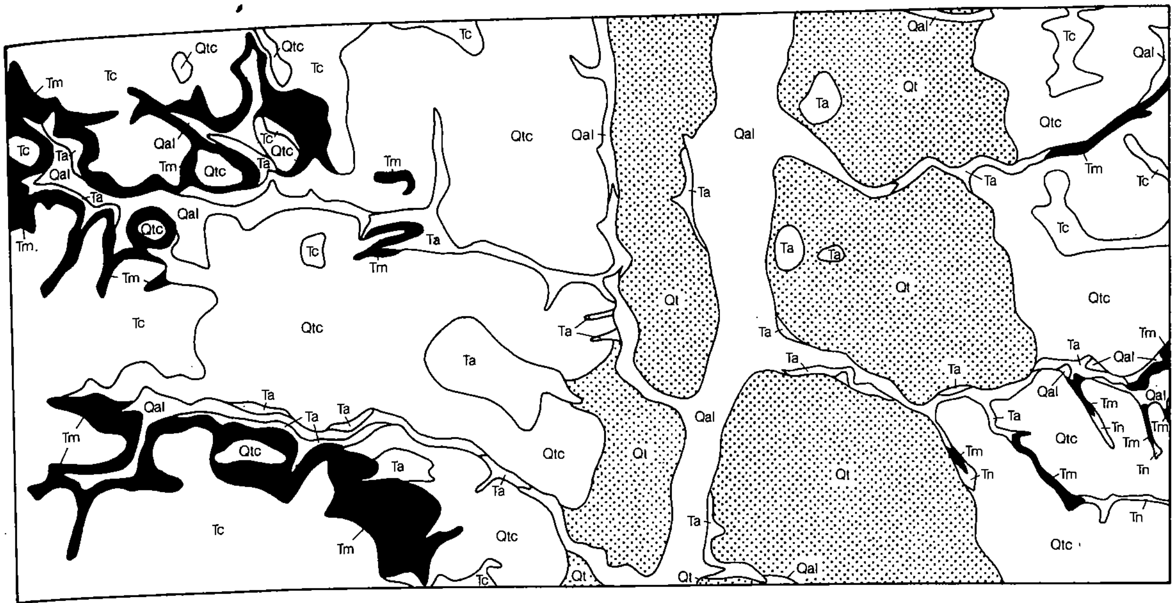
Qal: poorly sorted sand, silt, clay and gravel; Qtc: sand and silt, minor clay and gravel; Qt: sand, gravel, silt, and clay (stipple); Tc: Calvert Formation, fine-grained sand, silt, and diatomaceous silt; Tn: Nanjemoy Formation, poorly-sorted clayey glauconitic sand; Tm: Marlboro Clay, plastic clay with silt partings (black); Ta: Aquia Formation, well-sorted glauconitic sand

(b) Mineral Resources

clay (stipple); sand and gravel (open dot)

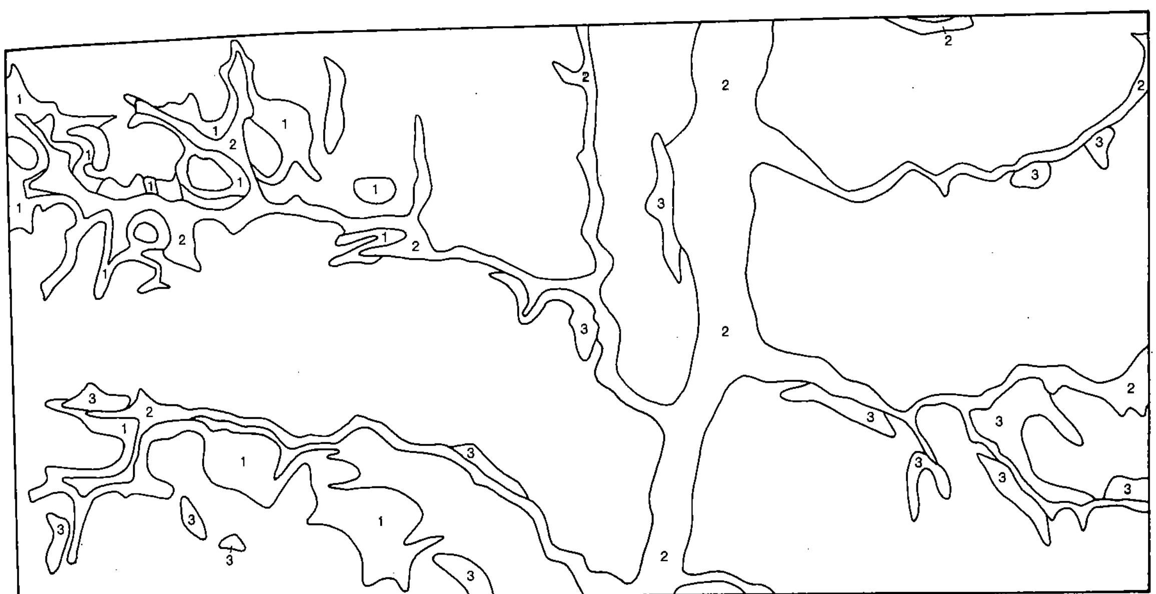
(c) Geologic Factors Affecting Land Modification

1, Marlboro Clay; 2, flood plain; 3, 15% slope



(a)

(b)



(c)

(a) (b)

Figure 3a. Towson Quadrangle Maps (adapted from Cleaves, and others, 1974).

(a) Geologic

Qal: Alluvium; Qac: Colluvium and alluvium; Kxs: Patuxent Formation, sand and gravel facies; pt: Pegmatite; gg: Gunpowder Gneiss; r: Raspeburg Amphibolite; o: Oella Formation; l: Loch Raven Schist; cm: Cockeysville Marble; s: Setters Formation; p€b: Baltimore Gneiss

(b) Mineral Resources

s-g: sand and gravel (open dot); p: pegmatite (black); m: marble (horizontal lining); q&g: quartzite and gneiss (stipple) which may occur in any geologic terrain in the humid east. The ground-water table is at or near the surface; swampy conditions are common — hence poor drainage; and part or all of the area is subject to flooding with coincident high velocity water movement and shifting substrate. The third unit (Figure 2c) represents slopes greater than 15%. This limit is an artificial, man imposed constraint, related to building codes, but is also related to the physiography of the area.

Having considered the Coastal Plain, let me turn to a Piedmont environment, illustrated by the Towson Quadrangle. The Quadrangle straddles three physiographic regions in the Piedmont: the Fall Zone, the Phoenix Domes, and the Harford Plateaus and Gorges. These physiographic regions are new designations, and will be formerly described in the literature at a later date. Coastal Plain sediments feather out and Precambrian and Paleozoic crystalline rocks are exposed at the surface (Figure 3a). This Quadrangle has a particularly varied suite of rocks including unconsolidated sand and gravel of the Patuxent Formation, quartzite and gneiss of the Setters Formation and marble of the Cockeysville Marble.

Mineral resource potential in the Towson Quadrangle is much more varied than that in the Bowie Quadrangle and includes sand and gravel, marble, pegmatite, gneiss, and quartzite (Figure 3b). This greater potential results from the location of the Quadrangle astride the physiographic transition between the Fall Zone Region and the Phoenix Domes Region in the Piedmont Province. However, mineral resource potential is not nearly as varied in most quadrangles in other physiographic regions of the Piedmont Province.

The sand and gravel resources occur in the unconsolidated sediments of the Patuxent Formation. The sediments were used for fill, aggregate, and construction sand as late as 1974, but the supply is now exhausted or pre-empted by urban development.

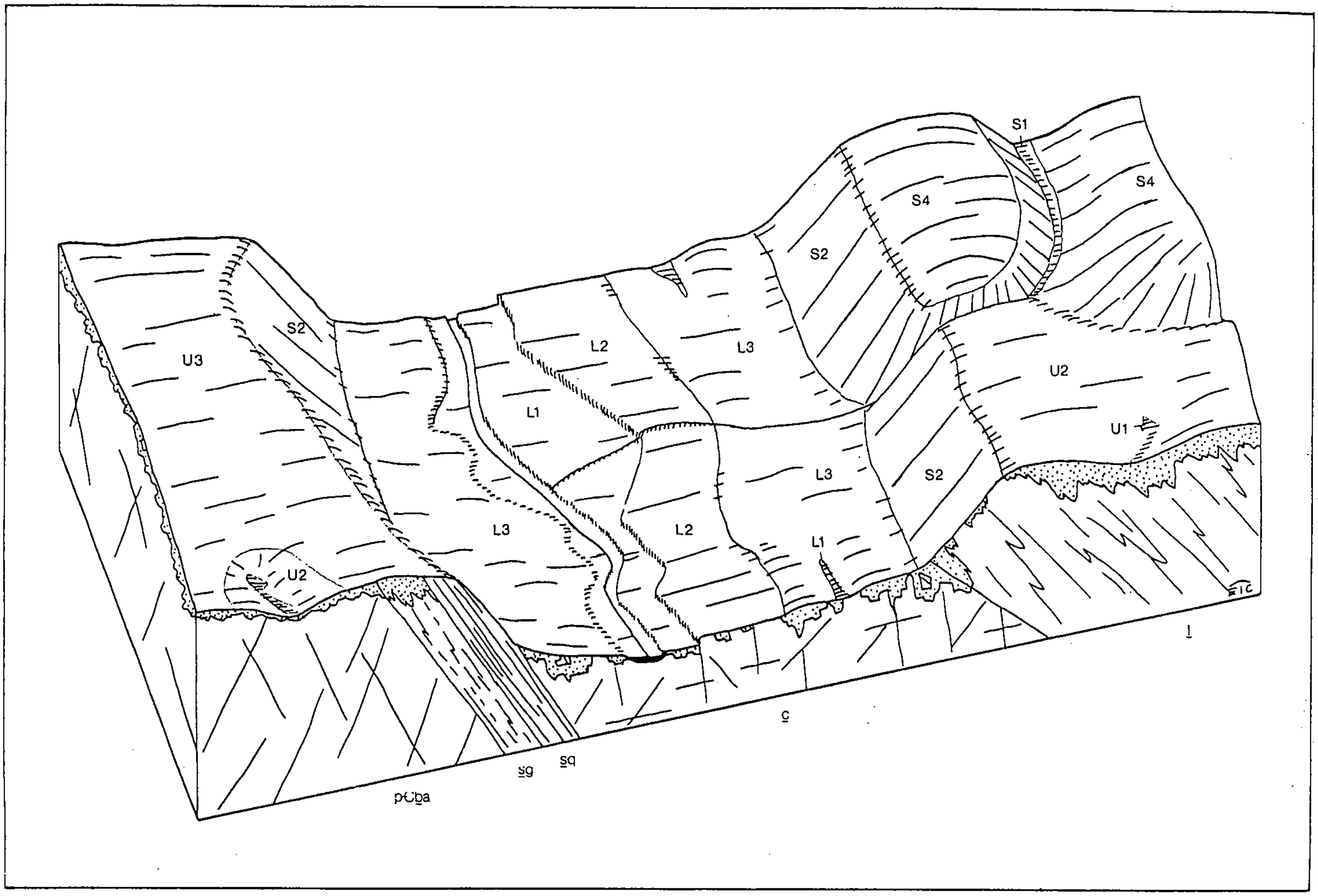


Figure 4. Block diagram illustrating lithology, saprolite, landform relationships

Landform units: L1, L2, L3, S1, S2, S3, S4, U1, U2, U3

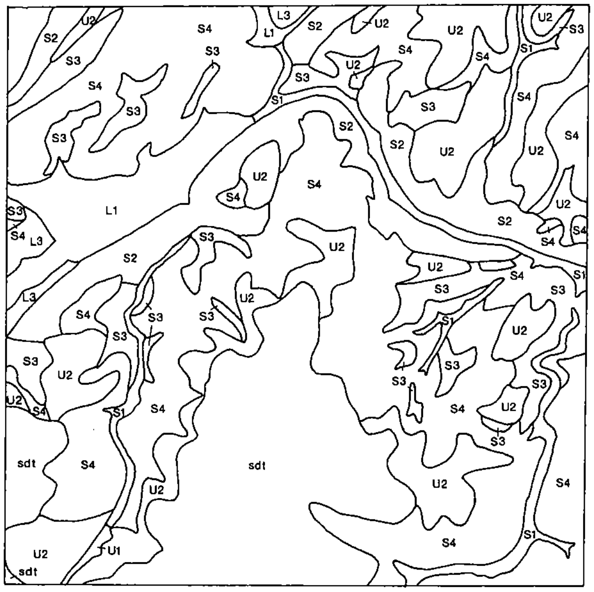
Geologic units: p€ba (Baltimore Gneiss, augen gneiss member); sq (Setters Formation, quartzite); sg (Setters Formation, gneiss); c (Cockeysville Formation); l (Loch Raven Schist); saprolite shown by stippling; alluvium colored black.

The crystalline rock mineral resources are not presently being quarried. These include gneiss and quartzite of the Setters Formation, pegmatite, and the Cockeysville Marble. These potential resources will probably remain undeveloped due to encroachment of the Baltimore suburbs and the presence of Loch Raven Reservoir, a major part of the Baltimore water supply system.

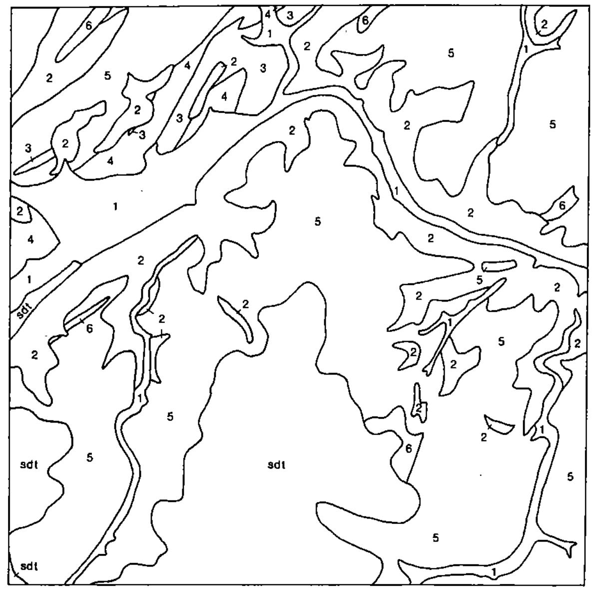
Marble for lime and crushed stone, and gneiss for crushed stone are mineral resources confined to the Piedmont and Fall Zone. They reflect the crystalline rock geology in the area. Sand and gravel are confined to the sediments of the Coastal Plain and Fall Zone. They reflect the geology of the sediments. The two resources are juxtaposed only in the Fall Zone.

Geologic factors affecting land modification in the Towson Quadrangle are keyed to overburden thickness in the crystalline rock terrain, and to lithofacies in terrain underlain by sediments. The situation in sedimentary terrain is discussed above (Bowie Quadrangle) so now I will consider the situation in crystalline terrain.

Overburden thickness was selected as the key geologic factor affecting land modification, and is directly related to thickness of saprolite which mantles the Piedmont crystalline rocks (Figure 4). Weathering of the marble produces a residuum of extremely variable thickness, in which pinnacles of fresh rock and residual boulders commonly occur. Slopes steeper than 12^{o} (Landform units S2, S3) are underlain by a thin saprolite usually less than 5 feet, regardless of rock type. Upland areas (U1, U2, U3) are underlain by saprolite, the thickness of which varies with lithology. On the Loch Raven Schist (l) saprolite thickness commonly exceeds 20 feet; on Baltimore Gneiss (pfba) the saprolite is generally 5 to 20 feet thick, and has numerous residual boulders. On the gneiss member of the Setters Formation (sg), 20 feet



(b)



(c)

Figure 5. Towson Quadrangle: Geologic Factors Affecting Land Modification (adapted from Cleaves, and others, 1974).

(a) Geologic Map. See Figure 3a.

(b) Landform Map. Compare with block diagram (Figure 4) for visual impression of relative slope and topographic position.

(c) Geologic Factors Map. (1) High water table, flooding; (2) overburden 0-5 feet, slopes exceed 12°; (3) overburden 0-5 feet, slopes less than 12°; (4) variable overburden thickness; rock pinnacles and residual boulders; (5) overburden 5-20 feet; (6) overburden exceeds 20 feet; (7) sdt, terrain underlain by sedimentary deposits.

or more of saprolite is common, but on the quartzite member (sq) saprolite is thin or absent.

Using these lithology, landform, overburden thickness relationships, it is possible to construct a "geologic factors" map on which thickness of overburden is the primary factor. Part of the Towson Quadrangle illustrates such an application (Figure 5). For example, wherever steep slopes occur (Figure 5b, unit S2, S3), overburden thickness is estimated to be 5 feet or less (Figure 5c, unit 2) regardless of rock type. Wherever pegmatite occurs (Figure 5a, pt), regardless of landform, overburden thickness is 5 feet or less (Figure 5c, unit 3). Wherever the Loch Raven Schist occurs in combination with landform unit U2, overburden thickness exceeds 20 feet. Compare Figures 5a, 5b, 5c. From a construction point of view the map provides, among other things, a general overview for estimating the presence of rock at or near the surface.

The map atlases considered above are essentially map exercises, an attempt to predict in general terms some of the geologic factors which may constrain man's modification of the landscape. Let me briefly review what actually may occur during a construction project. The project I am most familiar with is the Susquehanna Aqueduct, which carries Susquehanna River water from Conowingo Dam to Aberdeen and to Baltimore (Figure 1). My comments relate to a cut and cover trench between Aberdeen and Baltimore. This section lies entirely within the Fall Zone. The water pipeline is 108 inches in diameter, and trench depth varied from a minimum of 14 feet to 35 feet, and width averaged 14 feet.

Excavation rates in saprolite varied with the degree of decomposition of the parent rock. Subaerial, chemical weathering of the rock resulted in bouldery saprolite, or alternating layers of hard rock and saprolite, or in

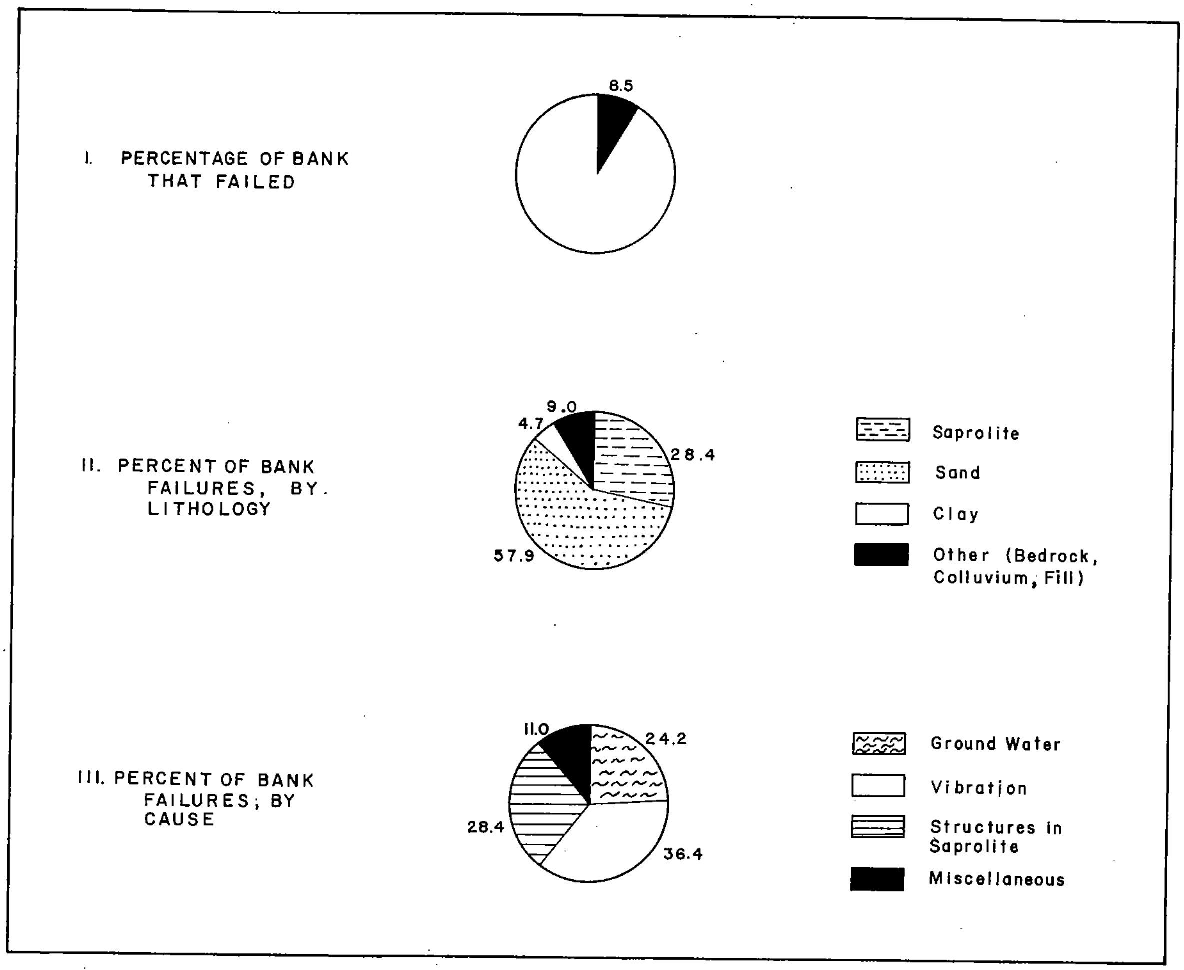


Figure 6. Bank Failures in Sediment and Saprolite (Cleaves, 1978, Figure 16)

Table 1. Excavation rates in saprolite (Cleaves, 1968, p. 17)

Quartz monzonite saprolite	270 yd^3/hr
Amphibolite saprolite, some residual boulders	230 yd^3/hr
Bouldery alluvium over amphibolite saprolite	200 yd^3/hr
Bouldery amphibolite saprolite	140 yd^3/hr

rock completely altered to saprolite. Quartz monzonite, which decomposed completely to a silty sand saprolite was excavated at 270 yd^3/hr (Table 1). Bouldery saprolite developed on the Aberdeen Metagabbro varied considerably in excavation rate depending upon the size and number of boulders, from 140 to 230 yd^3/hr. On occasion, rock ledges or very large boulders stopped excavation completely until the offending rock was drilled and blasted.

Saprolite ranked second after sand as unstable wall material (Figure 6). Bank failures in saprolite were related to three factors: preservation of primary rock structures such as joints, the presence of ground water, and vibration from construction equipment.

Excavation and wall stability of the sediments were related to engineering, hydrologic, and lithologic factors. Depth of excavation is an engineering factor, for example. One section of trench which averaged 24 feet deep was excavated in beds of clay, sand, and silty sand at a rate of 85 yd^3/hr. The same combination in a trench averaging 15 feet in depth was removed at a rate of 130 yd^3/hr. In general, however, excavation rates were mainly influenced by lithology. Clay was the most difficult, with rates varying from 120 yd^3/hr in a dense, tough clay to 230 yd^3/hr in a silty, sandy, clay; gravel was removed at rates from 150 to 210 yd^3/hr; sand excavation varied from 240 in clayey sand to 360 yd^3/hr in a gravelly sand.

Trench walls in which sand was exposed resulted in the most treacherous wall stability conditions encountered during construction. Readjustment of sand to its angle of repose plus contributing factors of ground-water saturation and/or vibration from construction equipment resulted in more failures than in any other sediment, saprolite, or rock. Sluffing and cave-ins in water-saturated sands were common. Cave-ins resulted from ground-water sapping at the base of the trench. Particularly dangerous situations developed where clayey sediments overlaid water-saturated sand and collapsed into the trench due to sapping of the sand. Equally dangerous, and perhaps more frustrating, were failures in dry sand triggered by vibration from the construction equipment. In some instances, the trench was 4 to 6 times greater in width than in depth.

Much more could be related about the Aqueduct, about the map atlases, and about the very significant differences between the crystalline rock-saprolite Piedmont Province and the sedimentary Coastal Plain Province. I hope this brief overview provides a geologic setting in which to place the subsequent papers of this Symposium.

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LANDSAT OVERVIEW OF FALL LINE GEOLOGY

ABSTRACT: The Fall Line is defined as a zone that separates the soft sedimentary material of the Coastal Plain from the harder rocks of the upland Piedmont Province. Consequently, construction within or along this zone is often influenced by various aspects of this diverse geology and the resultant geomorphology.

A review of the existing Landsat imagery along the Fall Line, in conjunction with supporting data from ground observations and aerial photography and imagery, show that the resolution of the Landsat imagery is too coarse for detailed geological engineering applications. And, the nature of the Fall Line is such, that even at larger scales and higher resolution this boundary may not be well mapped from the air or from space platforms. The Landsat imagery, however, is adequate for the delineation of the

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regional structural geological trends, lineaments, structural features, geomorphic forms and the changing landuse patterns. For example, within the Fall Line zone the vegetation boundaries, surface water, and sometimes even the scarp of the Fall Line can be inferred directly from the Landsat imagery.

INTRODUCTION:

Traditionally the Fall Line has been accepted as a zone separating young Cretaceous soft rocks and deposits of the Coastal Plain from harder Precambrian igneous and metamorphic rocks of the ancient Piedmont Province. (Darton, 1950; Johnston, 1964; USGS, 1967; and many others).

The morphology of this region is largely controlled by the properties of the underlying rocks. However, there is a scarcity of exposed bedrock except in stream, river, or road cuts. What one finds in most places, for example, is that the Coastal Plain/Piedmont boundary is overlain by clays and sands, resulting, in part, from the decomposition of the bedrock. Therefore, the topographic expression of a fall-line-scarp is, for the most part, worn down by erosion, or masked by weathered materials interleaved with a cover of Cretaceous marine sediments and younger terrigenous materials. Consequently, because of the distinct difference between the Precambrian and Cretaceous lithologies, the mapping of the surface gegology in this zone is of practical importance in connection with the siting of major construction projects such as dams, power plants, railroads and highways.

To date, numerous papers and studies document the utility of satellite remote sensing and specifically the use of Landsat imagery, for mapping. For example, before Landsat, Nimbus satellite imagery was used for geological purposes, and in particular, the delineation and mapping of the Fall Line boundary. However, since the launching of Landsat-1 in 1972, an identical Landsat-2 in 1975, and a somewhat modified Landsat-3 in March 1978, satellite images of the Fall Line zone can be routinely collected and available. And, it can be shown that Landsat imagery can provide a rapid and an economical means of observing the effects of many short duration phenomenon such as flooding and other dynamic events. In addition, lineaments, not visible on aerial imagery or even on the ground, can be mapped and analyzed on satellite imagery for engineering purposes. (Rabchevsky, 1970 and 1977; Eichen and Pascussi, 1975).

<u>BACKGROUND</u>:

Civil engineering, geological, and hydrological projects are becoming increasingly complex in their design and environmental and safety requirements. This situation, together with the ever shortening delivery schedules demanded by the client, compel the engineer to utilize the most efficient engineering technology available to produce his design/report on schedule. However, before any engineering design or exploration work begins, an accurate topographic map of the area must be available to the design staff.

Today, most, if not all, topographic maps are produced by aerial photogrammetry. Aerial mapping has a number of advantages over the conventional ground surveys, such as speed, cost, and accuracy. Using precision vertical photography and modern stereo compilation equipment, topographic maps up to scales of 1"=25', with 1 foot contours can be produced. Because the survey is prepared from aerial photography, large areas can be accurately surveyed without actually having access to the property. And, if the aerial mission is well planned, the aircraft may be used to collect remote sensing, and even multispectral and seasonal data on the same flight as the mapping mission. Thus, by preplanning the aerial survey sortie, and the utilization of the collected information, the engineer may gather much data before any money is spent on field studies and subsurface exploration.

A thorough approach to site evaluation with remote sensing techniques would involve a sequential study of small-scale (satellite, or high-altitude aircraft) and large-scale (medium or low altitude aircraft) imagery. However, the final site selection should certainly include a planned program of field observations, sampling and laboratory testing and analysis. The remote sensor data, thus provides an ideal base from which a field investigation can be planned. This results in efficient sampling, with the number of samples proportioned to the complexity of the situation, and field data which are complemented, but not duplicated, by remote sensor data.

The use of remote sensor data may provide information, relative to:

1. The type of bedrock, and its general structural relationship;
2. The presence of rocks or sediments that may introduce leakage, sinking and foundation problems;
3. The location of major faults and their bearing on earthquake problems;
4. The depth of weathering and its relationship to topographic features and to rock types;
5. The presence of possible sources of construction materials such as sand and gravel, or other fill; and
6. The general surface and ground water conditions in the various terrains.

The characteristics of Landsat-1, 2, and 3, and the data and imagery produced by their sensors are similar and by now well known (NASA, 1976). Table 1 compares the sensor characteristics of the three Landsat systems.

	SPECTRAL RANGE		AREA COVERAGE		SPATIAL RESOLUTION		TEMPORAL RESOLUTION
	MSS	RBV	MSS	RBV	MSS	RBV	MSS and RBV
LANDSAT—1 1972 to 1978	500-600 nm 600-700 nm 700-800 nm 800-1100 nm	475-575 580-680 690-830	185 km^2	185 km^2	80 m	80 m	18 days
LANDSAT—2 1975 to date	500-600 nm 600 700 nm 700-800 nm 800-1100 nm	475-575 nm 580-680 nm 690-830 nm	185 km^2	185 km^2	80 m	80 m	18 days
LANDSAT—3 1978 to date	500-600 nm 600-700 nm 700-800 nm 800-1100 nm 10.4-12.6 μm	505-750 nm	185 km^2	93 km^2	80 and 237 m	40 m	18 days

Table 1 Sensor characteristics of Landsat-1, Landsat-2, and Landsat-3, Multispectral Scanner System (MSS), and Return Beam Vidicon (RBV) camera systems.

The Landsat series, with their uniform repetitive coverage, have demonstrated capabilities for producing distortionless, generally continuous images of the Earth's surface on a defined map projection. Landsat imagery is even helping to correct and update certain features on existing U.S. maps at, or near, the national map accuracy standards (Colvocoresses, 1977). It is also possible to make photomap overprints to fit on the existing data of conventional maps. This fresh information on the overprints has made it possible to observe such engineering developments as urban sprawl and modifications to transportation networks. Further, a new type of small scale map has been made possible by the unique capacity of Landsat's Band 7 to delineate directly, with high reliability, water bodies as small as 200 m (656 ft.) in diameter. It can identify streams 20 to 50 m wide (65 to 165 ft.), if they are not overhung by trees. For land use purposes, the Landsat data has become a useful, and many times an indispensable complement, to field studies and aerial photographic inventories. For regional planning, a satellite scene can serve as a reconnaissance base on which planners can pinpoint areas of stress or of rapid change for which more detailed information is required. Finally sophistication or a historic perspective can be added by using data acquired during different times of the year - to take advantage of details revealed by seasonal differences in vegetation or soil moisture. Information on static features, however, can be obtained usually from a single cloud free pass over the area without the necessity for repetitive coverage.

This presentation focuses on a region of the Fall Line between Washington, D.C. and Baltimore, Maryland (Figure 1). In this region, the topographic expression of the Fall Line scarp is either worn down by erosion or is masked by a cover of Cretaceous marine sediments and younger terrigenous materials. (Brake, et al., 1977; Froelich and Heironimus, 1977; Reinhardt and Cleaves, 1978).

DATA ANALYSIS AND DISCUSSION:

The Landsat imagery used in this study was originally ordered from the EROS Data Center in a 70 mm format, black and white positive and negative transparencies (chips), at a scale of 1:3,369,000. Consequently, black and white prints and false color composites were produced at varous scales and seasonal coverage. Specifically, winter/fall and summer/spring coverage was used to aid the interpretation of this area. The advantages in using multi-seasonal and multi-scalar coverage have been well known (Rabchevsky, 1978 and many others).

Briefly summarized they are as follows:

1. The different sun angle elevations during the summer and winter seasons cause different

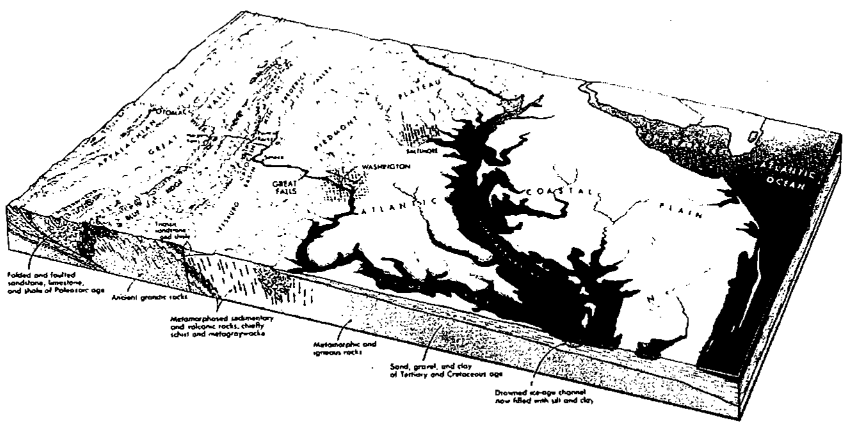

Figure 1. Block diagram of the study area showing the main geographical, geomorphological and geological features. The Fall Line zone extends approximately from Baltimore to Washington, D.C., separating the Coastal Plain from the Piedmont Plateau (U.S. National Park Service, 1970).

shadow and reflectance effects, so that when the sun is low the relief shadows are long, enhancing the topography; but when the sun is high during the summer, the reflectance characteristics (spectral signatures) of the surface cover are enhanced, instead of the topography.

2. The different position/azimuth of the Earth relative to the sun at different times provides new "look directions" at the same area, the sun's illumination moving clockwise from summer to winter.
3. Seasonal coverage provides a "stereoscopic" effect, partly due to points 1 and 2 above, if one winter and summer scene are viewed at the same time, and partly due to parallax in the imagery caused by the changing position and look direction of the satallite sensors.
4. Lack of vegetation in winter, and especially dusting of snow in this area, enhance lineaments and structural features that may otherwise be masked by the summer vegetation.

Figures 2 and 3 show the study area as it appears in the near infrared in the summer and fall. Note the higher reflectance of the summer scene caused by near infrared sensitivity to vegetation.

Figure 4 is a mosaic of the region. We found that even after examination of the distribution of water bodies and vegetation cover on images such as Figures 2 and 3, and of the mosaic, it was difficult to locate the Fall Line with confidence. However, Figure 5 shows the location of the Fall Line as interpreted with the aid of overlays such as an engineering map of coastal plain sediments and surficial deposits (Figure 6).

The direct identification of various lithologic or soil units, and of the sedimentary surface covers on either side of the Fall Line was not possible because of dense vegetation, the proliferation of man made features and the interfingering of the saprolite with more recent sediments.

A winter scene (Figure 7) was used for detailed interpretation of drainage patterns (Figure 8). Note that the NE-SW lineation pattern connecting the upper reaches of the Potomac River and the Chesapeake Bay also connects with the headwaters of the Delaware Bay (see Figure 5). Thus far only this pattern appears usable as an indicator of the Fall Line. Currently, we are investigating in more detail the erosional features of various streams and rivers as they flow from the hard rocks of the upland Piedmont Province onto the soft sediments of the Coastal Plain. It is anticipated that this approach will provide additional indications of the separation and location of the Fall Line lithologies.

Land use was also mapped using imagery, as in Figure 7. It was not significantly different on either side of the Fall Line (Figure 9), and consequently, land use does not appear to be an indicator of Fall Line geology either.

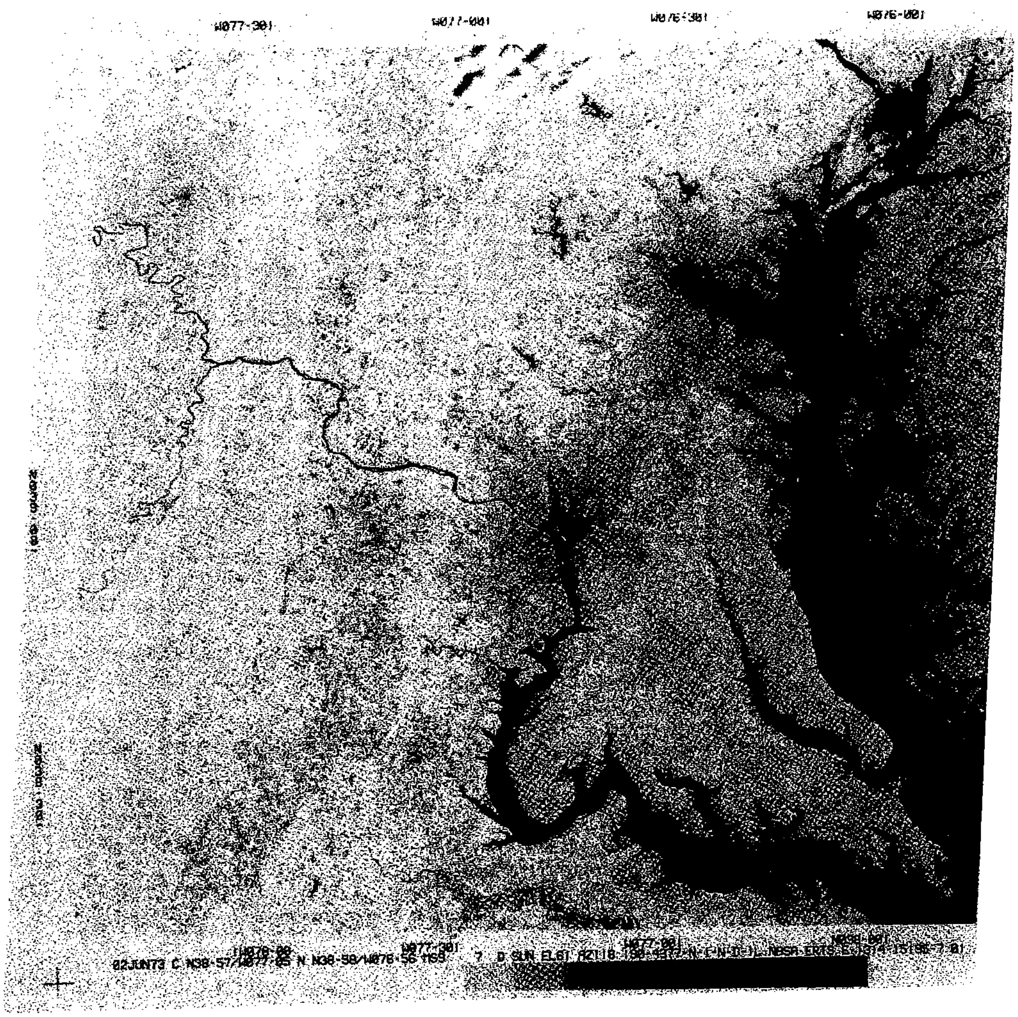

Figure 2. Black and white print of a Landsat-1 image of the study area during the summer (Landsat-1, 2 June 73, Band 7, ID. No. 1314-15195-7).

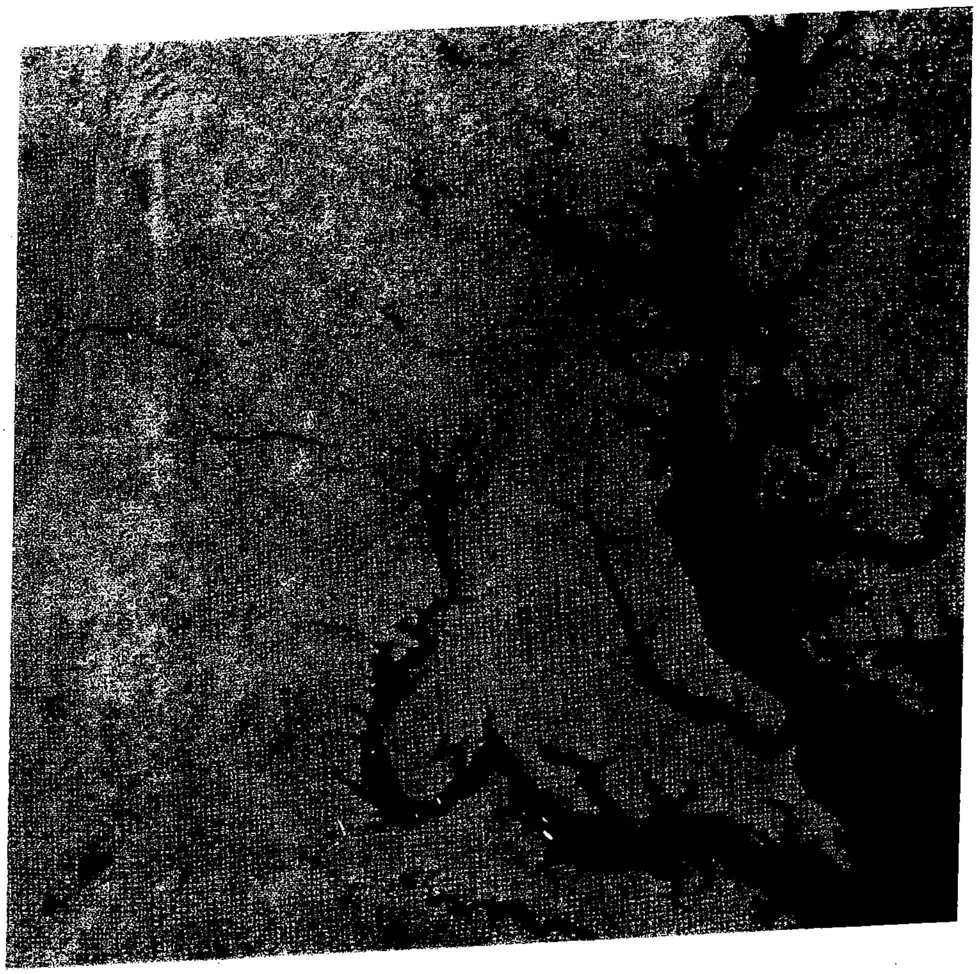

Figure 3. Black and white print of a Landsat-1 image of the study area during the Fall season (Landsat-1, 6 Oct. 73, Band 7, ID. No. 1440-15175-5).

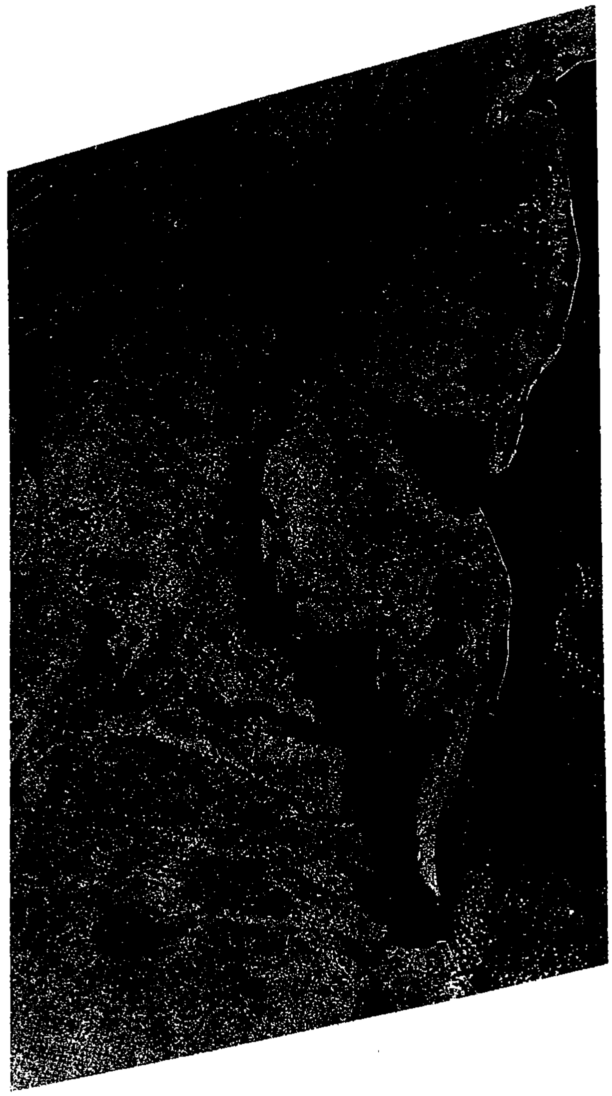

Figure 4. Black and white print of a false color Landsat mosaic (NASA Photo).

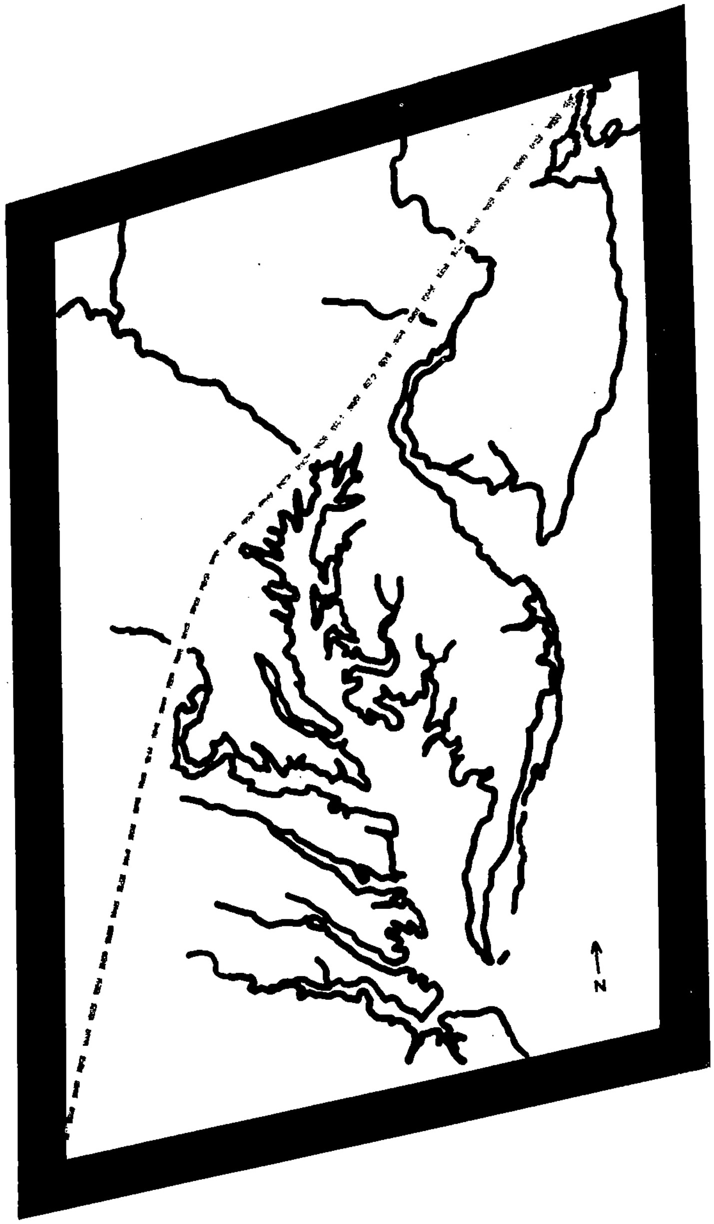

Figure 5. An overlay of figure 4, showing the location and extent of the Fall Line (see figure 4 for reference).

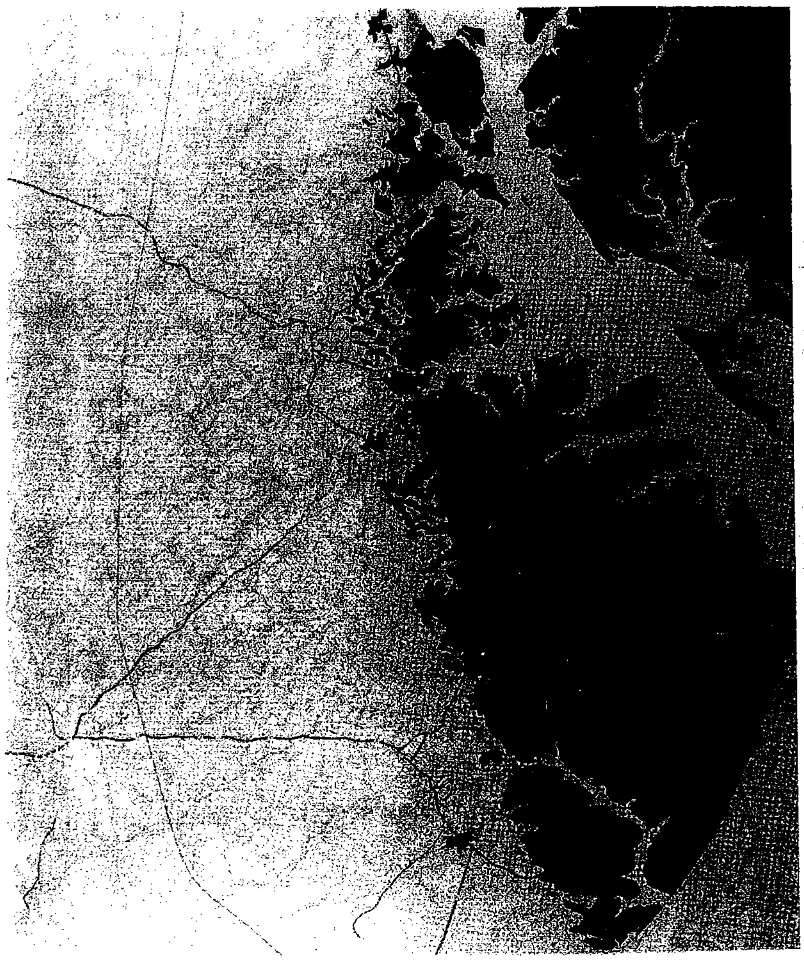

Figure 6. Geological map showing the westward extent and distribution of the Coastal Plain sediments (USGS, 1967).

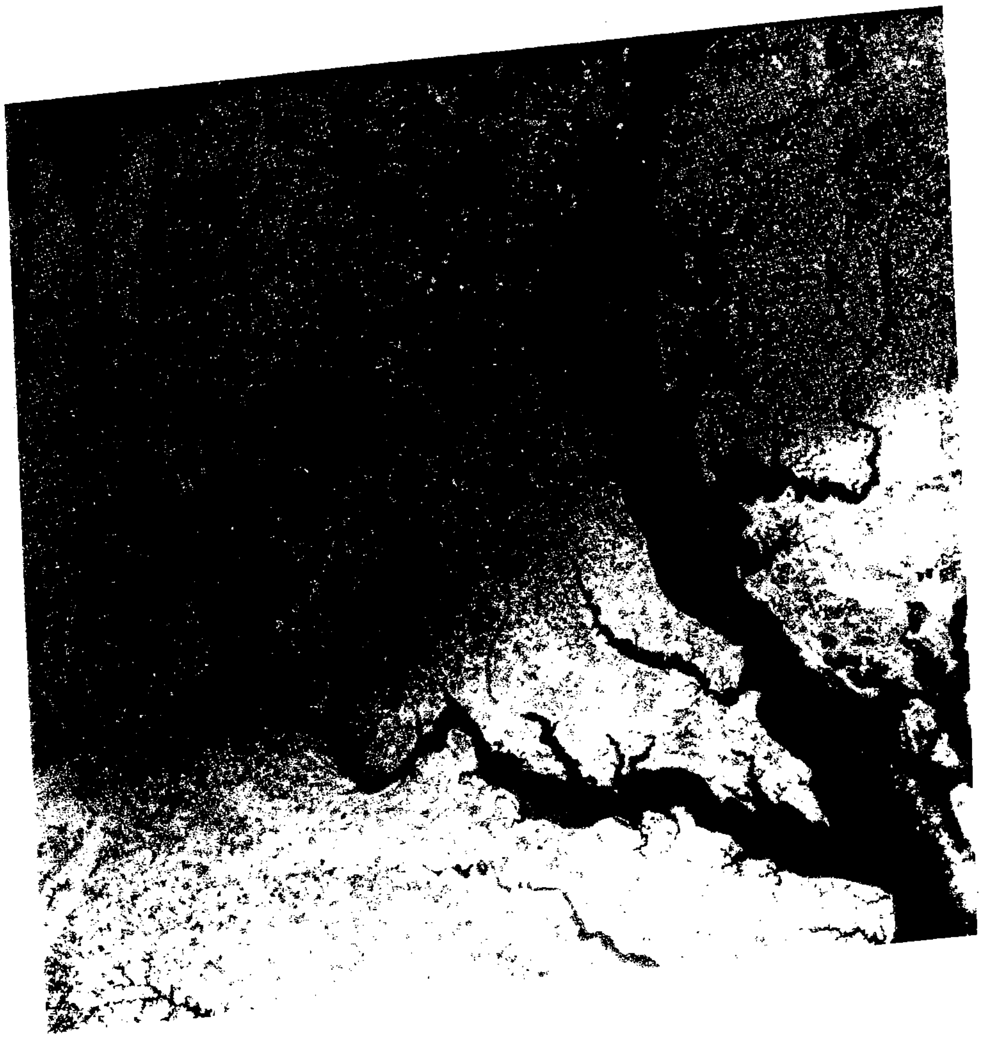

Figure 7. Black and white print of a false color Landsat-1 image of the study area during the winter (Landsat-1, 9 Jan. 73, ID. No. 1470-15193).

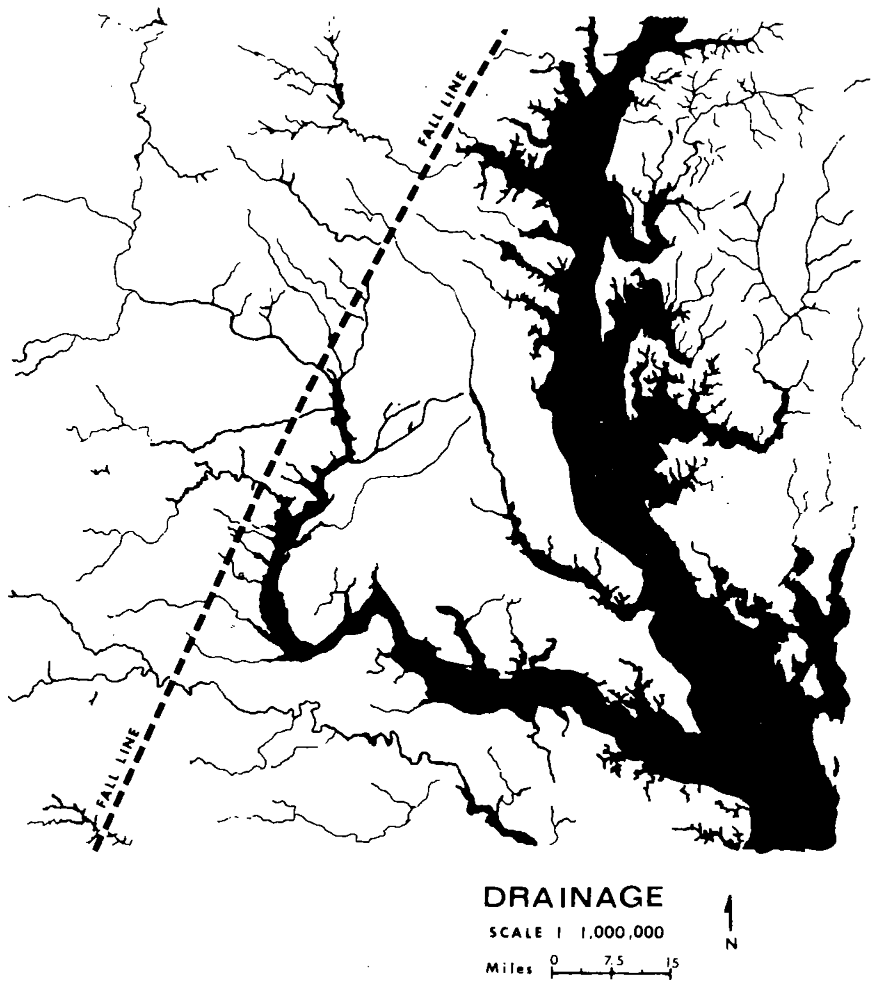

Figure 8. Major patterns of drainage within study area. The overlay was prepared from a winter season Landsat-1 false color composite at the scale of 1:1,000,000, 9 Jan. 73, ID. No. 1170-15193.

Interpretation of high altitude aircraft infrared imagery was also unproductive, even though smaller hydrological and smaller land use features could be identified more readily and more precisely.

Direct photogeologic interpretation of Landsat imagery produced a number of lineaments (Figure 10). Some of these lineaments cross the Fall Line. Most, however, were identified west of the Line, due to better bedrock exposures, geological structures and regional geomorphology. And, at this time, the only definite conclusion that might be drawn from these data is that unless the line connecting the upper portions of the Potomac, the Chesapeake and the Delaware is considered a continuous lineation, the interpreted lineaments also appear to be of no value for identifying or locating Fall Line geology.

SUMMARY AND CONCLUSION:

The direct identification and location of the Fall Line on standard Landsat imagery alone was for all practical purposes, ineffective. However, the boundary could be partially delineated with the aid of correlative overlays, and because of our familiarity with the area.

Still, the inadequacy of direct identification of Fall Line Geology should not be overly emphasized as a general limitation on Landsat imagery because the problem is very much due to local Geologic conditions. For example:

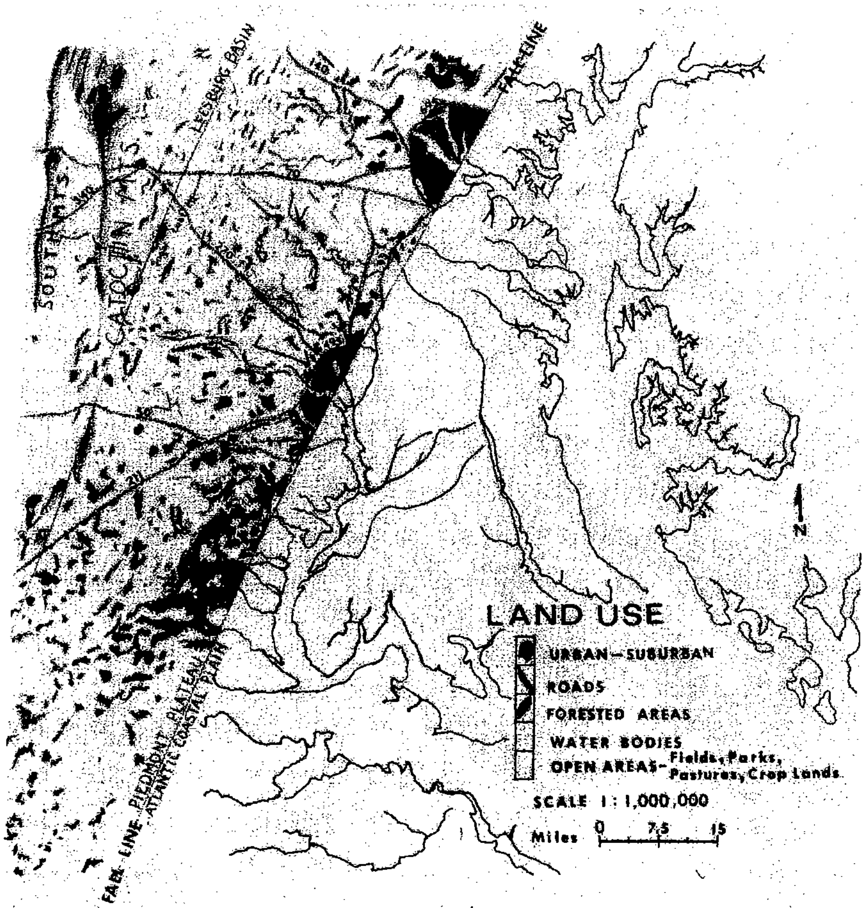

Figure 9. Black and white print of an interpreted color overlay of the major land use categories west of the Fall Line. The overlay was prepared from a winter season Landsat-1 false color composite at the scale of 1:1,000,000, 9 Jan. 73, ID. No. 1170-15193.

Figure 10. Distribution of the geologic lineaments as interpreted from Landsat imagery. The heavy dashed line shows the position of the Fall Line (Rabchevsky, 1978).

The Fall Line scarp is in many places either worn down by erosion or the "soft-hard" lithological contact has been covered by recent sediments.

Further, the common in-situ weathering of the Piedmont metamorphics produced a thick layer of saprolite which now subdues the morphologic expression of the Fall Line scarp.

In addition, the interfingering of the recent and cretaceous soft sedimentary lithologies and sediments east of the Line with the saprolite west of it further obliterates the Fall Line boundary. Also, human distrubance of the land contributes to the lack of differences on either side of the line.

Lastly, due to dense vegital cover in many places along the Fall Line, the scarp of the line and the reflectance differences (spectral signatures) that might be expected from the differing lithologies are not expressed. However, improved remote sensing satellites planned for the 1980's are to have better ground resolution, and scanners that can distinguish more than 200 gray level differences compared to the 64 that todays Landsat can distinguish. Consequently, much better results can be expected in the future.

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RELATIONSHIP OF LANDSLIDES TO FRACTURES IN POTOMAC GROUP DEPOSITS, FAIRFAX COUNTY, VIRGINIA

by

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United States Geological Survey
Reston, Virginia

1978

ABSTRACT

RELATIONSHIP OF LANDSLIDES TO FRACTURES IN POTOMAC GROUP DEPOSITS, FAIRFAX COUNTY, VIRGINIA

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Landsliding is a common problem in eastern Fairfax County, an area underlain by Potomac Group (Lower Cretaceous) Coastal Plain deposits of silt and clay interbedded and interfingered with sand and gravel. The slides commonly are present in clay and silt that, on the basis of laboratory tests, appear to be much too strong to have failed. However, the very plastic silt and clay deposits are commonly cut by long continuous to short discontinuous high-angle and subhorizontal joints, shears, and faults. These fractures can contribute to sliding. Failure along faults and shears takes place because the relative movement has greatly weakened and softened the clay and silt. The mechanism for failure along the joints is less obvious but may be related to infilling along joints, slight movement due to erosional unloading, swelling of clays, and softening along joints.

Field investigations suggest that the three best developed high-angle joint sets in eastern Fairfax County have trends in the northeast quadrant. Lineaments plotted on a lineament map prepared from Landsat imagery also trend northeast parallel to the regional joint orientation. This regional orientation suggests that some of the fractures are tectonic in origin. Therefore, they should be anticipated in a broad zone many miles

wide in the vicinity of the Fall Line. In addition, fractures of unknown origin contributing to landsliding, have been observed in massive Potomac Group clay and silt deposits. Irrespective of origin, these fractures require careful investigation and engineering to prevent slope failure at critical locations during and after major excavation.

INTRODUCTION

Landsliding is a widespread problem in the eastern part of Fairfax County, Virginia. This part of the county is underlain by Potomac Group deposits of silt and clay interbedded and interfingered with sand and gravel. The distribution of these deposits and their association with landslides are fairly well known (Am. Soc. Civil Engineers (ASCE), Natl. Cap. Sect., 1977), but the physical mechanisms causing many landslides are not fully understood. Conventional investigation and design procedures show that slightly weathered and unweathered materials are stiff to hard and that slopes cut in them should be stable at high angles. However, slopes cut into these strong Potomac Group silts and clays commonly fail at low angles and typically fail many years after excavation of a cut.

The authors suspected that fractures of diverse origin were at least partially responsible for the discrepancy between expected and observed slope behavior. However, because data concerning fractures in these soils* were generally unavailable, a reconnaissance study of the fractures was undertaken. The

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* In this text, "soils" is used in the engineering sense, to mean all unconsolidated material overlying bedrock.

purpose of this research was fourfold: 1) to determine the presence or absence of structural fractures (i.e., joints and shear zones) and weathering fractures (i.e., fractures related to surface weathering processes such as wetting/drying and freeze/thaw cycles) in the Potomac Group sediments of Fairfax County; 2) to describe the fractures; 3) to delineate regional patterns of the structural fractures; and 4) to determine the relationship of the fractures to landsliding.

METHODS OF STUDY

Approximately 25 outcrops that included Potomac Group sands, silts, and clays were investigated to relate joint and shear* formation and distribution to the geologic structure and stratigraphy. Fractures were sufficiently well developed** at 11 exposures to merit detailed studies; on average, 22 strike and dip measurements were taken at each exposure. As the best developed fractures are not always the most abundant, they were distinguished from the less developed fractures. Structural fractures were commonly obscured at the surface by weathering fractures. Therefore, to avoid measuring weathering fractures,

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* A "shear" is defined as a fracture along which there has been shear displacement related to surface weathering.

** Well-developed fractures are continuous in two dimensions for 1 ft (0.3 m) or more.

surface fractures were not measured except where they definitely extended below the local weathering horizon. As a result, extensive excavation with hand tools was necessary. Exposures were first cleared of debris and then excavated by use of a small shovel, pick, or geologic (masonry) hammer. Silty-clay and clay tended to break into small rectangular blocks, whose longest dimension was 3-6 in (7.5-15 cm) long. The planes along which the blocks broke out were considered to be joints and were measured only if they were planar for 1-2 ft (0.3-0.7m) in two dimensions.

While field studies were underway, an independent study was being conducted by Dr. Toru Iwahashi of Shizuoka University, Oya Shizuoka, Japan, to assess the value of Landsat imagery for mapping major linear geologic and topographic features in the Atlantic Coastal Plain. Dr. Iwahashi used a mirror stereoscope to examine two images at a scale of 1:500,000 in an attempt to map these lineaments in Fairfax County. The two images studied are NASA E-1944-15030 (Feb. 22, 1975) and E-2076-25080 (April 8, 1975). Principal points were marked on each image and aligned. The information was then transferred to a clear plastic film and enlarged to a scale of 1:48,000 (Iwahashi and Heironimus, 1978).

After the field study of joints and shears was completed, the data were compiled and plotted as rose diagrams on a map of Fairfax County. This map was combined with the map of Iwahashi and Heironimus (1978) in order to compare results (fig. 1). Additionally, laboratory tests to determine the physical properties of the Potomac Group soils included direct shear tests, determination of Atterberg limits and grain-size distribution, and, for selected samples, X-ray diffraction analyses. Atterberg limits and grain-size distribution data are particularly useful indicators of the strength of the soils. The results are discussed in following sections.

SEDIMENTARY DEPOSITS

The eastern quarter of Fairfax County is within the Coastal Plain province, the western margin of which is along the Fall Line. Most of the Coastal Plain province in Fairfax County is underlain by Potomac Group sediments of Cretaceous age, which are overlain in part by Tertiary and Quaternary sand and gravel deposits. The Potomac Group is the basal unit of the Cretaceous strata in Virginia, Maryland, Delaware, and southern New Jersey; in Fairfax County, it is the only part of the Cretaceous section present. The deposits range in thickness from a feather edge along the Fall Line to more than 600 ft (180 m) in the eastern part of the county. The Cretaceous deposits overlie metamorphic and igneous bedrock and saprolite of the Piedmont province.

The Potomac Group sediments in Fairfax County are mainly silt-clay (i.e., silt and/or clay) layers interbedded with sand-gravel layers. The silt-clay beds are iron stained to a reddish brown, or are gray to gray green. They range from massive widespread beds to irregular lenses having little lateral continuity. The clays are predominantly montmorillonite and montmorillonite/illite mixed-layer clay but contain some layers of almost pure montmorillonite or illite or of clay mixed with small amounts of vermiculite or kaolinite (Force and Moncure, 1978). The sand-gravel layers are generally white to buff and in places are also iron stained. The sands are generally trough cross-bedded and commonly have a silt-clay matrix. Most individual sand grains are quartz, and secondary amounts of feldspar are present.

Cross-bedding, and other primary features in the sand indicate that the Potomac Group sediments were deposited by a fluvial system. The lower part of the Potomac Group deposits commonly consists of arkose, quartz gravel, and montmorillonitic clay. The rest of the section generally decreases in grain-size upward, with the notable exception being extensive channel-fill sands. The increased weight added by each successive layer of sediment compacted the Potomac Group sediments to a consistency that is very stiff to hard. (Consistency descriptions in this text are semiquantitative; relationship of consistency to unconfined compression strength is shown in Table 1.)

During the late Tertiary and the Quaternary, hundreds of feet of Potomac Group sediments were eroded away. In places the deeply dissected Potomac Group terrane was subsequently capped by gravels deposited by the ancestral Potomac River. These gravel caps define a series of at least four terraces, each lower terrace being formed during successive depositional/erosional cycles (Force, 1975). In addition to this erosion and deposition, some tectonic deformation (flexing and faulting) took place along the Fall Line. This movement, possibly accompanied

QUALITATIVE AND QUANTITATIVE EXPRESSIONS FOR CONSISTENCY OF CLAYS AND CLAY-CEMENTED SANDS OF POTOMAC GROUP		
CONSISTENCY	FIELD IDENTIFICATION	UNCONFINED COMPRESSIVE STRENGTH (tons/sq ft)
Very soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25-0.5
Medium	Can be penetrated several inches by thumb with moderate effort	0.5-1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0-2.0
Very stiff	Readily indented by thumbnail	2.0-4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Table 1 (After Peck and others, 1953, pg. 29.)

by stresses associated with the erosional unloading of the Potomac Group sediments, caused joints and shear zones to form within the deposits.

Current geologic processes acting in the area are primarily surface weathering (including wetting/drying and freeze/thaw cycles) and natural and man-induced landslide and creep.

DESCRIPTION OF JOINTS

Joints are commonly well defined and best developed in massive, highly plastic (CH)* clays having thicknesses exceeding 3-5 ft (1-1.7 m). Highly plastic clays in thinner layers either have more poorly developed joints or no joints. Interbedded clays, and low-plasticity silts or sands less than 6 in (15 cm) thick do not have joints. Thick massive clays of lower plasticity (CL), and highly plastic silts (MH) have well-developed closely spaced (1-2 ft (0.3-0.7 m) horizontally) joints at some sites, but few or none at others. Thinly bedded silts and sands in soils of lower plasticity have no joints. Massive sandy units (SC or of lesser plasticity) and massive

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* Capitalized letters in parentheses refer to the Unified Soil Classification System.

silts of low plasticity (ML) are rarely jointed.

Vertical joints in plastic soils can sometimes penetrate at least 50 ft (16 m) beneath the surface of the Potomac Group (Withington, 1964). Vertically oriented joints 30-40 ft (10-13 m) beneath the surface of the Potomac Group have been noted by one of the authors in excavations for the Washington, D.C., rapid transit system. Laboratory test data and field observations show that at other sites, undrained shear strengths are often highly variable to depths of 50 ft (16 m) and greater because of slickensides and natural defects (Mueser, Rutledge, Wentworth, and Johnston, Co., 1971, 1973).

Although joints are very rare in soils of low plasticity,, at one locality, vertical, weak, infilled clay zones as much as 0.5 in (1.2 cm) wide traverse hard sands bounded by clays of low plasticity; these infilled clay zones and other rare joints in sands and silts of low plasticity are probably related to nearby shears of possible tectonic origin. As discussed below, shears almost certainly not of landslide origin are common, and whereas these shears have a preferential strike, associated zones of weakness such as secondary cracks, fissures, and splays at various strikes and dips must be anticipated. Thus although vertical joints are not likely for certain combinations of soil plasticity and thickness, zones of weakness having random strikes and variable dips must be expected to exist at some localities.

Most joints observed are nearly vertical because field investigation methods favor the observation of vertical joints rather than subhorizontal joints. Most exposures studied extended farther horizontally than vertically, thus exposing more vertical joints. The method of digging similarly biased the observations because the excavations also extended more horizontally than vertically. Nevertheless, at almost all sites having abundant high-angle joints, some low-angle joints, partings, or other zones of weakness were also found. For example, at a 20-ft- (7-m-) high vertical exposure of unweathered material having vertical joints spaced approximately 5 ft (1.7 m) apart, two or three subhorizontal zones of weakness are present. These zones of weakness may be true joints, zones that have been sheared slightly, zones that have been forced apart by tree roots, or sedimentary strata preferentially weakened because of unknown geologic factors, such as small changes in clay or silt content. Irrespective of origin, very thin subhorizontal zones of pronounced weakness traverse the massive clay. These zones are nearly planar, slightly undulating, and continuous for several tens of feet. On the basis of field observations at this and other sites, such subhorizontal zones are thought to be common in the clays and in highly plastic silts within approximately 20 ft (7 m) of the present surface of the Potomac Group.

JOINT DEVELOPMENT AND RELATION TO REGIONAL STRUCTURAL FRAMEWORK

Although no attempt was made to classify joints systematically, many joint types were observed and may have had several different origins. The subhorizontal joints previously mentioned are interpreted to be pressure-release joints resulting from the removal of overburden by erosion. Steeply dipping open joints having irregular, slightly curved surfaces may also have resulted from unloading. Consolidometer data from samples in the area show that 200-300 ft (65-100 m) of overburden has been removed by erosion. Removal of such great thicknesses of overburden must cause very high ratios of horizontal to vertical forces near the ground surface. Therefore, both types of pressure-release joints can be expected in highly plastic silts and clays in the area.

The best developed, most continuous joints observed (although not always the most abundant) are steeply dipping or vertical with smooth or slickensided planar surfaces. Although these characteristics are not indicative of any one mode of origin, they do suggest shear deformation (Nevin, 1942).

Joint origin was further examined by studying the regional pattern of the joints (fig. 1), which shows that the dominant sets of joints at each sampling locality are oriented in the northeast quadrant, the most common orientations being N60°-75°E, N30°-45°E and NS to N15°E. The orientation of N30°-45°E is approximately that of the Stafford fault zone (N30°E) mapped just south of the study area by Mixon and Newell (1976). Thus tectonism (compression) along this zone may have been at least partially responsible for the creation of the Fall Line. This fault zone extends along the Fall Line into Fairfax County. In this study, joints oriented N30°-45°E are interpreted to be of tectonic origin related to movement along the fault. In addition, the other two dominant sets of joints oriented 30° to the left and right of the N30°-45°E joints may be part of the same regional fracture systems.

The Landsat lineament map of Iwahashi and Heironimus (1978) was compared with the joint orientations as determined by our field measurements (fig. 1). Most dominant joint orientations parallel nearby Landsat lineaments. The fact that the orientations of the joints and the Landsat lineaments, which were determined by two independent studies using entirely different methods, are similar, lends great support to the suggestion that the joint pattern in eastern Fairfax County is related to regional structures such as the Stafford fault zone.

DESCRIPTION OF SHEARS, SHEAR DEVELOPMENT, AND RELATION TO REGIONAL STRUCTURAL FRAMEWORK

Shears (shear zones) along which relative motion exceeds a few inches (5 cm) on opposite sides of well-defined shear surfaces were observed at three localities; possible shear zones were observed at four other localities. Shear zones observed in the field have dips ranging from near vertical to near horizontal and are discrete zones ranging in thickness from 0.5 in (1.2 cm)

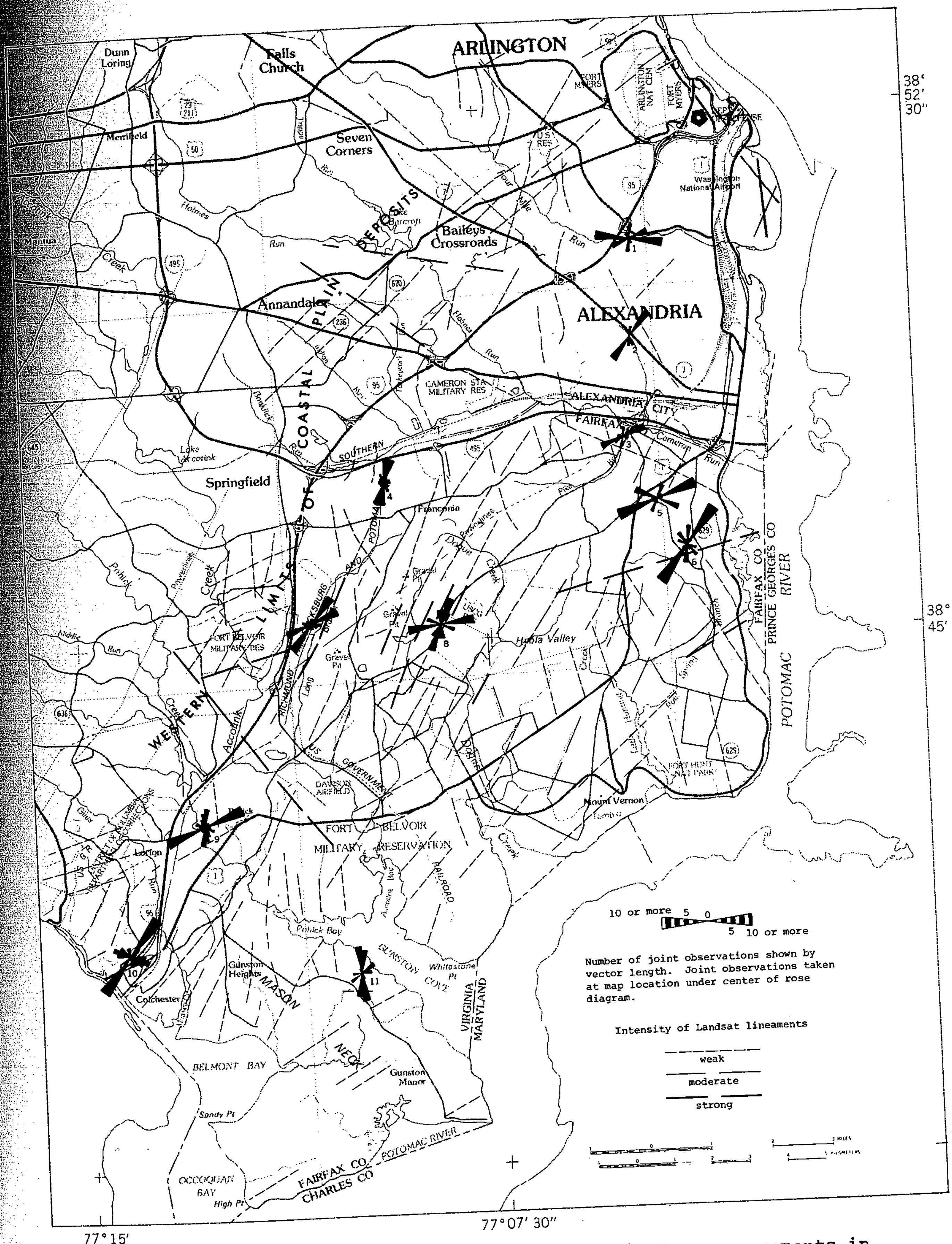


Figure 1.--Comparison of Landsat lineaments and joint measurements in eastern Fairfax County, Virginia. Lineaments modified from Iwahashi and Heironimus (1978). Joint measurements by W. H. Langer and S. F. Obermeier.

or less to 6 in (15 cm) or more. They may occur as one single shear, as a set of smaller shears 0.1 in (0.2 cm) apart, or as single shears or sets of shears spaced 3-6 ft (1-2 m) apart. The shear surfaces commonly have slickensides. The shear zones are commonly soft because of reworking of plastic silts or clays during shearing movement, because of infilling, or because of weathering or soaking with water. Fragments of hard unaltered original material surrounded by soft soil can also be found in the shear zones. The shears generally strike about N5°E. The lack of many long, continuous exposures oriented perpendicular to the shears precludes detailed measurements of the spacing of shear zones. However, at two localities (fig. 1, sites 2 and 8), shears having a displacement of at least 1 in (2.5 cm) are spaced within 100 ft (32 m) of each other; along Fourmile Run, nearly continuous exposures 6,000 ft (1,900m) long revealed two major shear zones. One measurement of the minimum length of a shear (along strike) was also made. The projection along the strike of a shear on Taylor Branch (site 2) nearly coincided with an observed shear 2,400 ft (800 m) away on Timber Branch; therefore, one shear is probably continuous between the two sites.

To summarize, most of these shear zones are believed to be of tectonic origin and to be related to the Stafford fault system for the following reasons:

(1) The sense of motion was completely independent of existing topography.

(2) The strikes of the shears have approximately the same orientation.

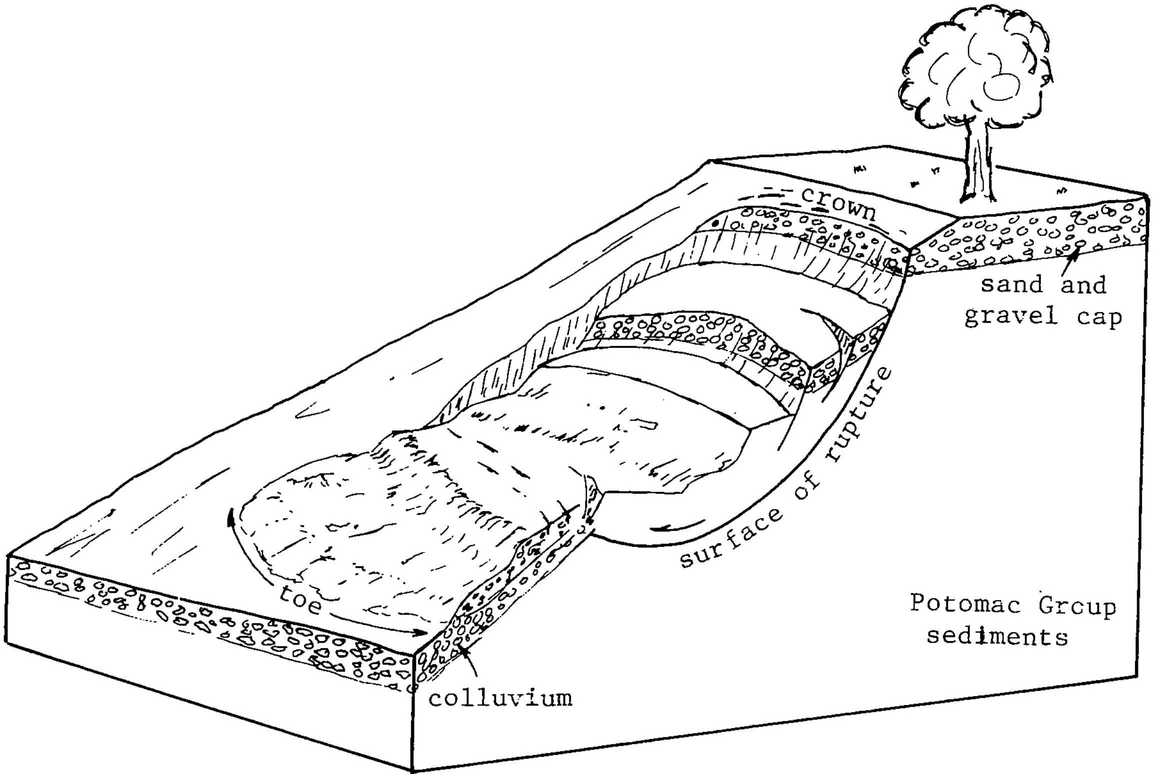
(3) The strikes and senses of motion of the shears conform to geologic structures in the Stafford fault zone.

(4) It was possible to predict the location of a major shear zone at a site thousands of feet from the exposure at which it was first observed.

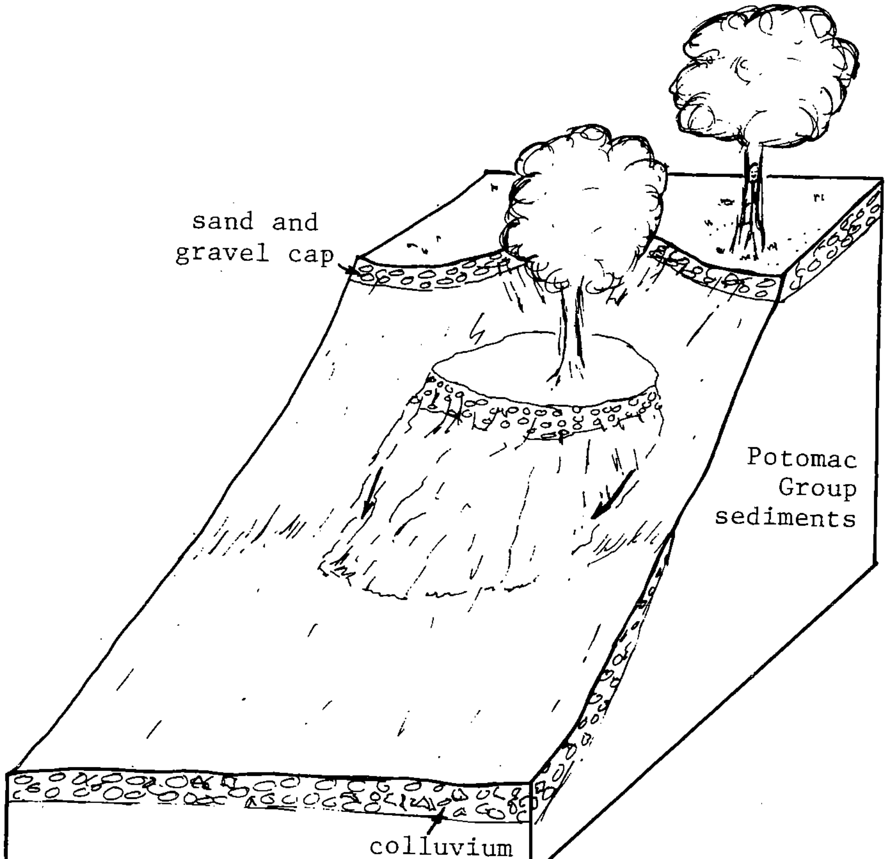
JOINT AND SHEAR RELATIONSHIP TO LANDSLIDING

Natural and construction-related landslides are very widespread and common in Potomac Group clay and sand-clay strata in Fairfax County. The slides are especially troublesome because they may occur many years after completion of construction. Both types of slides typically move noticeably soon after prolonged rainy spells or sudden downpours. Scarps of 0.5 in (2.5 cm) to several feet (1 m) can form within a few hours.

The landslides are normally small, having average lengths and widths on the order of a hundred feet (30 m). The form of most landslides is roughly that of a rotational slump, but planar glide blocks are also commonplace (fig. 2). The maximum depth of the rupture surface of rotational slumps is usually about 15-20 ft (5-7 m). Detailed examination of the terrain in eastern Fairfax County reveals that many slopes greater than about 15 percent and underlain by clay and sand-clay strata have been altered by landslides or landslide debris. A comparison of natural slope angles, abundance of natural landslides, and soil mechanics laboratory strength test data leads to the conclusion that landslides are a major factor in landform development.



Rotational slump

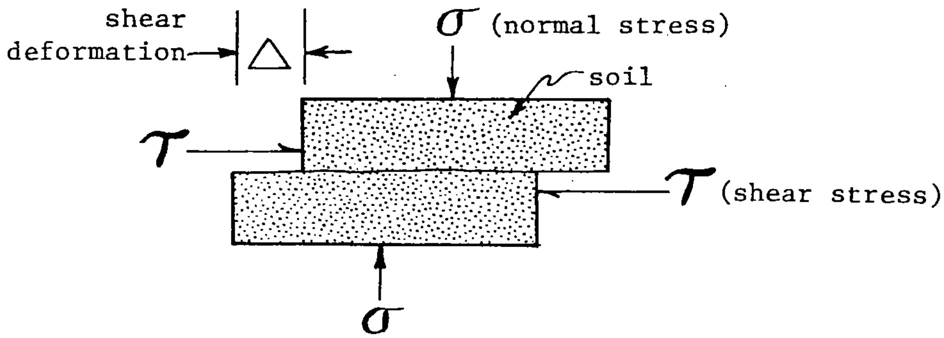


Planar glide block

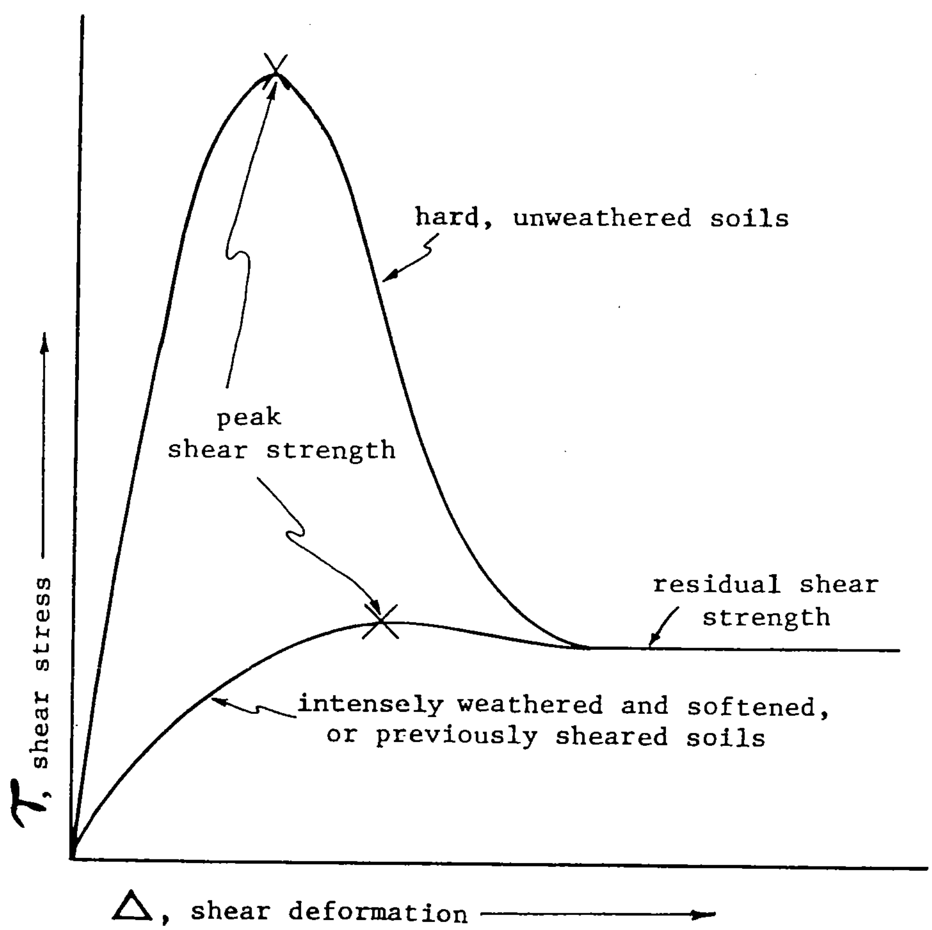
FIGURE 2 Typical landslide types in Potomac Group sediments (modified from Varnes, 1958).

These laboratory strength test data (J.S. Jones, Law Engineering Testing Company, McLean, Va., oral commun., 1976) also show that the slope angles are often controlled approximately by the residual drained shear strength, which is the shear strength of a soil sheared sufficiently to align the clays parallel to the rupture or shearing surface. For both Potomac Group clays and clayey sands, this strength is commonly much less than the peak strength. Figure 3 shows schematically the relation between peak strength and residual strength in a direct shear test for typical unweathered and intensely weathered or sheared clay-rich soils of the Potomac Group.

At the 1977 National Capital Section ASCE seminar, J. S. Jones presented limited data based on conventional slope stability calculation techniques (limiting equilibrium methods), showing that previously unfailed slopes in Potomac Group clays and silts sometimes slump at angles controlled approximately by strength properties as low as the residual drained shear strength parameters of the clays and highly plastic silts. Jones also presented data showing that many unfailed slopes exist at angles greatly in excess of those predicted by residual strength properties and noted that some of the factors causing these apparent discrepancies were unknown. Geologic settings where the soils are very close to the residual shear strength are at the unconformable contact between the Potomac Group and the overlying younger gravel cap and at the contact with colluvium. Both contacts are where rupture surfaces in landslides commonly originate. The Potomac Group clays and silts at these contacts



Direct shear test stress and deformation conditions.



Plot of shear stress vs. shear deformation, with a constant normal stress.

FIGURE 3 Typical strength relationships for drained shear tests on Potomac Group soils.

are commonly weathered and weakened to a medium, or even soft consistency, especially in the water-bearing channels and depressions often found at these contacts. The residual and peak shear strengths of these weakened soils sometimes are almost equal. Figure 4 shows the geologic setting where these and other weakened zones are found in Potomac Group sediments.

Jones also noted that landslides commonly have surfaces of rupture passing beneath these unconformities through unweathered or slightly weathered clays and silts. On the basis of the peak strength of the soil immediately beyond the limits of a thin zone of shearing along the ruptured surface, these soils often seem to be much too strong to have failed. Tests on soils from within the sheared zones show that landslides can initiate under conditions approximately predictable by the residual drained shear strengths, rather than by the much higher drained peak shear strengths or by the unconfined compression or unconsolidated-undrained shear strengths adjacent to the shear zones. Alternatively, as noted by Jones, other slides occur in soils that are apparently stronger than the residual strength state when failure first occurs, intermediate between the residual drained and unconsolidated-undrained or unconfined compression strengths. These seemingly contradictory observations pose great difficulty to the engineer trying to select shear strength properties of the soil that will result in a safe economical design.

The authors believe that much of this apparent discrepancy in strength properties is related to the presence of undetected

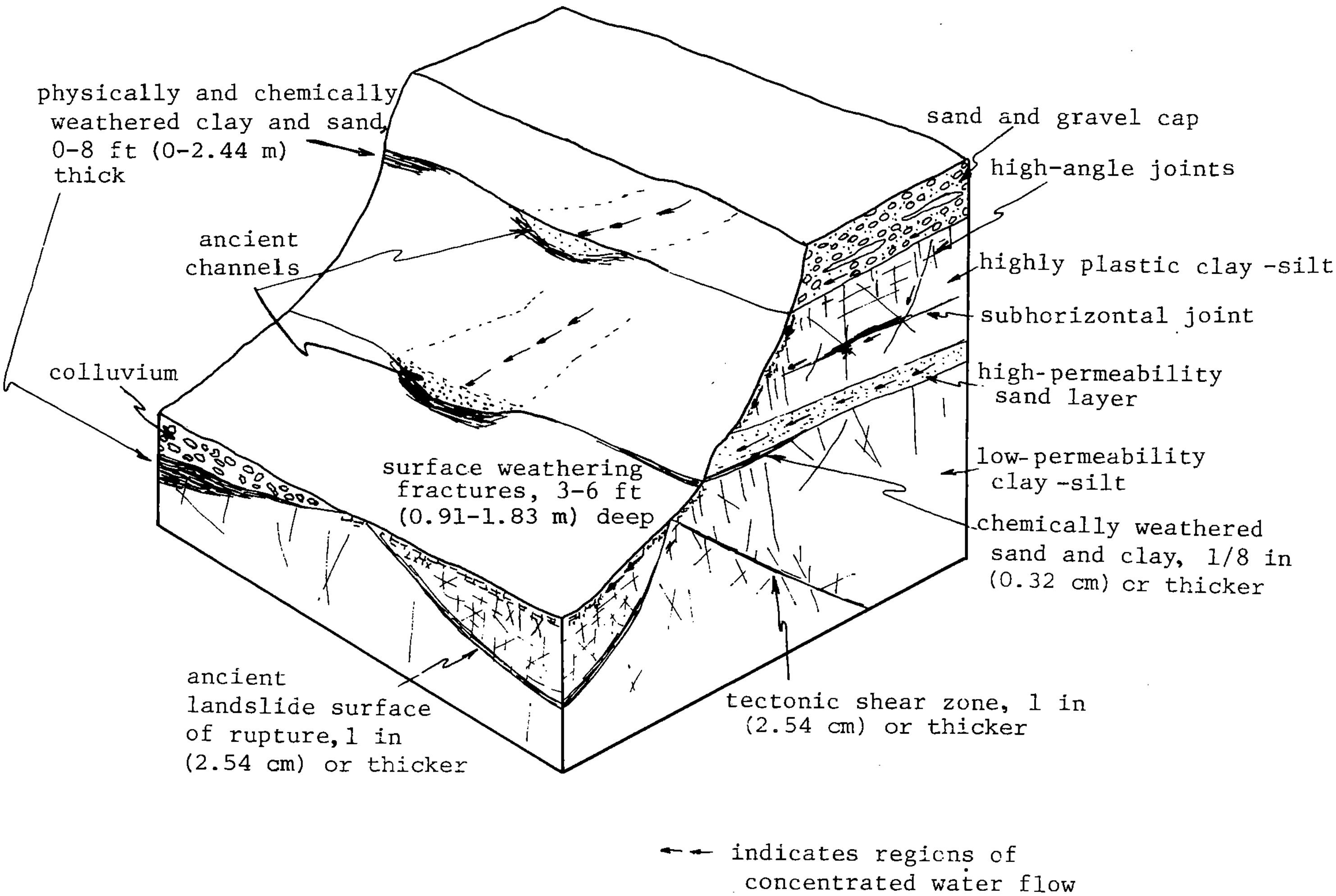


FIGURE 4 Critical zones of weakness

joints and shear zones. Quite clearly all these breaks are ready-made surfaces of rupture for any landsliding, providing they are oriented adversely. The strength of the soil along the joints is always less than that of the massive soil and commonly is much less. Often these breaks provide avenues through which water passes easily, generating higher pore pressure than would be found within massive deposits.

The authors also believe that the strength along many of the joints and shear zones must approach the residual state with little or no apparent cohesion. (The term "apparent cohesion" is used in the engineering sense, in which a cohesion value is associated with zero normal stress in the direct shear test.) Along the shear zones, relative movement could have reduced the strength to a residual condition, providing the movement was sufficient to align the clays. Laboratory tests by Skempton (1964) and Bishop and others (1971) showed that an inch or two (2.5-5 cm) of relative movement is sufficient to reduce the strength of the soil to its residual strength. Limited testing of undisturbed samples at our laboratory using reversed shearing motions supports this finding. Two of the shear zones observed in the field have relative shearing displacements exceeding 9 in (22 cm), and at least one other exceeding 2 in (5 cm); others have unknown relative movements. Most of the shears also have well-defined, shiny, slightly undulating planes of separation. Thus soil behavior in the shears probably approaches the residual strength case. Three somewhat related mechanisms that may explain how joints in silts and clays contribute to strength approaching the residual strength are: infilling of joints; movement along joints; and weathering along joints.

To examine how infillings operate, it is necessary to know how joints open. Where Potomac Group clays and silts are exposed to surface drying and wetting, joints and surface weathering cracks commonly open at random orientations to a depth on the order of 5 ft (1.7 m). These surface fractures are commonly filled with soft clayey soil, apparently washed in by surface water during rainstorms; some are partially filled by well-formed gypsum crystals that developed in-situ, to as much as 0.5 in (1.2 cm) in length and 0.06 in (.2 cm) in diameter. These types of near-surface fractures tend to be oriented vertically, or to have high-angle dips near the ground surface, curving toward lower dips, and even subhorizontal inclinations at depth.

Joints apparently are also opened and moved by tree and vine roots, which in some places penetrate to depths of at least 15-20 ft (5-7 m) beneath the ground surface. The joints may have been opened slightly before penetration by roots, but certainly the roots are very effective in further wedging the sides apart. The pattern of roots viewed in vertical section tends to be rectangular or trellised, following vertical joints and subhorizontal planes of weakness (which may or may not be joints as noted previously, but are called joints). Both vertical and subhorizontal root-penetrated joints commonly have soft, clayey soil that is apparently washed in.

The infilled soil in both the near-surface and root-opened fractures tends to have a slightly higher plasticity than the soil on either side of the joint, according to our laboratory test data. The infilled soil in subhorizontal joints is commonly not consolidated by the weight of the existing overburden, probably because the weight of the overburden is supported at random "points" such as strong soil-to-soil contacts and roots. This soft infilled material must also have drained shear strength properties at least approaching those of the residual case. Because the drained shear friction angles of these soils decrease as plasticity increases, and because joints are commonly oriented subhorizontally in the clayey materials to depths that agree with the observed depths of the landsliding, it is likely that infilling contributes to landslides at angles approaching those controlled by the residual strength properties of the soil.

The joints permit water to enter, causing the soil near the joints to swell or to weather and soften. Discolored and softened soil is sometimes observed to a depth of 0.5 in (1.2 cm) behind the joint surface. The swollen or softened soil has a lower strength than the undisturbed soil; near some joints, the soil is softened so much that the peak and residual strengths must be about the same.

Swelling and shrinking of the montmorillonite-rich clay during wetting and drying may also induce small horizontal movement along the subhorizontal joints to depths sufficient to influence landsliding. Upon wetting of the clay, complete reclosure of the vertical joints is probably prevented by the infilled clays, crystals, and roots, causing large horizontal forces and small differential horizontal movements along the joints. Well-developed slickensides from wetting and drying (shrinking and swelling) are commonly observed near the ground surface, and the same mechanisms that cause these slickensides almost certainly cause small movements to depths significant to landsliding.

Other strength-reducing mechanisms, such as the generation of movement along joints by horizontal stresses during erosion of Potomac Group sediments, may be related to joint formation, but the relationships discussed above are believed to be adequate to explain why many slopes in clays and silts are permanently stable only at very low angles.

A final note should be made about the role of tree roots on slope stability. Although roots are thought to be a slope-weakening mechanism, a very good possibility exists that the detrimental effects do not appear until after the tree or vine has died. While the roots are alive, their strength may contribute to the strength of the soil in maintaining a slope. After the plant dies, the roots decompose and weaken and also leave tubes that allow water to have easy access to a potential rupture surface.

Joints in the Potomac Group can also be important to short-term construction problems. In 1962, at nearby Greenbelt, Maryland, a clay slab that was 42 ft (14 m) long, 15 ft (5 m) high, and about a foot (0.3 m) thick toppled from an unsupported vertical cut, killing five men. Withington (1964), who visited the site, concluded that the slab failed along an incipient joint which opened when lateral support was removed. The base of the cut was 40 to 50 ft (13-16 m) below the surface of the Potomac Group, and multiple vertical joints extend the full depth of the cut.

Thus far, the landslide discussion has been directed to problems in Potomac Group clays and silts, because these are the deposits in which the problems are most commonplace. However, landsliding may be initiated less commonly along joints, fractures, and sheer zones in sand-rich facies (SC, SM, SP, SW). This conclusion is supported by the following field observations in the sand units:

(1) Vertical joints and tectonically related fractures and shears are present only near major shear zones in sand-rich facies; whereas they are widespread in clay units.

(2) Horizontal and subhorizontal joints and shears are rare in sand units.

(3) The joints and shears in sand units are often filled with oxides which have rehealed any previous ruptures. In contrast, joints and shears in clays are sometimes softened or are sometimes infilled with soft soil.

(4) Weathering-related, near-surface fractures in sand units are short and discontinuous and have high-angle dips. These are found primarily in intensely weathered deposits at hilltops (Milan J. Pavich, USGS, oral commun., 1977).

(5) Sands are more porous and permeable to water, and water tables in sand units tend to be lower than those in clay-rich units.

Because sand units have fewer weak zones and better internal drainage than clay units, natural and cut slopes in sand units are generally stable at angles much greater than would be predicted on the basis of residual shear strength.

SLOPE DESIGN CONSIDERATIONS

Some of the most important factors to consider in designing a stable slope are (1) the geologic setting; (2) plasticity and massiveness of the soils; (3) rate of formation of joints; and (4) orientation, continuity, and spacing of the structural defects.

The geologic setting is a major factor in establishing the probability that surface weathering types of defects exist in the soil. Potomac Group sediments are unconformably overlain by younger sediments of different ages. Each of these contacts must be suspected as a possible site of ancient weathering where shearing and softening may have affected the sediments to depths of at least 6 ft (2 m) beneath the contact. Use of residual or near-residual shear strength parameters should be considered for soils in these weathered contact zones, especially if the soils are massive and plastic, as discussed below. The geologic setting is also important in establishing where there has been ancient landsliding. Although landslides are a source of shear zones not discussed previously in this paper, shear zones of any origin and any age must be considered as having only residual shear strength, except where recemented in sands. Widespread ancient landsliding of the Potomac Group took place when Pleistocene glaciation caused lowering of sea level, resulting in deep downcutting into the sediments. Surface evidence of these slides is now buried beneath younger sediments, but probable sites are beneath dipping contacts of the Potomac Group and the overlying younger sediments.

Plasticity and massiveness of the soil has a pronounced influence on the probability of occurrence of joints and shear zones, and in turn, on the strength parameters selected for design. Massive units of clay and plastic silts (CH, CL, MH) commonly have discontinuous high-angle joints and associated discontinuous subhorizontal joints extending to a depth of four to six feet (1.3-2m) beneath the top of the unit; continuous vertical and subhorizontal joints are also present within the upper 4-6 ft (1.3-2 m) of the unit, and sometimes to much greater depths. On the basis of observations of slides and other data from the local area, we conclude that potential slides through the joints should be analyzed as though the soil were at or near the residual strength state. Less plastic and less massive soils tend to have fewer joints, and joints they do have are more likely to reheal. If joints, especially infilled joints and weathered and softened joints, are not rehealed, residual strength properties should be considered for any potential slide along the joint surface. It must be emphasized that this conclusion about strength properties is based on geologic observations and comparisons of field and laboratory data for the Potomac Group deposits. We are aware of only one detailed geologic and engineering investigation (Skempton, 1977) on the long-term slope stability of over-consolidated, stiff, fractured clays, and our conclusion is considerably different from those expressed by Skempton. In that report, Skempton described the behavior of the Brown London Clay in England and suggested that many other stiff fractured clays throughout the world may behave similarly. However, we believe that the Brown London Clay and Potomac Group clays of Fairfax County have very different characteristics. The geologic description of Brown London Clay by Skempton and others (1969) made no mention of infilled or weathered and softened joints (such as those observed in the Potomac Group), nor does the Brown London Clay have the reputation of being a highly expansive soil (such as the highly plastic silts and clays of the Potomac Group).

Data related to the rate of joint development in the Potomac Group are sparse, but observations in a railroad cut in massive clays demonstrate clearly that intensely developed jointing related to surface weathering can extend to a depth of at least 4 ft (1.3 m) within 50 years or less. For many design problems, a depth of at least 4 ft (1.3 m) would be a prudent selection for use of residual strength parameters on permanent slopes cut into massive unweathered clays and plastic silts. For soils of lesser plasticity, it is not necessary to use such low strength parameters.

Joints beneath the zone of weathering fractures are frequently discontinuous and not well developed, even on terrain little altered for hundreds or thousands of years. If these deep joints had developed rapidly after excavation, we would expect that they would be continuous. Thus, significant deep joints probably do not generally develop during the lifetime of a structure, unless unusual factors such as high intensity stresses on steep slopes or growth of tree roots are present.

The relation of continuity of the defects to the design problem has been noted in relation to other aspects; the influence of joint spacing and orientation remains to be considered. Joints can be as close as 3 in (7.5 cm) and as much as 3 ft (1 m) or more apart for a particular soil type at a given site. From a practical viewpoint for many design projects, however, if a joint is found at a given orientation, it must be presumed that many others at the same orientation are present unless detailed examination proves otherwise; if many joints were present the spacing would be irrelevant. A few fractures must also be expected to have orientations completely different from those of the well-defined joint sets; in design it must be kept in mind that a single critically oriented defect can initiate sliding. If a major defect traversing otherwise strong soils is close to an assumed, theoretically determined critical surface of rupture (i.e., the surface of rupture determined without consideration of the presence of the defect), an apparent cohesion of zero or approximately zero should be considered for slope design. However, for most slopes, the structural defects and the assumed critical surface of rupture will probably be at different orientations, in which case a small apparent cohesion could safely be included in the design of the slope.

In summary, it appears that design parameters should be quite conservative for major projects and projects where human life is involved, because of the difficulty in detecting the presence and orientation of the structural defects from scattered borehole data and from infrequent outcrops. The use of test pits is believed to be the only practical way to observe these structural defects in detail and to have a clear understanding of how they fit together. Without detailed field investigations based on existing concepts and knowledge, residual or at least near-residual strength parameters certainly should be considered for design of permanent slopes in clays and plastic silts for important projects; for sands and silts of low plasticity, strengths nearer the peak values usually can be used safely.

CONCLUSIONS

Preliminary evidence strongly suggests a tectonic or structurally controlled origin for regionally oriented joints in the plastic silt-clay deposits of the Potomac Group in Fairfax County, Virginia. Similar well developed joints can be expected to be present elsewhere in the Potomac Group plastic silt-clays of the county at similar, predictable orientations. In addition, randomly oriented joints formed by unloading are likely to be present in massive, highly plastic (CH) clays; they are somewhat less abundant but are still common in massive clays of lower plasticity (CL) and in silts of high plasticity (MH). The joints are inherently weak zones and may be further weakened and softened by shearing, infilling, and weathering. Joints, faults, and shear zones are surfaces of weakness within an otherwise much stronger soil and are potential surfaces of rupture for landsliding. It is important that excavation sites in Potomac Group clay-silts be carefully investigated and designed to prevent landsliding during and after construction.

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THE "O" STREET SLIDE AND ITS GEOLOGIC ASPECTS, WASHINGTON, D. C.

by: Ernest Winter[1]

and

Brian Beard[2]

ABSTRACT

Unstable slopes along O Street, S.E., Washington, D.C. have plagued the area for years. Evidence of slide activity predates any development in the area. The natural instability of the Potomac Clay slopes was aggravated by construction of roads and dwellings at the toe and top of the slope. Portions of O Street repeatedly became impassable from slide debris moving onto the road, and the safety of houses at the toe and top of the slope was threatened. A detailed subsoil investigation revealed a weathered zone of relatively soft clay and saturated sand layers in the upper soil profile. Water level observations indicated artesian water conditions in sand layers within the clay.

The depth of failure movements observed with slope inclinometers was found to be within 10 to 20 feet below grade, and typical soil parameters developed from these slides indicated a friction angle $\phi' = 21^{\circ}$ in clay. The geologic character of the Potomac Clay was related to observations on the site to identify the layers of most probable instability and to establish criteria for the design of protective structures. A slurry wall along the upper portion of the slide area is now under construction.

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INTRODUCTION AND SLIDE HISTORY

The subject of this paper is the stability of slopes along O Street, S.E., in Washington, D.C., where slope instability has plagued the area for many years, and where a retaining wall is being installed for protection against further slope movement.

The particular hillside is bordered by O Street, S.E., Branch Avenue, Highwood Drive and Carpenter Street. (Figure 1) Single family houses are located along Highwood and Carpenter Streets, and five houses are located along O Street. The area of instability is the interior of the block, behind the buildings. In the center portion of the site, along O Street, slides have extended down and onto the street.

The topography shows O Street on a moderate grade from El 94 Branch Avenue to El 142 at Carpenter Street, with the high points of the site along Highwood Drive at about El 183. The slopes behind the houses have a grade drop of about 40 to 60 feet.

The site has a long history of slide movements. An 1884 topographic map shows existing scarps probably resulting from slides on the hillside of O Street in the general area of the present instability. The area was not developed at that time, and the original instability was apparently not caused by human activity. When the site was later developed with single family homes built between 1941 and 1957, O Street was built along the bottom of the hill and Highwood Drive was built to serve the buildings on top. It is probable that some grading was made at the time of construction of the streets including some cuts for O Street and possibly some fills for Highwood Drive. More fill was placed for the development of buildings on the hillside.

Major slides occurring through the years were concentrated in an area about 300 feet southwest of Branch Avenue. Slides developed and extended down to O Street, sending debris over the street and making the area impassable. Also slide movements were occurring, in addition to the old slide areas, within other slopes on the site.

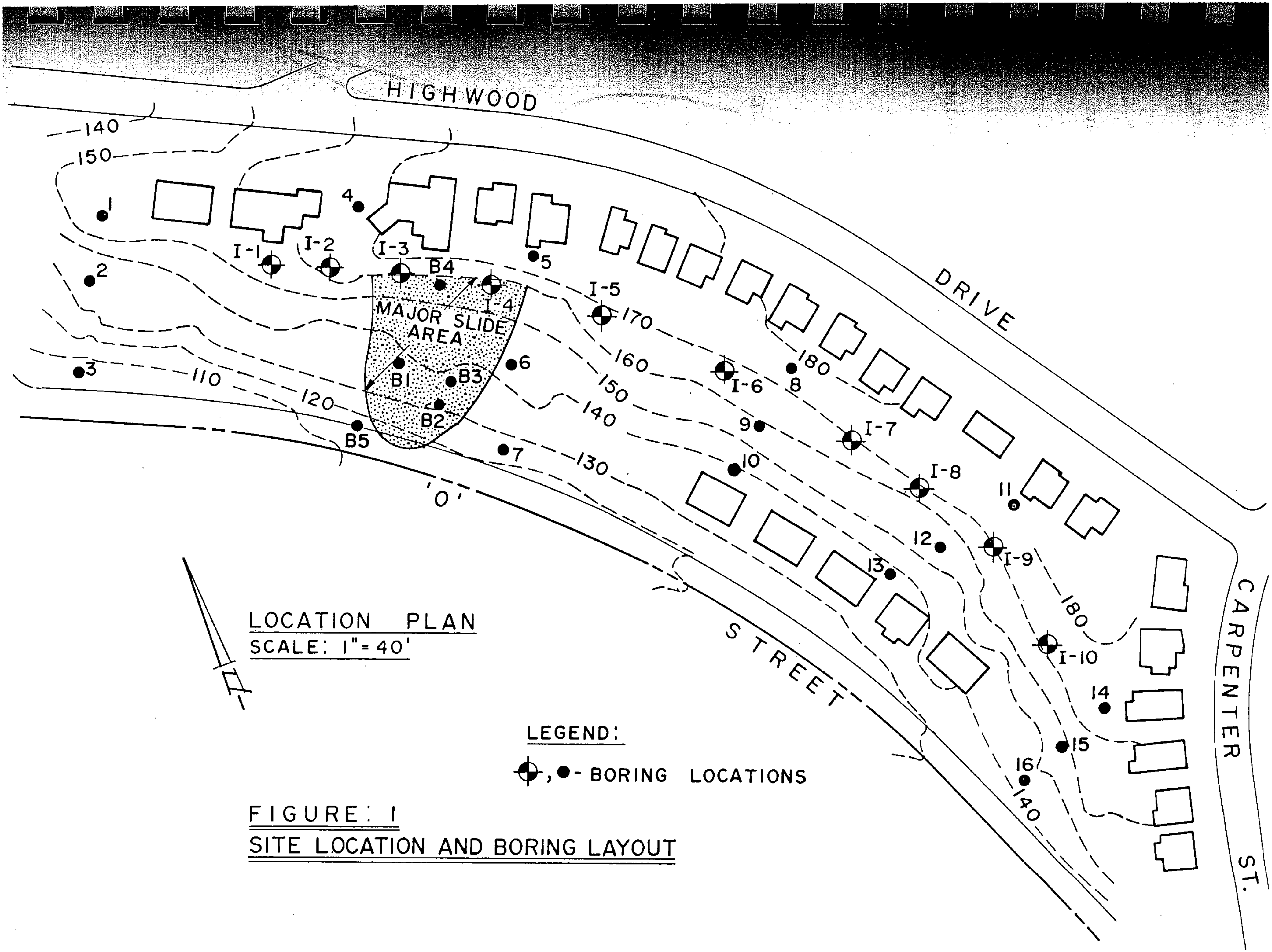


FIGURE: 1
SITE LOCATION AND BORING LAYOUT

GEOLOGIC SETTING

The site is located about 5 miles east of the geologic "Fall Line" within the outcrop area of the Potomac Group sediments of Cretaceous Age. Since the coastal plain deposits dip gently to the southeast, the site is in the upper Potomac Group - the Patapsco formation. (Figure 2) The upper Patapsco consists of interlayered sands, clays, and sand-clay mixtures, and is characterized by cross-bedding, channels filled with both clays and sands, and pinching and swelling of beds. Sand lenses within clay strata frequently develop quite high pore-water pressures.

Slope stability problems in the Potomac clays are common in southeast Washington and in Fairfax County, Virginia. Although massive, overconsolidated clays are present to the north, few stability problems are reported. This may be attributed to a change within the Potomac Group from a montmorillorite facies to an illite-kaolinite facies at about the location indicated on Figure 2. (Obermeier, pers. commun.) The clays of the illite-kaolinite facies are less susceptible to shrinkage and swelling and other weathering effects.

The clays of the Potomac Group are generally massive, highly preconsolidated and possess high plasticity. In southeast Washington and to the south, the predominant clay mineral is a calcium montomorillonite, which absorbs water causing the clays to shrink and swell cyclicly causing a considerable loss of strength that creates fissures and fractures in the upper part, which is generally referred to as the weathered zone.

Weathered zones typically have lower standard penetration resistance (blowcount), higher water content, and lower shear strength than the underlying materials. If sand lenses are present within the weathered zone, high pore water pressures develop within the confined sand lenses and accelerate the weathering process. Slides develop most frequently in these weathered zones.

SUBSOIL CONDITIONS AND THE SLIDE MOVEMENTS

The stratification of a typical test boring is indicated in Figure 3. The upper subsoil profile consists of silty and sandy clay, and fine silty and clayey sand interbedded

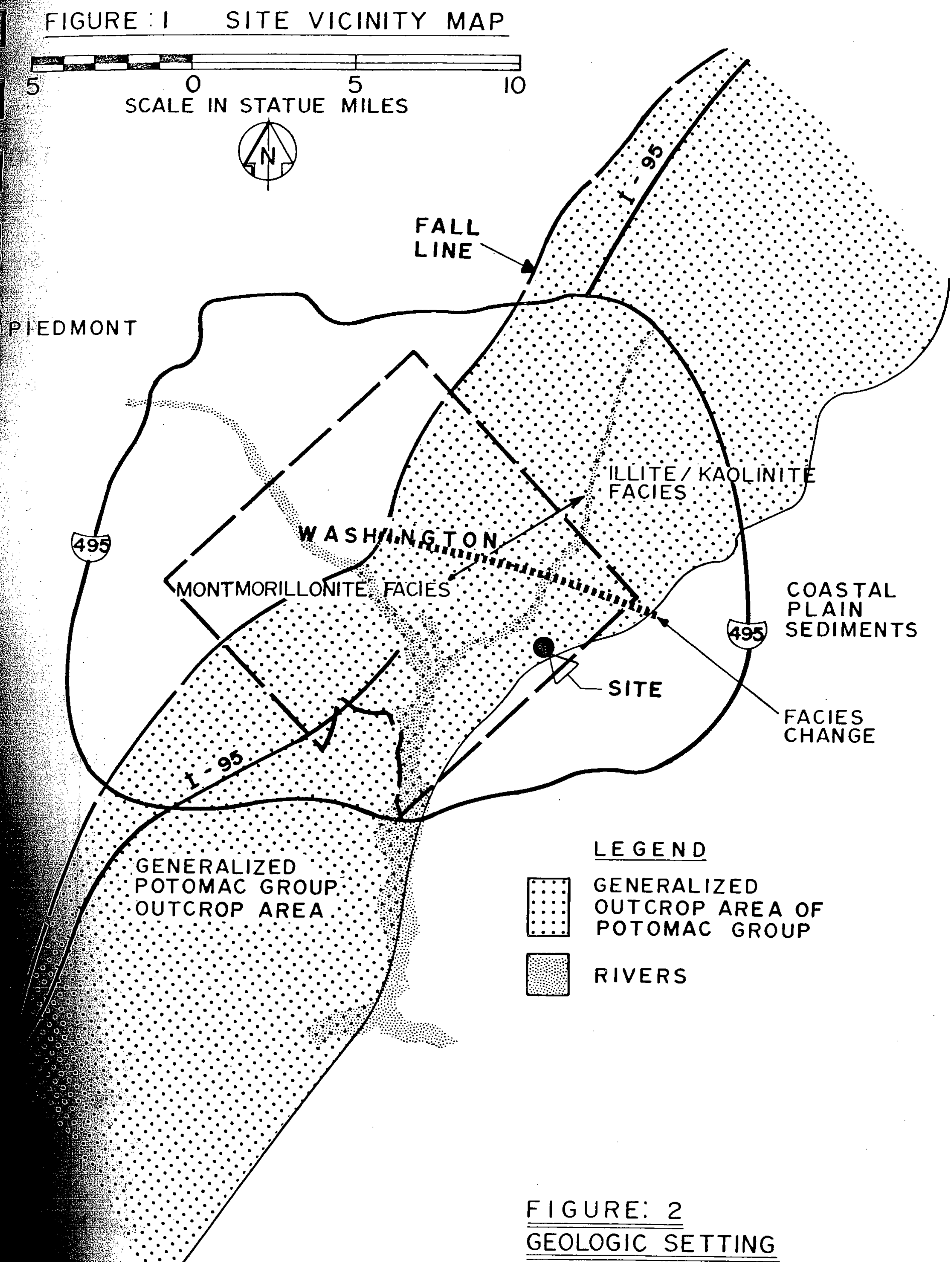


FIGURE: 2
GEOLOGIC SETTING

in layers, seams and pockets. Portions of these upper layers may have been placed by older slide activity and transported from the higher elevations. Clays in the upper formations were generally sandy and weaker due to the presence of water in the interbedded sand layers. These layers are believed to be generally horizontal or to dip slightly down slope. The depth to these soils seems to be less towards the lower elevations.

The deeper stiff clays and higher density sands showed marked increase in stiffness due to less weathering and less exposure to water, and the sand layers also have a higher density. There is also little evidence of fracturing or fissuring in the lower, unweathered layers. It was believed that potential failure surfaces would be less likely to penetrate into these soils, except possibly at the surface of the stratum.

Slides have apparently developed in the upper portion of the slopes, usually in the layers of clay and sand. These strata extend to about 30 feet maximum depth below present grade and include a complex stratification of layers with seams and pockets of both sand and clay. Water is apparently available in several of the sand layers and shear strength is reduced along the contact with clay layers, by softening of the clay surface. Movements may occur both along previous slide surfaces which developed from the natural conditions of the slope or along newly formed slip planes. The conditions on this site are aggravated by the artesian pressure encountered in some borings and in some layers. This pressure results in uplift along low permeability clay layers and reduces the frictional resistance and the overall stability of the slopes.

Conventional stability analyses based on the geometry of the site also indicated that the most probable location for slip surfaces to develop would be in the upper 10 to 20 feet depth of the soils profile, Figure 4. This coincided with measurements on slope inclinometers from the slide area. Figure 5 indicates typical movements prior to a slide, and locates a probable slide surface at a depth of 8 to 11 feet below existing grade. Extensive stability analyses were performed on the existing slide examining soil stratification indicated by the borings, piezometric water levels measured in the water observation wells, and the geometry of the slope to evaluate the shear strength

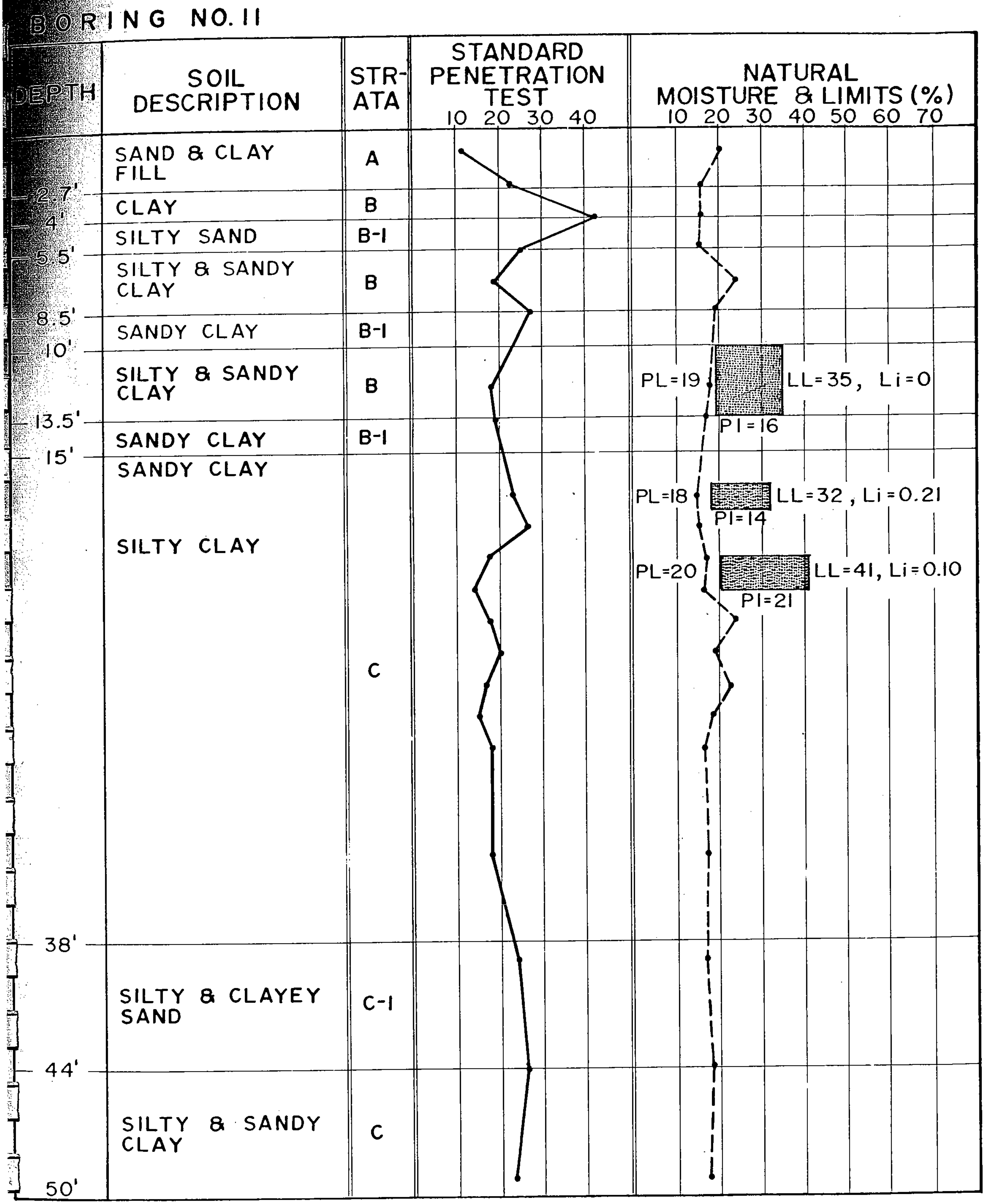
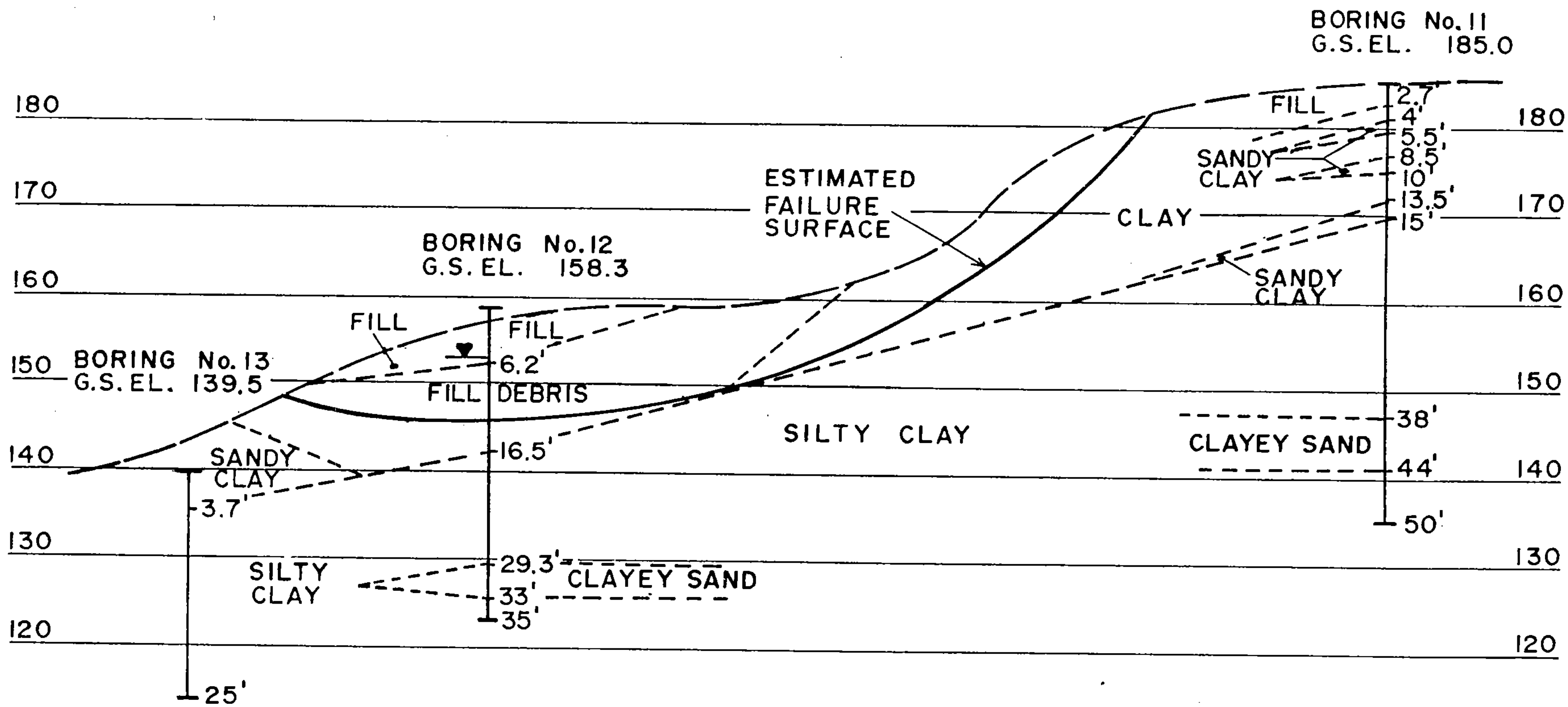
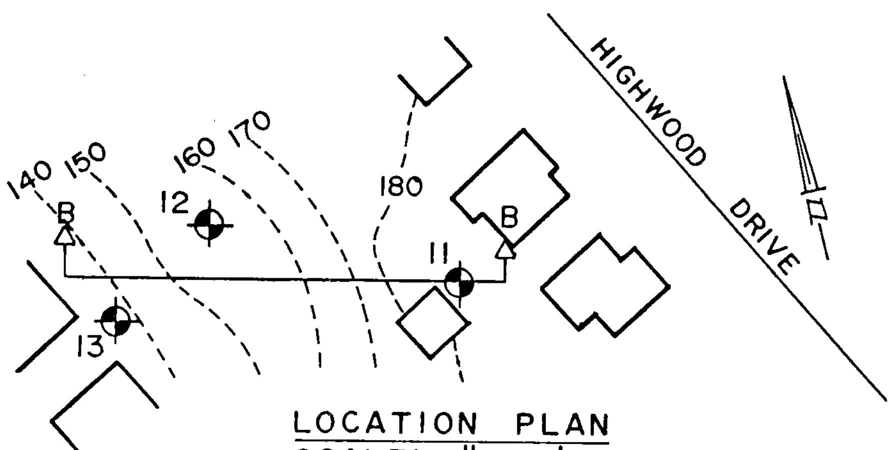


FIGURE: 3 TYPICAL SOIL TEST DATA

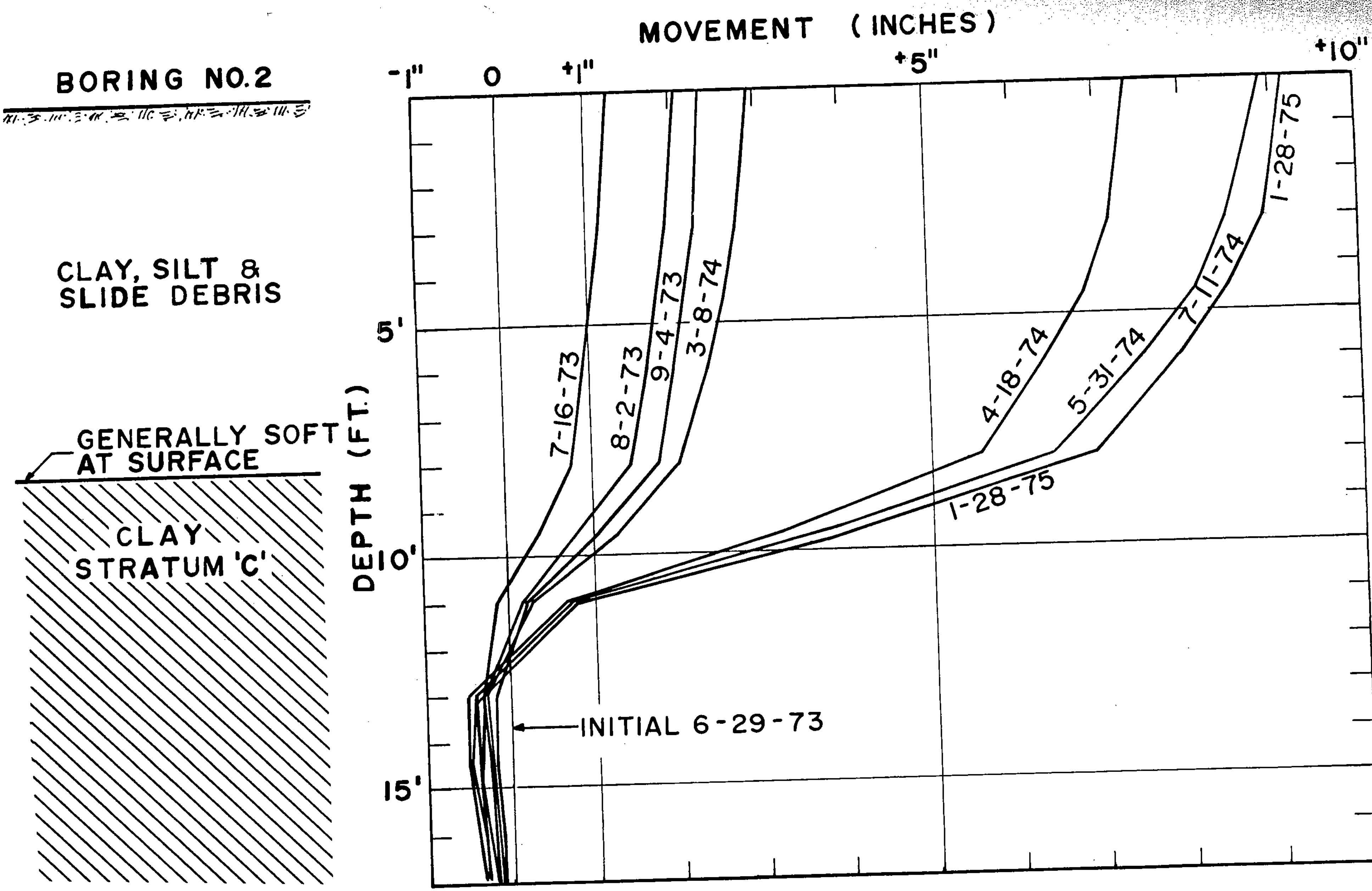


TYPICAL SECTION
SCALE: 1" = 20'



LOCATION PLAN
SCALE: 1" = 80'

FIGURE: 4
SLOPE STABILITY CONDITIONS



TYPICAL SLOPE INCLINOMETER DATA

SLIDE AREA

FIGURE: 5

of the clay - which resulted in the in situ shear strength available at the time of failure. With no cohesion, results indicated a friction angle of $\phi' = 21^{\circ}$ in the clay. It was recognized however that some cohesion may be present in areas where no previous slide occurred. For the design of new structures the shear parameters of $\phi' = 18^{\circ}$ and c' = 50 psf were used.

In summary several factors were seen to have contributed to slide activity:

1) Geologic conditions as found at this location were particularly unstable due to the clay surface sloping toward O Street; sand filled buried ravines in the clay; and the presence of layers and lenses of sand that provide sources of water. These conditions were probably responsible for the pre-existing instability found at this location.

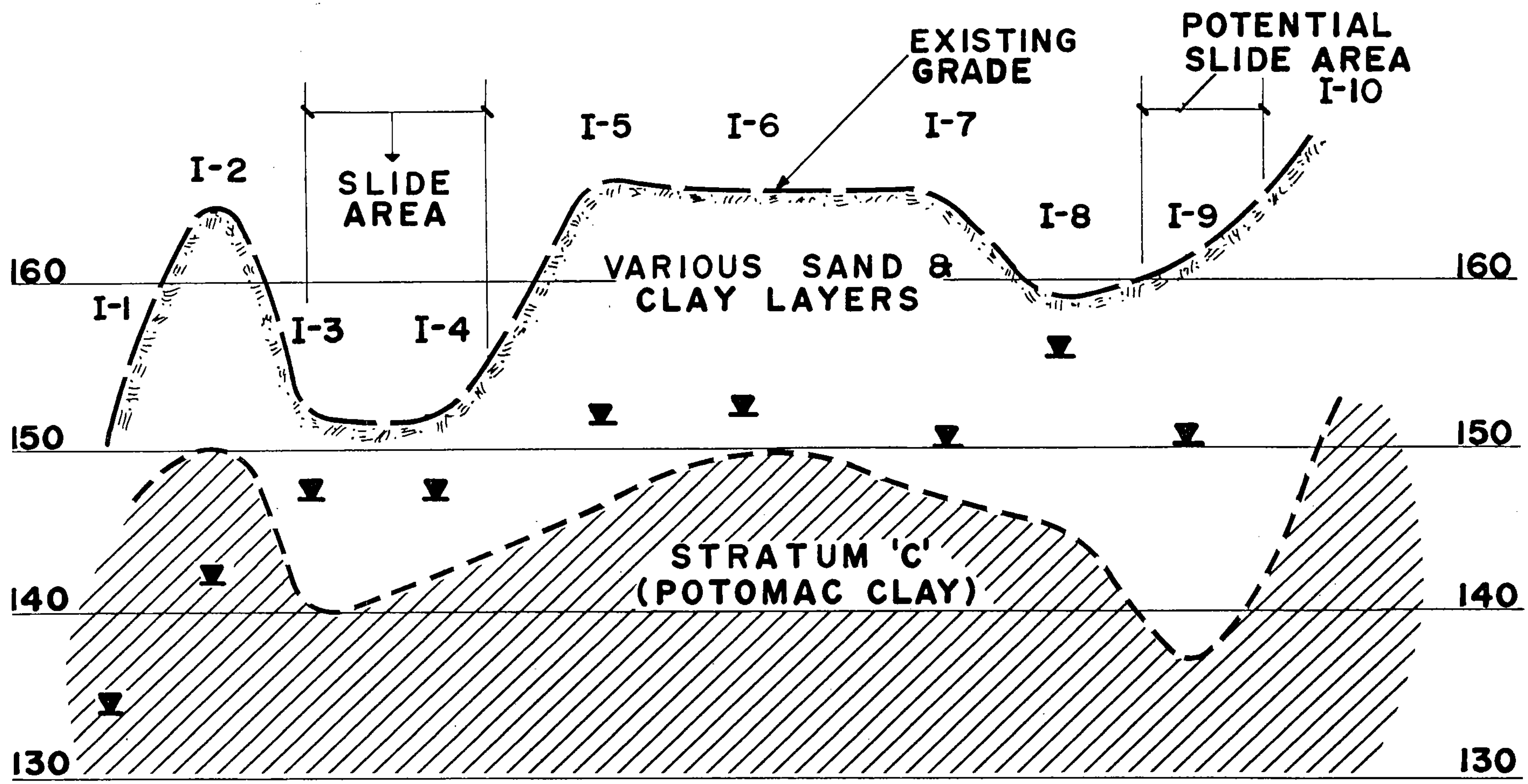
2) Hydrostatic pressures in sand layers reducing the effective friction and resistance to sliding.

3) Environmental changes with construction above the slope and leakage of water sources.

4). Cutting at the bottom of the slope and filling at the top.

An interesting feature of the site was that depressions were discovered in the surface of the lower clay. A section across the site is indicated by Figure 6. The considerable variation in elevation of the clay can be clearly seen with two deep valleys on the north and south side of the site. These locations were also found to be most sensitive to movement, and the north "valley" was in the area of the major instability zone. Apparently water is being collected in these areas on the surface of the lower clay, with evidence of discharge at the bottom of slopes. The water provides a constant lubrication to clay surfaces, reducing the shear strength along potential failure zones. In addition, hydrostatic water pressures develop in sand layers, seams, or pockets, where discharge is blocked due to discontinuity.

The remedial measures were designed considering these features of the slope.



TYPICAL SECTION ALONG HILLSIDE

FIGURE: 6

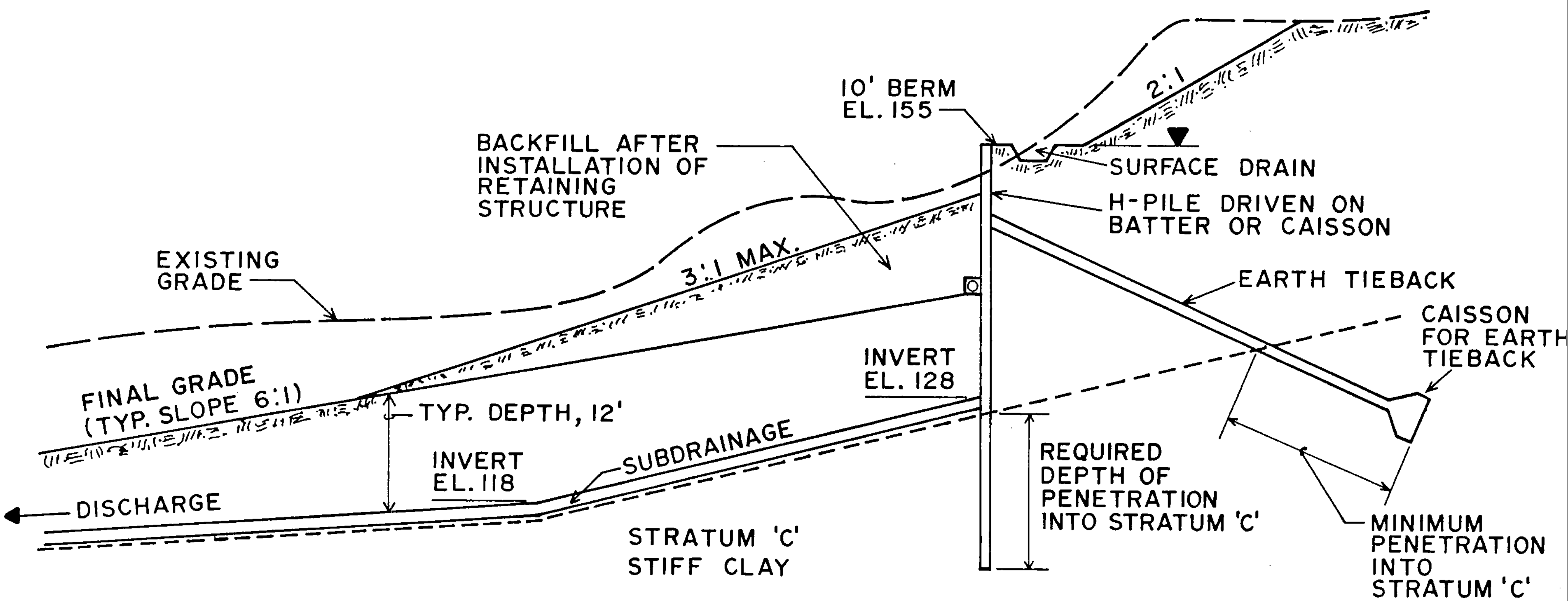


FIGURE: 7
CORRECTIVE MEASURES

TYPICAL DESIGN REQUIREMENTS FOR PROTECTIVE MEASURES

Retaining structures with regrading and drainage were considered the most effective combination for this site.

Corrective measures recommended generally included a retaining structure at the upper elevation of the slope, see Figure 7, to reduce the possibility of failures spreading toward the buildings and to allow the flatter development of slopes downhill, as well as excavation to the desired drainage depth. Typical wall height was about 12 to 15 feet. A drainage line at the bottom of the wall was designed to collect surface water. Installation of a subdrainage system was recommended to collect water from the ravines encountered in the lower clay and to relieve any hydrostatic pressure buildup. Regrading of the downhill slopes to the flattest feasible angle was also part of the remedial measures. It was expected that downhill slopes be graded to 5 horizontal to 1 vertical or flatter in most locations. In addition all surface water was required to be diverted from areas above the site. At the time of this writing a slurry wall is being installed at the site. This wall type was selected in lieu of the originally planned retaining structure using soldier beams and concrete lagging. The design called for a minimum wall penetration of 10 feet into the lower hard clay layer which was not affected by slide activity. The design of the walls was further based on pressures resulting from instability rather than conventional earth pressures. This was based on the consideration that stresses could be caused by unstable soil masses moving along pre-existing slides. Horizontal support was further provided by tiebacks also required to penetrate into the deep clay.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to Andrew White for drafting and to Toni McLaughlin for typing.

REFERENCES CITED

1. U.S. Coast and Geodetic Survey, 1884, Topographical Maps of the District of Columbia: Map No. 40.

2. Obermeier, Stephen, 1978, personal communication, U.S. Geological Survey.

ADDITIONAL COMMENTS - ENGINEERING GEOLOGY OF THE CHESAPEAKE BAY BRIDGES

Carl W. A. Supp, P.E., C.P.G.S.
Consulting Engineer-Geologist
Baltimore, Maryland

ABSTRACT

This paper is a relatively informal commentary intended to supplement the presentation by J.G. Lutz and S.M. Miller titled "Construction of the Chesapeake Bay Bridge - Parallel Span".

In 1947-48 subsurface investigations were completed for the original Bay Bridge, including preparation of two geologic sections extending across the full width of the Bay. The resulting data, augmented by additional studies as outlined by S.M. Miller, were again utilized for the design of the Parallel Span which was completed in 1973.

The salient features of the engineering geology of the site, such as the submerged and bruied former channel of the Susquehanna River, are briefly reviewed. Comments are offered regarding the permanent value, both practical and scientific, of a systematic and well-documented geotechnical investigation for a major engineering project.

INTRODUCTION

In 1947-48, while employed by J.E. Greiner Company, I was responsible for geological studies for the original Bay Bridge. As briefly discussed below, the substantial volume of resulting subsurface data was also found to be highly useful for the design of the Parallel Span. Therefore, Messrs. J.G. Lutz and S.M. Miller have thoughtfully invited me to participate in their presentation.

FIRST CHESAPEAKE BAY BRIDGE (1952)

Prior to 1948, the proposed construction of a Bay crossing was a highly controversial issue. One segment of public opinion vociferously insisted that the service provided by the then-existing Sandy Point - Matapeake Ferry was entirely adequate, and that any new crossing would be both redundant and an economic fiasco. A tunnel crossing had numerous adherents on the basis that it presumably would not obstruct vitally important marine commerce to and from the Port of Baltimore; also, rather widely circulated rumors suggested that construction of a bridge would not be technically or economically feasible because the mud underlining the Bay is virtually "bottomless". And finally, there was substantial support for a high-level bridge crossing.

To provide essential subsurface data for comparative studies of "bridge versus tunnel", and for final design purposes, an extensive test boring program was completed during the period December, 1947 - June, 1948, as follows:

16 Borings - Along and adjacent to the "Tunnel Line", extending in a straight line from the extreme tip of Sandy Point to Kent Island (see Plate 1)

33 Borings - Along the "Bridge Line", curving off Sandy Point adjacent to the existing Ferry Terminal, and thence extending on a tangent (about 4,000 feet south of the "Tunnel Line") to Kent Island (see Plate 2). (The location and the curve in the bridge were dictated by War Department criteria with respect to the main navigation channel - the Sandy Point to Thomas Point Shoal Sailing Course.)

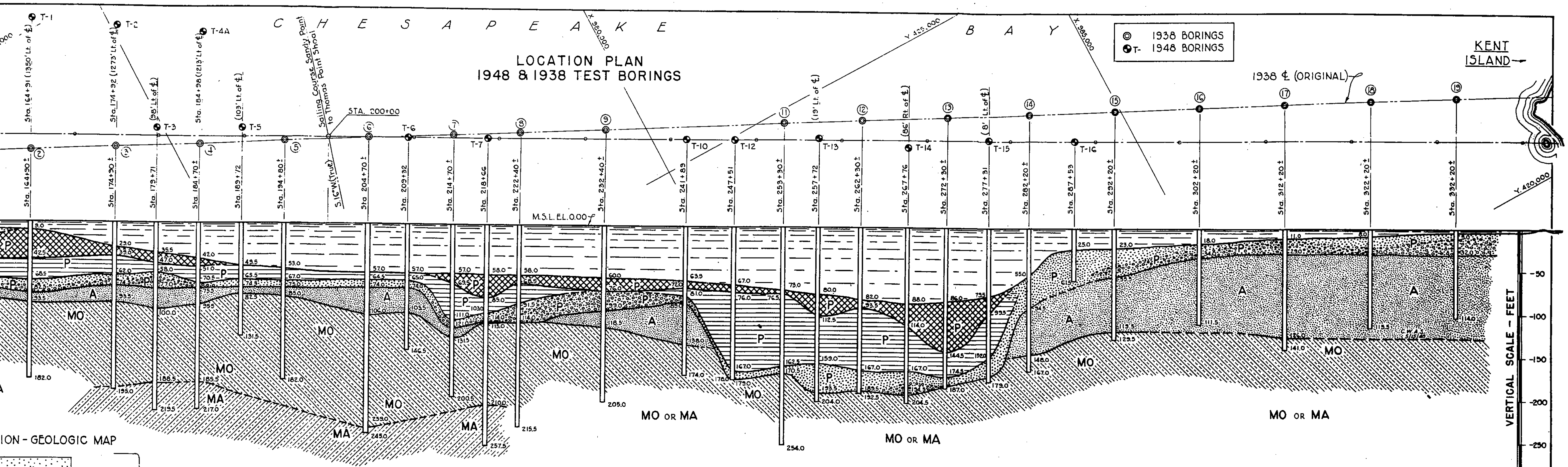
Additionally, some 24 borings had been made in 1938, for a preliminary bridge study, along a line roughly coinciding with the "Tunnel Line" (see <u>Plate 1</u>). The resulting samples were no longer available for examination; but the logs furnished valuable data for the layout and planning of the new program and for development of geologic cross sections as mentioned below.

More detailed discussions of the subsurface investigations have been presented elsewhere by Greiner Co. (1948, 1961) and Supp (1952, 1955, 1957).

During the early months of 1948 J. E. Greiner Company, in collaboration with several other engineering consultants, completed exhaustive studies covering the relative merits, feasibility, and costs of a bridge as compared to a tunnel crossing. In June, 1948, Greiner Company prepared a statement for the Governor, the State Roads Commission, and the State Highway Advisory Council which recommended the bridge alternate for the reasons that it would (a) offer superior traffic serviceability; (b) involve significantly lower initial and maintenance and operating costs; (c) require a shorter construction period; and (d) present less interference to commercial shipping and small craft. The State accepted the recommendation; and in July Greiner Company issued a formal engineering report on the Chesapeake Bay Bridge (Greiner Co., 1948) and was authorized to proceed with final design studies and preparation of Contract Documents.

More or less concurrently with the aforementioned developments, the author was responsible for all geological studies which included:

1. Classification of all boring samples, based upon examination under the microscope and laboratory test data.
2. Development of a geologic cross section along the "Tunnel Line" (<u>Plate 1</u>)



ION - GEOLOGIC MAP

posits, beach sands, sand
swamp deposits in areas
e coast, where the older
d Cretaceous deposits
above sea level. Surficial
errace deposits which
of the remainder of the
g upon the Cretaceous and
rmations, are disregar-
napping.

QUATERNARY

Marys Formation

ptank Formation

vert Formation

munkey Group
d Nanjemoy Formations)

TERTIARY

, Magothy, Matawan,
onmouth Formations

tomac Group
ent, Arundel, and
apsco Formations)

CRETACEOUS

ed from Plate 1, XVI International
s Guidebook 5, "Chesapeake Bay
n, Cooke, and Mansfield.

PRELIMINARY GEOLOGIC SECTION
SANDY POINT TO KENT ISLAND

HORIZONTAL SCALE

500 0 500 1000 1500 2000 2500

FEET

-EXPLANATION-

P — RECENT AND PLEISTOCENE "SEMI-LIQUID" TO VERY SOFT, BLACK SILT AND CLAY. Laboratory analyses of undisturbed samples disclose composition of a typical sample to be: SILT, COARSE TO FINE (55%), CLAY (43%), trace Fine Sand (2%). Material has considerable Organic fraction (est. approx. 15%).

P — RECENT AND PLEISTOCENE SOFT, GRAY, SILT AND CLAY. Laboratory analyses of undisturbed samples indicate that composition and physical characteristics are somewhat variable. SILT fraction predominates and in some samples CLAY fraction drops to 10-15%. Where determined, Organic content ranges from 6-12%.

P — RECENT AND PLEISTOCENE SAND AND/OR GRAVEL DEPOSITS. These deposits form old terraces, now submerged, and the terraces currently in process of being built up. They also occupy former stream channels.

A — AQUIA FORMATION - GLAUCONITIC SAND. Glauconite is a soft green mineral, essentially a hydrous silicate of iron and potassium, originally formed under marine conditions. It gives to the sands, where they are comparatively unweathered, a greenish, greenish blue, or greenish gray color ("Greensand"). In this area, the Aquia has been subjected to more or less weathering, with consequent alteration of some or all of the Glauconite to Iron Oxide (Limonite), causing the characteristic coloration of the sands - yellowish green, greenish brown, reddish brown, mottled and variegated reddish brown, orange, green, gray, etc. Over restricted areas, the Limonite has been dissolved and redeposited, forming thin crusts, ledges, and layers of iron-cemented Sandstone. Occasionally a gray, hard, silica-cemented Sandstone, also purely local in occurrence, is encountered. Sand is predominantly Medium to Fine, with varying amounts of Inorganic Silt and Clay from trace to approx. 20%; occasional trace of Mica.

MO — MONMOUTH FORMATION - GLAUCONITIC SAND. A uniformly Medium to Fine Sand, color generally dark greenish gray, greenish black, or dark brownish or yellowish green. Glauconite is mostly dark green to greenish black, with occasionally some altered to greenish brown, and varies in amount from a trace to approx. 40% of the sand. Inorganic Silt fraction varies from a trace to approx. 15%, with an occasional very slight trace of Mica.

MA — MATAWAN FORMATION - GLAUCONITIC SAND. Very similar in character to the Monmouth. Generally greenish black in color. Usually somewhat finer than Monmouth, and carries less Glauconite in smaller grains, more Silt and clayey material, and usually considerably more Mica.

NOTES - GEOLOGY

① Geologic Section is based primarily on 1948 borings from which samples are available for thorough study. Logs of 1938 borings, from which samples are no longer available, were carefully correlated with the 1948 data and used to extend the section and provide additional detail.

② In this vicinity, the Monmouth and Matawan Formations are quite similar in character. The location of the contact between them as shown is probably somewhat approximate and is based entirely on slight lithologic differences. The line of contact has not been extended beyond the point where identification from samples is reasonably certain. For engineering purposes the slight differences in character between the two formations are considered to be relatively unimportant.

NOTES - GENERAL

① Profile of Bay bottom is based on sonic soundings by J. E. Greiner Co. and has been verified by direct measurements at boring locations.

② All surveys are referred to the Maryland State Coordinate System.

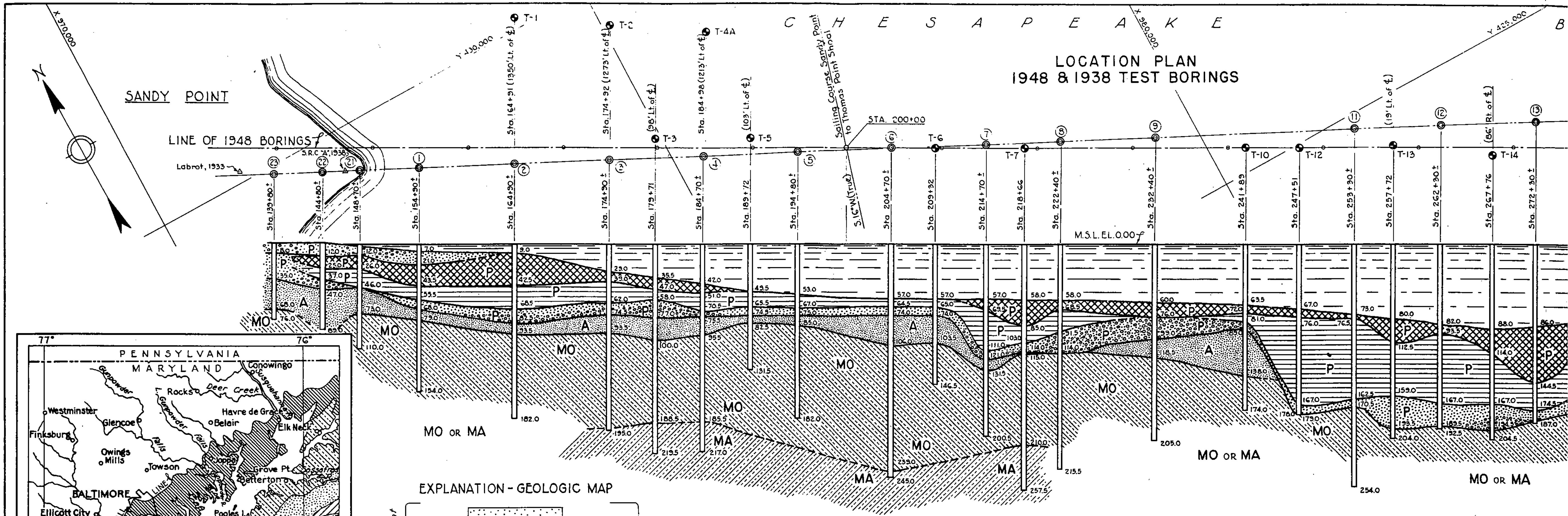
"TUNNEL LINE"

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY CROSSING
SANDY POINT-KENT ISLAND SITE
GEOLOGIC SECTION

SCALES - AS SHOWN — JUNE, 1948

J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MARYLAND

PLATE I



PRELIMINARY GEOLOGIC SECTION
SANDY POINT TO KENT ISLAND

HORIZONTAL SCALE

500 0 500 1000 1500 2000 2500

FEET

-EXPLANATION-

P (cross-hatched) — RECENT AND PLEISTOCENE "SEMI-LIQUID" TO VERY SOFT, BLACK SILT AND CLAY. Laboratory analyses of undisturbed samples disclose composition of a typical sample to be: SILT, COARSE TO FINE (55%), CLAY (43%), trace Fine Sand (2%). Material has considerable Organic fraction (est. approx. 15%).

P (horizontal lines) — RECENT AND PLEISTOCENE SOFT, GRAY, SILT AND CLAY. Laboratory analyses of undisturbed samples indicate that composition and physical characteristics are somewhat variable. SILT fraction predominates and in some samples CLAY fraction drops to 10-15%. Where determined, Organic content ranges from 6-12%.

P (dotted) — RECENT AND PLEISTOCENE SAND AND/OR GRAVEL DEPOSITS. These deposits form old terraces, now submerged, and the terraces currently in process of being built up. They also occupy former stream channels.

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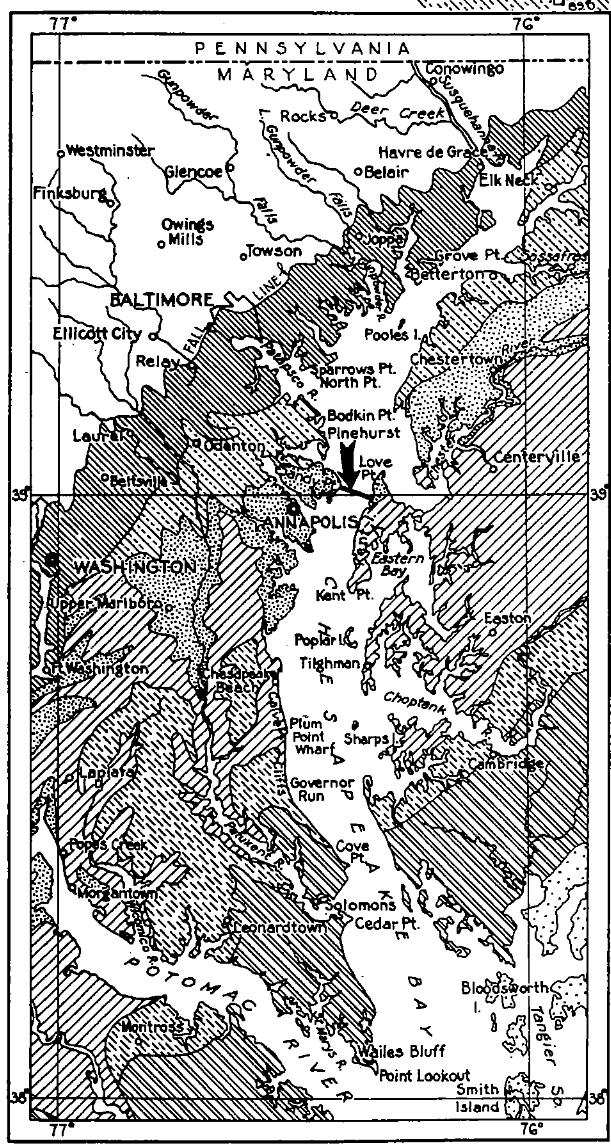


GEOLOGIC MAP
NORTHERN CHESAPEAKE BAY AREA

5 0 5 10 15 20 25 30 35 MILES

EXPLANATION - GEOLOGIC MAP

QUATERNARY

Pleistocene and Recent: Terrace deposits, beach sands, sand dunes, and swamp deposits in areas bordering the coast, where the older Tertiary and Cretaceous deposits do not rise above sea level. Surficial Pleistocene terrace deposits which cover most of the remainder of the area, resting upon the Cretaceous and Tertiary formations, are disregarded in the mapping.

TERTIARY

Miocene: St. Marys Formation; Choptank Formation; Calvert Formation

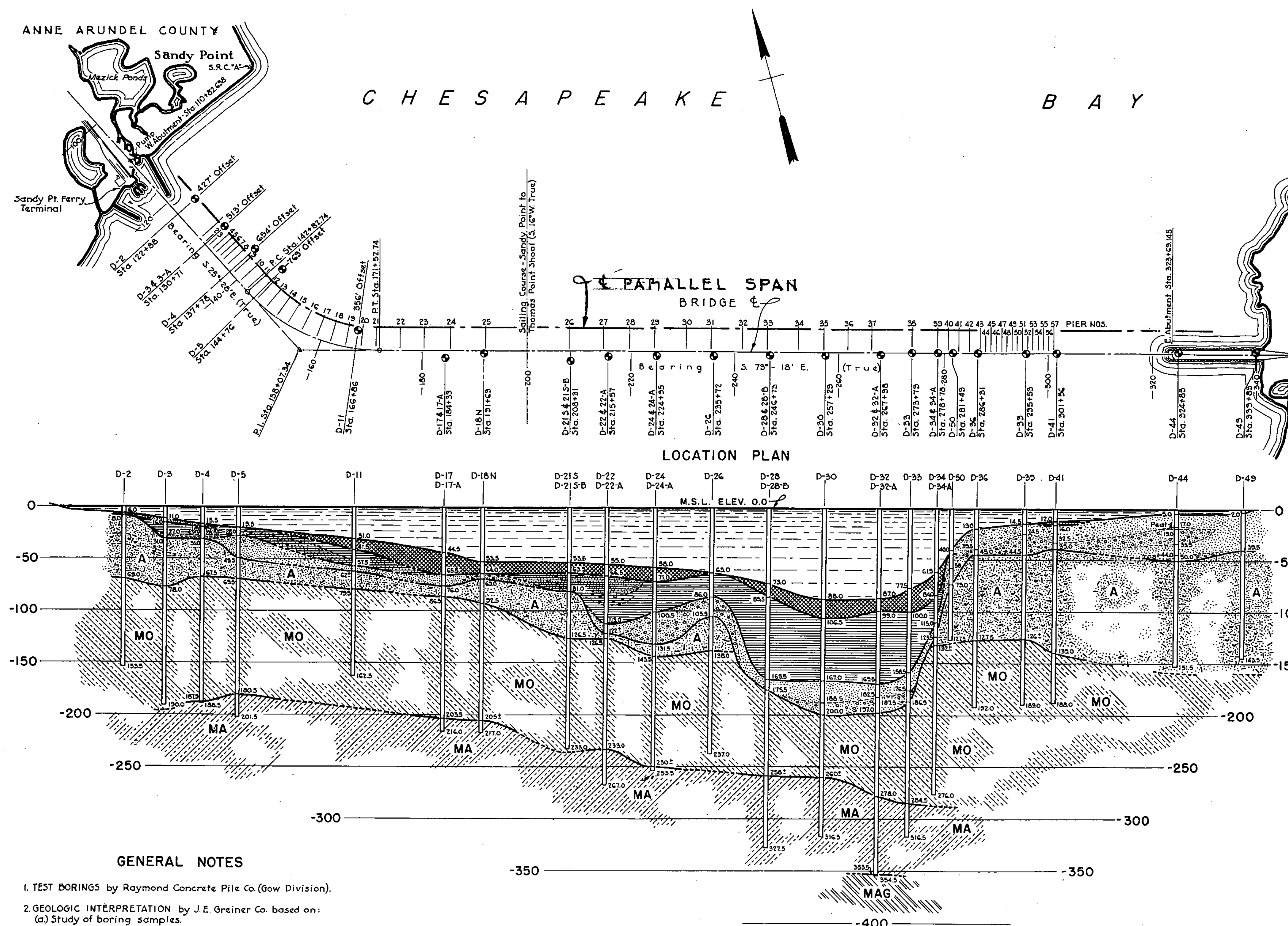
Eocene: Pamunkey Group (*Aquia and Nanjemoy Formations*)

CRETACEOUS

Upper Cretaceous: Raritan, Magothy, Matawan, and Monmouth Formations

Lower Cretaceous: Potomac Group (*Patuxent, Arundel, and Patapsco Formations*)

NOTE: Geologic Map adapted from Plate 1, XVI International Geological Congress Guidebook 5, "Chesapeake Bay Region", Stephenson, Cooke, and Mansfield.



GENERAL NOTES

1. TEST BORINGS by Raymond Concrete Pile Co. (Gow Division).
2. GEOLOGIC INTERPRETATION by J. E. Greiner Co. based on:
   (a) Study of boring samples.
   (b) Correlation with published data by Maryland State Dep't of Mines, Geology and Water Resources, U.S. Geological Survey, and others; with previously accumulated data pertaining to geology of vicinity of site in files of J. E. Greiner Co.; and with data derived from Soil Mechanics studies.

LEGEND

QUATERNARY — *Pleistocene & Recent*

- "SEMI-LIQUID" BLACK & GRAY ORGANIC SILT
- SOFT GRAY ORGANIC SILT
- SANDY SILT
- SHELL
- SAND GRAVEL
- CLAY

TERTIARY — *Eocene*

- A — GLAUCONITIC SAND ("GREENSAND")
- A — GREENSAND-SHELL MARL

AQUIA FORMATION - Silty Glauconitic Sand, usually reddish brown or mottled dark green & reddish brown. Sometimes contains fossil material ranging from minute to large shells and calcareous fragments (Greensand-Shell Marl). Sand is often locally cemented by iron oxide or silica to form thin crusts of Sandstone.

CRETACEOUS — *Upper Cretaceous*

- MO — MONMOUTH FORMATION - Glauconitic Sand, uniformly Medium to Fine, dark green, greenish gray to black, quite compact.
- MA — MATAWAN FORMATION - Glauconitic Sand, somewhat finer than Monmouth, with more Silt & Clay, greenish gray to black, compact.
- MAG — MAGOTHY FORMATION - Coarse to Medium bluish gray Sand, no Glauconite, Sands and Clays containing Lignite & Pyrite.

GEOLOGY BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: E.G. Ruark
IN CHARGE: R. Archibald

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND

GEOLOGIC SECTION AT SITE

SCALE: AS SHOWN DATE: JANUARY, 1949 CONTRACT

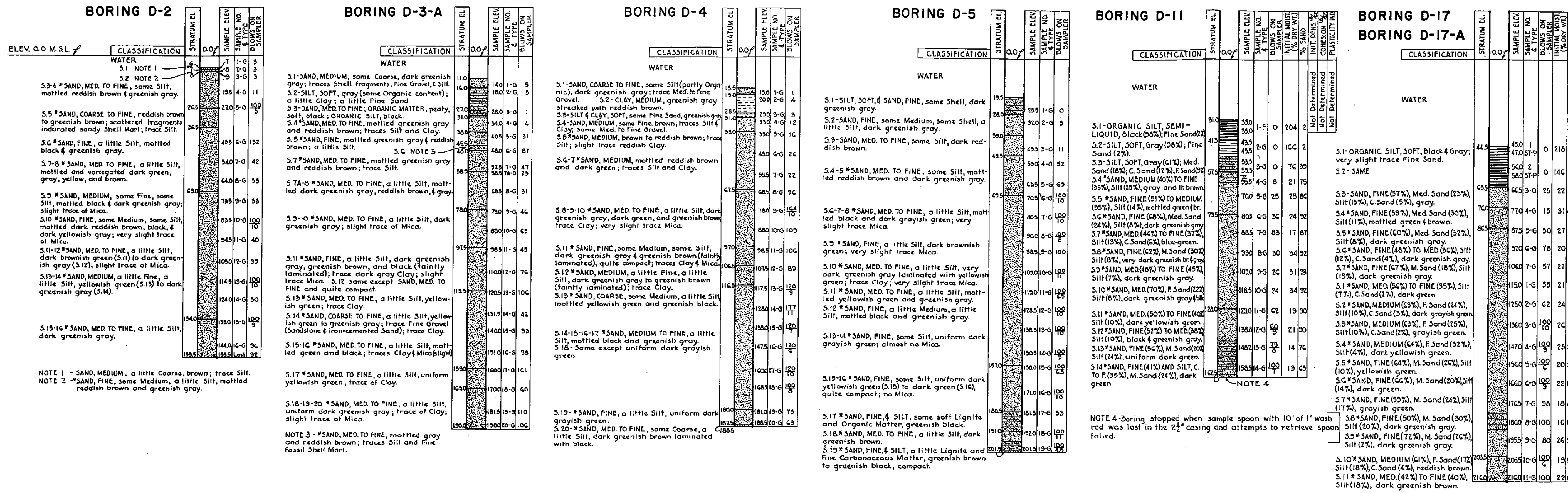
J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MD.

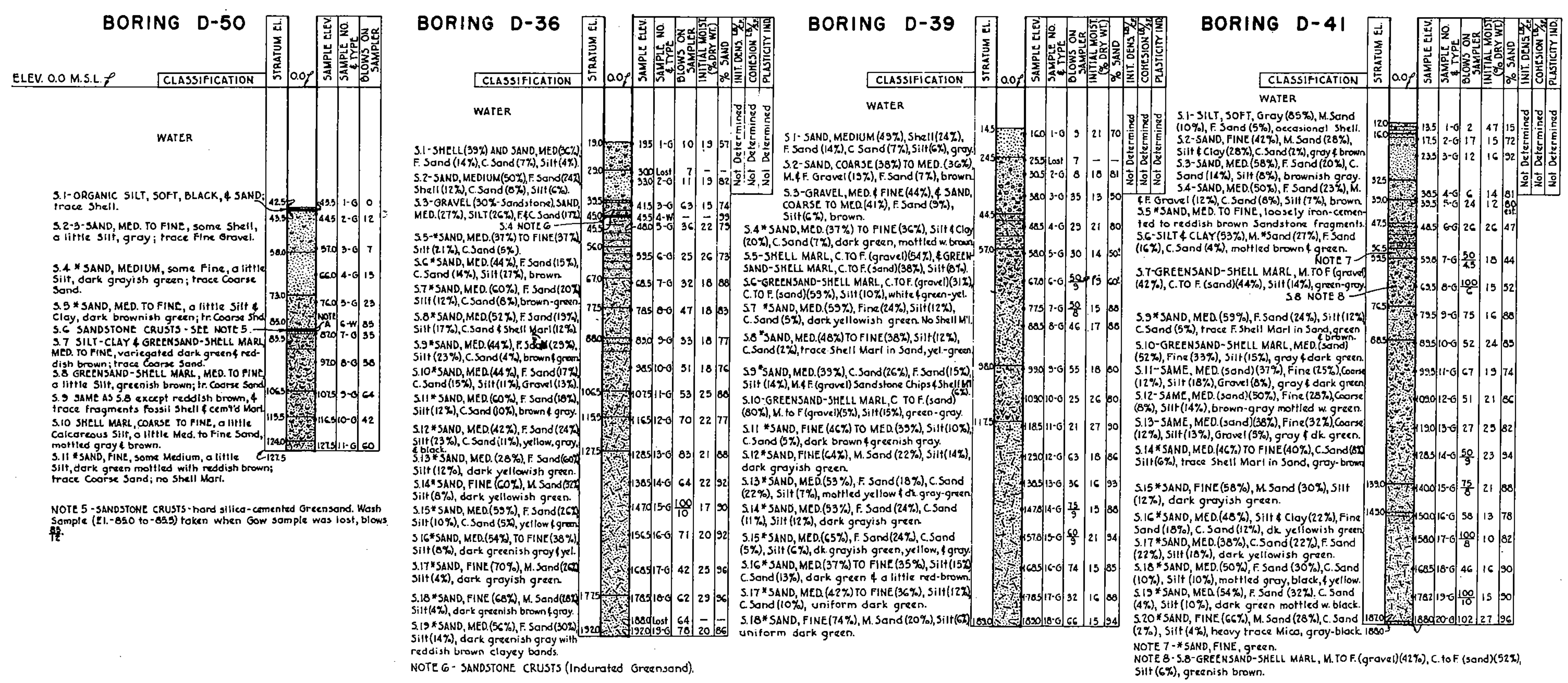
DWG. NO. 181
SHEET 7 OF 7
C-508-

REVISIONS

PLATE 2

DESIGNED BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: E.G. Ruark
IN CHARGE: R. Archibald





Legend

TYPES OF SAMPLES - CORRESPONDING SYMBOLS

- G - Ordinary GOW Type Dry Sample, 1½" I.D. Split Spoon.
- ST - Undisturbed Sample, SHELBY TUBE, Seamless Thin Wall, 3" I.D.
- STP - Same as ST, except using PISTON Type Sampler.
- F - FLAP VALVE Sample, 2½".
- W - WASH Sample.
- OP - Ordinary Gow Type, except using 1" OPEN END PIPE.

435 / 455 2-G — Continuous Sample, 2' interval, using ordinary Split Spoon, described above.

435 / 455 2 ST — Continuous Undisturbed Sample, 2' interval, using Shelby Tube, described above.

BLACK ORGANIC SILT — GRAY SILT & INORGANIC SILT

SAND — GRAVEL

GLAUCONITIC SAND, *SAND ("Greensand") See Below — SHELL & SHELL MARL

CLAY — PEAT & ORGANIC MATTER

SANDSTONE CRUSTS (Generally Locally Indurated Sand). — Symbols are used both singly & in combinations.

BLOWS ON SAMPLER - Sampling spoon used to recover Gow type samples was driven by 140 lb. weight falling 30 in. on all "D" Borings except D-3-A, D-4, & D-5, on which 128 lb. weight was used. Whole numbers in column record number of blows required to obtain standard 12 in. spoon penetration. Fractional numbers record number of blows (Numerator) over penetration in inches (Denominator).

*SAND, or GLAUCONITIC SAND ("Greensand") indicates sand having an appreciable content of Glauconite, a soft mineral, essentially a silicate of iron and potash. When fresh and unaltered, it imparts a typical bluish or greenish color to sands of which it is a constituent, giving rise to common term "Greensand". When more or less weathered and oxidized, these sands are typically reddish brown, mottled and variegated brown and green, etc. Iron oxide derived from weathering of Glauconite sometimes cements the sands, forming Sandstone crusts and ledges, usually quite local in extent.

% CLAY, where stated, is based on results of laboratory tests which determine %(of dry wt.) of Clay Size particles present. This amount may not necessarily be entirely composed of true Clay minerals.

General Notes

For LOCATION PLAN of borings see Dwg. No. 137.

TEST BORINGS were made by Gow Division, Raymond Concrete Pile Co., 1947-1948, under supervision of J.E. Greiner Co. For transcript of original boring logs see Dwg. No. 138.

CLASSIFICATION OF MATERIALS was made by J.E. Greiner Co., based on:

(a). Mechanical Sieve Analyses and other standard laboratory tests performed at Soil Mechanics Laboratory, University of Maryland, except for Borings Nos. D-2, D-3-A, D-4, D-5, and D-50, for which no such tests were made. Classifications of material from the latter borings were based on (b), following.

(b). Study (by J.E. Greiner Co.) of all samples (except Shelby Tube samples) with low power microscope, to determine geologic formation and mineral composition; also approximate grain size distribution in samples for which mechanical sieve analyses were not made. Approximate quantitative notation used in latter cases, and based on visual examination, is as follows:

Predominating Component (Usually 50% or more) - in capitals, eg."SA
35-50% - "and" — 10-20% - "a little"
20-35% - "some" — 1-10% - "trace" (1% - "very slight"; 1-5% - "slight"; 5-10% - "trace" or "heavy trace

(c) M.I.T. STANDARD, shown below, is used in all cases to describe grain sizes. Grain size curves for all mechanically sieved sample were prepared, from which particle size distribution was determined. For samples classified by visual examination only, application of standard is necessarily approximate.

M.I.T. STANDARD

Gravel			Sand			Silt			Clay
C	M	F	C	M	F	C	M	F	

60 20 6 2 .6 .2 .06 .02 .006 .002
Grain Size in Millimeters

REVISIONS

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.

CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND

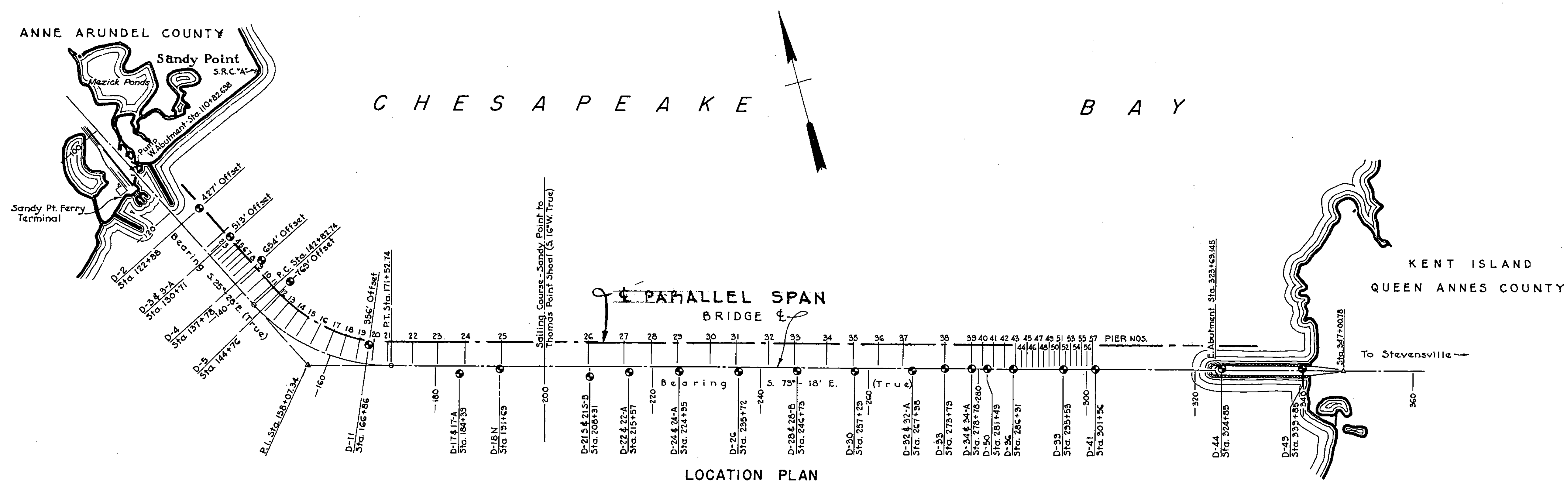
LABORATORY & TECHNICAL DATA "D" BORINGS

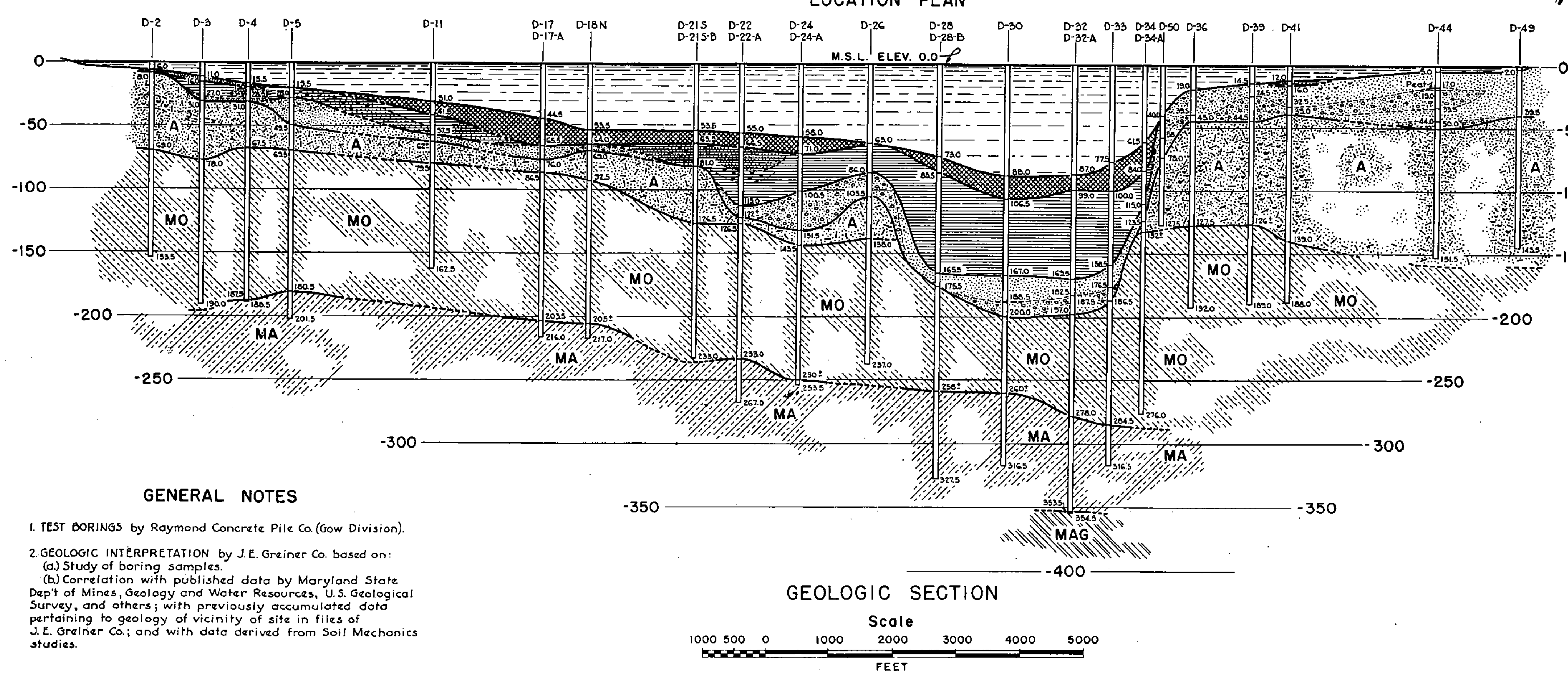
SCALE: VERT. 1" = 25' DATE: JANUARY, 1949 CONTRACT

J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MD.

DWG. NO. 139
SHEET 4 OF 7
C-508-

PLATE 3





GENERAL NOTES

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2. GEOLOGIC INTERPRETATION by J. E. Greiner Co. based on:
(a) Study of boring samples.
(b) Correlation with published data by Maryland State Dep't of Mines, Geology and Water Resources, U.S. Geological Survey, and others; with previously accumulated data pertaining to geology of vicinity of site in files of J. E. Greiner Co.; and with data derived from Soil Mechanics studies.

LEGEND

QUATERNARY — Pleistocene & Recent
- "SEMI-LIQUID" BLACK & GRAY ORGANIC SILT
- SOFT GRAY ORGANIC SILT
- SANDY SILT
- SHELL
- SAND GRAVEL
- CLAY

TERTIARY — Eocene
- GLAUCONITIC SAND ("GREENSAND")
- GREENSAND-SHELL MARL

AQUIA FORMATION - Silty Glauconitic Sand, usually reddish brown or mottled dark green & reddish brown. Sometimes contains fossil material ranging from minute to large shells and calcareous fragments (Greensand-Shell Marl). Sand is often locally cemented by iron oxide or silica to form thin crusts of Sandstone.

CRETACEOUS — Upper Cretaceous
- MO — MONMOUTH FORMATION - Glauconitic Sand, uniformly Medium to Fine, dark green, greenish gray to black, quite compact.
- MA — MATAWAN FORMATION - Glauconitic Sand, somewhat finer than Monmouth, with more Silt & Clay, greenish gray to black, compact.
- MAG — MAGOTHY FORMATION - Coarse to Medium bluish gray Sand, no Glauconite, Sands and Clays containing Lignite & Pyrite.

GEOLOGY BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: E.G. Ruark
IN CHARGE: R. Archibald

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND

GEOLOGIC SECTION AT SITE

SCALE: AS SHOWN   DATE: JANUARY, 1949   CONTRACT

J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MD.

DWG. NO. 181
SHEET 7 OF 7
C-508-

REVISIONS

PLATE 2

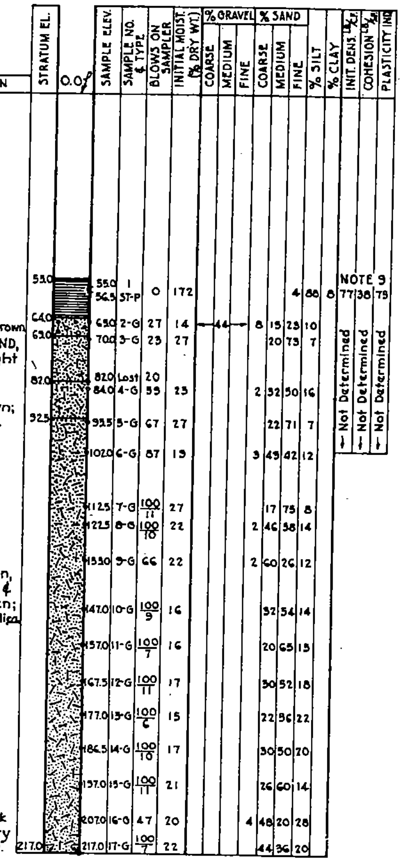
DESIGNED BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: F.H. Burghardt
IN CHARGE: R. Archibald

BORING D-18N

ELEV. 0.0 M.S.L. — CLASSIFICATION

WATER

S.1- SOFT BLACK ORGANIC SILT
S.2-SAND & GRAVEL, brown
S.3-GLAUCONITIC SAND, greenish brown; slight trace of Mica.
S.4-SAME, mottled green & reddish brown; slight trace of Mica.
S.5-S.15-SAME, green, dark greenish gray, & dark yellowish green; very slight trace of Mica.
S.16-S.17-SAME, dark greenish brown; very slight trace of Mica.

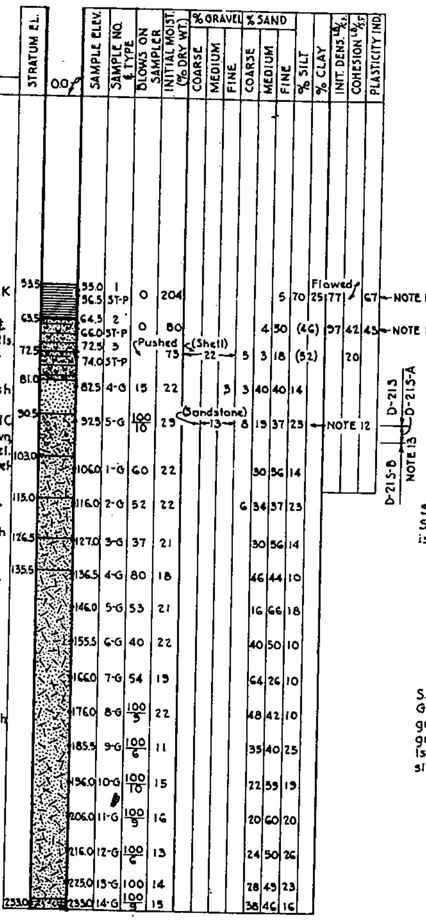


NOTE 9-S.1-Liquid Limit 116; Plastic Limit 41; Spec. Grav. 2.74; Init. Voids Ratio 5.05.

BORING D-21 S
BORING D-21 S-B

WATER

S.1-SEMI-LIQUID BLACK ORGANIC SILT
S.2-SOFT GRAY SILT & SAND; occasional Shells.
S.3-GRAY SILT, numerous large Shells.
S.4-SAND, light greenish gray.
S.5(D-21S)-GLAUCONITIC SAND, dark reddish brown; a little Sandstone Gravel.
S.1(D-21S-B) SAME, dark yellowish green.
S.2-SAME, dark green & reddish brown.
S.3-SAME, dark greenish gray; occasional thin crusts of Sandstone (cemented sand); tr. Mica.
S.4-S.13-SAME, dark green, bluish green, greenish brown, yellowish green, etc.; occasional thin SILT-CLAY seams & slight traces of Mica.
S.14-SAME, dark green thinly banded with black.

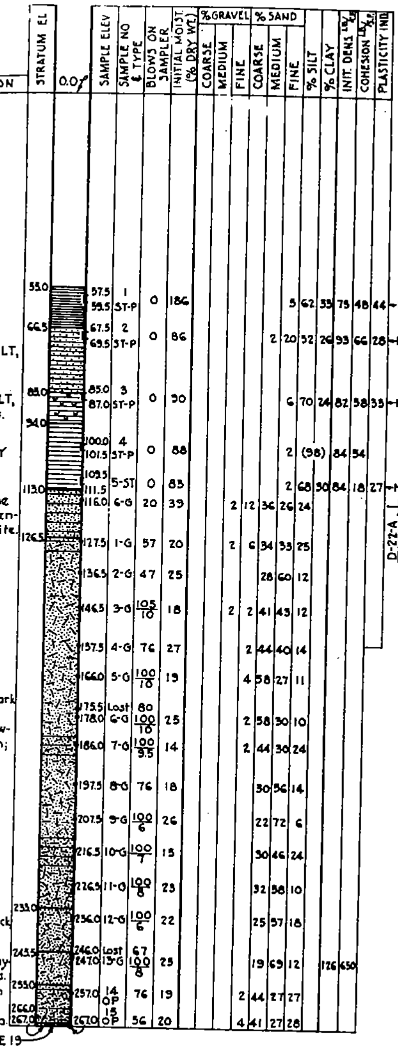


FOR NOTES pertaining to above borings, see below log of BORING D-22-A.

BORING D-22
BORING D-22-A

WATER

S.1-SOFT BLACK ORGANIC SILT
S.2-SOFT GRAY SILT, some Sand.
S.3-SOFT GRAY SILT, occasional Shells.
S.4-S.5-SOFT GRAY SILT.
S.6(D-22)-SAND, some Silt, dark green & greenish black; trace of Lignite.
S.1 to S.11(D-22-A) GLAUCONITIC SAND, dark green, brownish green, greenish black, & yellowish green; Silt as shown; slight trace of Mica.
S.12-SAME, greenish black; trace of Mica.
S.13-SAME, very dark grayish green; trace of Mica.
S.14-SAME, dark greenish brown banded with thin SILT-CLAY seams; tr. of Mica.
S.15-NOTE 19



	SAMPLE NO.	INITIAL VOIDS RATIO	SPECIFIC GRAV.	LIQUID LIMIT	PLASTIC LIMIT
NOTE 14	1	5.48	2.72	85	45
NOTE 15	2	2.44	2.75	45	25
NOTE 16	3	2.88	2.68	66	33
NOTE 17	5	2.63	2.67	60	33

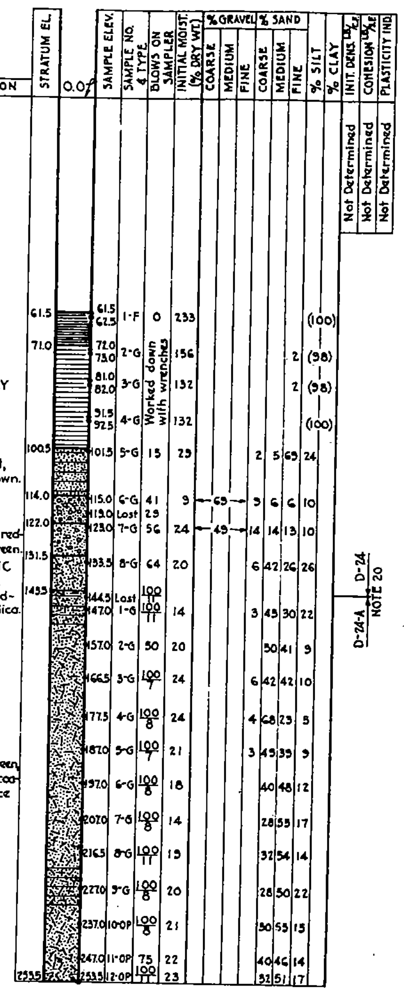
NOTE 18-BORING D-22 was abandoned at El.-116.0 to avoid loss of casing in storm. BORING D-22-A furnishes data from El.-116.0 to -267.0.
NOTE 19-S.15-GLAUCONITIC SAND, some Silt and Clay, greenish black; slight trace of Mica.

NOTE 10-S.1-Spec. Grav. 2.66; Init. Voids Ratio 5.55; Liquid Limit 107; Plastic Limit 40.
NOTE 11-S.2- " " 2.68; " " " 2.10; " " 66; " " 23.
NOTE 12-S.5(D-21S)-SANDSTONE is locally cemented Glauconitic Sand.
NOTE 13-BORING D-21S was abandoned at El.-92.5 when casing was lost in storm. BORING D-21S-A was lost at El.-57.5 under similar conditions. BORING D-21S-B furnishes data from El.-103.0 to -233.0.

BORING D-24
BORING D-24-A

WATER

S.1-SEMI-LIQUID SOFT BLACK SILT.
S.2-S.4-SOFT GRAY SILT.
S.5-SAND, some Silt, dark gray & red-brown.
S.6-GRAVEL, some Sand, gray.
S.7-SAND & GRAVEL, reddish brown & dark green.
S.8(D-24)-GLAUCONITIC SAND, some Silt, dark green mottled with reddish brown; slight tr. Mica.
S.1 to S.11(D-24-A) SAME, Silt as shown, black, dark grayish green, greenish brown, etc., occasional very slight trace of Mica.
S.12-SAME, greenish black; trace of Mica.

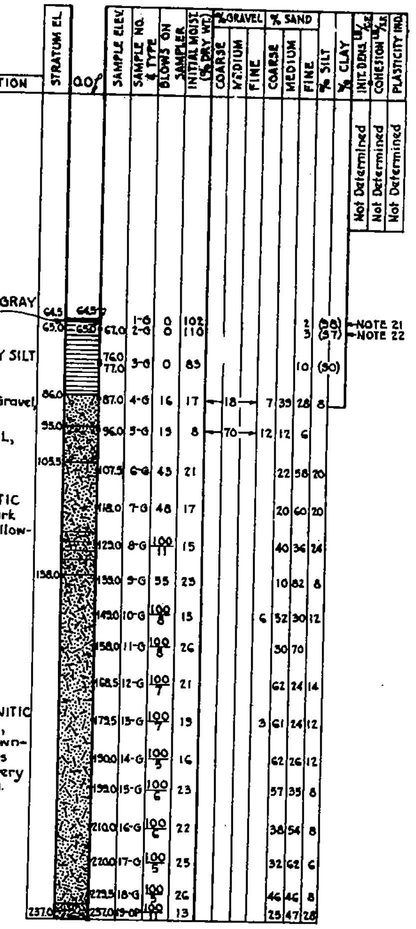


NOTE 20-BORING D-24 was abandoned at El.-141.5 when casing broke during storm and had to be withdrawn. BORING D-24-A furnishes data from El.-141.5 to -253.5.

BORING D-26

WATER

S.1-SEMI-LIQUID GRAY ORGANIC SILT.
S.2-S.3-SOFT GRAY SILT
S.4-SAND, a little Gravel, greenish brown.
S.5-SAND & GRAVEL, brown.
S.6-S.8-GLAUCONITIC SAND, some Silt, dark brownish green & yellow-
S.9 to S.15-GLAUCONITIC SAND, greenish gray, greenish black, brownish green, etc.; Silt as noted; occasional very slight trace of Mica.

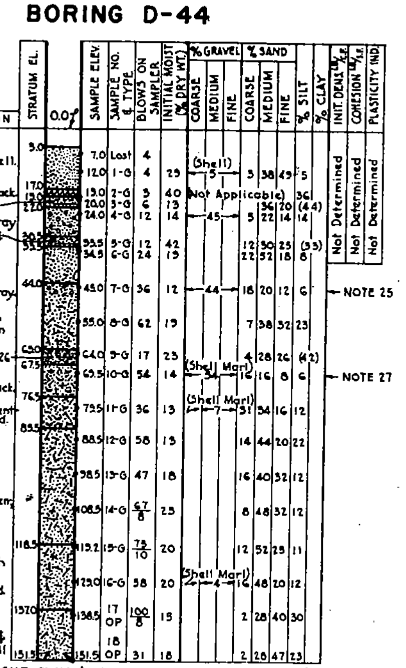


NOTE 21-S.1-Sample interval El.-64.5 to -65.0. Obtained with split spoon.
NOTE 22-S.2- " " -65.0 to -67.0. " " " "

BORING D-44

WATER

S.1-SAND, gray; trace Shell.
S.2-SOFT PEAT, brown-black
S.3-NOTE 23
S.4-SAND & GRAVEL, lt. gray
S.5-NOTE 24
S.6-SAND, dark gray.
S.7-SAND & GRAVEL, dk. gray.
S.8-GLAUCONITIC SAND, mottled reddish brown & dark green.
S.9-NOTE 26
S.10-GREENSAND-SHELL MARL, gray, green, & black.
S.11-SAME, slightly cemented.
S.12-S.14-GLAUCONITIC SAND, reddish brown mottled with dark green; trace Shell Marl.
S.15-S.16-SAME, reddish brown; a little Shell Marl, partly cemented.
S.17-S.18-SAME, light gray, reddish brown, & dark green; trace Shell Marl.

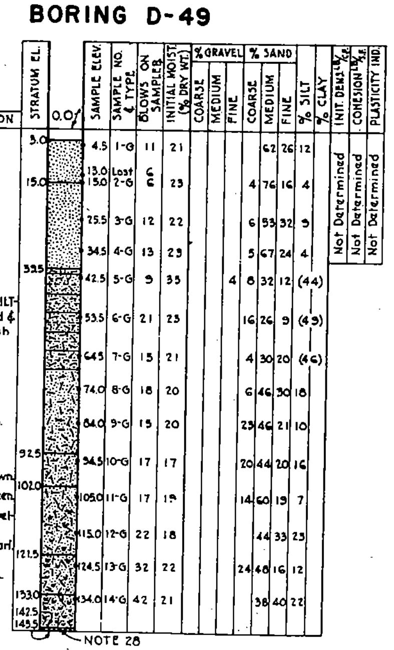


NOTE 23-S.3-SILT, CLAY, & SAND, light gray.
NOTE 24-S.5-STIFF SILT, CLAY, & SAND, dark gray.
NOTE 25-S.7-Sample probably not representative because of small size.
NOTE 26-S.9-GLAUCONITIC SILT, CLAY, & SAND, green mottled with brown.
NOTE 27-Gravel sizes composed of fragments of cemented Shell Marl.

BORING D-49

WATER

S.1-S.2-SAND, brown.
S.3-S.4-SAND, reddish brown.
S.5-S.7-GLAUCONITIC SILTY CLAY & SAND, variegated & mottled dk. green & reddish brown.
S.8-S.9-GLAUCONITIC SAND, variegated dark green & reddish brown.
S.10-SAME, dark green mottled with reddish brown.
S.11-SAME, brownish green.
S.12-SAME, variegated yellowish green, dk. green, & gray; some Fine Shell Marl.
S.13-GREENSAND-SHELL MARL, brown & gray, occasionally cemented.
S.14-SHELL MARL, some Greensand, light gray.



NOTE 28-SANDSTONE CRUSTS. Washed and chopped with chopping bit, no sample obtainable.

FOR GENERAL NOTES AND LEGEND REFER TO DWG. NO. 139.

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND
LABORATORY & TECHNICAL DATA
"D" BORINGS

SCALE: VERT. 1"=20' DATE: JANUARY, 1949 CONTRACT

J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MD.

DWG. NO. 140
SHEET 5 OF 7

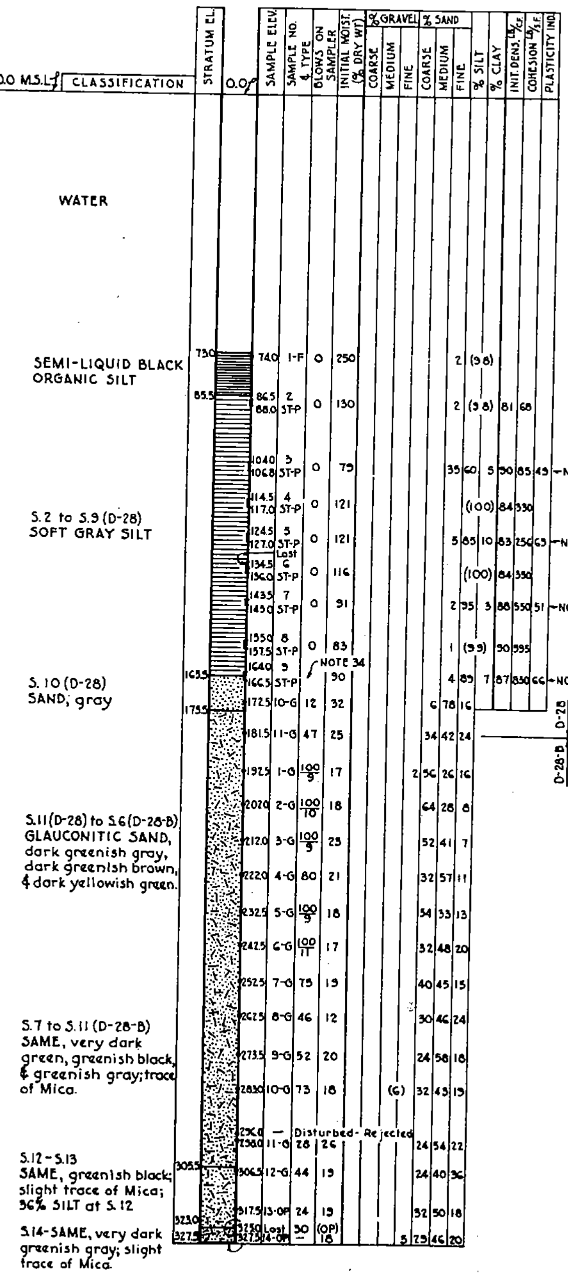
REVISIONS

C-508-

PLATE 4

DESIGNED BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: E.H. Burghardt
IN CHARGE: R. Archibald

BORING D-28
BORING D-28-B

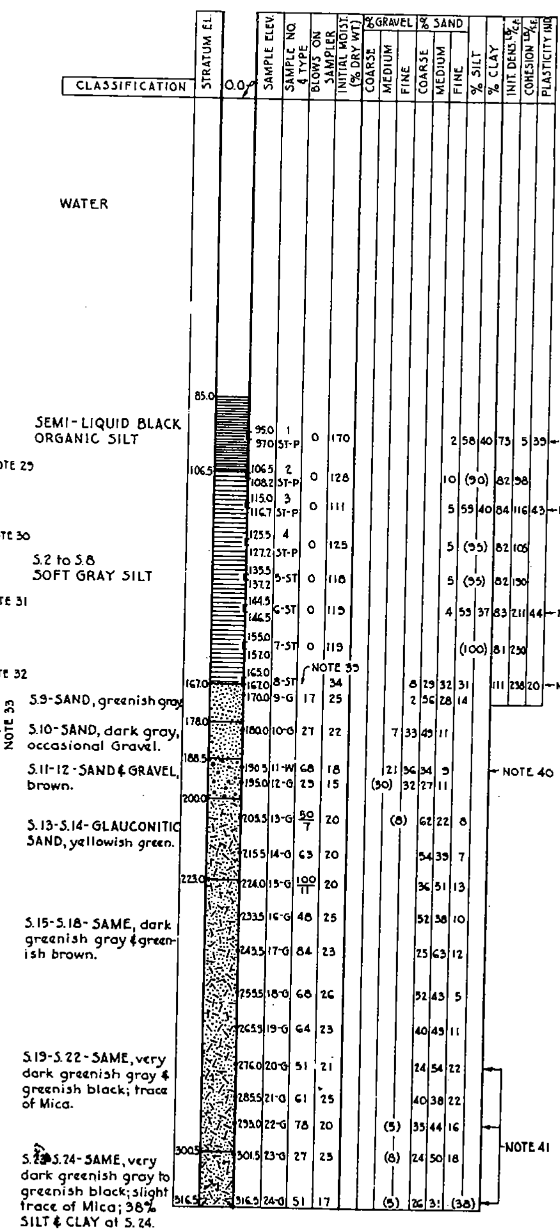


	SAMPLE NO.	INITIAL VOIDS RATIO	SPECIFIC GRAV.	LIQUID LIMIT	PLASTIC LIMIT
NOTE 29	3	2.45	2.78	81	32
" 30	5	3.52	2.72	114	45
" 31	7	2.65	2.69	92	41
" 32	9	2.70	2.71	112	46

NOTE 33-BORING D-28 went to El.-181.5 when 4" casing broke and 2½" casing bent during strong S.W. storm. BORING D-28-A was lost at EL-155.0 when 4" casing parted. BORING D-28-B furnishes data from El.-181.5 to -327.5.

NOTE 34-Sampler went down under weight of tools in 2 seconds for 20", under weight of tools plus 350 lb. for 10" Sample is not entirely representative, since it includes 2 types of material.

BORING D-30



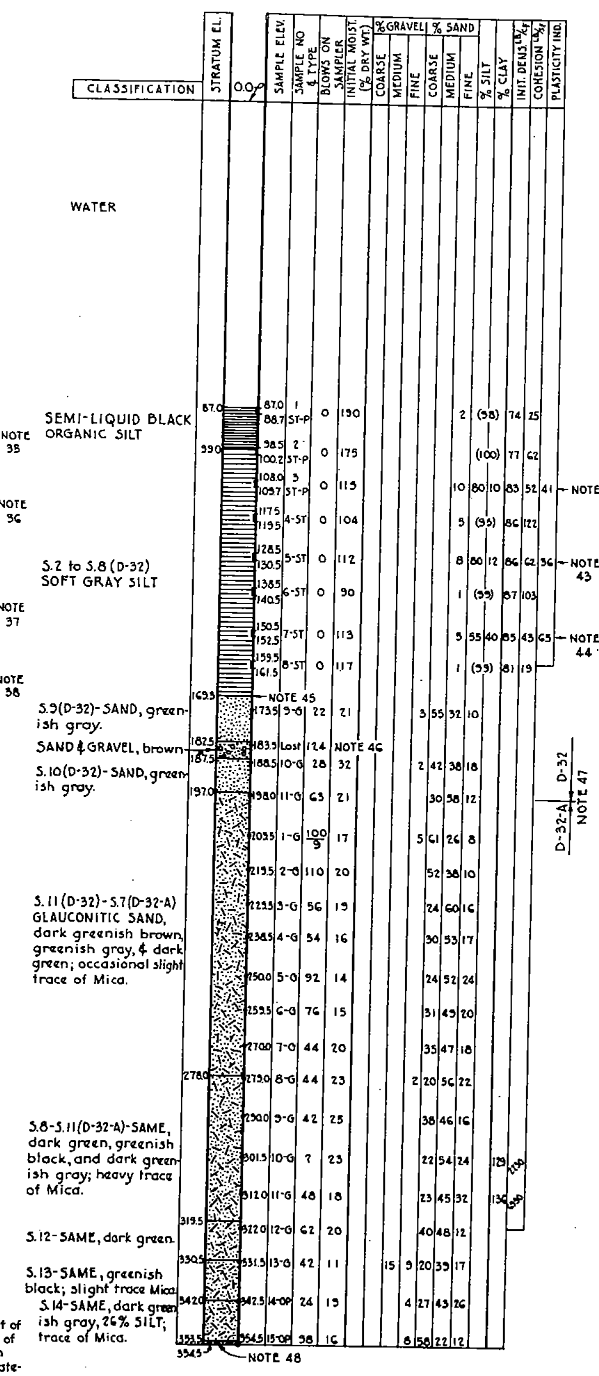
	SAMPLE NO.	INITIAL VOIDS RATIO	SPECIFIC GRAV.	LIQUID LIMIT	PLASTIC LIMIT
NOTE 35	1	5.28	2.72	82	43
" 36	3	3.23	2.70	78	35
" 37	6	3.50	2.74	86	42
" 38	8	1.04	2.71	41	21

NOTE 39- Sampler went down under weight of tools for 19" in 6 seconds, under weight of tools plus 500 lb. for 1", and pulled down with block and tackle for 4". Last 4" of material, probably sand, was not recovered. Sample may not be entirely representative, since it probably includes 2 types of material.

NOTE 40- Lost Gow sample at El.-190.5, blows 68/12. Wash sample was taken at approximately same elevation.

NOTE 41- Compressive Strength (As Received):
Sample 20 - 4,000 lb/sq. ft.
" 22 - 1,300 "
" 24 - 16,000 "

BORING D-32
BORING D-32-A



	SAMPLE NO.	INITIAL VOIDS RATIO	SPECIFIC GRAV.	LIQUID LIMIT	PLASTIC LIMIT
NOTE 42	3	3.41	2.68	74	33
" 43	5	3.15	2.70	67	31
" 44	7	3.31	2.76	104	39

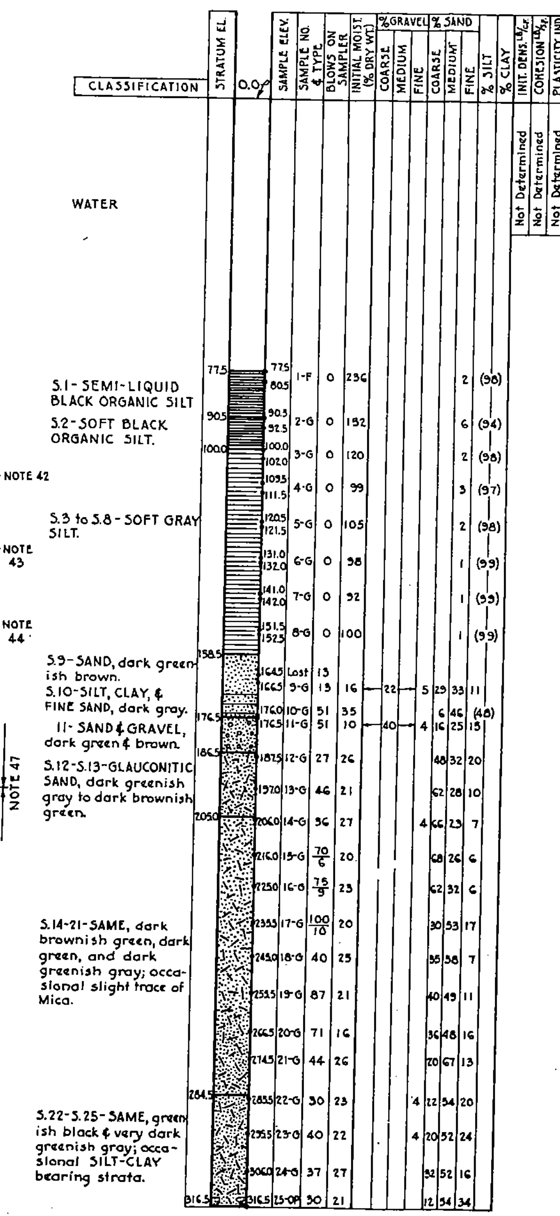
NOTE 45-ST sample was attempted at EL-169.5. Sand was encountered and no sample could be recovered.

NOTE 46-Failed to recover Gow sample at El.-183.5 because piece of gravel lodged in end of spoon. Drive record is probably not representative.

NOTE 47-BORING D-32 was abandoned at El.-198.0 to save casing in storm. BORING D-32-A furnishes data from El.-197.0 to -354.5.

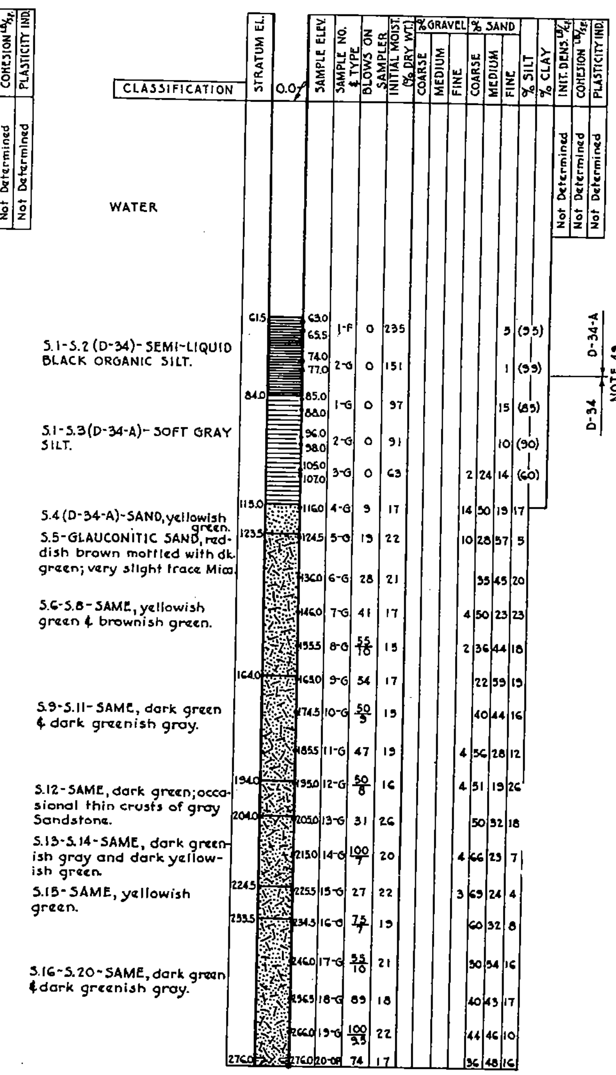
NOTE 48-S.15-SAND, greenish gray to purplish gray; NO Glauconite. Probably top of Magothy formation.

BORING D-33



FOR GENERAL NOTES AND LEGEND REFER TO DWG. NO. 139.

BORING D-34
BORING D-34-A



NOTE 49-BORING D-34 was lost at EL-77.0 because of severe storm. BORING D-34-A furnishes data from EL-77.0 to -276.0.

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND
LABORATORY & TECHNICAL DATA
"D" BORINGS

SCALE: VERT. 1" = 25' ..... DATE: JANUARY, 1949 ..... CONTRACT

J. E. GREINER COMPANY
CONSULTING ENGINEERS
BALTIMORE, MD.

DWG. NO. 141
SHEET 6 OF 7
C-508-

REVISIONS

PLATE 5

DESIGNED BY: C.W.A. Supp
DRAWN BY: C.W.A. Supp
CHECKED BY: E.G. Ruark
IN CHARGE: R. Archibald

BORING D-2

ELEV. 0.0 M.S.L.

CLASSIFICATION — WATER

S.1 NOTE 1
S.2 NOTE 2

S.3-4 *SAND, MED. TO FINE, some Silt, mottled reddish brown & greenish gray.

S.5 *SAND, COARSE TO FINE, reddish brown to greenish brown; scattered fragments indurated sandy Shell Marl; trace Silt.

S.6 *SAND, FINE, a little Silt, mottled black & greenish gray.

S.7-8 *SAND, MED. TO FINE, a little Silt, mottled and variegated dark green, gray, yellow, and brown.

S.9 *SAND, MEDIUM, some Fine, some Silt, mottled black & dark greenish gray; slight trace of Mica.
S.10 *SAND, FINE, some Medium, some Silt, mottled dark reddish brown, black, & dark yellowish gray; very slight trace of Mica.
S.11-12 *SAND, MED. TO FINE, a little Silt, dark brownish green (S.11) to dark greenish gray (S.12); slight trace of Mica.
S.13-14 *SAND, MEDIUM, a little Fine, a little Silt, yellowish green (S.13) to dark greenish gray (S.14).

S.15-16 *SAND, MED. TO FINE, a little Silt, dark greenish gray.

NOTE 1 - SAND, MEDIUM, a little Coarse, brown; trace Silt.
NOTE 2 - *SAND, FINE, some Medium, a little Silt, mottled reddish brown and greenish gray.

BORING D-3-A

CLASSIFICATION — WATER

S.1-SAND, MEDIUM, some Coarse, dark greenish gray; traces Shell fragments, Fine Gravel, & Silt.
S.2-SILT, SOFT, gray (some Organic content); a little Clay; a little Fine Sand.
S.3-SAND, MED. TO FINE; ORGANIC MATTER, peaty, soft, black; ORGANIC SILT, black.
S.4 *SAND, MED. TO FINE, mottled greenish gray and reddish brown; traces Silt and Clay.
S.5 *SAND, FINE, mottled greenish gray & reddish brown; a little Silt.
S.6 NOTE 3
S.7 *SAND, MED. TO FINE, mottled greenish gray and reddish brown; trace Silt.

S.7A-8 *SAND, MED. TO FINE, a little Silt, mottled dark greenish gray, reddish brown, & gray.

S.9-10 *SAND, MED. TO FINE, a little Silt, dark greenish gray; slight trace of Mica.

S.11 *SAND, FINE, a little Silt, dark greenish gray, greenish brown, and black (faintly laminated); trace dark gray Clay; slight trace Mica. S.12 same except SAND, MED. TO FINE and quite compact.
S.13 *SAND, MED. TO FINE, a little Silt, yellowish green; trace Clay.
S.14 *SAND, COARSE TO FINE, a little Silt, yellowish green to greenish gray; trace Fine Gravel (Sandstone & iron-cemented Sand); trace Clay.
S.15-16 *SAND, MED. TO FINE, a little Silt, mottled green and black; traces Clay & Mica (slight).

S.17 *SAND, MED. TO FINE, a little Silt, uniform yellowish green; trace of Clay.

S.18-19-20 *SAND, MED. TO FINE, a little Silt, uniform dark greenish gray; trace of Clay; slight trace of Mica.

NOTE 3 - *SAND, MED. TO FINE, mottled gray and reddish brown; traces Silt and Fine Fossil Shell Marl.

BORING D-4

CLASSIFICATION — WATER

S.1-SAND, COARSE TO FINE, some Silt (partly Organic), dark greenish gray; trace Med. to Fine Gravel. S.2-CLAY, MEDIUM, greenish gray streaked with reddish brown.
S.3-SILT & CLAY, SOFT, some Fine Sand, greenish gray.
S.4-SAND, MEDIUM, some Fine, brown; traces Silt & Clay; some Med. to Fine Gravel.
S.5 *SAND, MEDIUM, brown to reddish brown; trace Silt; slight trace reddish Clay.

S.6-7 *SAND, MEDIUM, mottled reddish brown and dark green; traces Silt and Clay.

S.8-9-10 *SAND, MED. TO FINE, a little Silt, dark greenish gray, dark green, and greenish brown; trace Clay; very slight trace Mica.

S.11 *SAND, FINE, some Medium, some Silt, dark greenish gray & greenish brown (faintly laminated), quite compact; traces Clay & Mica.
S.12 *SAND, MEDIUM, a little Fine, a little Silt, dark greenish gray to greenish brown (faintly laminated); trace Clay.
S.13 *SAND, COARSE, some Medium, a little Silt, mottled yellowish green and greenish black.

S.14-15-16-17 *SAND, MEDIUM TO FINE, a little Silt, mottled black and greenish gray.
S.18 - Same except uniform dark grayish green.

S.19 - *SAND, FINE, a little Silt, uniform dark grayish green.
S.20 - *SAND, MED. TO FINE, some Coarse, a little Silt, dark greenish brown laminated with black.

BORING D-5

CLASSIFICATION — WATER

S.1-SILT, SOFT, & SAND, FINE, some Shell, dark greenish gray.
S.2-SAND, FINE, some Medium, some Shell, a little Silt, dark greenish gray.
S.3-SAND, MED. TO FINE, some Silt, dark reddish brown.

S.4-5 *SAND, MED. TO FINE, some Silt, mottled reddish brown and dark greenish gray.

S.6-7-8 *SAND, MED. TO FINE, a little Silt, mottled black and dark grayish green; very slight trace Mica.

S.9 *SAND, FINE, a little Silt, dark brownish green; very slight trace Mica.

S.10 *SAND, MED. TO FINE, a little Silt, very dark greenish gray laminated with yellowish green; trace Clay; very slight trace Mica.
S.11 *SAND, MED. TO FINE, a little Silt, mottled yellowish green and greenish gray.
S.12 *SAND, FINE, a little Medium, a little Silt, mottled black and greenish gray.

S.13-14 *SAND, FINE, some Silt, uniform dark grayish green; almost no Mica.

S.15-16 *SAND, FINE, some Silt, uniform dark yellowish green (S.15) to dark green (S.16), quite compact; no Mica.

S.17 *SAND, FINE, & SILT, some soft Lignite and Organic Matter, greenish black.
S.18 *SAND, MED. TO FINE, a little Silt, dark greenish brown.
S.19 *SAND, FINE, & SILT, a little Lignite and fine Carbonaceous Matter, greenish brown to greenish black, compact.

BORING D-11

CLASSIFICATION — WATER

S.1-ORGANIC SILT, SEMI-LIQUID, Black (98%); Fine Sand (2%).
S.2-SILT, SOFT, Gray (98%); Fine Sand (2%).
S.3-SILT, SOFT, Gray (61%); Med. Sand (18%); C. Sand (12%); F. Sand (9%).
S.4 *SAND, MEDIUM (40%) TO FINE (35%), Silt (15%), gray and lt. brown.
S.5 *SAND, FINE (51%) TO MEDIUM (35%), Silt (14%), mottled green & br.
S.6 *SAND, FINE (68%), Med. Sand (24%), Silt (8%), dark greenish gray.
S.7 *SAND, MED. (44%) TO FINE (37%), Silt (13%), C. Sand (6%), blue-green.
S.8 *SAND, FINE (62%), M. Sand (30%), Silt (8%), very dark greenish brown & gray.
S.9 *SAND, MED. (48%) TO FINE (45%), Silt (7%), dark greenish gray.
S.10 *SAND, MED. (70%), F. Sand (22%), Silt (8%), dark greenish gray & black.
S.11 *SAND, MED. (50%) TO FINE (40%), Silt (10%), dark yellowish green.
S.12 *SAND, FINE (52%) TO MED. (36%), Silt (12%), black & greenish gray.
S.13 *SAND, FINE (56%), M. Sand (20%), Silt (24%), uniform dark green.
S.14 *SAND, FINE (41%) AND SILT, C. TO F. (35%), M. Sand (24%), dark green.

NOTE 4

NOTE 4 - Boring stopped when sample spoon with 10' of 1" wash rod was lost in the 2½" casing and attempts to retrieve spoon failed.

BORING D-17
BORING D-17-A

CLASSIFICATION — WATER

S.1-ORGANIC SILT, SOFT, Black & Gray; very slight trace Fine Sand.
S.2-SAME

S.3-SAND, FINE (57%), Med. Sand (23%), Silt (15%), C. Sand (5%), gray.
S.4 *SAND, FINE (59%), Med. Sand (30%), Silt (11%), mottled green & brown.
S.5 *SAND, FINE (60%), Med. Sand (32%), Silt (8%), dark greenish gray.
S.6 *SAND, FINE (48%) TO MED. (36%), Silt (12%), C. Sand (4%), dark greenish gray.
S.7 *SAND, FINE (67%), M. Sand (18%), Silt (15%), dark greenish gray.
S.1 *SAND, MED. (56%) TO FINE (35%), Silt (7%), C. Sand (2%), dark green.
S.2 *SAND, MEDIUM (63%), F. Sand (24%), Silt (10%), C. Sand (3%), dark grayish green.
S.3 *SAND, MEDIUM (63%), F. Sand (25%), Silt (10%), C. Sand (2%), grayish green.
S.4 *SAND, MEDIUM (64%), F. Sand (32%), Silt (4%), dark yellowish green.
S.5 *SAND, FINE (64%), M. Sand (26%), Silt (10%), yellowish green.
S.6 *SAND, FINE (66%), M. Sand (20%), Silt (14%), dark green.
S.7 *SAND, FINE (59%), M. Sand (24%), Silt (17%), grayish green.
S.8 *SAND, FINE (50%), M. Sand (30%), Silt (20%), dark greenish gray.
S.9 *SAND, FINE (72%), M. Sand (26%), Silt (2%), dark greenish gray.
S.10 *SAND, MEDIUM (61%), F. Sand (17%), Silt (18%), C. Sand (4%), reddish brown.
S.11 *SAND, MED. (42%) TO FINE (40%), Silt (18%), dark greenish brown.

BORING D-17-A — BORING D-17 — 106.0

BORING D-50

ELEV. 0.0 M.S.L.

CLASSIFICATION — WATER

S.1-ORGANIC SILT, SOFT, BLACK, & SAND; trace Shell.

S.2-3-SAND, MED. TO FINE, some Shell, a little Silt, gray; trace Fine Gravel.

S.4 *SAND, MEDIUM, some Fine, a little Silt, dark grayish green; trace Coarse Sand.
S.5 *SAND, MED. TO FINE, a little Silt & Clay, dark brownish green; tr. Coarse Sand.
S.6 SANDSTONE CRUSTS - SEE NOTE 5.
S.7 SILT-CLAY & GREENSAND-SHELL MARL, MED. TO FINE, variegated dark green & reddish brown; trace Coarse Sand.
S.8 GREENSAND-SHELL MARL, MED. TO FINE, a little Silt, greenish brown; tr. Coarse Sand.
S.9 SAME AS S.8 except reddish brown, & trace fragments fossil Shell & cem'td Marl.
S.10 SHELL MARL, COARSE TO FINE, a little Calcareous Silt, a little Med. to Fine Sand, mottled gray & brown.
S.11 *SAND, FINE, some Medium, a little Silt, dark green mottled with reddish brown; trace Coarse Sand; no Shell Marl.

NOTE 5 - SANDSTONE CRUSTS - hard silica-cemented Greensand. Wash Sample (El. -85.0 to -85.5) taken when Gow sample was lost, blows 82.

BORING D-36

CLASSIFICATION — WATER

S.1-SHELL (35%) AND SAND, MED. (30%), F. Sand (14%), C. Sand (7%), Silt (4%).
S.2-SAND, MEDIUM (50%), F. Sand (24%), Shell (12%), C. Sand (8%), Silt (6%).
S.3-GRAVEL (30%-Sandstone), SAND, MED. (27%), Silt (26%), F. & C. Sand (17%).
S.4 NOTE 6
S.5 *SAND, MED. (37%) TO FINE (37%), Silt (21%), C. Sand (5%).
S.6 *SAND, MED. (44%), F. Sand (15%), C. Sand (14%), Silt (27%), brown.
S.7 *SAND, MED. (60%), F. Sand (21%), Silt (12%), C. Sand (8%), brown-green.
S.8 *SAND, MED. (52%), F. Sand (19%), Silt (17%), C. Sand & Shell Marl (12%).
S.9 *SAND, MED. (44%), F. Sand (29%), Silt (23%), C. Sand (4%), brown & green.
S.10 *SAND, MED. (44%), F. Sand (39%), C. Sand (5%), Silt (11%), green (13%).
S.11 *SAND, MED. (60%), F. Sand (18%), Silt (12%), C. Sand (10%), brown & gray.
S.12 *SAND, MED. (42%), F. Sand (24%), Silt (23%), C. Sand (11%), yellow, gray.
S.13 *SAND, MED. (26%), F. Sand (60%), Silt (12%), dark yellowish green.
S.14 *SAND, FINE (60%), M. Sand (27%), Silt (8%), dark yellowish green.
S.15 *SAND, MED. (55%), F. Sand (32%), Silt (10%), C. Sand (5%), yellow & green.
S.16 *SAND, MED. (54%), TO FINE (38%), Silt (8%), dark greenish gray & yel.
S.17 *SAND, FINE (70%), M. Sand (26%), Silt (4%), dark grayish green.
S.18 *SAND, FINE (68%), M. Sand (28%), Silt (4%), dark greenish brown & gray.
S.19 *SAND, MED. (56%), F. Sand (30%), Silt (14%), dark greenish gray with reddish brown clayey bands.

NOTE 6 - SANDSTONE CRUSTS (Indurated Greensand).

BORING D-39

CLASSIFICATION — WATER

S.1-SAND, MEDIUM (49%), Shell (24%), F. Sand (14%), C. Sand (7%), Silt (6%), gray.
S.2-SAND, COARSE (38%) TO MED. (36%), M. & F. Gravel (15%), F. Sand (7%), brown.
S.3-GRAVEL, MED. & FINE (44%), & SAND, COARSE TO MED. (41%), F. Sand (9%), Silt (6%), brown.
S.4 *SAND, MED. (37%) TO FINE (36%), Silt & Clay (20%), C. Sand (7%), dark green, mottled w. brown.
S.5-SHELL MARL, C. TO F. (gravel) (54%), & GREENSAND-SHELL MARL, C. TO F. (sand) (38%), Silt (8%).
S.6-GREENSAND-SHELL MARL, C. TO F. (gravel) (31%), C. TO F. (sand) (59%), Silt (10%), white & green-yel.
S.7 *SAND, MED. (55%), Fine (24%), Silt (12%), C. Sand (5%), dark yellowish green. No Shell M'l.
S.8 *SAND, MED. (48%) TO FINE (38%), Silt (12%), C. Sand (2%), trace Shell Marl in Sand, yel.-green.
S.9 *SAND, MED. (39%), C. Sand (26%), F. Sand (15%), Silt (14%), M. & F. (gravel) Sandstone Chips & Shell (6%).
S.10-GREENSAND-SHELL MARL, C. TO F. (sand) (80%), M. to F. (gravel) (5%), Silt (15%), green-gray.
S.11 *SAND, FINE (46%) TO MED. (35%), Silt (10%), C. Sand (5%), dark brown & greenish gray.
S.12 *SAND, FINE (64%), M. Sand (22%), Silt (14%), dark grayish green.
S.13 *SAND, MED. (55%), F. Sand (18%), C. Sand (22%), Silt (7%), mottled yellow & dk. gray-green.
S.14 *SAND, MED. (53%), F. Sand (24%), C. Sand (11%), Silt (12%), dark grayish green.
S.15 *SAND, MED. (65%), F. Sand (24%), C. Sand (5%), Silt (6%), dk. grayish green, yellow, & gray.
S.16 *SAND, MED. (37%) TO FINE (35%), Silt (15%), C. Sand (13%), dark green & a little red-brown.
S.17 *SAND, MED. (42%) TO FINE (36%), M. Sand (13%), C. Sand (10%), uniform dark green.
S.18 *SAND, FINE (74%), M. Sand (20%), Silt (6%), uniform dark green.

BORING D-41

CLASSIFICATION — WATER

S.1-SILT, SOFT, Gray (65%), M. Sand (10%), F. Sand (5%), occasional Shell.
S.2-SAND, FINE (41%), M. Sand (28%), Silt & Clay (28%), C. Sand (3%), gray & brown.
S.3-SAND, MED. (58%), F. Sand (20%), C. Sand (14%), Silt (8%), brownish gray.
S.4-SAND, MED. (50%), F. Sand (23%), M. & F. Gravel (12%), C. Sand (8%), Silt (7%), brown.
S.5 *SAND, MED. TO FINE, loosely iron-cemented to reddish brown Sandstone fragments.
S.6-SILT & CLAY (53%), M. Sand (27%), F. Sand (16%), C. Sand (4%), mottled brown & green.
NOTE 7
S.7-GREENSAND-SHELL MARL, M. TO F. (gravel) (42%), C. TO F. (sand) (44%), Silt (14%), green-gray.
S.8 NOTE 8
S.9 *SAND, MED. (59%), F. Sand (24%), Silt (12%), C. Sand (5%), trace F. Shell Marl in Sand, green & brown.
S.10-GREENSAND-SHELL MARL, MED. (sand) (52%), Fine (33%), Silt (15%), gray & dark green.
S.11-SAME, MED. (sand) (37%), Fine (25%), Coarse (12%), Silt (18%), Gravel (8%), gray & dark green.
S.12-SAME, MED. (sand) (50%), Fine (28%), Coarse (8%), Silt (14%), brown-gray mottled w. green.
S.13-SAME, MED. (sand) (38%), Fine (32%), Coarse (12%), Silt (13%), Gravel (5%), gray & dk. green.
S.14 *SAND, MED. (47%) TO FINE (40%), C. Sand (6%), Silt (6%), trace Shell Marl in Sand, gray-brown.
S.15 *SAND, FINE (58%), M. Sand (30%), Silt (12%), dark grayish green.
S.16 *SAND, MED. (48%), Silt & Clay (22%), Fine Sand (16%), C. Sand (12%), dk. yellowish green.
S.17 *SAND, MED. (38%), C. Sand (22%), F. Sand (22%), Silt (18%), dark yellowish green.
S.18 *SAND, MED. (50%), F. Sand (30%), C. Sand (10%), Silt (10%), mottled gray, black, & yellow.
S.19 *SAND, MED. (54%), F. Sand (32%), C. Sand (4%), Silt (10%), dark green mottled w. black.
S.20 *SAND, FINE (66%), M. Sand (28%), C. Sand (2%), Silt (4%), heavy trace Mica, gray-black.

NOTE 7 - *SAND, FINE, green.
NOTE 8 - S.8-GREENSAND-SHELL MARL, M. TO F. (gravel) (42%), C. to F. (sand) (52%), Silt (6%), greenish brown.

Not Determined (INIT. DENS., COHESION, PLASTICITY IND.)

Legend

TYPES OF SAMPLES - CORRESPONDING SYMBOLS
G- Ordinary GOW Type Dry Sample, 1½" I.D. Split Spoon.
ST- Undisturbed Sample, SHELBY TUBE, Seamless Thin Wall, 3" I.D.
STP- Same as ST, except using PISTON Type Sampler.
F- FLAP VALVE Sample, 2½".
W- WASH Sample.
OP- Ordinary Gow Type, except using 1" OPEN END PIPE.

43.5 / 45.5 2-G	Continuous Sample, 2' interval, using ordinary Split Spoon, described above.
43.5 / 45.5 2 ST	Continuous Undisturbed Sample, 2' interval, using Shelby Tube, described above.

BLACK ORGANIC SILT — GRAY SILT & INORGANIC SILT
SAND — GRAVEL
GLAUCONITIC SAND, *SAND ("Greensand") See Below — SHELL & SHELL MARL
CLAY — PEAT & ORGANIC MATTER
SANDSTONE CRUSTS (Generally Locally Indurated Sand). — Symbols are used both singly & in combinations.

BLOWS ON SAMPLER - Sampling spoon used to recover Gow type samples was driven by 140 lb. weight falling 30 in. on all "D" Borings except D-3-A, D-4, & D-5, on which 128 lb. weight was used. Whole numbers in column record number of blows required to obtain standard 12 in. spoon penetration. Fractional numbers record number of blows (Numerator) over penetration in inches (Denominator).

* SAND, or GLAUCONITIC SAND ("Greensand") indicates sand having an appreciable content of Glauconite, a soft mineral, essentially a silicate of iron and potash. When fresh and unaltered, it imparts a typical bluish or greenish color to sands of which it is a constituent, giving rise to common term "Greensand". When more or less weathered and oxidized, these sands are typically reddish brown, mottled and variegated brown and green, etc. Iron oxide derived from weathering of Glauconite sometimes cements the sands, forming Sandstone crusts and ledges, usually quite local in extent.

% CLAY, where stated, is based on results of laboratory tests which determine % (of dry wt.) of Clay Size particles present. This amount may not necessarily be entirely composed of true Clay minerals.

General Notes

For LOCATION PLAN of borings see Dwg. No. 137.

TEST BORINGS were made by Gow Division, Raymond Concrete Pile Co., 1947-1948, under supervision of J.E. Greiner Co. For transcript of original boring logs see Dwg. No. 138.

CLASSIFICATION OF MATERIALS was made by J.E. Greiner Co., based on:
(a) Mechanical Sieve Analyses and other standard laboratory tests performed at Soil Mechanics Laboratory, University of Maryland, except for Borings Nos. D-2, D-3-A, D-4, D-5, and D-50, for which no such tests were made. Classifications of material from the latter borings were based on (b), following.
(b) Study (by J.E. Greiner Co.) of all samples (except Shelby Tube samples) with low power microscope, to determine geologic formation and mineral composition; also approximate grain size distribution in samples for which mechanical sieve analyses were not made. Approximate quantitative notation used in latter cases, and based on visual examination, is as follows:
Predominating Component (Usually 50% or more) - in capitals, eg. "SAND"
35-50% - "and"; 10-20% - "a little"
20-35% - "some"; 1-10% - "trace" (1% - "very slight"; 1-5% - "slight"; 5-10% - "trace" or "heavy trace")
(c) M.I.T. STANDARD, shown below, is used in all cases to describe grain sizes. Grain size curves for all mechanically sieved sample were prepared, from which particle size distribution was determined. For samples classified by visual examination only, application of standard is necessarily approximate.

M.I.T. STANDARD

Gravel		Sand			Silt			Clay
C	M	F	C	M	F	C	M	F

60 20 6 2 .6 .2 .06 .02 .006 .002
Grain Size in Millimeters

REVISIONS

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MD.
CHESAPEAKE BAY BRIDGE
SANDY POINT - KENT ISLAND
LABORATORY & TECHNICAL DATA "D" BORINGS

SCALE: VERT. 1" = 25' DATE: JANUARY, 1949 CONTRACT
J. E. GREINER COMPANY CONSULTING ENGINEERS BALTIMORE, MD.
DWG. NO. 139
SHEET 4 OF 7
C-508-

PLATE 3

3. Preparation of drawings showing detailed technical logs of borings along the "Bridge Line" (Plates 3, 4, and 5)
4. Development of a geologic cross section along the "Bridge Line" (Plate 2)

The above data were invaluable for the determination of the geometrics of the bridge superstructure (e.g. span lengths and types) as well as the design of the substructure. Additionally, they were later presented as "Information Drawings" to accompany the Contract Drawings and were thus available to prospective bidders on substructure construction contracts. In that context, it may be reasonably surmised that the detailed subsurface information reduced the element of risk to contractors, thus encouraging lower unit bid prices and fewer claims based upon "unforeseen subsurface conditions".

The most prominent feature delineated by the geologic cross sections (Plates 1 and 2) is the drowned and buried former channel of the Susquehanna River. Aside from its geologic interest and implications (e.g. post-Pleistocene rises in sea level), the configuration of the channel (and the physical characteristics of the deep deposits of soft organic silt nearly filling it) directly affected the design of both the superstructure and substructure of the major cantilever span which crosses it and the overlying East Navigation Channel.

As a final footnote, it is appropriate to mention that the detailed geological investigations outlined above were largely instrumental in substantiating the feasibility of a bridge crossing and therefore contributed to the final resolution of the "bridge versus tunnel" controversy. Some 20 years later there was no question regarding the feasibility of the Parallel Span.

CHESAPEAKE BAY BRIDGE - PARALLEL SPAN (1973)

The alignment of the Parallel Span is 450 feet north of the center line of the original bridge (see Plate 2).

As discussed by S.M. Miller, the subsurface data developed for the first crossing were reviewed with particular reference to the record of foundation conditions actually encountered during construction; and the proximity of the two alignments was considered. It was concluded that a relatively limited program of supplemental borings and related studies, added to the 1948 data, would suffice for preparation of Contract Drawings and "Information Drawings" to accompany the bidding documents for the Parallel Span.

(Author's Note - Prior to preparation of the final manuscript it is proposed to procure from S.M. Miller more specific data regarding the supplemental borings, etc. for inclusion in this section. As an alternate, he has recently indicated (5/10/78) that he may find it feasible to submit his own text (which was not originally planned), in which case this brief section may be modified slightly to "dovetail" with his paper.)

SUMMARY AND CONCLUSIONS

The data developed from the 1948 subsurface investigations confirmed the feasibility of the original Chesapeake Bay Bridge; and they were invaluable for the preparation of final design drawings and information for prospective bidders for construction contracts. More than 20 years later the data were again effectively utilized for similar purposes in connection with the design and construction of the Parallel Span.

It is concluded that carefully planned and executed geotechnical investigations, if properly documented, are of permanent practical and scientific value.

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In retrospect, this paper is respectfully and gratefully dedicated to the memory of the late Dr. Joseph T. Singewald, Jr., former Director of the Maryland Department of Mines, Geology, and Water Resources, and the late Dr. Robert M. Overbeck, former Ground Water Geologist on the staff of the same agency. In 1947-48, they both offered kindly encouragement to my relatively early efforts in the field of engineering geology (the term "geotechnical engineering" would not be coined until many years later). Additionally, Dr. Overbeck spot-checked identifications and descriptions of test-boring samples; reviewed the geologic sections in considerable detail; and made many helpful suggestions.

HEALTH AND ENVIRONMENTAL ASSESSMENT
A MAJOR GEOLOGIC CONCERN ON THE FALL LINE

By

Earl G. Hoover[1/]

ABSTRACT

Until recently very little importance was given to the health and environmental aspect of crushed stone and natural aggregates. With the advent of major landmark environmental legislation and its stabilization by the regulatory agencies, great concern has surfaced that certain rock types are potentially hazardous to the public health when used in highway construction, parking lots and similar structures.

In the Fall Zone there are commonly thick series of ultra-mafic rocks, and it is the association with this geology that has precipitated a telescopic series of press releases, electronic media coverage, proposed Federal and State regulations, local health scares, and a redefinition of mineral terminology.

The major issue focuses on the presence of "asbestos" in these rocks. There is a marked disagreement between the various governmental agencies, public interest groups, health researchers, geologists and industry representatives over the mineralogical aspects; the relationship between chemical and physical properties and health effects; and the analytical methods used vis-à-vis the regulatory aspect of "asbestos."

This paper presents an overview of some of the major events having reference to geology, that have affected crushed stone producers during the past two years.

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Historically, very little concern has been directed toward any health implications associated with the use of stone for highway construction and other engineering applications. However, the ascendency of environmental activism, along with landmark legislation and regulation to clean up and protect the environment, has caused controversy between health and geological scientists. The heart of the controversy lies with improper use of the word "asbestos." Instead of being confined to the six asbestiform minerals, namely, chrysotile, amosite, crocidolite, tremolite asbestos, anthophyllite asbestos and actinolite asbestos, the term "asbestos" is extended to many other minerals in airborne dusts, that have an aspect ratio of 3 to 1 and are longer than 5 micrometers. Practically all experimental and epidemiologic evidence linking asbestos inhalation with asbestosis and various malignancies, has been derived almost entirely from studies with commercial grades of asbestos.

Health Implications - An Overview

In a report prepared by W.C. Cooper for the American Iron Ore Association[1] the point is made that "the biological effects of fibrous minerals are determined not only by dose, but also by fiber diameter, lengths and shapes; surface properties; and internal crystal structure. These latter characteristics determine respirability, deposition, retention, clearance, translocation and influence action at target sites in the body." Timbrel[2] found that the diameter of a fiber is the major determinant of its falling speed in air. The diameter also determines whether or not the fiber actually gets into the lung, in what location it is deposited, whether it is retained and how the body cellular defenses respond to it. Harris, et al.[3] noted that fiber length is important, since many very long fibers are found in the lung. The effect of length is related to site of deposition, retention and clearance and is very important in its influence upon biologic effects at the site of implantation.

There are several biologic effects in humans that have been associated with commercial asbestos, namely, asbestosis, benign pleural plaques, lung cancer, malignant mesothelial tumors and possible tumors in other sites. Studies in lower animals have shown that all types of commercial asbestos are capable of producing non-malignant and malignant changes analogous to those seen in humans, but there are demonstrable quantitative and qualitative differences in response.

Gilson[4] determined that health studies suggest that of the four economically important forms of "asbestos", crocidolite has been responsible for the greatest health risks, followed by amosite, then chrysotile and anthophyllite.

Ross[5] in Table 1, compares the proportional mortality from lung cancer and mesothelioma for the Quebec and North Italian chrysotile miners and millers and also for the entire populations of various countries in the year 1970. The figures do not support a higher cancer incidence for the male employees in the Quebec chrysotile industry when compared to the male cancer incidence in the whole of Canada. However, the incidence of cancer among those employed in "asbestos" trades is very high; incidence of lung cancer being 3 to 4 times that of the average population, incidence of mesothelioma being 130 to 220 times. Asbestos trades generally utilize a variety of "asbestos" minerals including amosite and/or crocidolite.

Additionally, Fears[6] made an epidemiological study of cancer risk, including respiratory cancer, in 97 U.S. counties in 22 States known to be mining chrysotile or amphibole "asbestos". He found no excess of cancer mortality compared with cancer mortality rate in 194 demographically matched

TABLE 1. Proportional mortality from lung cancer and mesothelioma for selected male populations.

GROUP	COHORT NO. MEN	DEATHS ALL CAUSES	% LUNG CANCER	% MESOTHELIOMA
General Population[a]				
Canada (1970)		82,052	5.3	0.03
USA (1970)		988,620	5.1	0.03
Finland (1970)		22,332	7.1	0.04
Italy (1970)		252,795	4.7	-
England - Wales (1970)		278,617	8.9	0.06
Chrysotile mining-milling[b]				
Quebec (1936-73)	10,951	3,938	5.7	0.18
N. Italy (1932-70)	1,098	270	2.2	0
Anthophyllite mining-milling[c]				
Finland (1936-67)	900	216	9.7	0
"Asbestos" trades[d]				
Insulators	26,505	2,137	19.6	6.7
Asbestos factory	10,781	1,422	15.0	3.1

a/ *McDonald, J.C., and McDonald, A.D., Epidemiology of mesothelioma from estimated incidence, Preventive Medicine 6, pp. 426-446, (1977), Entire male population over 24 years of age, Table 13.*

b/ *do Table 12; McDonald, J.C., and Becklake, M.R., Asbestos-related disease in Canada, Hefte z. Unfallheilkunde, 126, 2. Deutsch-Osterreichisch-Schweizerische, Unfalltagung in Berlin 1975, Springer-Verlag, Berlin, P. 525, (1976).*

c/ *do Table 12.*

d/ *do Table 12; Composite figures from Table 12.*

counties in which such minerals are not known to be mined; cancer mortality in both counties was significantly below the national average.

Although all types of asbestos can cause fibrosis, it is significant that according to Gilson[7] an excess of bronchial cancers is closely related to cigarette smoking. In addition, he notes that crocidolite is especially related to cancers of the surface of the lung as well as the gut. However, the mining and milling of chrysotile have caused few mesotheliomas, despite heavy dust exposures in the past. The lower incidence of disease caused by chrysotile, than by the amphiboles, may, in part, be due to the greater solubility of chrysotile and, in part, the difference in shape of the fibers.

The National Institute for Occupational Safety and Health in its program plans for 1978[8] have included several health related studies on worker exposures to various types of mineral dusts - among these are: morbidity and industrial study of workers handling fibrous mineral, mortality study of crushed stone exposure, chronic inhalation of short asbestos fibers, mortality study of workers exposed to clay fibers and a joint study with the Mine Safety and Health Administration concerning morbidity and industrial hygiene of cement workers.

It is clear that much research is needed. Rohl, et al[9] based on their studies done in Montgomery County, Maryland conclude: "while the data we have collected are limited to five single measurements, they suggest that a possible public health hazard exists associated with the distribution and use of quarried asbestos-containing serpentinite. Similarly, the exploitation of crushed amphibolite and a variety of ultra-mafic rocks also raises the possibility of contamination by actinolite, tremolite, crocidolite, asbestos or their asbestiform analogs. The use of such quarried rock for road surfacing and similar purposes may result in widespread asbestos contamination of community air." Authors, Rohl, et al. note that "the evaluation of the possible health hazard that may be associated with this exposure requires information that is not yet known in the scientific community (i) the biological activity of short chrysotile fiber, (ii) the level of exposure to asbestos which is safe insofar as human cancers are concerned, if a safe level exists, and (iii) the biological activity of asbestiform silicates, not necessarily asbestos."

Essential to a resolution of the health implications associated with mineral fiber exposure is a precise definition of what is "asbestos".

Geological Considerations

Because many of the health studies related to inhalation and ingestion have been essentially limited to well defined commercial types of asbestos, it is now vitally important that the biologic effects of cleavage fragments be compared with those of true mineral fibers. Campbell, et al.[10] in their study observed that cleavage fragments will generally have a frequency maximum of aspect ratio less than 3 to 1, whereas asbestiform varieties will fall between 10 to 1 and 20 to 1, or higher depending on the characteristics of the mineral and the history of the sample, particularly the type and degree of milling. The case is made that with respect to medical evidence, shapes and sizes of the particles should be specified.

In contrast, some health researchers and judges have advocated eliminating ambiguity by defining all mineral particles with 3 to 1, or greater, aspect ratios that are longer than 5 micrometers as "asbestos". Technical problems in defining and characterizing fine mineral particles and the unknown health effects on humans by minerals not generally regarded as "asbestos" appear to be indirectly responsible for rather broad definitions for "asbestos" - an example is the following from the Sixth Minnesota Judicial District, August 11, 1977:

> "Fibers" are defined as durable asbestiform mineral particles including chrysotile, amphibole, and silicate mineral particles of approximately parallel sides with 3 to 1 or greater aspect ratios. In considering, promulgating or applying any fiber standard which may be adopted, the Agency shall act consistent with the Supreme Court's decision including the requirement that there be a single standard for all plants causing air emissions which may be potentially dangerous to health, wherever such plants may be located in Minnesota.
>
> "Amphibole" is a group of hydrated silicate minerals usually containing two or more metals such as iron, magnesium and/or calcium. Amphibole minerals share a common crystalline structure with a double chain of linked silica tetrahedra.
>
> "Chrysotile" is a fibrous magnesium silicate mineral in the serpentine group with a characteristic scroll-like structure which often gives the unit fibers a hollow tube appearance.
>
> "Silicate minerals" include but are not limited to serpentines, Minnesotaite, stilpnomelane, greenalite, pyroxenes and talcs."

The preceeding definition is the type of response that Erickson[11] makes note of when he states that perhaps the severest restriction upon asbestos and mineral fiber research is the absence of an accurate and reliable analytical methodology. Scientific opinion supports the fact that present day "fiber" identification and counting techniques do not produce either accurate or reproducible results.

The two Federal agencies regulating "asbestos" exposure to workers are Mine Safety and Health Administration (MSHA) and Occupational Safety and Health Administration (OSHA). Although both agencies have similar definitions for "asbestos", MSHA is more precise in its definition in the Code of Federal Regulations, Title 30, revised as of July 1, 1976, p. 224 - " 'Asbestos' is a generic term for a number of hydrated silicates that, when crushed or processed separate into flexible fibers made up of fibrils. Although there are many asbestos minerals, the term 'asbestos' as used herein is limited to the following minerals: chrysotile, amosite, crocidolite, anthophyllite asbestos, tremolite asbestos, and actinolite asbestos."

Chrysotile, a sheet silicate, is the only commercial asbestos in the serpentine class. Fibers of chrysotile are bundles of hollow cylindrical fibrils, which may have diameters in the range of 30 to 40 nanometers (300 to 400 A).

The other commercial types of asbestos are amphibole minerals, which are chain silicates. Some of the most important series of amphibole minerals, each of which includes fibrous varieties, as shown in parenthesis are: cummingtonite-grunerite (amosite and anthophyllite); glaucophane-riebeckite (crocidolite) and tremolite-actinolite-ferroactinolite (actinolite). Figures 1 and 2 compare nonasbestiform actinolite with actinolite asbestos.

Chidester and Shride[12] summarized the known asbestos deposits in the U.S. with the associated basic rock formations. Their map shows how widespread are naturally occurring asbestiform minerals. Serpentinites, containing varying amounts of chrysotile, occur as outcrops or in deposits just below the surface of the earth, vary extensively. It is apparent that in such areas excavations for houses, road construction, mining, etc. would result in fibers becoming airborne. Because most of these fibers are in the micron-size range, they would tend to remain airborne for long periods. Further evidence that asbestiform minerals occur widely in nature is provided by their presence in many natural water sources.

Mining and milling of asbestos in the U.S. is confined to California, Vermont, Arizona and North Carolina.

Because of the ubiquitous nature of asbestos in the natural environment, the Environmental Protection Agency (EPA) contracted the Battelle Columbus Laboratories[13] to define the potential seriousness of asbestos emissions that result from man's disturbing geological formations in which asbestos occurs as an accessory mineral.

Limited sampling and analysis done during the study did not support the hypothesis that a health hazard exists. The conclusion was based on limited sampling of persons living in the vicinity of two large mining operations working asbestos — containing ore. They found that the workers were not exposed to asbestos concentrations above those frequently encountered in ambient air.

Battelle's extensive literature search covered each State with a general description, and highlighted deposits of potentially "asbestos" containing rock, gave a map of the State outlining areas of igneous and metamorphic rock, and listed the value of mineral production and the specific location of mining activities.

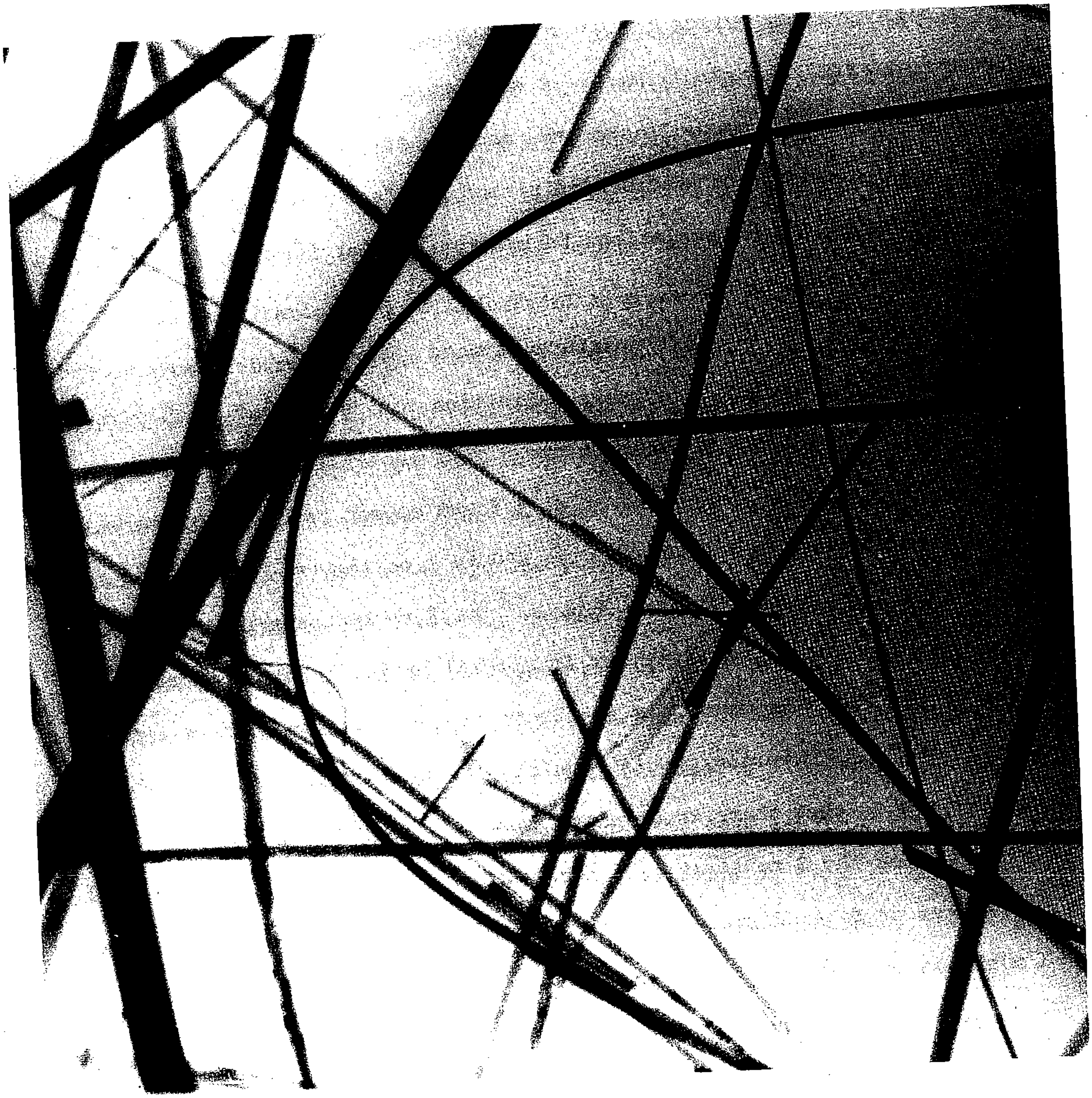
Ross[5] comments that if the definition of "asbestos" from the point of view of a health hazard does include the common nonfibrous forms of amphibole, particularly the horneblende and cummingtonite varities, then we must recognize that so called "asbestos" is present in significant amounts in many types of igneous and metamorphic rocks covering perhaps 30 to 40% of the U.S. rocks within the serpentinite belts; other rocks within the meta-

Figure 1. Actinolite, Showing Blocky Shape

(SEM photomicrograph at x7000)

Figure 2. Actinolite Asbestos Fibers

(SEM photomicrograph at x19,700)



morphic belts higher in grade than the greenschist facies, including amphibolites and many gneissic rocks; and amphibole-bearing igneous rocks such as diabase, basalt, trap rock and granite.

What Ross stated was prophetic in that Zumwalde, et al.[14] are considering a host of other minerals that commonly possess a fibrous or acicular habit to be health hazards; minerals such as the zeolite group, pyroxenes, sepiolites, halloysite, wollastonite, and the fibrous form of calcite.

It is clear that a precise definition for "asbestos" is needed in order that government regulations be written so that they do not ban the earth's crust.

Regulatory Aspects

Once a universally acceptable definition for "asbestos" is developed, it will be essential to re-examine the 3 to 1 aspect ratio as a criterion for a mineral fiber. Campbell, et al.[10] found aspect ratios for fibers from asbestiform minerals to be as much as 200 to 1 or higher, whereas the ratio for cleavage fragments is about 3 to 1. The 3 to 1 aspect ratio may be a valid conservative lower limit for industrial hygiene control in asbestos-processing plants; however, its applicability to existing nonasbestos mining and ore processing plants requires critical evaluation. Campbell further states that there is no experimental or epidemiologic evidence to indicate that the 3 to 1 ratio is necessarily a valid definition over all the ranges of fiber diameters, and that perhaps an aspect ration of 10 to 1 might be more appropriate.

Historically, the 3 to 1 aspect ratio arose out of a need for standardized methods of counting airborne fibers. When the U.S. Public Health Service[15] described its light-microscope membrane filter method, it used this definition of a fiber; it also defined countable fibers as those over

5 micrometers in length. An important criticism of the 3 to 1 ratio as a sole criterion is that it ignores the basic shape of the particle. Some irregularly shaped particles, while literally having a 3 to 1 aspect ratio, are clearly not fibrous.

The background and validity for the 5 micrometer length in defining a countable fiber under an occupational and environmental standard was a result of technical problems in recognizing and counting fibers with the light microscope. Additionally, a considerable body of information indicated that longer fibers were more fibrogenic than short fibers.

It has been recognized in industrial hygiene practice that the counting of "asbestos" fibers by light microscope provides only an index of "asbestos" exposure, and does not give an absolute measure because so many fibers are too small to be seen by light microscopy. The expense and lack of standardization of electron microscopy (EM) methods have led to their being regarded as not feasible for industrial hygiene purposes.

Prior to OSHA (1970) there was no federal standard for "asbestos". In 1972, OSHA (29 CFR 1910-1001) published the first standard establishing an eight-hour time-weighted average (TWA) concentration exposure limit of five fibers longer than 5 micrometers per cubic centimeter of air, and a ceiling limitation against any exposure in excess of ten such fibers per cubic centimeter. The present standard, which became effective in July 1976, is two fibers. The MSHA standard will soon be lowered to two fibers.

OSHA proposed in 1975[16] that the eight-hour exposure limit (TWA) be lowered to 0.5 fibers per cubic centimeter, and that ceiling exposure limit be 5 "asbestos" fibers per cubic centimeter for any period up to 15 minutes. It further noted that OSHA recognizes that there is no assurance of a safe exposure for a substance with a known carcinogenic property and that the

0.5 fiber standard is to prevent asbestosis. Soon OSHA will adopt the 0.5 fiber standard.

The trend is toward setting lower and lower limits on the acceptable amount of "asbestos" permitted in the environment. The National Institute for Occupational Safety and Health[17] recommended a standard of no more than 100,000 fibers greater than 5 micrometers in length per cubic meter (0.1 fiber per cubic centimeter) as a protection against the noncarcinogenic effects of asbestos, materially reduce the risk of asbestos-induced cancer and be measurable by techniques that are valid, reproducible, and available to industry and official agencies. The measurement technique referred to is light-optical microscopy.

Such is the state of the art in standards development for worker exposure. However, there is no standard either existent or proposed covering the general public. In fact, there were no reports of analyses of ambient air for asbestos prior to 1960. The first major study was on 187 samples from 49 U.S. cities during 1969-1970 by the National Air Pollution Control Administration. Of the 187 samples, only 3 had more than 19 nanograms of asbestos per cubic meter, while 163 had under 5. Probable sources for asbestos in the ambient air included natural occurrence in ultra basic rock formations, asbestos mining and milling, transport of asbestos, manufacture of products containing asbestos, use of products containing asbestos, demolition and waste disposal.

In order to protect the general public from "asbestos" released from crushed stone the Environmental Protection Agency[18] has undertaken a study of the use of crushed serpentinite rock for roadway surfacing. Accordingly, EPA has underway a monitoring program and they report that results to date indicate standards will be proposed in January 1979.

In a draft[19] prepared for EPA by Research Triangle Institute (RTI) the specific goals of a study to determine "asbestos" content in quarries suspected of containing serpentine "asbestos" were: to determine whether "asbestos" is present in a quarry, to determine the amount (proportion) of asbestos in the total rock and to identify the sources - rock types and locations of asbestos in the quarry.

The program as proposed by RTI will consist of two phases. In Phase I, three chip samples are to be taken of any and every rock type in the quarry that would possibly contain "asbestos". If asbestos is found in Phase I, the quarry would be sampled under Phase II. This phase would consist of "representative" belt sampling at random for twelve consecutive weeks. The product selected for sampling is to be one that would ordinarily be used in an unbound form for highway use.

A major drawback in the RTI proposal is the apparent inability of electron microscopists to duplicate results. This coupled with the difficulty in obtaining representative chip samples would make the entire sampling and analytical procedure very suspect for the results obtained. To predict with certainty what future mineralogy will be, would require detailed mapping and systematic core drilling - a very expensive option.

Such is now the state of the "asbestos" problem. No confirmed health risk has been associated with low level exposure, namely from crushed stone in an unbound form; a methodology for sampling and analyzing for "asbestos" has not been developed; a precise definition of what is "asbestos" does not exist; nor does a concensus among the various government groups as to what course of action to pursue.

Crushed Stone and the "Asbestos" Controversy

September 1976 marked the start of the "asbestos" controversy for crushed stone operators with the focal point being the quarry of Rockville Crushed Stone, Inc. on the Fall Line and about 15 miles from downtown Washington, D.C. The rock types quarried here include serpentinite and rodingite. A brief chronology of events associated with the controversy are highlighted.

On 11 September, 1976, there appeared in a Washington, D.C. newspaper a story entitled, "Cancer-Causing Asbestos Found in Rockville Quarry -- Maryland Quarry Yields Carcinogenic Fibers." The article revealed that two local amateur rockhounds had found chrysotile in the serpentinite in the quarry.

What the local rockhounds had done was to bring several mineralogists/ geologists from Mt. Sinai Environmental Sciences in New York City, to collect a series of dust samples from various locations on the quarry property. These samples formed the basis for the news story.

Spring 1977 began a series of kaleidoscopic events triggered by a public interest group, the Environmental Defense Fund, petitioning the State of Maryland before the Montgomery County Council to suspend the use of serpentinite for resurfacing roads and parking areas. The council's response was to propose convening a public information forum under the sponsorship of the National Institute of Health (NIH) to educate all parties on how much was known, what was not known and what needed to be done to evaluate and address the asbestos problem. The forum was scheduled for June 8, 1977.

In the meantime, however, the Environmental Defense Fund didn't wait for the facts expected to be developed by the NIH Forum, and on May 10, petitioned EPA to take Emergency Action against the Maryland quarry and to undertake an immediate study of other quarries in six States.

With all the publicity the asbestos issue received in the Washington D.C. area, it was only natural that the Maryland congressman closely associated with suburban Maryland would find it politically expedient to introduce bills in the House and Senate to provide for Federal Standards to protect the public against asbestos contamination.

On the eve of the NIH conference, the EPA sent a mailgram to the Montgomery County Executive recommending a comprehensive plan for immediate action to minimize asbestos emmissions into the ambient air. EPA also ordered the quarry to release company information covering customer identity, quantity sold, and application of the product.

The remainder of June was marked with a flurry of activity and included the closing of playgrounds, walkways, bicycle trails and so forth. Also during June, many other Maryland counties became involved with similar activities which prompted the State of Maryland to begin preparation of an emergency regulation to control so-called asbestos material on roads and parking lot surfaces on a statewide basis. All of this activity went forward without any medical evidence to indicate that a hazard existed.

In mid-August, EPA released a press notice on results of a study in which eight laboratories voluntarily analyzed air samples collected in Montgomery County, Maryland, to determine the comparability of data that would result when laboratories using different techniques independently analyzed air samples to measure the presence of asbestos. The conclusions were that <u>there is no correlation between laboratories and little agreement on a practical method of consistently measuring the presence of asbestos</u>.

In early November, the EPA published in the Federal Register the advance notice of proposed rule-making in the development of an asbestos standard for the crushed stone industry. EPA requested that all interested persons submit factual information concerning crushed stone produced from serpentinite rock, particularly information as to its production, sale, and public use in various applications; its asbestos content and public exposure to ambient air asbestos emmissions resulting from its use in various applications. NCSA* members reviewed the development document and found it to be totally unacceptable since the procedures outlined by EPA could not produce any valid scientific conclusions, but could instead result in highly inflamatory and prejudicial news releases.

In February, 1978, the State of Maryland Department of Health and Mental Hygiene issued proposed regulations which would prohibit loose serpentinite on "any right-of-way, driveway, parking lot, recreational area or similar area", whether public or private. In addition, vehicles transporting crushed serpentinite would have to be covered, and signs of warning of a health hazard would have to be posted at entrances and exits of plants quarrying the rock.

The Maryland quarries responded to the State's proposed regulations by pointing out that there is no substantive medical evidence that the relatively low levels of asbestos purportedly measured in the air around areas paved with unbound serpentinite posed a hazard to anyone. It was also noted that EPA tests of stone from the Maryland quarries were not precise enough to prove levels of airborne asbestos were higher than normal.

It is evident that all of the various legislative actions and public concerns within the past few years have had, and will continue to have, an impact on the crushed stone industry. Additional fuel for the foregoing interests will be provided by investigations and research into asbestos and elongate mineral particles which is burgeoning in every level of government as well as the private sector.

* National Crushed Stone Association

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CONTROL OF VIBRATIONS FROM COMMERCIAL BLASTINGS IN URBAN AREAS

Presented at the 29th Annual Highway Geology Symposium in Annapolis, MD, May 3-5, 1978.

by

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Abstract

Highway construction and excavation in areas of outcropping bedrock require the use of explosives. Safe blasting is accomplished by keeping the vibrational peak velocity below 2 in/sec. The peak velocity can be reduced by delayed charges. The scaled distance relates the charge per delay to the distance by the equation: $S.D. = D/\sqrt{W}$. If S.D. is larger than 50, the blasting is considered to be safe according to recommendations of the U.S. Bureau of Mines.

For urban construction projects such stringent safety limits cannot be maintained, as there is little choice to increase the distance. Blasting with too many delays makes the project expensive as the effectiveness is reduced. In order to help clients in the planning stage of their construction we frequently make use of predictive propagation equations of the form: $V=K\ (S.D.)^{-\beta}$. V is the peak velocity, K is a constant that is mainly dependent on rock properties and some details of the blasting, and β is related to the dissipation and absorption of the vibrational energy. There is a certain interchangeability between K and β, if there are no simultaneous observations from the same blast for different S.D. Rather than to attempt an accuracy in the prediction of V by finding the correct value for β, we prefer to use the equation with optimal constants as a guide for a safe upper limit of V. Its usefulness is demonstrated for some examples. Even for similar values of S.D. the K-factor can vary considerably for the same β in the same material. We have analyzed a large number of observations and normally use maximum K-factor for prediction. In the process of preliminary investigations maximum K-factors

are replaced by other, usually smaller ones, that are obtained from test shots. Particular care is exercised if the test shots occur under different conditions from the planned blasting operation. In many cases it is possible to conduct controlled safe blasting at a scaled distance of 10.

Control of Vibrations from Commercial Blastings in Urban Areas

1. Introduction

Highway construction and excavation in areas of outcropping bedrock require extensive use of explosives. In urban areas the protection of constructions from the vibrations of the explosions is of major concern, as short distances between explosions and endangered sites are unavoidable. Any amount of explosives can be detonated, if they are fired with delayed charges. The instantaneous charge per delay is decisive for the final vibrational effect.

2. Criteria for Safe Blasting

Most industrial nations have accepted the peak particle velocity as a reliable indicator of potential damage for blasting. In the USA it is recommended to keep peak velocities below 2 in/sec to avoid possible damage. This tolerance for safe blasting is large in comparison with safety limits adopted by other countries. In the Federal Republic of Germany, for instance, there are not only lower limits for safe blasting but also recommended distinctions of various types of constructions (Baule, H. 1967). Peoples' complaints occur usually at a lower peak velocity starting at about 0.4 in/sec particularly in the case of permanent quarry operations (Nicholls & Duvall 1971). Interesting legal questions of liability arise, if, say a house, was built with unsafe foundations and negligence of the builder can be proven. Vibrational damage from blasting may occur below 2 in/sec; yet the blaster may argue that the damage would have occurred anyhow due to natural causes at a possibly later time.

When comparing the limits for safe blasting that have been adopted by other countries with our own 2 in/sec, the question arises whether we should also lower our tolerable peak velocity. While this suggestion would undoubtedly find support from an environmentally conscious public, there is little justification for a change, if it cannot be shown that damage occurs at velocities below 2 in/sec. We are more concerned about the lack of enforcement of our present safe blasting limits. Vibrations from blasts are controlled only if the builder or blaster suggests this service, or if third parties concerned about their property insist on control. We feel that the 2 in/sec - limit should be thoroughly enforced by continuous control, if the explosions occur within dimensions that are in a range where sizable peak velocities are expected at endangered sites. An occasionally observed error is the assumption that explosions conducted under similar circumstances will cause approximately the same particle velocities. It is certainly wrong to categorically expect vibrational damages that may have occurred at peak velocities way above 2 in/sec to disappear once the tolerable level for safe blasting has been reduced below the present level.

3. Control of Blast Vibrations

As the vibrational peak velocity from a blast decreases with distance and increases with the charge per delay, a practical unit for estimating the possible effect of an explosion is the scaled distance. It is defined as S.D. = $D/\sqrt{W}$ where D is the distance measured in feet and W the charge per delay measured in lbs. It was found that at scaled distances larger than 50 a peak velocity close to 2 in/sec is unlikely (Nicholls and Duvall 1971). The vibrations are recorded at the endangered site with a calibrated 3-component seismometer. With one vertical, V_z and two mutually prependicular horizontal ($V_{\parallel}$ and $V_{\perp}$) components one can record that resultant peak velocity V_{res}, which is:

$$V_{res} = \sqrt{V_z^2 + V_{\parallel}^2 + V_{\perp}^2} \qquad (1)$$

Fig. 1 shows a seismogram with three components. The upper fourth trace shows the air pressure from the blast, which explains why people often complain about comparatively low ground vibrations. A typical explosion work in an urban environment is shown in Fig. 2. A total of 300 lbs of explosives was detonated with 10 delays. After test shots at scaled distances between 60 and 90 most of the shots were detonated at scaled distances near 10.

4. Peak Velocity-Scaled Distance Relationship

The larger the scaled distance the smaller is the peak velocity under equal circumstances. Physically there is a charge-to-distance relationship with the peak velocity, since the spreading of the vibrations is subject to dissipation and absorption of elastic energy. A predictive formula that relates peak velocity to distance and charge per delay is most desirable for the following reasons:

1. The planning of construction or excavation programs.
2. A faster approach to an optimally high charge per delay at a given distance.
3. The estimation of a risk, if the owner agrees to go higher than 2 in/sec on his property.
4. The estimation of the potential damage, which a house may experience, if there were no control measurements but if the scaled distance is known.

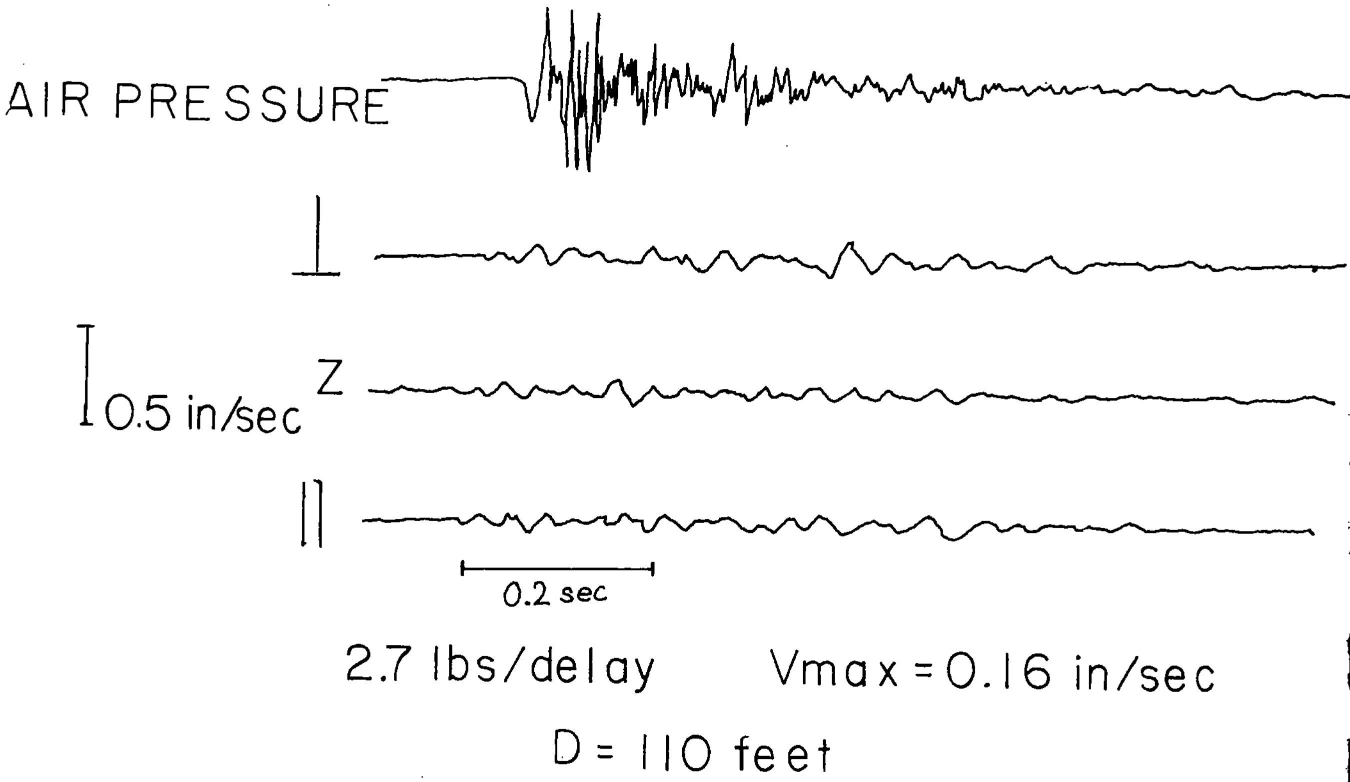

Fig. 1: Example of a seismogram recorded with a Sprengnether VS-1200 seismograph. Upper trace: air pressure.

Fig. 2: Explosion work for excavation in an urban setting near Stanford Connecticut. Left of center driller prepares boreholes for explosive.

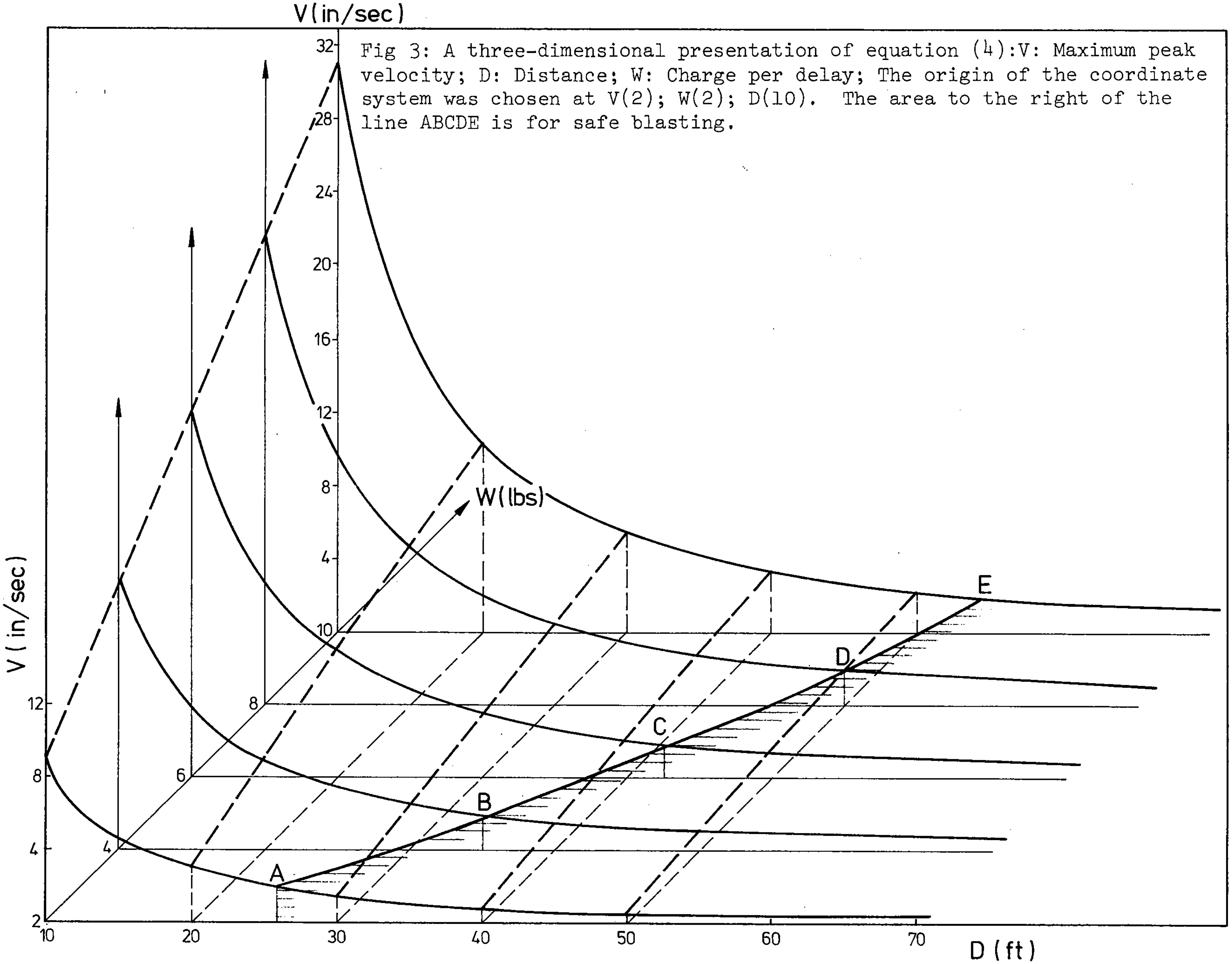

Fig 3: A three-dimensional presentation of equation (4):V: Maximum peak velocity; D: Distance; W: Charge per delay; The origin of the coordinate system was chosen at V(2); W(2); D(10). The area to the right of the line ABCDE is for safe blasting.

In the most general form the maximum peak velocity depends on the distance, D, and the charge per delay, W, as:

$$V_{MAX} = K \cdot W^a \cdot D^b \qquad (2)$$

where K, a, and b are physical constants. Schubart and Thümmel (1976) analyzed approximately 3600 explosions conducted by the cement industry. A least squares solution for the physical constants resulted in the following values:

$$K = 195 \qquad a = 0.66 \qquad b - 1.21 \qquad (3)$$

Devine (1966) suggested a dependence on the scaled distance of the form:

$$V_{MAX} = K \cdot (S.D.)^{-\beta} \qquad (4)$$

where β ranges between 1.5 and 1.7. Figure 3 shows a three-dimensional presentation of equation (4) for $\beta = 1.7$. To the right of the hatchured line peak velocities are below 2 in/sec. Another expression has been suggested by Edwards and Northwood (1960).

$$V_{MAX} = \frac{K \cdot W^{2/3}}{D} \qquad (5)$$

Most investigators agree that the constants determined in (3) have to be determined for each location separately. Even then the prediction of the peak particle velocity from the scaled distance is most unreliable. Schubart and Thümmel found their velocities to be ± 50% off the predicted value. There is no theoretically exact way, at this time, to determine the constants K, a, and be accurate as they depend on a number of factors that cannot be reliably quantified. Without claiming completeness they can be grouped into three categories:

1. Conditions given by the explosives in the borehole

   The K-factor depends mainly on the type of explosive, the condition of the rock in the immediate surrounding of the charge, the compaction of the material above the charge, and the distance to the face and the spacing between boreholes.

2. Physical parameters of the rock

   The elastic constants of the rock mainly near the shot but also near the site that is to be protected will determine the propagation and intensity of the vibration.

   Wave absorption due to fractional losses and the mechanical strength of the rocks will most likely influence the decay-constants a and b.

3. Geological influences and irregularities

   Such effects are most difficult to quantify but are probably quite significant. The development of a weathered surface layer may facilitate the formation of powerful surface waves. Irregularities in the surface topography, and structural features - such as bedding and jointing and faulting and folding - can substantially alter the propagating intensity of vibrational energy from that which would be expected on the basis of homogeneity and isotropy of the material.

   Most significant of all we see no justification to statistically treat alike the outcome of every blast-observation for the following reason: bedrock is subject to weathering, which is a process that decomposes the rock by weakening the bonding. With an explosion in a borehole, the same effect is achieved within a very short time. The intensity of elastic energy spreading from an explosion will depend on the amount of weathering and disintergration that occurred prior to the explosion. A partly disintergrated rock will allow much smaller amounts of energy to spread than a fairly compact and unweathered rock.

Therefore, we are primarily interested in peak vibrational velocities that are unexpectedly large at a given scaled distance. Identification of them is possible by comparing various observations of peak velocities with scaled distances by means of equation (4) with a constant negative exponent β.

5. <u>The Prediction of the Highest Possible Peak Velocity</u>

Table I shows the result of two series of blasts that were shot under similar circumstances. They are used as calibration-shots that allow us to evaluate the K-factor. Solving equation (4) for K, we obtain:

$$K = V_{MAX} \cdot (S.D.)^{\beta} \tag{6}$$

K is different for each β. Similarly one can determine a K-factor from (5)

$$K = V_{MAX} \cdot D \cdot W^{-2/3} \tag{7}$$

Series II that represents four explosions that showed unexpectedly large peak velocities, their K-factors are consequently larger than those obtained from other explosions at scaled distances of similar size.

In figure 4 Vmax is plotted versus the scaled distance for β = 1.5, 1.6, and 1.7 according to equation (4) with the average K-factor obtained from series I. Curve 1 was obtained from equation (5) with a K-factor of the

	V_{max}	D(ft)	W(lbs)	S.D.	K1	K(1.5)	K(1.6)	K(1.7)
SERIES I	1.36	50	4.34	24.0	25.6	159.9	219.7	301.9
	1.40	44	4.34	21.1	23.1	135.9	184.4	250.1
	1.52	49	6.51	19.2	21.4	127.9	171.9	230.9
	1.46	34	2.30	22.4	28.5	154.9	211.5	288.5
average	1.44	44	4.37	21.7	23.7	145.6	198.0	269.4
SERIES II	1.66	28	1.00	28.0	46.5	246.0	343.2	478.9
	1.56	34	1.50	27.8	40.5	228.2	320.0	443.5
	2.01	47	3.25	26.1	43.1	267.6	370.9	513.6
	1.63	57	3.25	31.6	42.4	289.7	409.6	577.9
average	1.72	42	2.25	28.4	42.1	260.3	363.8	508.4

Table I : Determination of K factors from scaled distances and observed peak velocities. K1 : charge distance relation 7. K(1.5), K(1.6), K(1.7) : K factors for equation 6 with exponents of 1.5, 1.6, and 1.7 respectively.

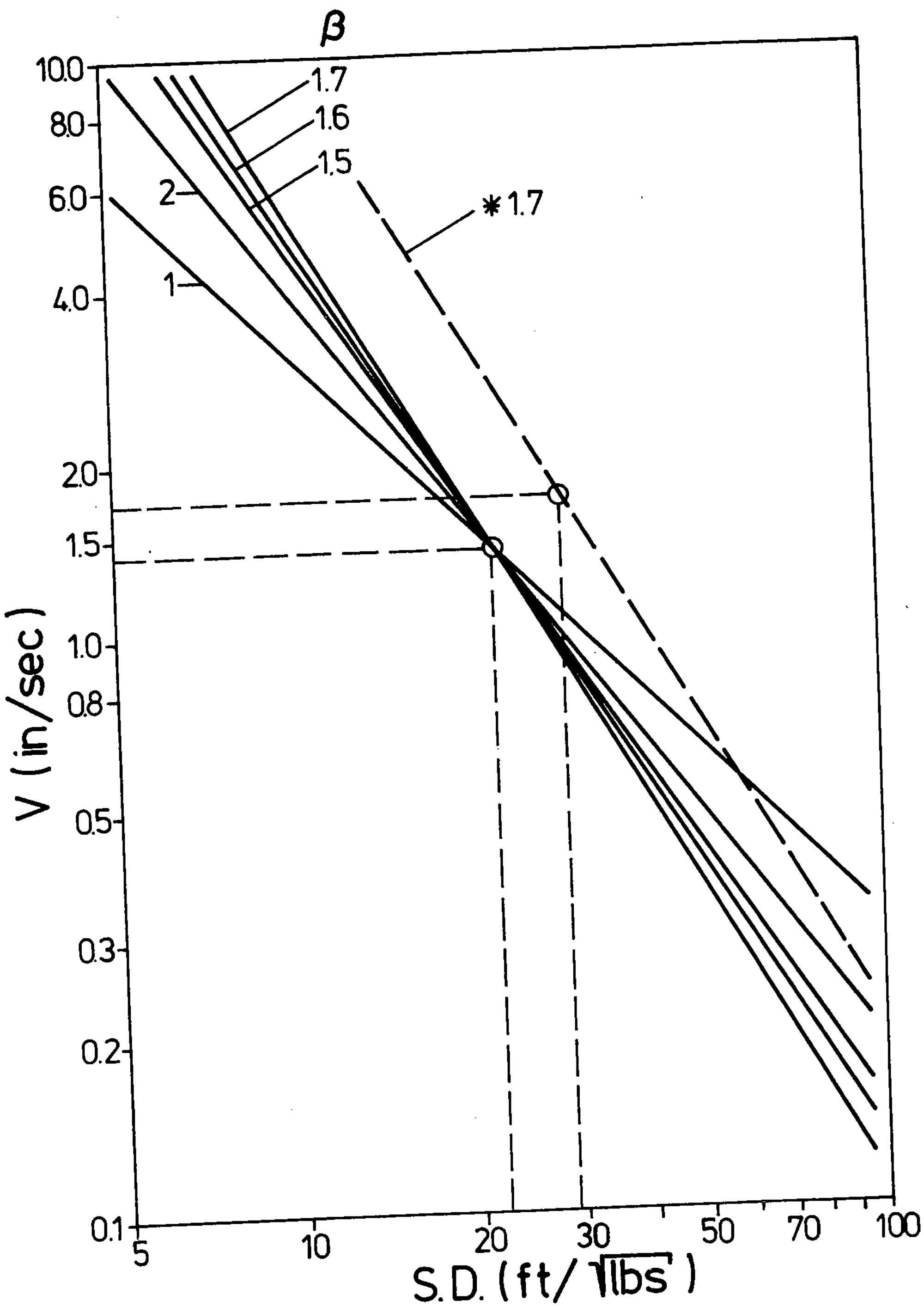

Fig. 4: Peak velocity versus scaled distance according to equation (4). with different values for β. Curves 1 (constant D) and 2(constant W) after equation (5).

same series I. The scaled distance was changed by keeping W constant. Curve 2 relates to the same equation and K-factor as curve 1, except that the scaled distance was changed by keeping D constant. In most practical cases a series of test shots at sufficiently large scaled distances can be used to establish decay - curves such as in figure 3. If the scaled distance has to be reduced, one can move from the intersection of the curves (circle) towards smaller scaled distances and see whether the curves remain below 2 in/sec. Equation (4) shows the most rapid increase for $\beta = 1.7$ and should provide us with the largest expected peak velocity at a smaller scaled distance. Without prior knowledge of the outcome of a test shot it is more advisable to use equation (4) for $\beta = 1.7$ based on a K-factor obtained from series II, at which the vibrational velocity was unexpectedly high. One can consider this as a relatively safe curve, which is shown in figure 3 (curve-parameter 1.7 with an asterisk). This curve shows a peak vibrational velocity of 0.8 in/sec at a scaled distance of 50. Blasting at a scaled distance larger than 50 can therefore be considered to cause, by far, smaller peak velocities than 2 in/sec.

6. An Example of Vibration Control at Small Scaled Distances

In the following we show an example of vibration control where the owner agreed to take a risk, if necessary, to go beyond the 2 in/sec, near his property. The sequence of vibrational control is shown in Table II. The first three shots at relatively large scaled distances between 27 and 38 resulted in small peak velocities below 0.4 in/sec. The largest K-factor for $\beta = 1.7$ was 153.8. With this value a predictive curve was calculated that is shown in figure 5. The scaled distance was then reduced to 16 at which the next four explosions were detonated. Guided by the curve for K=154 in figure 5 the observed resultant

Project Watertown, Conn.

DATE	D(ft)	W(lbs)	V(in/sec)	S.D.	K(1.7)
3/25/75	150	31	0.36	26.9	97.0
3/25/75	185	24	0.32	37.8	153.8
4/07/75	240	70	0.34	28.7	102.3
maximum possible charge for next shot at 96 ft based on S.D.=15 (V=1.55 in/sec)					
4/07/75	96	40	1.21	15.2	123.6
maximum charges for next 3 shots were predicted not to exceed 2 in/sec for an S.D. = 16					
4/14/75	135	75	1.51	15.6	161.2
4/14/75	114	46	1.34	16.8	162.2
4/14/75	50	10	0.94	15.8	102.5
firing of the final charges					
4/30/75	126	20.8	0.44	27.6	123.9
4/30/75	87	13.9	0.66	23.3	139.3
4/30/75	67	16.25	1.70	16.6	201.7
4/30/75	48	20.0	1.35	10.7	76.5
4/30/75	30	10.0	0.84	9.5	38.6

Table II : Successive estimates of the peak velocities for an optimal use of explosives at small scaled distances.

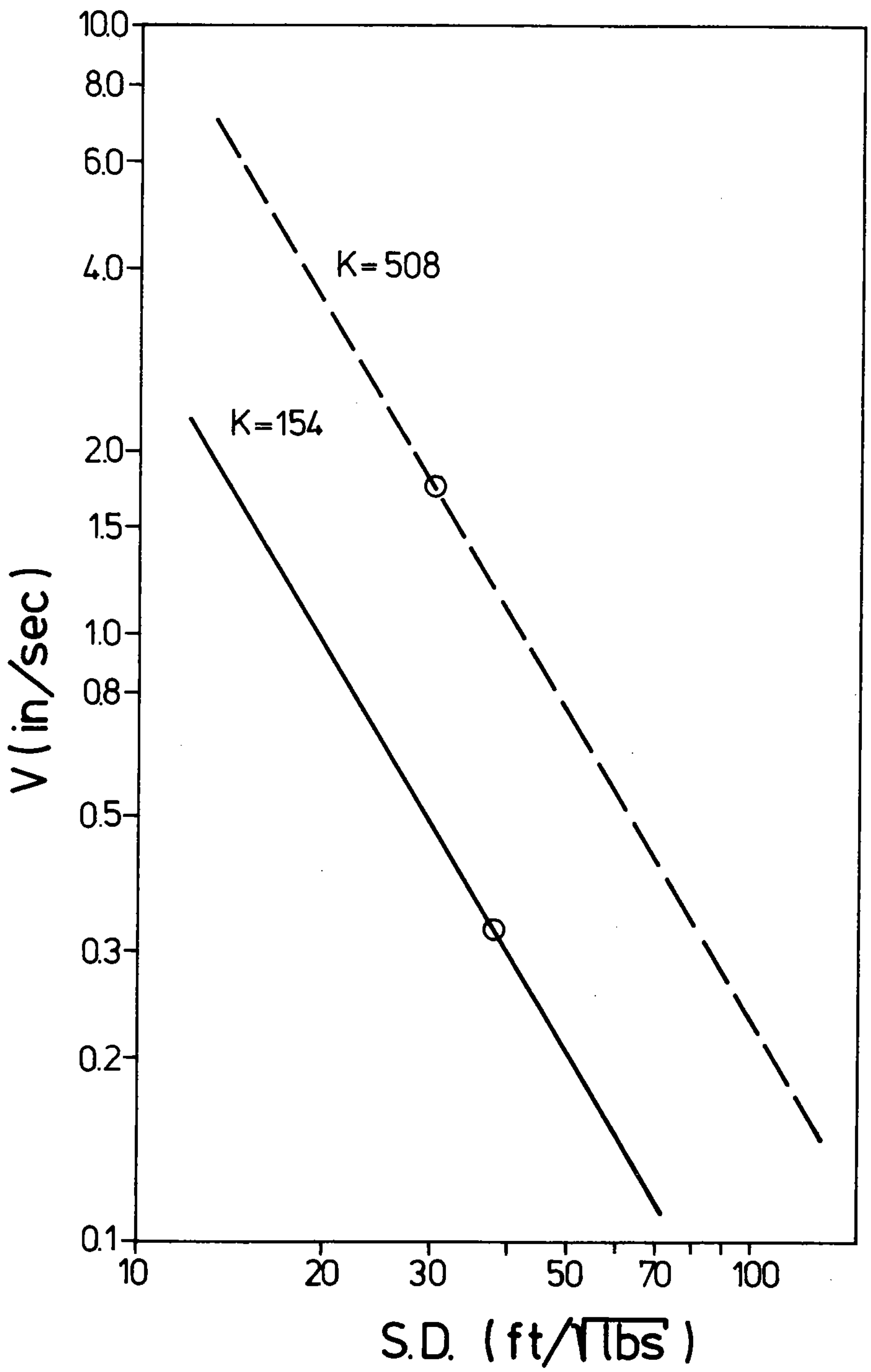

Fig. 5: Prediction of peak velocities according to equation (4; β= 1.7) for different K-factors.

peak velocities were below 2 in/sec (see Table II), through subsequent K-factors were in two cases larger than 154. Fig. 6 shows the spatial distribution of the shotholes at the site. With the last three shots a risk was taken as the K-factor of 154 was assumed to be valid. A safer curve for K=508 would not allow such small scaled distances. The last two shots had a surprisingly small K-factor mainly because they had more than one open face and because of jointing that may have occurred from preceding blasts.

7. An Example for a Damage-Estimate

During an excavation project blasts were conducted in the vicinity of a stone house. Vibrations were controlled from two blasts as shown in Table III. K-factors were relatively high. The construction firm proceeded to blast without further control in spite of the concern of the firm that conducted the seismic control. Considerable cracks occurred after an uncontrolled shot at a scaled distance of only 1.9. To allow for the benefit of doubt, the equation (5) is now applied to predict possible peak velocities. Notice that with equation (5) K-factors are generally smaller than those obtained with equation (4), see also Table I. Equation (5) showed the smallest increase of peak velocity with a decrease of the scaled distance. Assuming various K-factors with equation (5) we can see in Table III that the house must have undergone vibrations far in excess of 2 in/sec from the blast. As the previous controlled shots had rather large K-factors between 30 and 40, it is most likely that the house experienced peak velocities ranging between 10 to 15 times the maximum tolerable level. It therefore was very probable that the cracks were a result of the blasting.

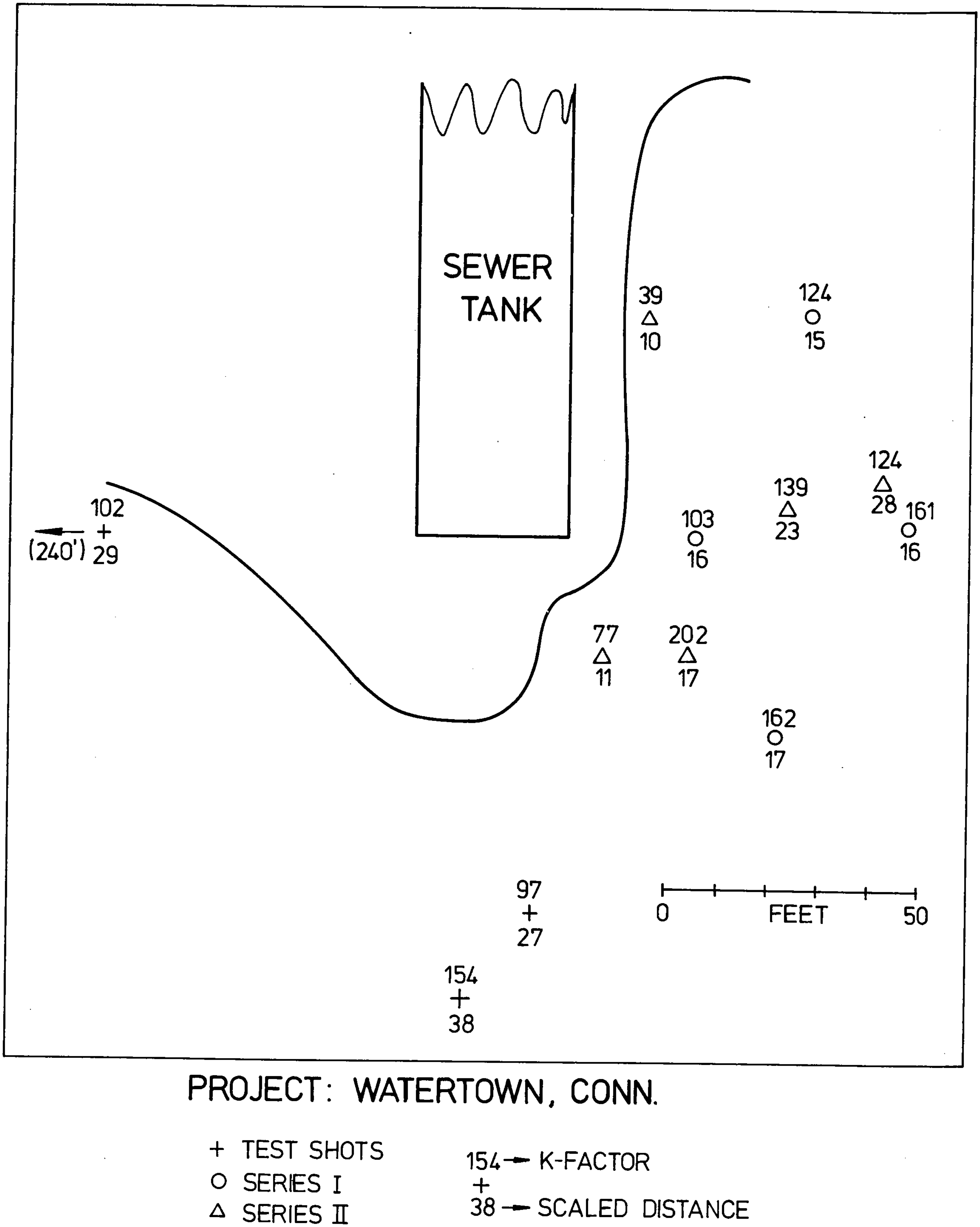

Fig. 6: Charge-distance relationships for a project at small scaled distances,

	D (ft)	W(lbs)	V(in/sec)	S.D.	K
controlled shots	75	6.83	1.97	28.7	41
	90	6.83	1.36	34.4	34
uncontrolled shot	5	6.83	?	1.9(!)	?

based on $V = K \frac{W^{2/3}}{D}$ the possible peak-velocities of the uncontrolled blast were for different K factors:

K	4	10	20	30	40
V(in/sec)	2.90	7.25	14.5	21.7	29.0

Table III : Estimation of the possible peak velocities that a house may have suffered from due to explosion damage. Estimates were based on two controlled explosions.

8. Conclusion

The use of explosives for construction of highways and foundations is a necessary, effective method of breaking rocks. In urban areas the control and protection of nearby structures is of major concern to keep cost over-runs low. Seismic control cannot be limited to mere measurements of the vibrational velocity. Predictive equations that allow one to estimate the maximum peak velocities at a given scaled distance can be used in the planning stage of an explosion program. Because of the large variance of statistical means obtained from test shots, the predictive equations for maximum peak vibrational velocities have to be used with caution. We emphasize that particular attention be directed to vibrational velocities that are above the predicted values, if K-factors are obtained from an average of a large number of shots. We prefer to use constants in the equations so that they can be useful as a guide leading to the highest possible peak velocity.

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EFFECT OF FALL LINE GEOLOGY ON DESIGN OF US 64

ROCKY MOUNT, NORTH CAROLINA

by

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ABSTRACT

The proposed US 64 By-pass of Rocky Mount, North Carolina is located in the Fall Line Physiographic Zone. A detailed geotechnical investigation was performed along the project to identify all design and construction problems related to the subsurface conditions.

Geotechnical survey data indicated that the stratigraphy of the project corridor consists of surficial beds of sand and highly plastic clay that overlie extensive deposits of soft marine clay probably belonging to the Yorktown Formation. The base of the Yorktown rests on granitic bedrock or saprolite, and forms a buried terrace with an associated scarp near the project's western terminus.

The following potential construction problems were recognized by the geotechnical study: a potential slope stability problem was recognized for a major cut section occurring within the Yorktown, potential embankment stability and settlement problems were recognized where approach fills rest on the Yorktown, highly plastic clays were encountered within several cut sections, and the water table was found to be close to grade in many cut sections. Acceptable borrow sources are expected to be difficult to locate because of urbanization within the project corridor.

As a result of geotechnical recommendations, major changes were made in

the design concept of this project. The grade and typical section was adjusted in order to avoid slope stability problems and large quantities of plastic clay. A special box cut with stabilization of the subgrade was proposed for a major cut section in the Yorktown Formation. Soft clay will be removed from underneath bridge approaches in order to achieve an adequate safety factor against failure. In order to reduce construction costs, the typical section was narrowed.

INTRODUCTION

The US 64 project at Rocky Mount, North Carolina consists of constructing a multilaned freeway around the north side of the city during the Highway Improvement Program of 1978-1984. A detailed geotechnical investigation was performed along the project in order to locate and identify all design and construction problems related to the subsurface conditions and to submit remedial design recommendations for the potential problem areas. Major potential construction problems recognized during the geotechnical investigation included the following: a slope stability problem in a major cut section, embankment stability and settlement problems on a bridge approach fill, highly plastic clay encountered in cut sections, and the water table close to grade in several cut sections.

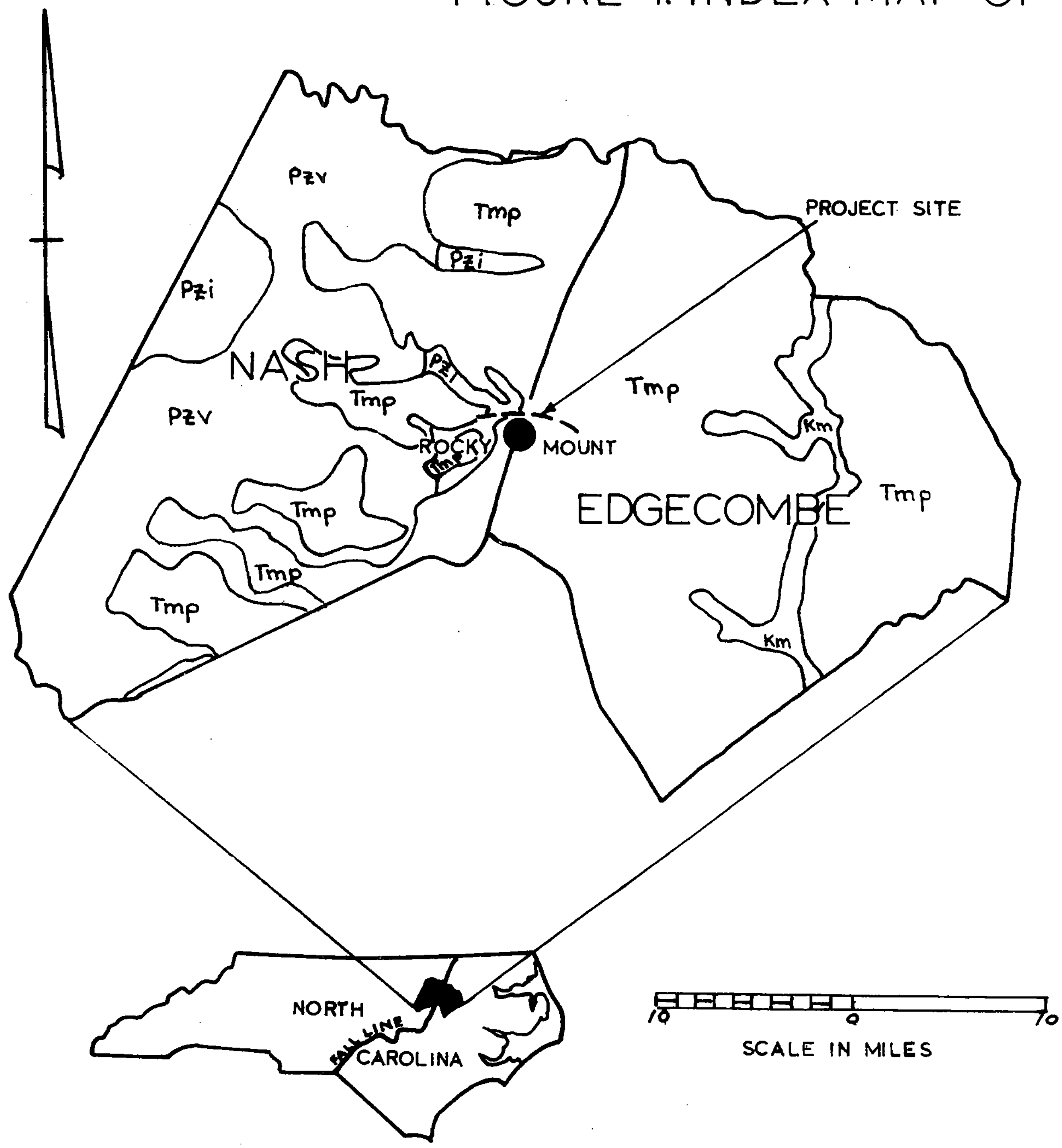
The purpose of this paper is to review the geology and physiography of the project corridor; the potential construction problems encountered, and to present the resulting remedial changes made in the design concept of the project.

LOCATION AND PHYSIOGRAPHY

The US 64 project is located in Nash and Edgecombe Counties at Rocky Mount, North Carolina. The city of Rocky Mount is located in the eastern portion of the state approximately 50 miles north east of Raleigh and 50 miles south of the Virginia - North Carolina state line (fig.1).

The project is approximately 6 miles in length, beginning in eastern Nash County on the Piedmont Plateau and ending in western Edgecombe County on the Upper Coastal Plain. Along its corridor, the project traverses the width of the Fall Zone, which is the boundary between the Piedmont and the Coastal Plain (Stuckey, 1965, p.8). Topography of the western half of the project is characterized by rolling hills, broad floodplains and well developed drainage systems typical of the eastern Piedmont. The eastern half of the project contains large areas of nearly flat topography and poor drainage indicative of the Coastal Plain.

FIGURE 1. INDEX MAP OF PROJECT SITE



ADAPTED FROM THE
GEOLOGIC MAP
OF
NORTH CAROLINA

DEPARTMENT OF CONSERVATION
AND DEVELOPMENT
1958

LEGEND

CENOZOIC	Tmp	YORKTOWN FORMATION MIOCENE–PLIOCENE
MESOZOIC	Km	MIDDENDORF FORMATION CRETACEOUS
PALEOZOIC	Pzi	GRANITIC INTRUSIVES
	Pzv	METAVOLCANICS

The Tar River is the principal drainage feature in the region and crosses the project near its center just north of Rocky Mount. The river is entrenched at this point and has produced a well developed alluvial terrace that is approximately 2 miles wide.

REGIONAL GEOLOGY

The basic geology of the region west of the Fall Line consists of Pleistocene terrace deposits underlain by a thick sequence of folded metavolcanic rocks consisting of slate, phyllite and breccia. Age of the metavolcanics is generally placed in the early Paleozoic. Wide zones of granitic intrusives are present within the metavolcanic sequence and are considered to be of late Paleozoic age. Along the Fall Zone and eastward to the Upper Coastal Plain, the surficial sands and sandy clays become underlain by extensive deposits of marine sands and clays belonging to the Yorktown Formation (fig. 1). The Yorktown sediments have a characteristic blue-gray color and are often fossiliferous. In earlier times the fossiliferous clays were extensively dug in this area and used as a source of agricultural lime. (Mundorff, 1946, p. 6) However, today the Yorktown is best known among highway geologists and engineers for the extremely poor engineering characteristics that it exhibits, and the resulting construction problems that it often causes.

The stratigraphic relationships of the Yorktown Formation in North Carolina are somewhat confused in existing geologic literature, and a complete biostratigraphic analysis is needed to accurately correlate the stratigraphy to the typical sections further north. Age of the Yorktown Formation traditionally has been assigned to the Upper Miocene, but recent studies by the Smithsonian Institution and others have indicated that it may be as young as Pliocene.

Underlying the Yorktown Formation throughout the project area is a base of granitic bedrock or dense sandy saprolite. Outcrops of granitic rock are exposed occasionally along the Fall Zone in stream valleys and other topographic low points.

INVESTIGATIVE TECHNIQUES

During the subsurface investigation for this project, several types of tests were conducted to completely evaluate the subsurface conditions. The field investigations included auger borings, standard penetration tests, undisturbed sampling and rock coring. The water level was also measured in all borings during the course of the investigation.

Auger borings were generally taken at intervals of 200 feet along the project in fill sections and 100 feet in cut sections. These borings were made to determine the depth and lateral extent of the various soil horizons encountered on the project, and to provide bulk samples for soil classification tests.

Hollow stem augers were used when making standard penetration tests, and obtaining both split spoon and undisturbed Shelby tube samples. The "N" values obtained from the standard penetration tests were used in conjunction with the sample test results in making foundation, slope stability and settlement analyses.

Rock coring was performed where it was felt that the presence of bedrock would influence the design of a bridge foundation. Cores were taken to determine the type, degree of weathering, Rock Quality Designation (RQD), and percent recovery.

Soil samples obtained from the various field tests were submitted to the Department of Materials and Tests labratory where the appropriate tests were made. Bulk samples and split spoon samples obtained from standard penetration tests were tested for grain size distribution and Atterberg limits, and assigned the appropriate AASHO soil classification. The undisturbed samples obtained from Shelby tubes were given the same classification tests on the other soil samples, but were also tested for void ratio, moisture content, specific gravity and density. Triaxial shear strength tests were also conducted in the labratory to obtain values for the cohesion and phi angle of the soil. Where it was felt that settlement would be a problem, Shelby tube samples were given consolidation tests to obtain an e-log p curve and the coefficient of consolidation for the questionable clay layer.

Personnel from the Geotechnical Unit were able to test for the moisture content of any soil sample through the use of the "Speedy Moisture Tester". Unconfined compression tests were also run on some Shelby Tube samples by our personnel to obtain the cohesion of questionable clay soils.

POTENTIAL CONSTRUCTION PROBLEMS

A. Stability Problems at -Y- Interchange

The -Y- interchange is located at the project's western terminus on gently rolling terrain at an elevation of 120 to 135 feet, and consists of the junction between the proposed U.S. 64 bypass and existing U.S. 64. Included in this interchange is a major cut section and a flyover for the connector Ramp B to existing eastbound US 64 (fig. 2).

An evaluation of the boring logs from the interchange indicated that the area is underlain by both Piedmont and Coastal Plain stratigraphy. Soils in the western half of the interchange are typical of those found in the eastern Piedmont, and consist of clayey sands with interbedded stiff plastic clays. Granitic rock is above the preliminary grade in the western half at depths of 5 to 15 feet, but plunges abruptly near the middle of the interchange forming a buried scarp like feature. East of the plunging rock face, the surficial sands are underlain by the soft (N=1-3) and wet (50 percent moisture) clay of the Yorktown Formation. A gentle eastward sloping surface of granitic rock and dense saprolite underlies the Yorktown at an elevation of 90 to 95 feet, and forms what is postulated to

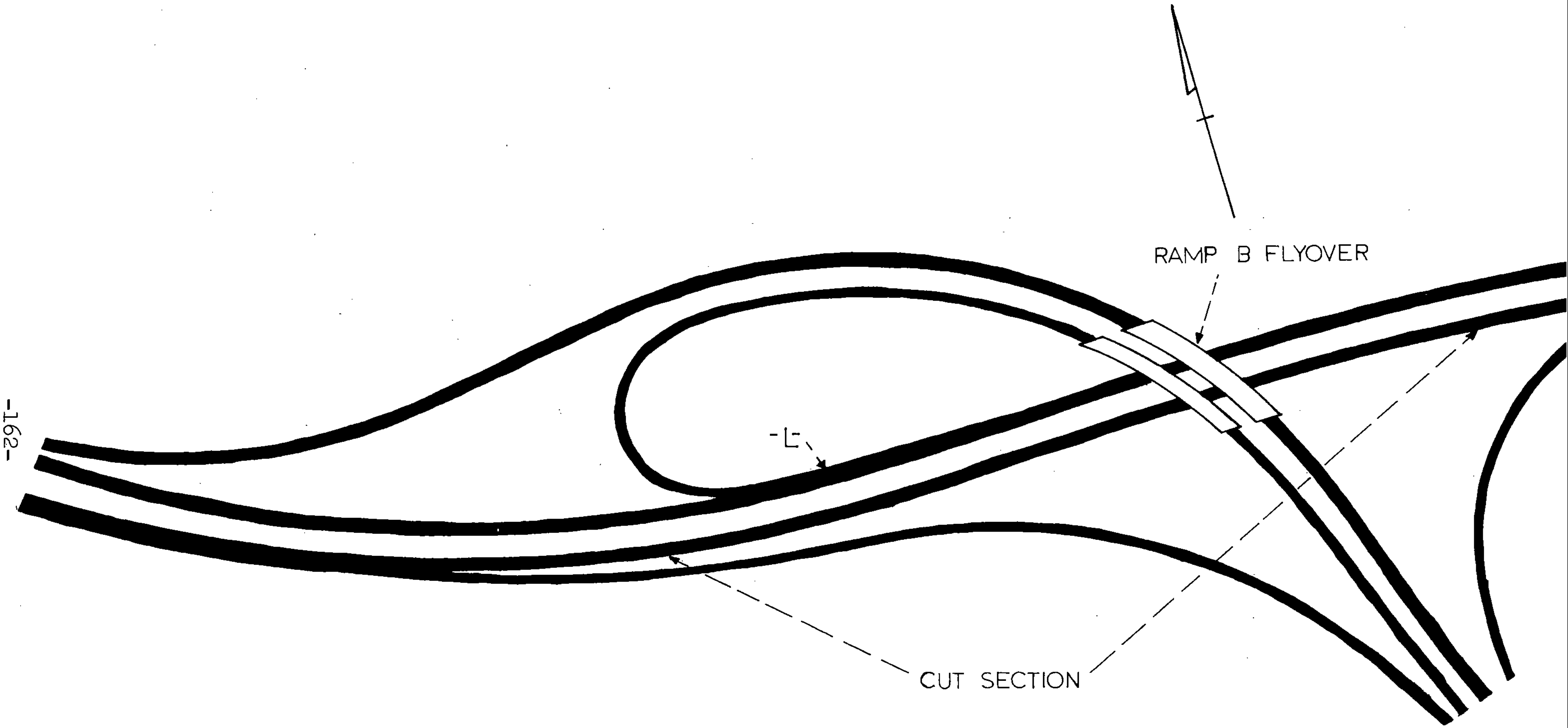


FIGURE 2. LAYOUT SKETCH OF -Y- INTERCHANGE

FIGURE 3. PROFILE VIEW OF CUT SECTION THROUGH -Y- INTERCHANGE

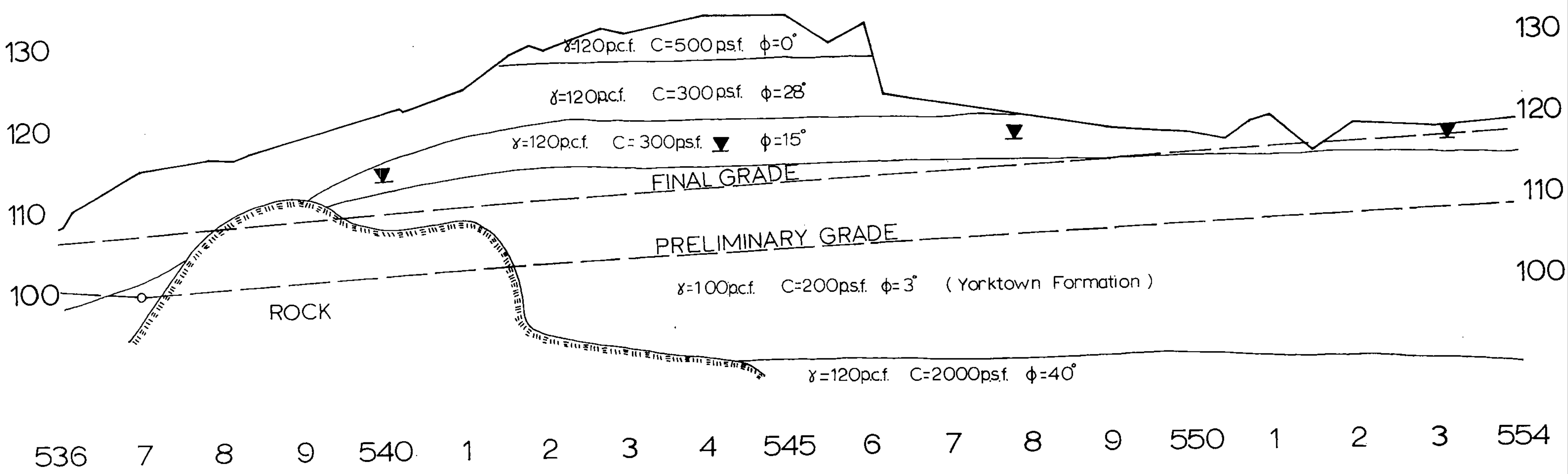
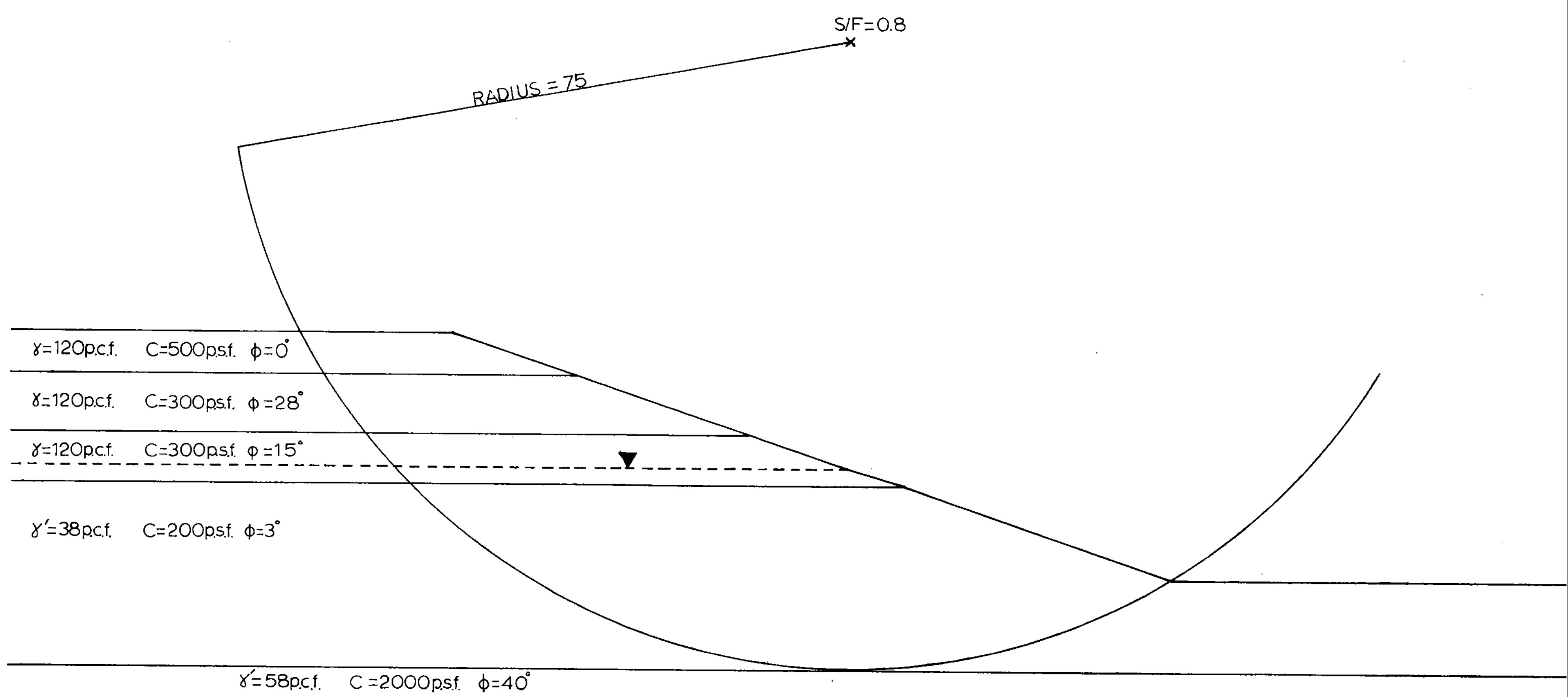


FIGURE 4. SLIP CIRCLE ANALYSIS OF PRELIMINARY GRADE DESIGN (LOW GRADE)



be a buried marine terrace (fig.3). It is the presence of the Yorktown, which is approximately 25 feet thick in this area, that causes concern for the following potential construction problems: (1) slope stability and unsuitable material, (2) embankment stability.

1. Slope Stability and Unsuitable Material

The preliminary grade for the proposed roadway in this area was designed as low as possible through the major cut section on the project due to a clearance problem with an existing overpass just west of the interchange. Also, it was assumed that the excavation would supply material needed for the high fills further down the project.

As a result of the geotechnical investigation for this project, which included a slope stability analysis, it was determined that the preliminary grade with its maximum depth of 29 feet would result in a major slope stability problem with a safety factor of 0.8 (fig. 4). In addition, it was determined from routine testing that the Yorktown clay with its extremely low shear strength and high void ratio would be unable to support the proposed roadbed.

2. Embankment Stability

The approach fills for the proposed Ramp B flyover reach a height of 20 feet, and are situated directly on a thin bed of stiff, highly plastic clay that is underlain by the Yorktown Formation. Most of the surficial granular soil that formally covered this area was removed for borrow during the construction of existing US 64.

Using the test information obtained during the bridge foundation investigation and from Shelby tube sample results, calculations using New York slope analyses indicated that the proposed embankment would exhibit an unsatisfactory safety factor of 0.7 (fig. 5). In addition, the embankment would experience a severe settlement problem. Calculations indicate that the total settlement would be

10.53 inches, and it will take 9.5 years for 90 percent consolidation.

B. Additional Problems

The remaining potential construction problems on this project are more typical of those found on the average construction project in the Upper Coastal Plain. The problems consist of (1) highly plastic clay near or at grade, (2) adverse ground water conditions and (3) borrow quanities.

1. Highly Plastic Clay Near or at Grade

Several cut sections on this project encounter sections where stiff clays of high plasticity are near or at grade. Based on the sieve analyses, Atterburg limits and "N" values of these clays, it was determined that they would be unsuitable for use in the subgrade of the proposed roadway due to their high plasticity (50), liquid limit (80), and 90 percent silt-clay content.

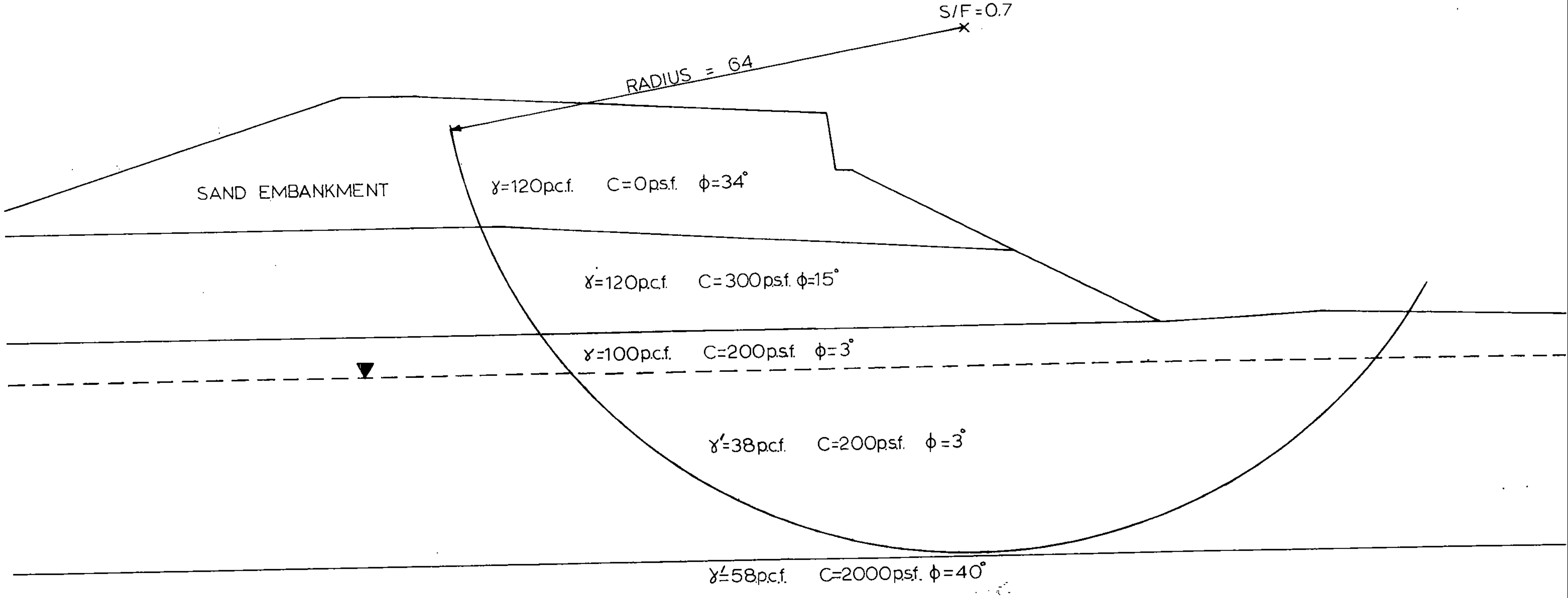
2. Adverse Ground Water Conditions

Several sections of this project, including the -Y- interchange area, had water levels that indicate potential compaction and stability problems in the subgrade. The areas of potential ground water problems were encountered in the following situations: where the grade of a cut section intersects or is near the water table, and where the grade is situated on nearly flat terrain with poor drainage and an extremely high water table.

3. Borrow Quantities

Although there are extensive deposits of granular material nearby consisting of granitic saprolite and terrace sands, increasing urbanizations of the project corridor will make acceptable borrow sources difficult to locate. Preliminary estimates indicated that approximately 6,000,000 cubic yards of borrow would be required for construction of this project. The high borrow figure is a result of the following factors:

FIGURE 5. SLIP CIRCLE ANALYSIS OF APPROACH FILL



A. The 20 to 30 foot high fills required for bridges overpassing 3 main highways, 2 secondary roads and a main line of the Seaboard Coast Line Railroad.

B. The large quanity of unsuitable excavation due to undercutting soils that exhibit such poor engineering properties as high organic content, high plasticity and low compressive strength.

C. The large quanity of excavation material from the cut sections that are unsuitable for use in construction due to high plasticity and poor workability.

REMEDIAL DESIGN CHANGES

A. Slope Stability and Unsuitable Material at -Y- Interchange

Upon the recommendation of the Geotechnical Unit, the grade through the interchange was raised 8 feet to increase the safety factor of the cut slopes and reduce the large quanity of unsuitable clay and rock, but still maintain the necessary bridge clearance. However a slope stability analysis at the new proposed grade with 3:1 slopes indicated that a slope stability problem with a safety factor of 0.9 would still exist (fig. 6). As a result, the slopes were flattened to 4.5:1 giving an acceptable safety factor of 1.1 (fig. 7).

In order to eliminate the problem with the soft Yorktown clay below grade, the material will be undercut 6 feet below grade in a box cut including the median and backfilled with select borrow. Settlement of the resulting 6 foot thick roadbed is estimated to be less than 0.1 foot.

B. Embankment Stability and Settlement

The remedial recommendations for the unstable approach fills for the proposed Ramp B flyover at -Y- consist of undercutting the soft Yorktown clay beneath the proposed front slopes of the embankment down to the surface of granitic rock or hard saprolite at an elevation of 92 feet. Select borrow will then be backfilled in the undercut areas, resulting in a safety factor of 1.9 and virtually eliminating settlement (fig. 8).

FIGURE 6. SLIP CIRCLE ANALYSIS OF FINAL GRADE WITH 3:1 SLOPES

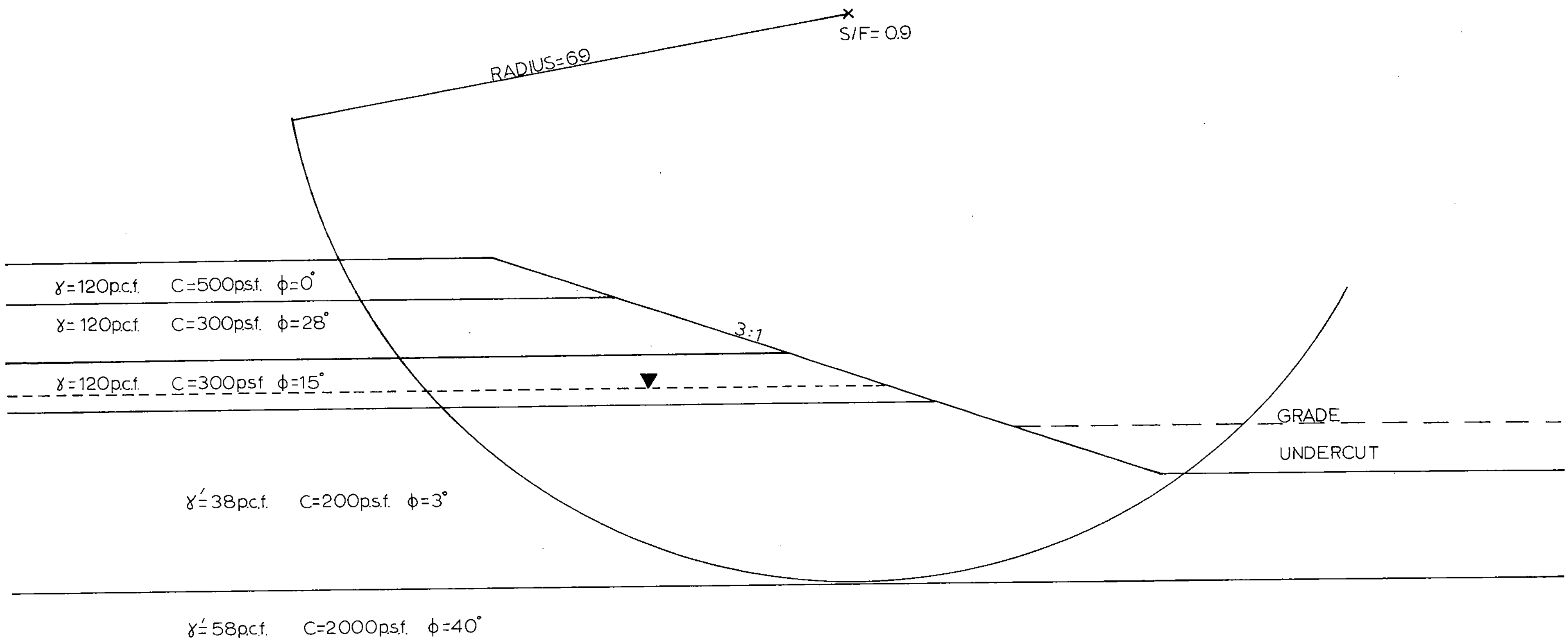


FIGURE 7 SLIP CIRCLE ANALYSIS OF FINAL GRADE WITH 4.5:1 SLOPES

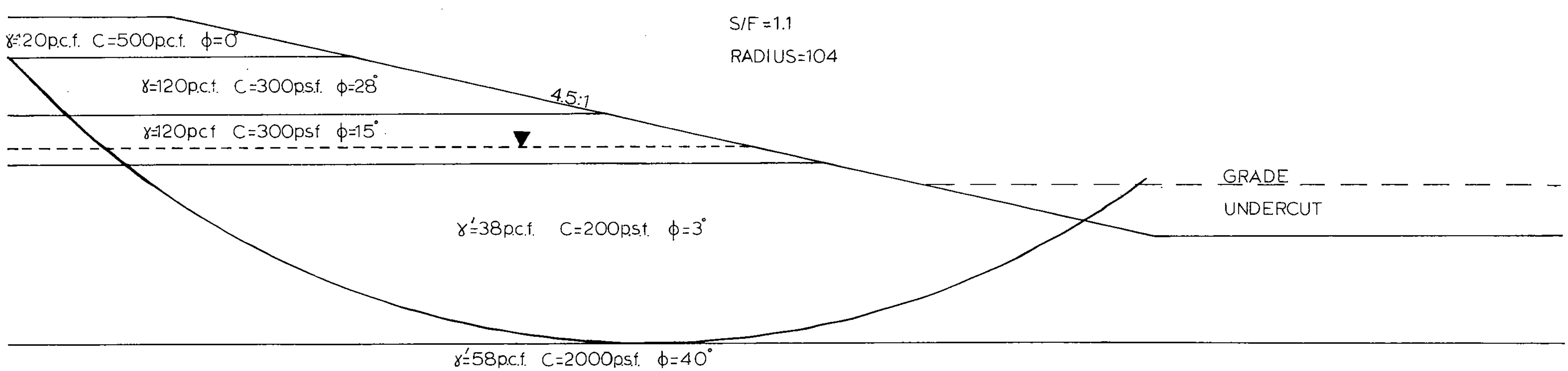
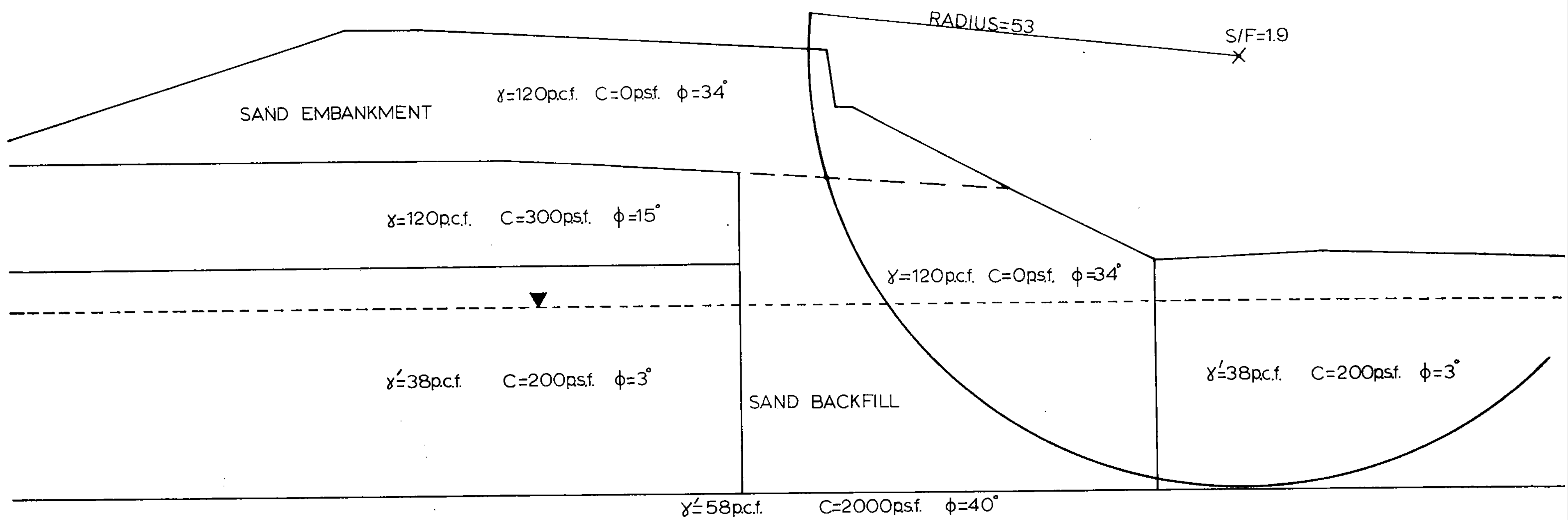


FIGURE 8. SLIP CIRCLE ANALYSIS OF REMEDIAL DESIGN FOR APPROACH FILL



C. Highly Plastic Clay

Where highly plastic clay is encountered below grade, the material will be undercut 3 feet below grade and backfilled with select borrow.

D. Adverse Ground Water Conditions

Where the water table is near or at grade and there is an adequate gradient for outfill, approximately 12,500 feet of perforated underdrain will be used as follows:

1) Depth of underdrains should be a minimum of 6 feet below grade.

2) Where practical, grades of underdrains should be a minimum of 0.5 percent.

3) In dual lane sections, underdrains should be installed within the outer shoulder of each lane.

4) In single lane sections, underdrains should be installed within the shoulder on the upslope side.

In the areas of nearly flat topography where the water table is near or at grade, special lateral deep ditches have been designed to eliminate water from the subgrade.

E. Borrow

In order to reduce the large estimated borrow quanity on this project, the grade was raised and the median width was narrowed from 68 to 46 feet. These design changes resulted in a reduction of the estimated borrow quanity to approximately 2,350,000 cubic yards.

LIME STABILIZATION

In order to reduce the depth of undercut in unsuitable clay soil, an item of lime stabilization has been included in this project. It is estimated that there are 7,000 square yards of clay subgrade that can be stabilized by using 60 tons of lime. The following procedure will be used during the stabilization process: Hydrated lime, approximately 3 to 5 percent, will be used to stabilize the clay subgrade to depths of 8 to 12 inches depending on conditions. After compaction, the lime stabilization layer will be cured for 7 days using bitumnous curing seal consisting of cut back asphalt, grade RC-250 or asphalt emulsion, grade SS-1 applied at a rate of 0.10 to 0.22 per square yard.

SUMMARY

As a result of the geotechnical investigation and subsequent recommendations, major changes were made in the design concept of this project. The grade was raised and the typical section was adjusted in order to avoid slope stability problems and large quantities of rock and plastic clay. A special 6 foot deep boxcut with stabilization is proposed for a major cut section in the Yorktown. Soft Yorktown clay will be undercut from underneath bridge approaches where necessary to achieve an adequate safety factor against embankment failure. Stiff plastic clay will be undercut to a depth of 3 feet below grade and stabilized. In areas where ground water is anticipated to be a problem, underdrains will be placed where there is sufficient gradient and special deep lateral ditches will be constructed in areas with poor drainage. In order to reduce construction costs on this project, the typical section was narrowed.

ACKNOWLEDGEMENTS

The writer expresses his appreciation to the Geotechnical Design Section of the North Carolina Department of Transportation for their assistance with the stability and settlement portions of this paper. Also, to the Soils Investigation Section of the North Carolina Department of Transportation for the lime stabilization data included in this paper.

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DESIGN ALTERNATIVES FOR CONSTRUCTION OVER COMPRESSIBLE MATERIALS

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May 1978

ABSTRACT

Similarities of terrain conditions of the Atlantic Coastal Plain, east of the Fall Line, and the Gulf Coastal Plain, south of the Pleistocene terraces, are presented. The geological setting and history of late Pleistocene and recent deposits of the Louisiana Coastal marshland are described.

Discussed in detail are the roadbuilding conditions typical of Louisiana marshland, the roadbed design options considered, the economic factors involved, and the construction problems posed. The rationale for using construction materials and techniques which are departures from the conventional is presented. The concept adopted, of using lightweight embankment materials to minimize settlement, is not new; however, the use of clam shell as a lightweight embankment material for a major highway project is.

Controlling factors requiring consideration when lightweight materials are deemed feasible as embankment, such as terrain conditions, cost, and availability are discussed. Essential properties of a lightweight embankment material are presented.

It is proposed that the use of lightweight materials for construction of embankments over compressible materials in both the Gulf Coast and the Atlantic Coastal Plain be considered as a viable practice.

GEOLOGICAL SETTING

The construction site under discussion is a fresh water marsh in coastal Louisiana, 40 miles southwest of New Orleans. The terrain is flat, soft alluvium approximately 350 feet thick, of recent age, overlying the Prairie Terrace of Pleistocene age. The surface and near surface soils are highly organic, and have natural moisture contents nearly equal to their liquid limits. This results in a consistency comparable to soft ice cream. The site was probed by a 3/4 inch pipe and was easily penetrated to 12 feet, pushed with one hand. Deep foundation borings revealed these soft soils exceed 50 feet in depth.

The depositional environment for this alluvium was a basin, some six miles across, between the natural levees of two streams. The natural levees are themselves partially buried below these backswamp alluvial deposits, and these levees are the best foundations along the coast below the Pleistocene terrace outcrops. Similar depositional environments exist along the entire Gulf Coast, as well as the Atlantic Coastal Plain.

Unlike the Piedmont - Coastal Plain contact, the Gulf Coastal Plain lacks a well defined fall-line separation from the interior uplands. However, there exists a well defined hinge line flexure in the subsurface which runs along the northern periphery of the Gulf of Mexico geosyncline. This hinge line is just north of New Orleans and slightly south of Baton Rouge. Pleistocene terraces that have a normal dip of 1 - 5 feet per mile north of this hinge line plunge to a dip of 10 to 30 feet per mile just south of the line, and to much steeper gradients at the coast line. It is this change in dip south of the hinge line which created an environment of deposition resulting in several hundred feet of underconsolidated alluvium from the Lafourche course of the Mississippi River. Based on radiocarbon dating and Indian culture artifacts, the Mississippi River abandoned its Lafourche Delta course 450 years ago and occupied its present course below New Orleans. It is estimated that the upper 50-100 feet of alluvium at the project site is from 200 to 800 years of age, and is a completely saturated heavy clay with humus.

The Gulf Coast, like the southern part of the Atlantic Coastal Plain, is still experiencing active subsidence. There is evidence of this subsidence in the vicinity of the project under discussion.

There are stream systems with their associated levees completely buried under recent alluvial deposits. There exist trees indigenous to dry ground environments which have died within the last 20-30 years due to their root systems becoming completely submerged below water. There is evidence of two to three feet of subsidence in the last 80-100 years in this area. Elevations at the job site vary from zero to four feet above mean sea level. Changes in vegetation are good clues of the location of buried streams and levees in this type environment.

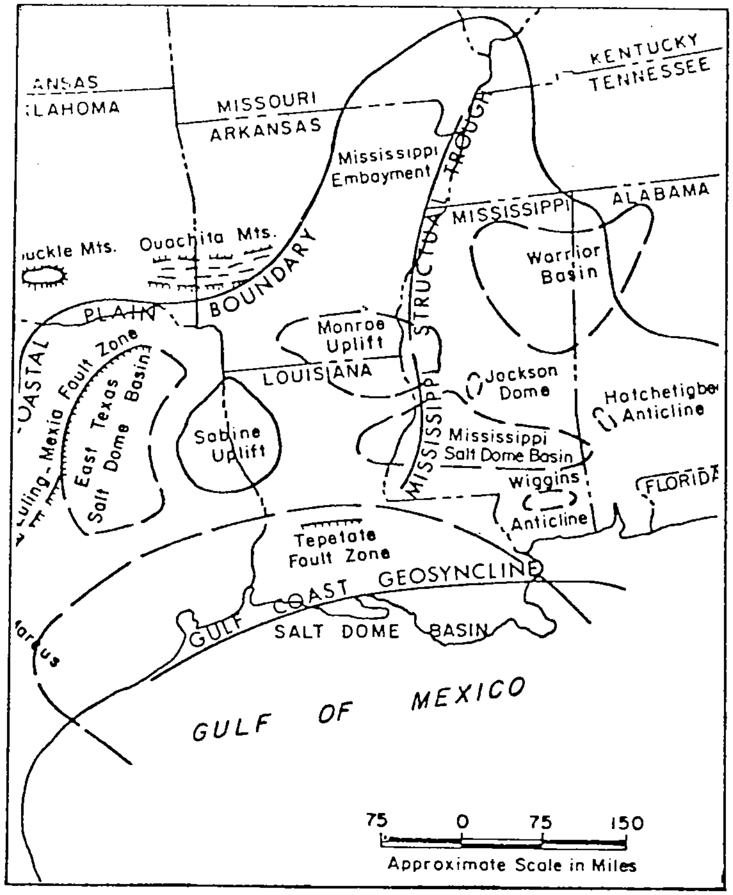

Figure I - Major Structural Features of Gulf Coastal Plain

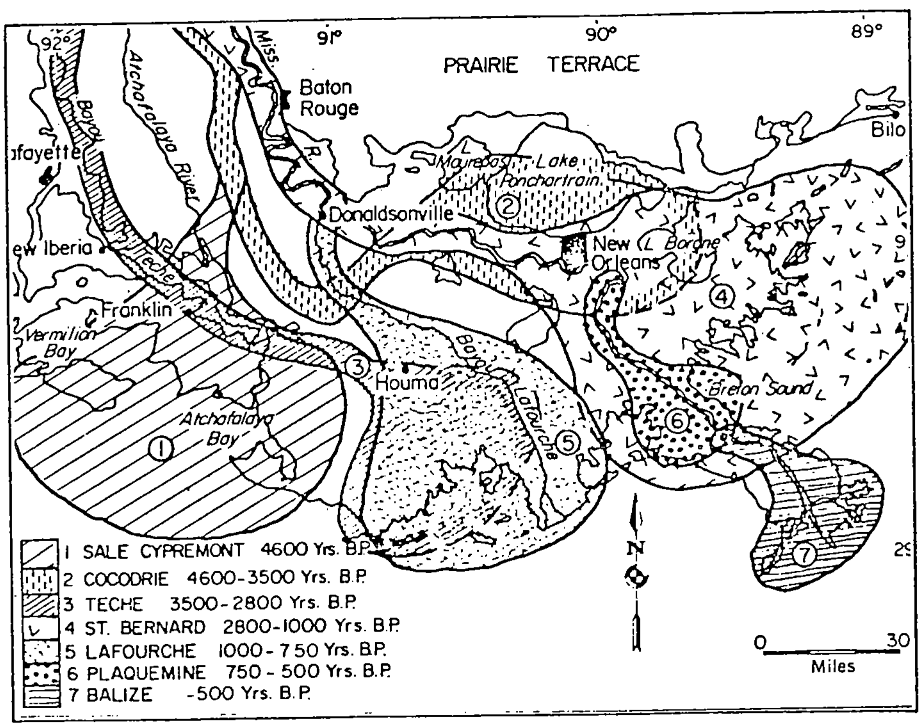

Figure 2 - Succession of Mississippi Deltas

SIMILARITIES OF ATLANTIC AND GULF COASTAL PLAINS

These physiographic provinces share many likenesses. They are similar in surface features, such as beaches, bays, estuaries, swamps and marsh lands. Both coastal plains have little surface relief, and elevations near the coast are very low.

Additionally, both plains are composed of relatively young, unconsolidated sedimentary deposits near the coast line. Many of these deposits consist of soft clays, some with high organic contents.

Another likeness is that both provinces are presently experiencing subsidence, with the area near the Mississippi River delta having a very high rate.

TYPICAL DESIGNS

Design practices for roadway embankments are usually dictated by terrain conditions, availability of materials, economics, and long established practices of the area.

Designing embankments for coastal marsh and swampland is one of the bigger challenges facing practitioners in the areas of soils and geotechnical engineering. Attempting to develop design alternatives that will prove structurally sound over terrain that consists of mushy clay and organic soil to depths in excess of fifty feet can hardly be construed to be a routine exercise. This is the problem we were faced with.

Our early road system along the coast usually followed ridges or natural levees of streams. This often resulted in a road that twisted and wound its way between terminal points, increasing travel distance over straight line distance by as much as 50 - 75 percent. But, leaving these ridges and levees and traversing marsh or swamps resulted in design changes and special construction techniques and practices.

For decades, our usual practice was to use side borrow for an embankment, and allow it to settle for as much as ten years. With modern traffic demands, this settling period became impractical. Our second generation practice was to muck out compressible materials and then fill with side borrow or pumped sand. This was done when the soft materials were fifteen feet or less in depth. In some cases, we mucked out the worst material, but were faced with soft, compressible soil below the muck line. In order to minimize future settling, we used a surcharge.

Regardless of the construction techniques specified, the embankment materials always consisted of side borrow clay or sand, or pumped in sand.

Route U.S. 90 west of New Orleans is one of the older highways that followed old ridges and levees, resulting in high excess mileage over straight line distances. This facility is being upgraded to a four lane highway for some 150 miles west of New Orleans. It was decided to bypass urban areas and shorten the route. To do this requires traversing some of the worst terrain for foundations in the state.

Upon analyzing the results of the subsurface exploration of the area west of the town of Raceland, we realized that the typical designs and practices would not work in this area. We were forced to rethink our approach to embankment design. The result was an innovation that appears to be working.

PROJECT DESIGN ALTERNATIVES

The objective of constructing the project under discussion is to relocate some twenty-nine miles of U.S. 90 through some of the worst road building terrain along the Gulf Coast. The proposed new highway is to be a four lane facility divided by a depressed median. The desired finished grade elevation is six feet above sea level. Rigid pavement designs are not feasible for this terrain, thus the pavement section selected must be a flexible section.

The soils exploration borings indicated three foundation conditions for the project. One section, nearest the natural levee of Bayou Lafourche, would support a soil or sand embankment constructed in a conventional manner. This type construction would usually cost 1 1/2 to 2 million dollars per mile for embankment alone.

The second section, west of the first, if constructed of sand or soil, would require mucking 10 feet deep by 250-300 feet wide. This type construction would cost 2 - 3 1/2 million per mile.

The third section, worse than the other two, would have to be built similar to the second section, but the estimated cost of embankment would exceed 5 million per mile, due to subsidence and difficulty of access.

The design studies revealed a projected cost of 85-100 million dollars for embankment construction alone for 29 miles of highway. These costs did not include acquisition of right-of-way, bridges and drainage structures, utility relocation, or the base and pavement structure.

The Department had experience in marsh and swamp construction using the technique of mucking out highly compressible surface soils, and replacing them with sand pumped

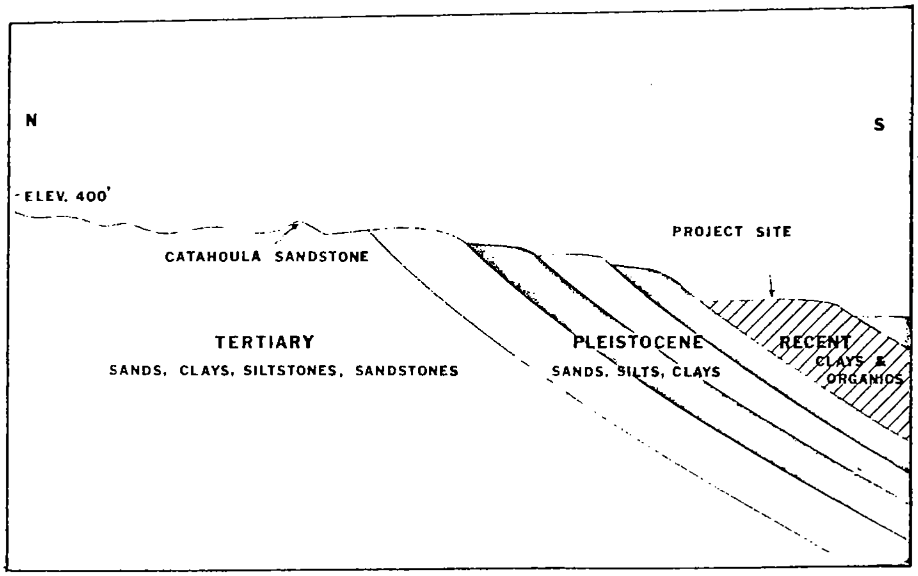

Figure 3 - Generalized North-South Stratigraphic Section of Louisiana

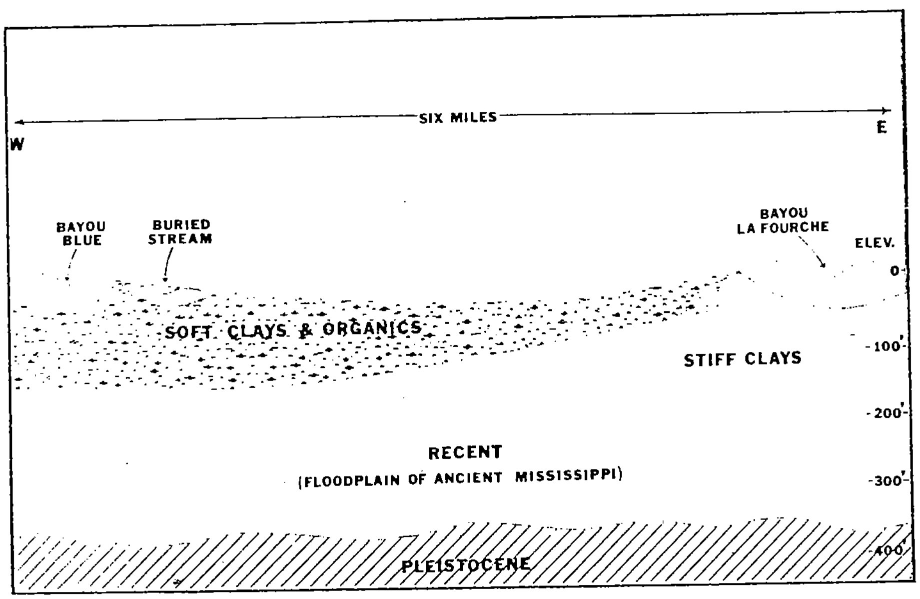

Figure 4 - Stratigraphic Section of Project Site

from the Mississippi River. The farthest we had pumped sand hydraulically was some 15 miles, but this required four booster pumps. The project under study is 21 miles from the Mississippi. It was determined that such an operation was technically feasible, but was an economic albatross. Mobilization for such an endeavor would run about 10 million dollars.

At this point, soils design and materials personnel began searching for alternates to soil or sand as embankment materials over the worst two sections. It was recognized that the wet unit weight of soil and sand, 110-130 lbs/ft^3, was one of the major problems, necessitating mucking and berms.

There are several lightweight materials which are feasible for use as embankment over compressible soils. Many porous volcanic rocks, such as pumice or scoria, porous Florida limestone, as well as vitrified clay or slag would be acceptable. In some cases, sawdust, wood fiber, or bagasse, the waste pulp from sugar cane, would be satisfactory as a mat below the embankment.

In the evaluation process, we developed a set of desirable physical properties for lightweight fill material. These are:

1. Loose dry weight density of less than 80 lbs. cu. ft.
2. High permeability
3. Adequate shear strength (particle interlock)
4. High crushing resistance
5. Inert to the effects of water and organic matter.

As a result of these evaluation processes, we concluded that we were left with one lightweight material, shell, which met our criteria. In order to comply with state law, we developed two design alternates, a section with shell embankment, and one with sand embankment which required muck excavation.

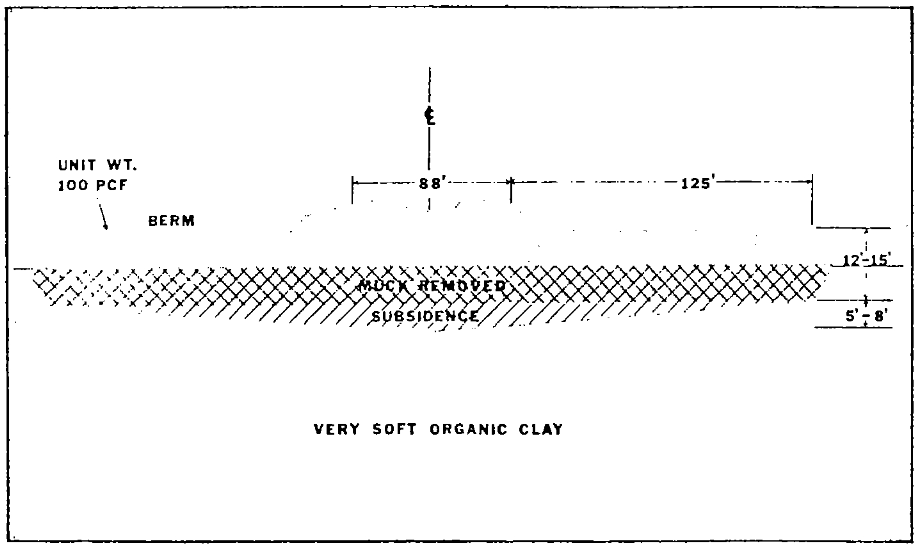

Figure 5 – Proposed Typical Section – Sand

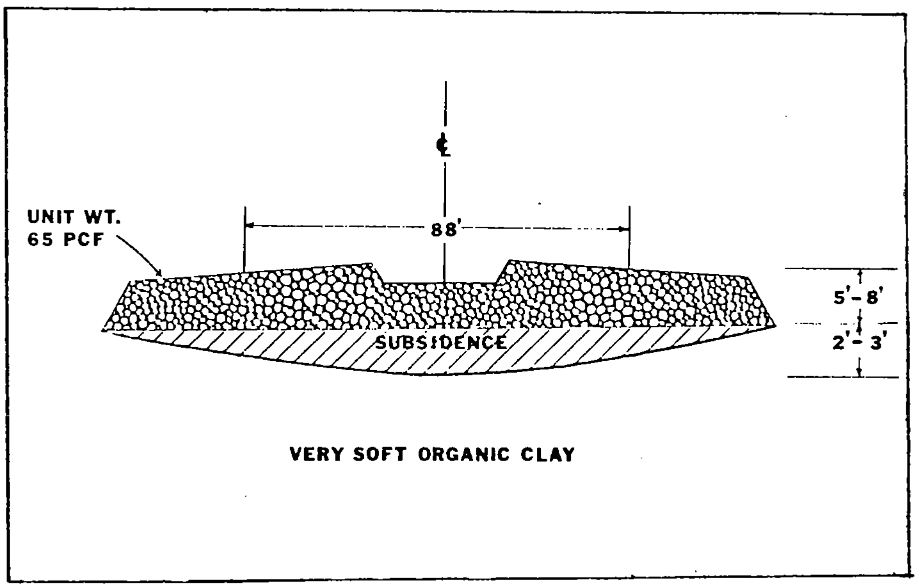

Figure 6 – As-Built Typical Section – Shell

Experience in Louisiana with shell and sand as construction materials indicate that shell, due to its size and shape, has greater shear strength. Triaxial and unconfined compressive strength tests in the laboratory confirm this. The combination of lighter unit weight and superior shear strength are factors favoring the use of shell over sand. However, the controlling factor is usually the comparison of total cost.

The only abundant, natural lightweight aggregate or soil in Louisiana is the shell found in lakes and bays near the coast. There are both clam and oyster shell beds in these brackish lakes. Clam shell is more resistant to degradation than oyster shell, and relatively light at about 60 lbs/ft^3. Its natural angle of repose (friction) is about 45 degrees, compared to only 30 degrees for fine sand, and it is very permeable and unaffected by submergence in water.

In order to make a valid evaluation of the clam shell embankment, it was decided to build a test section on representative terrain. A section 350 feet long was built in the marsh for this purpose, and a D-6 dozer made 2000 passes on the shell embankment to simulate traffic loads, but more important, to induce vibration.

Measurements on the test section indicated that the shell embankment of 5-6 foot thickness immediately compressed the underlying marsh, settling one to 1 1/2 feet. It was found that settlement virtually ceased after four months, and the total final settlement was roughly two feet. A significant thing about the settlement is the fact that it was uniform, not differential. The computed settlement of the sand alternate design was of the same magnitude. Plans and specifications were developed for the 4.5 miles of highway over the worst terrain encountered on relocated U.S. 90. Included were two alternates, a sand typical section requiring mucking and berms, and a shell section.

COST AND TIME COMPARISON

The actual bid price for 4.5 miles of clam shell embankment for a four lane, divided highway, was 5.6 million dollars. This was based on a unit price of $6.50 per cubic yard for 870,000 yards.

The alternate bid for 2.8 million yards of sand at $5.20 per cubic yard would have been $14.5 million, and there would have been 1.9 million yards of muck removal at $2.00, adding another $3.8 million to the price for this type construction.

The total bid cost for this 4.5 miles of highway, including drainage, but not including base course and surfacing, was $9 million for the shell alternate. The sand alternate was estimated to be $21.7 million.

A consideration which ranks nearly equal to project cost is time required for completion. In this aspect, lightweight materials enjoy a definite advantage over conventional fill materials for soft, compressible terrain.

Conventional techniques include either muck removal or surcharging, or both processes. These are time consuming procedures, especially when surcharging is required. Surcharges of six months to one year duration are not unusual in Louisiana.

On this particular project, the contract allotted 700 work days. The contractor completed the lightweight shell embankment in less than 300 work days, and in just over one year of calendar time. We estimate that had the contractor bid sand fill with muck removal, the elapsed time would have been three to four years.

The political and public relations merits of such time savings are obvious, and in some cases, could be the deciding factor in the selection of embankment typical sections.

CONSTRUCTION TECHNIQUES & PROBLEMS

The techniques employed included:

1. No mucking -- the marsh vegetation was left in place.
2. End dumping -- the embankment was advanced by truck end dumping.
3. Full thickness -- lift construction was not used.
4. No compaction control -- no density was specified, no compaction was used, except loaded haul trucks.
5. Arrow point advance -- the shell was advanced along the centerline with a 45 degree point maintained.

The test section revealed that 4-5 feet of shell would be required to support the hauling equipment, and this thickness was needed to maintain grade. It was found that the marsh vegetation provided a mat for the shell and helped prevent breakthroughs. We also discovered that if the embankment was advanced uniformly, a mud wave would form, the marsh surface would rupture, and tremendous quantities of shell would be required to bridge these ruptures. The technique developed to avoid the ruptures was the arrow-shaped point advance.

The lack of density requirements was intended for reduction of static load on the marsh. The top foot of the shell densified under the weight and repetitions of the loaded haul trucks. We believe that this dense surface of the embankment will provide the stability needed for construction of the base and surface courses.

CONCLUSIONS

The use of lightweight materials for construction over yielding terrain should be given serious consideration regardless of geographic location. Materials which possess the required properties, whether they are naturally occurring or synthetic, deserve evaluation.

Possible advantages to be realized include money, due to elimination of muck excavation and reduced cross-section, and time, as a result of the lack of mucking and the elimination of surcharging.

Weight-Credit Foundation Construction
Using Foam Plastic as Fill

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Abstract

Methods of foundation construction are described which are based upon the attainment of a weight-credit by the replacement of weak but heavy soils (soft, saturated clays; peats) with lightweight foam plastic.* A case study involving the construction of bridge abutments and approach embankments is reviewed. Potential applications for building foundations, retaining walls, route construction, airport construction, and applications to trenching and pipeline construction techniques are outlined. A sewer project is described where over $2,000,000 could have been saved. Also included are special applications to pile foundations. Costs and methods of cost analyses are described. Pertinent physical, mechanical, and chemical properties of common foam plastics are reviewed, such as compressive strength, water absorption, and density. Aspects of permanency and durability in the "buried environment" are considered.

* Based upon U.S. Patents No. 3,626,702 (Dec. 14, 1971) and No. 3,747,353 (July 24, 1973), E.J. Monahan.

Introduction

If a hole is dug and the material which is removed is immediately used to refill the hole, the state of stress below is unchanged. It follows that if a lightweight backfill is used, there will accrue a "weight credit" for a proposed structure. In an extreme case, if the weight of the backfill and the structure is equal to the weight of the soil removed, then no settlement of the structure can occur. This principle is used extensively in Mexico City to produce so-called "floating foundations" on the highly compressible volcanic clays underlying the region.

In less dramatic cases, lightweight backfills (e.g., cinder fills) may be used to effect a lesser weight credit, thus reducing the net stress increase on compressible materials below.

Apparently because of the scarcity of natural lightweight aggregates and the consequent infrequency of fortuitous circumstances which might dictate its use, the application of the technique has not been extensive.

It is proposed in this paper to use foam plastic as backfill. Technology exists or can easily be developed for on-site production of the foam plastic, or alternatively, the material can be precast for installation at the site.

Illustration of Weight Credit

Consider a site which is overlain by compressible material of soft consistency, i.e., a swamp muck.

Unit weight of muck:		100 pcf
Depth of excavation:		15 feet
Stress release	:	1500 psf
Foam density	:	3 pcf
Stress addition	:	45 psf
Stress credit	:	1455 psf

It is common that in areas where soils of such marginal supporting capacity exist, the typical methods of soil stabilization (e.g., preconsolidation with or without sand drains, the use of compacted sand backfill, etc.) are suitable for developing an improved condition such as to allow building pressures of about 500 psf, corresponding to one-story light industrial buildings or office buildings. It is seen that the stress credit in this example is about three times this value. And this trebled allowable building pressure could theoretically be applied without causing any settlement.

The compressive strength of such a foam is about 30 psi. Thus the building pressure of 1425 psf (10 psi±) would stress the plastic to a level of about 1/3 its compressive strength, a typical safety factor in foundation construction.

A perhaps oversimplified conclusion is that a three- or four-story building would now be possible where only one-story construction would be technically feasible using standard techniques.

Foam Materials

The only foam plastic which is known to have been used in weight-credit engineered construction is a (solid) precast extruded polystyrene foam designated STYROFOAM HI, one of approximately twelve STYROFOAM materials made by the Dow Chemical Company. The STYROFOAM HI was used on the construction of the Pickford Bridge (1).

To illustrate the broad potential for the general use of foam plastics (precast and blown-in-place) in weight-credit construction, consider the

(1) "Polystyrene Foam is Competitive, Lightweight Fill", CIVIL ENGINEERING ASCE, Feb.1974, pp. 68-69. T. Coleman.

following quotation from Modern Plastics Encyclopaedia (1968) (Urethane Foams, p. 346): "... foams may be produced which have densities ranging from less than one pcf to about 70 pcf, with an almost limitless range of chemical and mechanical properties."

In light of the above, it would seem feasible that for very large jobs, or very special circumstances, one could justify the expense of producing a foam plastic of special formulation to suit the particular use. Additionally, one could use combinations of existing products (i.e. STYROFOAM products of different properties) to effect a design. As an example, one could use a "high-quality" STYROFOAM immediately beneath a footing where stress levels are relatively high and a lower-quality (and presumably cheaper) foam plastic below. This would be analogous to the manner in which pavement systems are designed.

STYROFOAM HI*

Selected Technical Data:

1. Compressive Strength, 35 psi minimum (2.5 TSF)
   (at yield or 5% deflection)
   Test Method: ASTM D 1621 - 59T
2. Water Absorption, % by volume, 0.25 maximum
   Test Method: ASTM C-272-53
3. Density - approximately 3 lb/cu. ft.

Permanency, Durability

As already noted, there is only one known instance of large - volume use of foam plastic in a weight-credit construction application, that of the

* All data from available printed brochures of The Dow Chemical Company, unless otherwise noted.

Pickford Bridge where a maximum thickness of five feet was installed; this construction was completed in Dec., 1972. As reported to the writer, there was an immediate settlement of about two inches - apparently an adjustment of the stacked "bundles" of STYROFOAM - but no subsequent movement of significance has been observed since.

With respect to other evidence of permanency and durability, the following can be cited:

(a) Since 1962, STYROFOAM HI has been used in at least 39 installations as insulation for highway pavement systems. In most cases, the amount of plastic used has been about 1 to 3 inches (typically placed directly on the subgrade). Samples of the foam taken from various highways after several years of service have shown very little moisture pick-up.

(b) Accelerated laboratory tests such as freeze-thaw cycling and soak tests have shown very little increase in water pick-up.

(c) The structural performance of a highway insulated with STYROFOAM HI was investigated by personnel of the Maine State Highway Commission (2). Deflections were measured in two insulated sections and one non-insulated (control) section. Deflections were of the order of 0.02 inches in the insulated sections and were well below those of the control section during the spring thaw, the latter reaching a maximum of 0.03 inches.

Other Foam Plastics

As suggested by the earlier quotation from Modern Plastics Encyclopaedia, there are virtually an unlimited number of foam plastics which can be made.

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(2) Maine State Highway Commission, "Insulation of Subgrade-Evaluation of First Year Data", Soil Mechanics Series Technical Paper 66-1, January, 1966.

One product which has recently come to the attention of the writer is "beadboard", a rigid material made by pressing plastic "beads" together into a coherent solid. It is understood that typically the beads are sold by a supplier to a fabricator, who in turn makes the boards and sells them to the contractor.

It is the understanding of the writer that these boards are generally cheaper than Dow's STYROFOAM, but have less desirable properties, notably water absorption (values to 12 per cent, and higher).

As a general (early) conclusion, it would appear that beadboard would be "competitive" in weight-credit applications where loadings are light, larger differential settlements are tolerable, and inexpensive structures are involved.

Cost and Cost Analyses

The in-place cost of STYROFOAM HI at the Pickford Bridge was $40./yd. This is high as compared to the cost of compacted fill in-place (perhaps $8./yd.). However, a comparison of unit costs is not usually valid in appraising the feasibility of the weight-credit method.

Perhaps the best context in which to gauge the worth and economic feasibility of a particular application is to compare the cost of stabilization (or improved stabilization) with that of the real estate value. For example, experts have estimated the value of Lower Manhattan real estate at $400-500 per square foot (about $16,000,000 per acre). The cost of placing Battery-Park fill is estimated at about $14-15 per square foot, with a $40 upper limit when utilities and other costs are considered. Thus, it is clear that "making" this new gound (93 acres) is a tremendously sound investment.

In areas such as The New Jersey Meadows, where the land is also inherently valuable (in a real-estate context) because of its geography, but is undeveloped because of geologic (foundation) shortcomings, a similar type of cost comparison would be valid. Thus, if a foundation construction technique can yield a three- or four-story capability as opposed to the present one-story capability, the client gets three or four times as much floor space.

In summary, it is felt that the use of foam plastic subfoundations is best suited for areas of weak surficial soils which have poor drainage qualities, and especially those areas which are inherently valuable by virtue of their locale. Where deep foundations are precluded by either technical reasons, cost, construction restrictions, or other factors, the technique can be used to excellent advantage. Special cases may also dictate its use.

Case Histories, "Hypothetical" Case Histories, and Suggested Applications

Case Histories

1. Michigan State Highway Department Engineers designed and supervised the construction of a bridge for a stream crossing at Pickford, Michigan, using STYROFOAM HI as backfill for the approach embankment. The bridge was completed and opened to traffic in December, 1972. Details of construction are described in CIVIL ENGINEERING, February, 1974.

2. STYROFOAM has been used in at least 39 installations as insulation for highway pavement systems (starting 1962).

3. Thirteen utility installations were insulated with STYROFOAM between 1964 and 1966. These included sewer lines and water lines. No case of a pipeline application involving the purposeful use of foam plastic for weight credit is known.

4. A foam plastic compound called "POLESET" (trademark) is being used to install poles in the ground. The injected compound is used in lieu of the

usual technique of compacted soil backfill. Users, notably power companies, report in their literature that the material is stronger than the _in situ_ soil, based upon pulling tests in the field. In such a usage, weight-credit is not a factor. Apparently, the scarcity of select borrow, the "neatness" of the operation, and reduced labor costs, combine to make the technique workable and economically feasible.

Hypothetical Case Histories

In order to illustrate further the applications of the weight-credit technique, a number of "hypothetical case histories" follow. Some are general and merely conceptual; but some suggested applications are for the solution of specific, real problems in foundation construction.

1. Route Construction in General

If a route of any type (highway, railroad, pipeline) is to be constructed between two points A and B and a large area of marginal soil exists between these points, the usual solution would be to go around the area. With the dramatic weight credit that can be attained by the use of foam plastic (as an artificial subbase for a highway, for example), it is conceivable that the marginal soil might be crossed. The length of the route would thus be shortened and the resultant savings might make the straight route economically superior. Possibly the loads imposed by rail equipment would be so high as to preclude its application to railroads, but highway and pipeline loadings seem within the scope of application. When it is considered that Interstate systems cost in excess of $1,000,000 per mile (depending of course on cuts, fills, locale, etc.), the cost of the plastic foam and its installation could be justified.

2. Airport Construction

Foam plastic subbases might be used to establish a weight credit in airport pavement design. Because many airports must be located in areas of marginal soil support -- bay muds, for example -- the technique may have wide applicability.

3. Pipelines

(a) General. The use of foam plastic as an injected backfill under and around pipelines is thought to be practicable. Envisioned is a type of wheel-mounted machine which would move in a straddling fashion along the trench, producing and injecting plastic foam into the trench. This application might be particularly appropriate in large, congested cities for two reasons: First, the existing pavement would provide the needed support for the machinery, and, second, the trucking of soil fill to the site (through congested city streets) could be eliminated.

In the case of fluid in a pipeline which is designed for gravity flow, it is conceivable that the size of the pipe wall can be reduced to that of essentially a form since the only reason for a pipe of any substantial strength in such a case is to withstand the pressure of the backfill and any live loads. Since the pressure exerted by a foam plastic fill would be negligibly small, the required strength of the pipe would be governed principally by live loads. In fact, it is speculated that a slip-form of some type could eliminate the need for a pipe; the slip-form could be moved forward after the plastic foam has created a conduit through which the fluid would flow.

Thus, it is conceivable that, for the case of urban pipeline construction, the costs of soil fill, its transportation, and the costs of the pipe (gravity flow) could be reduced or even eliminated, i.e., weighed against the costs of the foam plastic weight-credit approach.

(b) Because of the well-known heat insulating properties of foam plastics, its use to encase the oil pipelines through the permafrost is suggested.

(c) Following is an excerpt of a report prepared for the writer by a graduate student at New Jersey Institute of Technology, Mr. Michael J. Healy, who is an engineer with the City of New York.

"A recent example of a case where foam plastic backfill would have provided a better and less expensive solution to a soil problem was the construction of a major arterial highway in Staten Island, New York. The author was directly involved with this project which bisected a tidal marsh. The method specified in the contract for sub-grade stabilization was to excavate unsuitable material and backfill with compacted 1½ inch broken stone. The purpose of using 1½ inch broken stone instead of sand fill was to have larger voids, thereby generating a larger weight credit. The contract unit price for stabilization was \$33.00 per cubic yard - \$13.00 for excavation and \$20.00 for in-place 1½ inch broken stone fill.

The subgrade was so poor that it was necessary to excavate as much as 10 feet in certain areas. The final excavation quantity for stabilization was 83,350 cubic yards which generated a total stabilization cost of \$2,750,550.00.

However, if foam plastic was utilized as backfill for the purpose of gaining weight credit, as was suggested to the New York City Highway Department during the early stages of the project, a substantial savings would have been realized. Only 10% of the final excavation quantity (8,335 cubic yards) would have been required to achieve an equivalent weight credit. The total cost for stabilization if polyurethane was utilized would have been approximately $441,755.00 (8,335 cubic yards at $13.00 per cubic yard for excavation plus $40.00 per cubic yeard for foam plastic backfill).

Therefore, it would seem that a savings of about 2.31 million dollars could have been achieved if foam plastic backfill was used. In addition, foam plastic probably would have produced a better overall project."

4. Pile Foundations

(a) The author worked on a pile load-test program in which steel H-piles were driven to bedrock to depths of about 70 feet, with specifications for equipment and driving such as to produce a design pile of 175 tons.

Above the bedrock were thick deposits of soft clay. Finished first-floor grade was at a level several feet above existing grade; and the design called for the space between existing grade and the slab to be a fill material. The high floor loads dictated a structural floor slab, so the only reason for the soil was to fill the space under the building. The weight of this fill would, however, cause the consolidation of the thick, compressible clay below, and it was estimated that the "drag" on each pile caused by the settling clay would be about 50 tons.

The use of a foam plastic fill could have eliminated the substantial effect of drag on the piles, and smaller piles could have been used to support the building-imposed pile loading of 125 tons.

(b) Another excerpt from Healy's paper follows:

"The weight credit obtained if polyurethane foam plastic was substituted for concrete in piles would be substantial. The pile bearing capacity could then be increased by the resulting weight credit.

For example, an average 40 ton capacity, 18 inch diameter, barrel shell friction pile which would probably be driven to a depth of about 40 feet, would have a volume of approximately 71 cubic feet. If concrete was utilized to fill the pile, it would weigh approximately 11,000 pounds. On the other hand, polyurethane plastic, which weighs 40 pounds per cubic foot for an equivalent compressive strength (4000 psi), would yield a pile weight of 2,840 pounds or 4 tons. As a result the capacity of the pile could now be considered as 44 ton, an increase of 10%.

In general, the weight credit generated by substituting foam plastic in any deep foundation system which utilizes concrete, would produce significant increases in allowable bearing capacity. In addition, it should be noted that the cost of materials are approximately equal - 4,000 pounds per square inch concrete costs about \$37.00 per cubic yard and 4,000 psi polyurethane foam costs about \$40.00 per cubic yard."

5. A Grade-Separation Case Study

The author is aware of plans to effect a grade separation at an important intersection in a major eastern city. In discussions with the chief soils engineer it was learned that the principal complication of the design was that a subway ran directly beneath the intersection at a relatively shallow depth. And the subway at that point is presently loaded "about to its limit," so the approach fills necessary to make the grade separation create a problem of over-stressing the structural elements of the subway system.

The solution was to excavate some of the material above the subway and backfill to the planned elevated grade with lightweight fill. The fill to be used has been inspected by the author and appears to be some form of ash material containing considerable black silty material. It appears to be a rather poor fill by normal standards (poor drainage, difficult compaction, probably frost susceptible), but its unit weight will be about 65-70 pcf. Thus a weight credit will be attained and apparently the material is acceptable for the solution of this special problem.

The use of a foam plastic approach fill (at least in part) could have provided a much greater weight credit and would have the advantages of ease of construction and elimination of drainage and frost problems.

6. The Catskill Aqueduct Case Study

Within a 48-mile section of the Catskill Aqueduct System there are included approximately fifteen miles of pressure tunnels. This construction was used to cross filled valleys where no suitable material was available to support other kinds of construction. In general, the depth of these pressure tunnels was governed by the depth of pre-glacial gorges which contained glacial drift and other recent unconsolidated alluvium. The deepest pressure tunnel was for the Hudson River crossing; it was founded at a depth of 1111 feet below the level of the river. The length of the tunnel at this depth is over 3000 feet.

No cost figures are available for the construction of these pressure tunnels, but the figures must be impressive for such an effort. Had the technology been available, it is probable that great savings might have been possible by using plastic foam backfill to "float" across the pre-glacial gorges with cut-and-cover construction.

7. Retaining Structures

The use of foam plastics behind retaining structures is a logical extension of the weight-credit concept. In this case the lateral pressure on the retaining structure caused by the pressure of the backfill itself will be reduced to a negligible value with the use of the foam plastic in place of the usual soil backfill. Thus, where no live loads are involved, the size and strengthening of the wall required to retain the backfill will be reduced substantially. Drainage design can also be greatly simplified.

A possible application would be the use of wedge-shaped sections of precast foam plastic for various size excavations adjacent to basement walls. Or a system might be developed to fabricate or inject the foam on-site. For large construction projects, the latter technique would be preferable.

Technical Problems

The presently-recognized technical questions relating to the use of foam plastics in weight-credit applications to foundation construction are related to permanency and durability under the influence of load and exposure to natural and man-made environments.

Evidence of permanence and durability, such as exists, is presented elsewhere in this paper.

Special problems during service relate to chemical resistance, flotation, and icing.

Chemical Resistance

Foam plastics (e.g. STYROFOAM) generally have poor resistance to some materials, most notably gasoline. For the Pickford Bridge construction, polyethylene sheets were used to cover and protect the STYROFOAM bundles against possible gasoline spills.

The possibilities of exposure to other materials should be considered for each case and for each type of foam plastic used. Undoubtedly, suppliers can supply detailed information and recommendations.

Flotation

At Pickford Bridge, five feet of soil fill, placed above the plastic, was sufficient pressure to "pin down" the foam material in the event of flooding and accompanying rising water table.

Soil "pins", analogous to anchor bolts in tunnel construction, might also be employed where larger weight credits are needed.

In cases of support of buildings (as opposed to embankments, pipelines, etc.), flotation problems would not generally be acute; the design approach would be to create a positive pressure in the subsoils consistent with (their) allowable pressure tolerances. In general "zero stress increases" would be avoided.

Icing

On highway insulation installations, the purpose of the foam plastic was to prevent or minimize frost action in susceptible subgrade soils (i.e. silts). Unfortunately, in some instances, the insulation can protect the subgrade, but cause the pavement surface to become icy along treated sections of the roadway, thus creating a hazardous driving condition.

This possibility should always be considered. Precautions could include "heat bridges" to allow some heat to leave the subgrade (through, for example , appropriate aluminum sheets or pipes installed as part of the system), or by providing sufficient cover of soils <u>over</u> the foam plastic.

Obviously, the problem of icing would generally be of concern only for highways in cold regions. Southern-tier states would not be affected.

Sunlight

Direct exposure of foam plastic to sunlight for extended periods of time should be avoided.

MEASURE THE VOIDS, NOT THE SOLIDS

by

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MEASURE THE VOIDS, NOT THE SOLIDS

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ABSTRACT

Characterization of soils and rocks has long focused on the size and concentration of solids, as in the quantities of grain size, density, and specific gravity. Where voids were known, they were expressed in total (rather than distributional) form in terms of absorption water content, void ratio, or porosity. Now it is practical to measure the distribution of voids, and to correlate material responses with parameters of such distributions. This is being done with the permeability, frost susceptibility, compactability and stress-strain behavior of compacted soils, as well as the physical durability of mineral aggregates of carbonate rocks. While the testing is currently concentrated on materials of the Midwest, the techniques can be applied to soils and rocks along the Fall Line, as well.

The measurement of the distribution of porosity is achieved through the technique of pushing a non-wetting liquid (viz., mercury) into the dry soil or rock. The pressure to intrude and the smallest diameter of pore intruded are related by a capillary type equation. By measuring the volumes of mercury intruded under increments of increase in pressure, the distribution is defined and may be expressed in either cumulative or differential form.

An additional challenge when working with clayey soils is the need to remove the soil water without changing the volume and hence the porosity. This

is usually done by freeze drying, where the quick-frozen moisture is sublimated from the sample.

The paper briefly describes the experimental procedures, which can be carried out routinely in a first-rate materials or geotechnical laboratory, and shows some of the correlations achieved to date. Engineers and engineering geologists practicing along the Fall Line may wish to use the techniques for new and original specialized predictions.

INTRODUCTION

The behavior of soils and rocks is controlled by the concentration and arrangement of solids and by the amount and distribution of voids among these solids. The former characteristics are stated in terms of density, specific gravity, grain size distribution, and fabric; the latter are described as porosities, void ratios, and (more recently) pore size distributions. The content of the pores is air, water or ice, and the nature of this content may greatly affect the performance of the material in situ or in an engineered structure.

It is simpler to measure the gravimetric quantities than the volumetric ones, and easier to determine the distribution of particle sizes, as opposed to void sizes. However, the determination of the arrangement of the solids, i.e., the soil fabric, becomes very difficult for fine-grained materials, even with modern X-ray diffraction and scanning electron microscopes. Conversely, the technique of mercury intrusion porosimetry gives a relatively simple, and quantitative, measure of the fabric in terms of the distribution of pores within it.

Parameters of pore size distribution (PSD) are enormously useful when material properties involving the movement of water in the pores are under study. Obvious examples are freezing, shrinking, swelling, permeability, capillarity, and rate of compression. It may be further possible to correlate PSD with liquefaction, sensitivity, and nondurability. The very real prospect of being able to develop empirical prediction equations, based upon PSD, which are vastly superior to extant ones based on grain size distribution or total porosity, fascinates the authors.

MERCURY INTRUSION POROSIMETRY

The most popular technique for measuring the distribution of pore sizes in rocks or compacted soils is that of mercury intrusion. The basic principal is that a nonwetting liquid requires pressure to be intruded into pores, and the smaller the pore, the greater the pressure required. Mercury is a good choice, since it has a very low compressibility and does not react with the usual earthen minerals.

The relation between pressure (P) and an equivalent diameter of pore (d) was first given by Washburn (1921) as

$$P = - \frac{4\sigma \cos\theta}{d}$$

where σ = surface tension of mercury; and

θ = contact angle between mercury and the pore wall

A variety of values of surface tension have been reported {Bhasin (1975)} with the value of Kemball (1946) of 484 dynes/cm at 25^{o}C being a commonly used one. The value of the contact angle is perhaps more critical to the calculation, and it may be measured by optical microscopy {Bluhm (1974)}. Diamond (1970) recommends a value of 147^{o} for kaolinitic and illitic soils and 139^{o} for montmorillonitic ones. In some materials, it is possible to drill small holes, measure the pressure required to intrude them, and solve for $- 4\sigma \cos\theta$ as the dependent variable, see Winslow and Diamond (1970).

The porosimeter shown in Figure 1 has a pressuring capacity of 15,000 psi, and is generally suitable for soils. The small soil sample is a cube about 1 cm. on a side. It is dried at constant volume, placed in the apparatus, evacuated, surrounded by mercury, and subjected to increments of pressure. Each pressure will correspond to an equivalent pore diameter in the Washburn

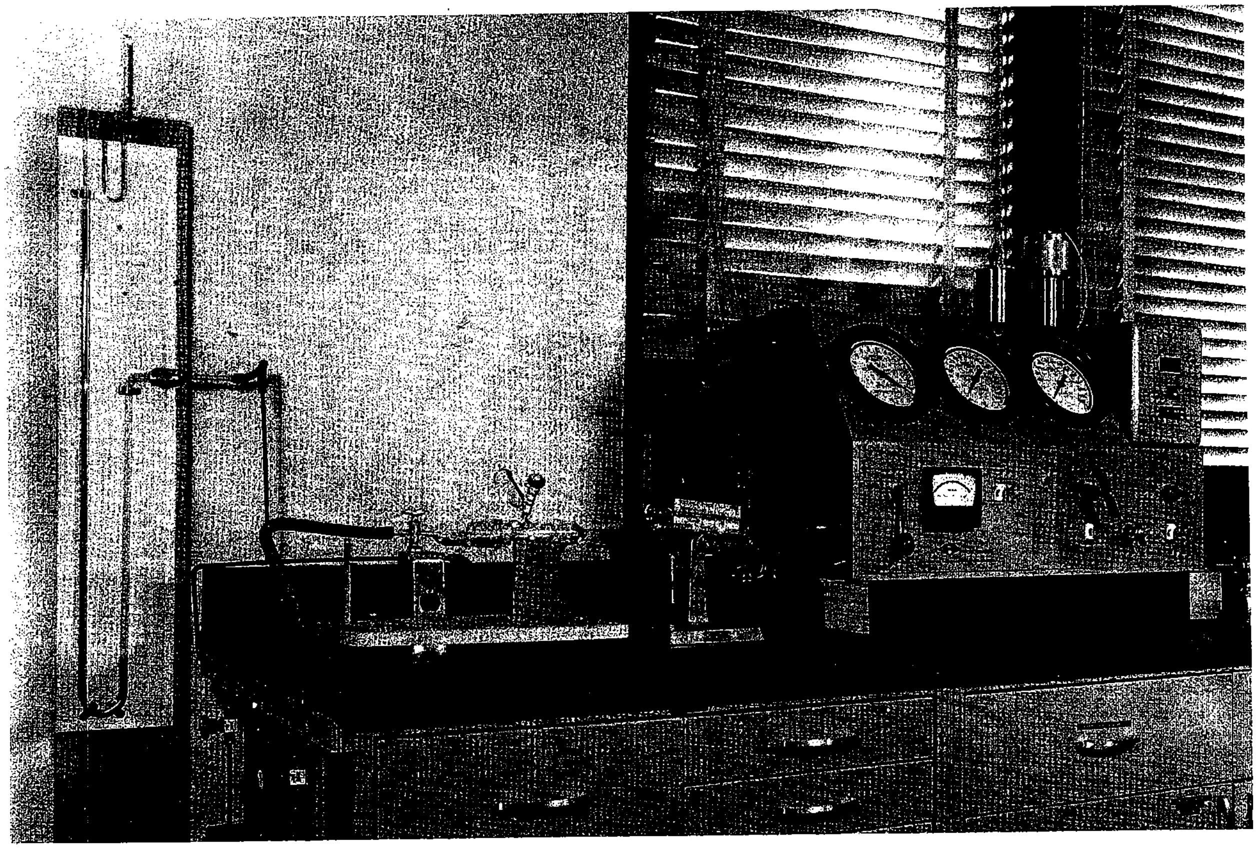

FIGURE I MERCURY INTRUSION EQUIPMENT SHOWING (from left to right) MANOMETER, VACUUM MANIFOLD, FILLING DEVICE, AND POROSIMETER (from Bhasin, 1975) equation, and a succession of these values allows a pore size distribution to be plotted in either cumulative or differential form.

CONSTANT VOLUME DRYING

The sample for mercury intrusion must be dry, yet the porosity and its distribution cannot be affected by the drying. The shrinkage forces induced by evaporation will generally produce serious volume changes in soils, and more sophisticated drying techniques must be followed. One of these is critical region drying which consists of three steps {Bhasin (1975)}: (1) raising the pressure and temperature of the soil water such that changes in the specific volume of water are slow in the compressed liquid region; (2) transforming the soil water into vapor within the critical region; and (3) removing the vapor within the superheated vapor region. Since no phase interface occurs within the critical region where the drying takes place, no surface tension forces are brought into play. Since temperatures of roughly 400^{o}C and pressures of almost 4000 psi are involved in this form of drying, the technique is not particularly inexpensive or simple, although Bhasin (1975) successfully dried a number of compacted soils in this way.

A much simpler drying technique is by freezing. As Ahmed (1971), Zimmie and Almaleh (1976) and Reed (1977) have shown, the soil sample can be quickly frozen in liquid nitrogen and placed in a vacuum where the moisture is removed by sublimation. The freeze drying equipment is very inexpensive and the test can be completed within a working day.

It is relatively easy to determine that the drying process has or has not produced a change in the sample volume and hence the sample porosity. Verification of the freedom on the pore size distribution from drying effects is much less certain and direct, and must usually rely upon "before" and "after" X-ray diffraction patterns and scanning electron micrographs.

PORE SIZE DISTRIBUTIONS

Since these distributions are not simple, they are thought of primarily as graphical representations. Cumulative distribution curves are the most common; the dependent variable is either volume of voids per unit weight of solids,or volume of voids per unit volume of sample. The independent variable is the equivalent pore diameter in microns, plotted onto a logarithmic scale. The lower portion of Figure 2 is a sample plot. The plots are interpreted in the same manner as grain size distribution plots, viz., steep portions denote a relative abundance of those sizes and flat portions indicate a relative scarcity. In Figure 2 (lower) there is shown a concentration of pore sizes between about 1 and 6 μm, while there is little pore space below 0.04 μm or above about 10 μm.

Note that the differential distribution in the upper portion of the Figure shows the same trends, even clearer than the cumulative plots. Peaks on the differential distributions are described as to their frequency (ordinate value) and mode (abscissa value). Garcia-Bengochea (1978) has shown that the most faithful differential distribution plots are obtained when readings are taken according to a pressure change (P_{i-1} to P_i) of

$$\frac{P_i}{P_{i-1}} = 10^c$$

where c = the logarithmic interval constant.

RESULTS

Pore size distributions (PSD) have provided improved predictions of material behavior to date (1978) in four principal areas: soil compaction, frost heave, and permeability, and aggregate durability. Former predictions were based upon:

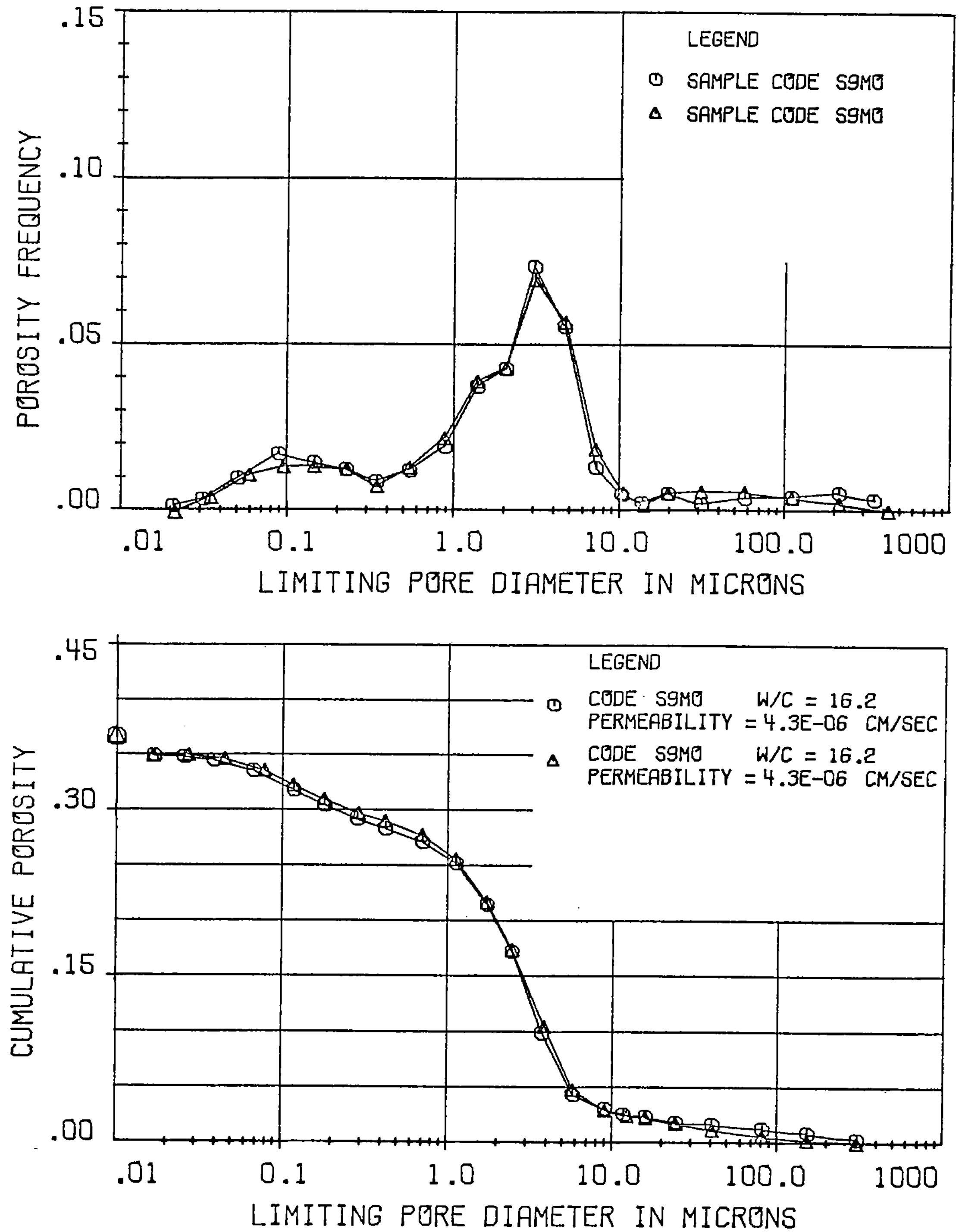

FIGURE 2 REPLICATE PORE SIZE DISTRIBUTION CURVES FOR SAMPLE S9MO (from Garcia-Bengochea, 1978)

grain size (or textural classifications based upon grain size), total porosity (or density), and largely qualitative fabric descriptions.

Compaction

Recent studies by Ahmed (1971), Hodek (1972), Bhasin (1975), Reed (1977) and Garcia-Bengochea (1978) have greatly improved our understanding of the compaction of fine grained soils. The particulate model is one of sand and silt grains and clay aggregations. The pore system exists between these units (interaggregate pores) and within the clay ones (intraaggregate pores). The interaggregate pores might typically be 10 to 100 times as large as the intra-aggregate ones, and the former are the ones affected most obviously by the compaction process.

Densification of a collection of sand, silt and relatively dry clay aggregates is achieved by pushing these units together without really changing their shape. Even with some particulate unit breakage, relatively large pores remain. Conversely, if the clay aggregates are at or slightly above the lower plastic limit water content, they can be pushed together and deformed, to minimize the interaggregate space. In the latter case, there may be some strong orientation of the clay platelets, as well, within the aggregate and normal to the compacting load.

Figure 3 from Garcia-Bengochea (1978) demonstrates the point. The dry side sample (S5MD) has a large (interaggregate) pore mode at about 2 μm. The other two samples of the same soil are at standard compactive effort optimum and wet of optimum, and they lack the interaggregate mode. Ahmed (1971) and Bhasin (1975) demonstrated the same trends with other soils. Although the "deformable aggregate" model, described by Hodek (1972), is a bit more sophisticated than the "ultimate particle" approach of Lambe (1958) and others, there is good general agreement between them. Further, the pore size

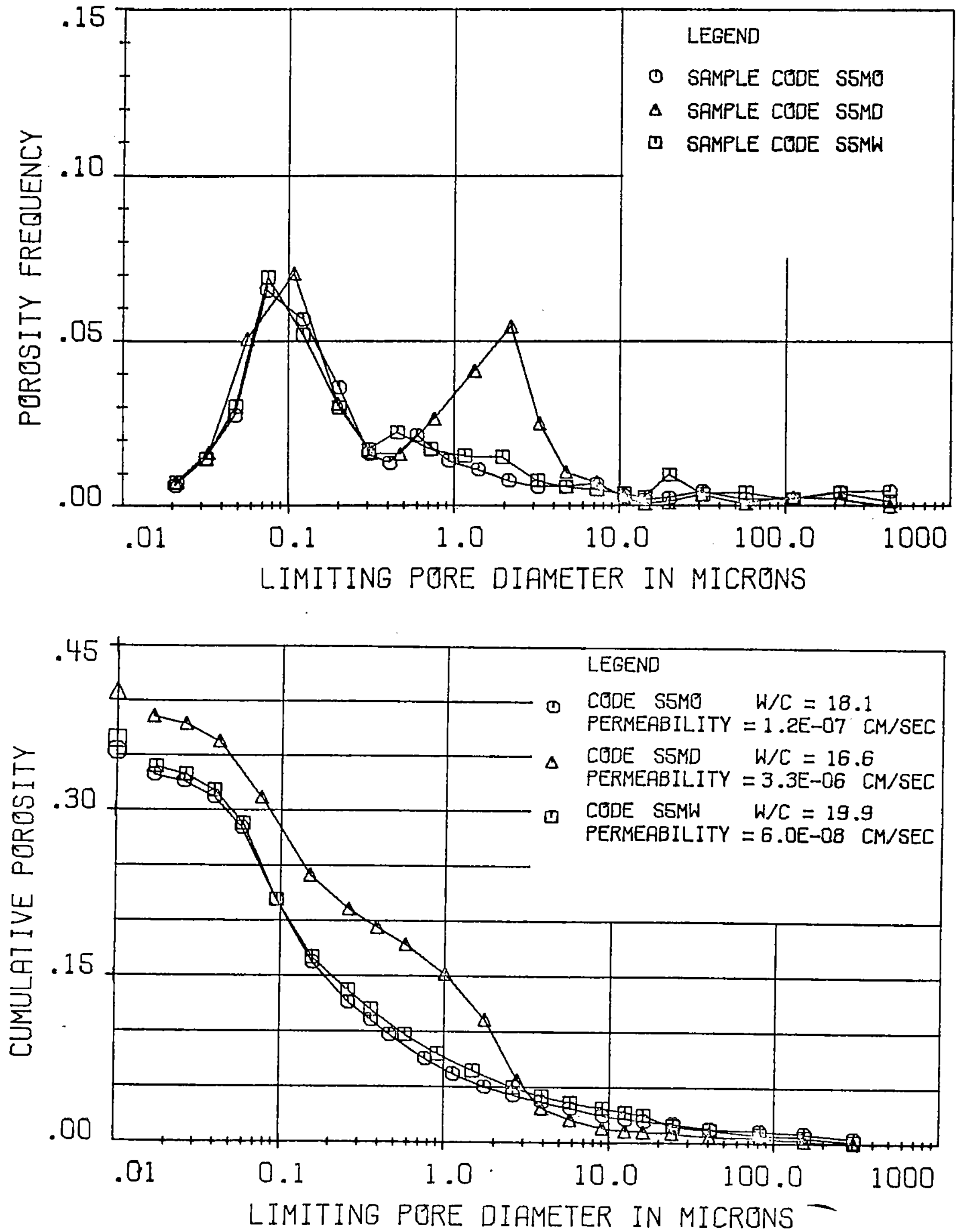

FIGURE 3 DIFFERENTIAL AND CUMULATIVE PORE SIZE DISTRIBUTION CURVES FOR 50% SILT-50% KAOLIN COMPACTED AT MEDIUM EFFORT (from Garcia-Bengochea, 1978)

measurements tend to affirm the common postulations about the fabric of fine grained soils as it is affected by compaction water content and compaction effort.

Frost Heave

The conventional way of predicting the frost susceptibility of a soil is by the Casagrande 0.02 mm criterion or by the Corps of Engineers textural "F-rating". Direct evaluation of this characteristic is through observation of the heaving rate in an open system freezing test.

There is a basic problem with a prediction based upon grain size distribution or texture, viz., a given soil has a single grain size distribution, but many values of pore size distribution depending upon the details of its placement.

Reed (1977) found that he could vary the frost heaving rate of "highly frost susceptible" soils almost at will, by changing the compaction variables. Figure 4 shows the compaction curves for one soil and Figure 5 shows the frost heaving measures for the same soil. Decreased compaction increased the frost heave rate, as did dry side compaction, but compaction at optimum could produce either more or less heave than on the wet side. However, the pore size distributions were good predictors, e.g., as the quantities of pores larger than about 3 μm increased, the frost heaving rate also increased. Thus in Figure 6, the dry side sample heaved the most, followed by the wet side sample, followed by the optimum moisture sample.

Although Reed (1977) correlated the frost heave of a given soil with the amount of the large pores only, correlations for different soils required consideration of the smaller pores as well. He showed an R^2 value of 82% for the equation:

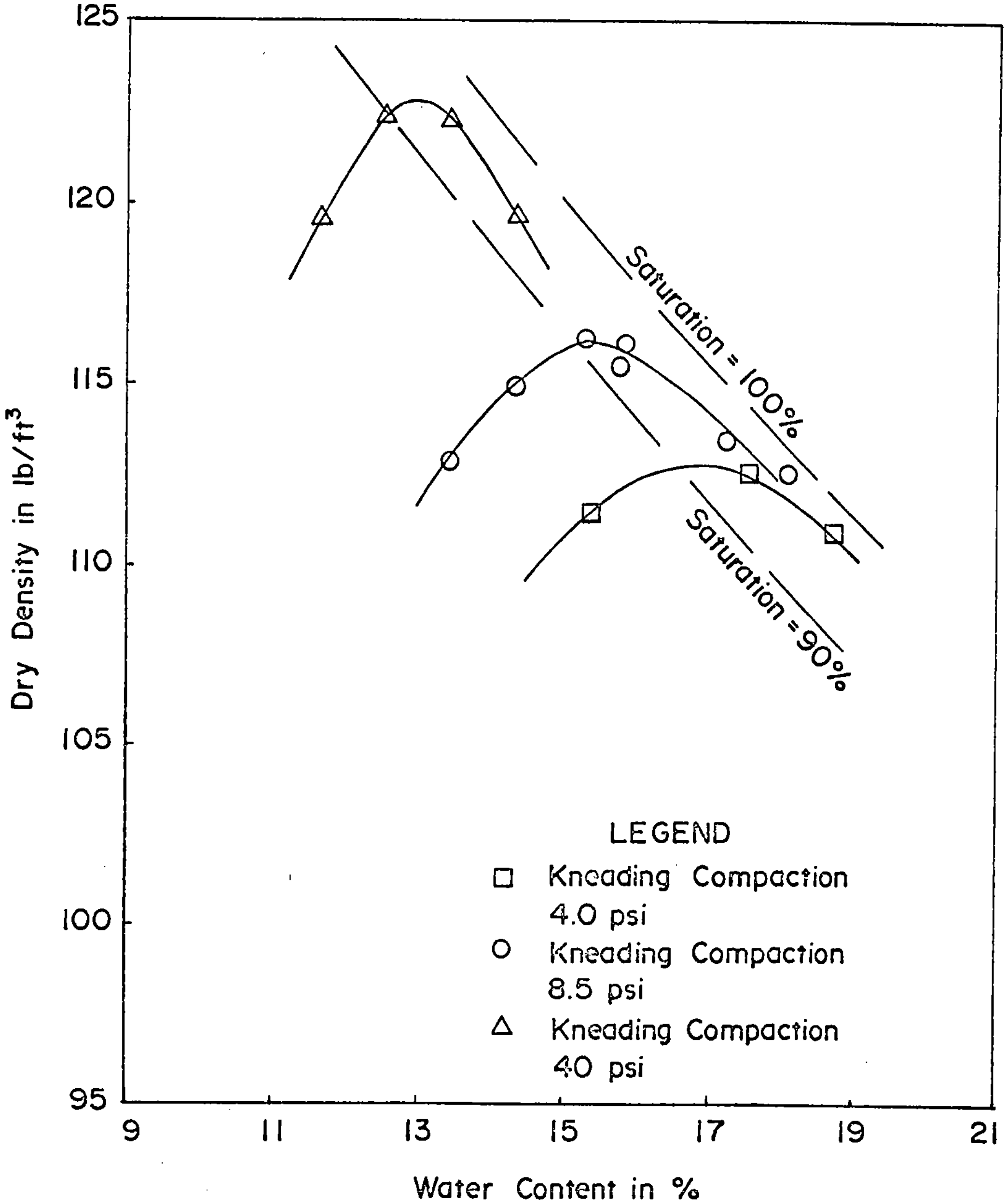

FIGURE 4 COMPACTION CURVES FOR 70% SILT-30% KAOLIN (from Reed, 1977)

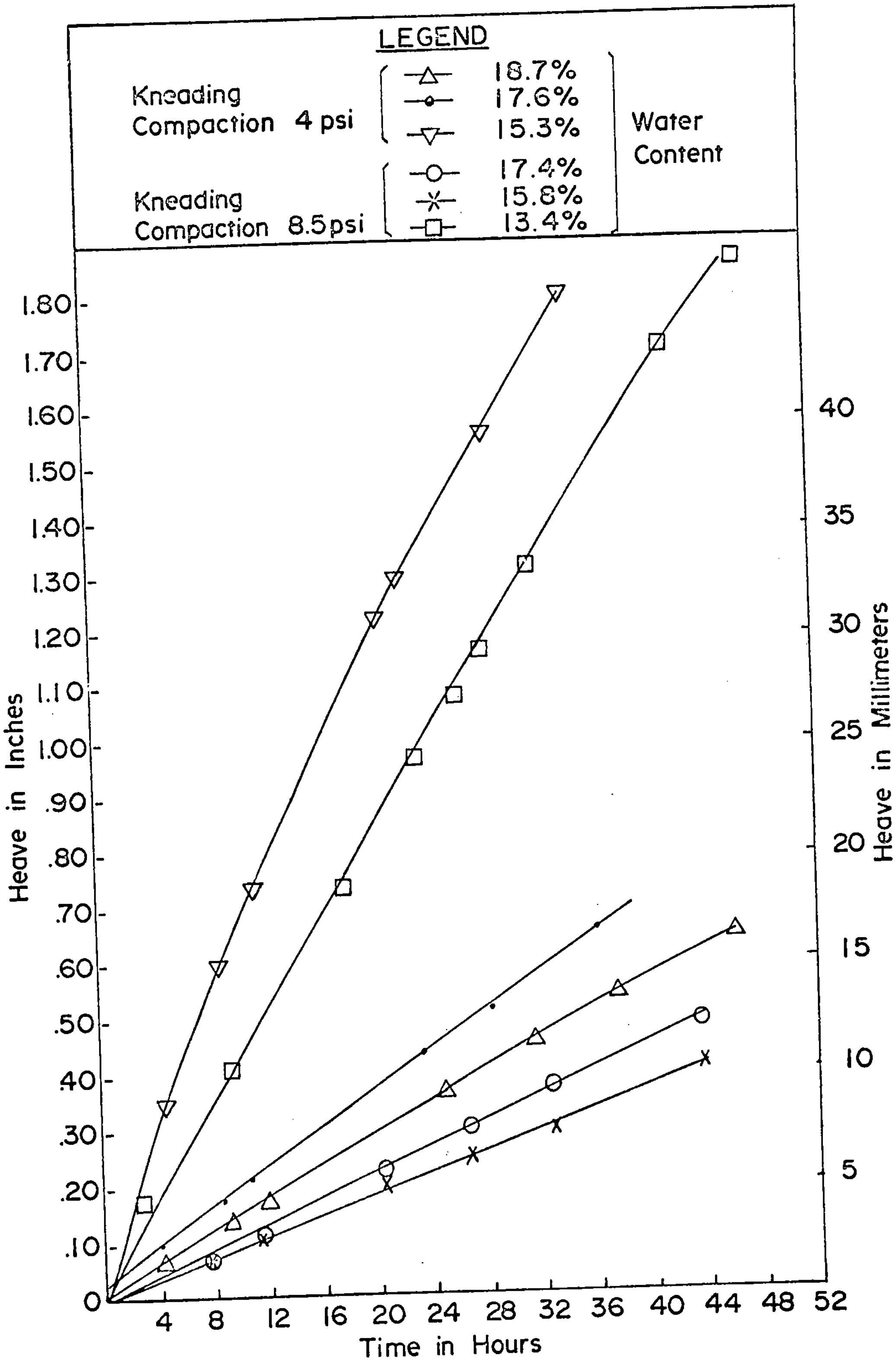

FIGURE 5 TYPICAL FROST HEAVE CURVES FOR 70% SILT-30% KAOLIN (from Reed, 1977)

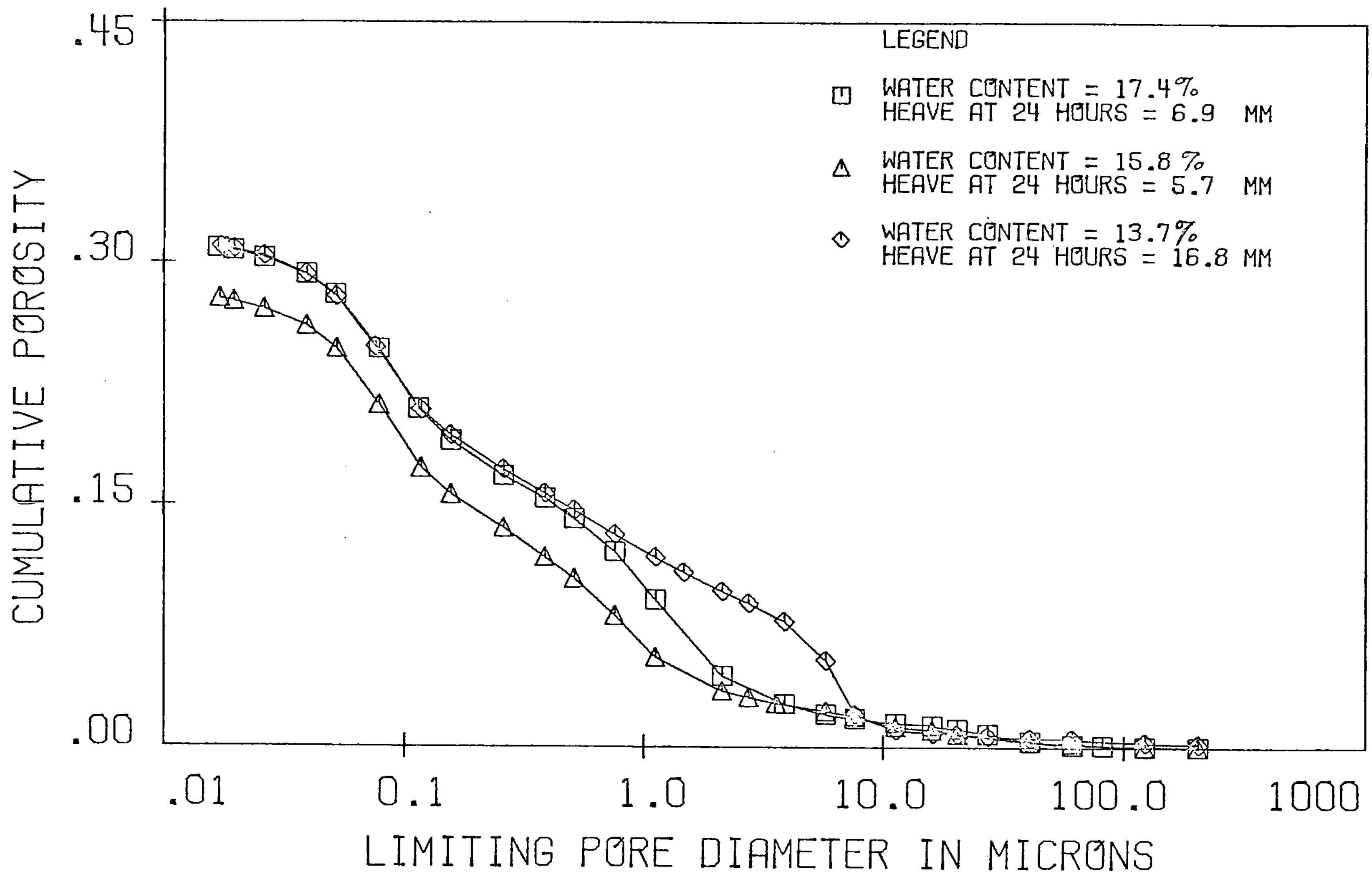

FIGURE 6 PØRE SIZE DISTRIBUTIØN FØR 70% SILT - 30% KAØLIN CØMPACTED AT 8.5 PSI (from Reed, 1977)

$$\hat{Y} = -5.46 - 29.46 \frac{X_{3.0}}{X_0 - X_{0.4}} + 581.1 \, (X_{3.0})$$

where $\hat{Y}$ = frost heave rate in mm/day

$X_{3.0}$ = cumulative porosity for pores > 3.0 μm but < 300 μm

X_0 = total cumulative porosity

$X_{0.4}$ = cumulative porosity for pores > 0.4 μm but < 300 μm.

Permeability

Popular forms of permeability prediction equations contain soil parameters such as effective gain size or a measure of the total porosity. For example, the logarithm of permeability is reported to vary linearly with the void ratio. Yet, as shown by Garcia-Bengochea (1978) in Figure 7, the relation is poor for either a single soil or a group of soils; pore size distribution parameters form a better basis for prediction. Referring back to Figure 3, the dry side compacted sample (S5MD) not only has a higher frequency of the large pore mode, but also has a permeability which is an order of magnitude or two greater than the permeabilities of the optimum (S5MO) and the wet side (S5MW) samples. Garcia-Bengochea (1978) inserted PSD parameters into 3 models for permeability. An example is the variable diameter capillary model, where the appropriate parameter is "$E(d^2)n$" or the second moment about the origin of the PSD. The resultant correlation is shown in Figure 8, with an r^2 value of 0.83.

Durability

The only Purdue research which can be reported is the on-going correlation of PSD with shale slaking by Surendra. Extensive measurements of PSD have also been made on Pennsylvania shales, according to R. H. Howe of the Bureau of Materials, Testing, and Research of the Pennsylvania Department of Transportation.

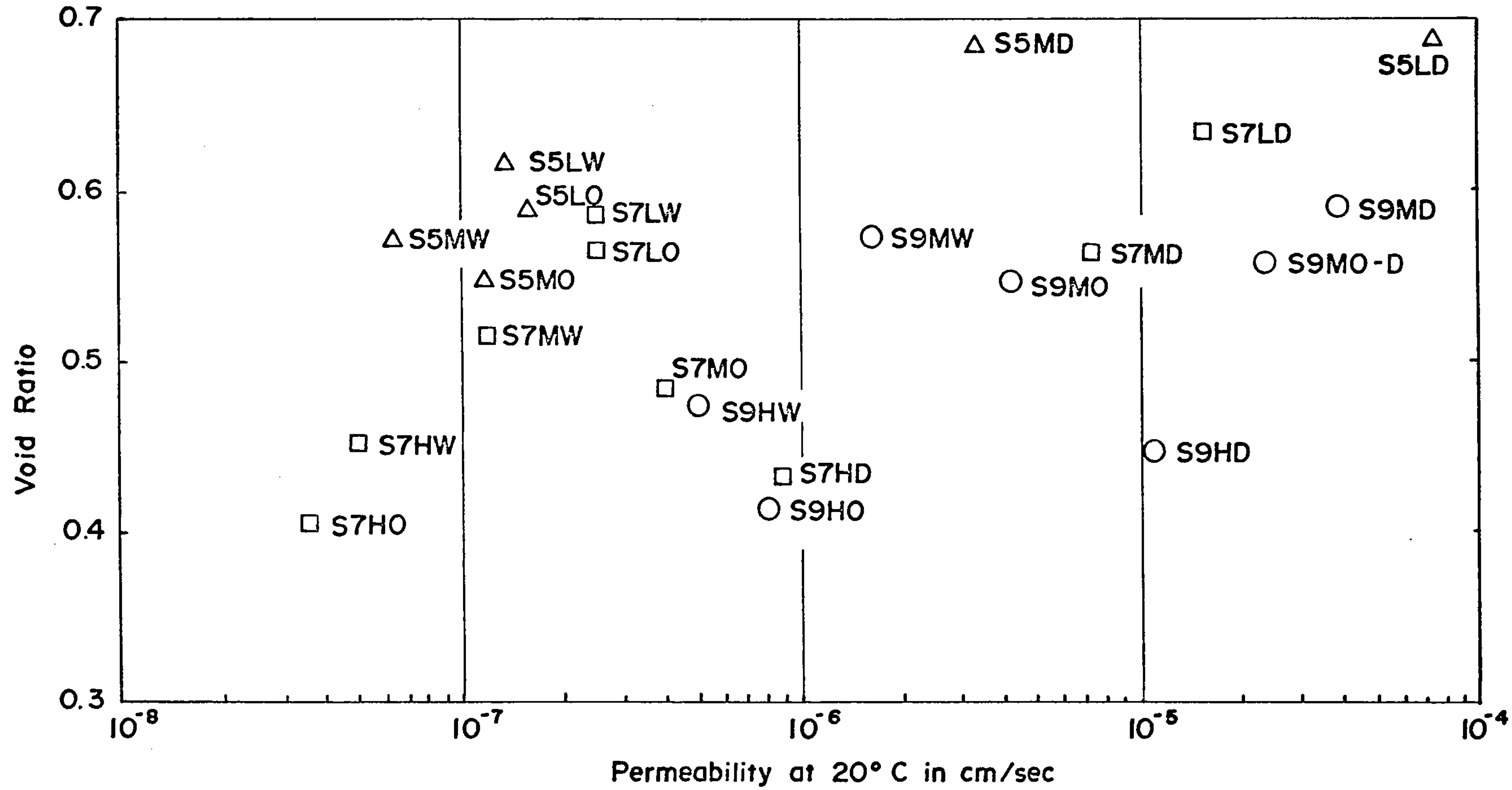

FIGURE 7 PERMEABILITY VS VOID RATIO (from Garcia-Bengochea, 1978)

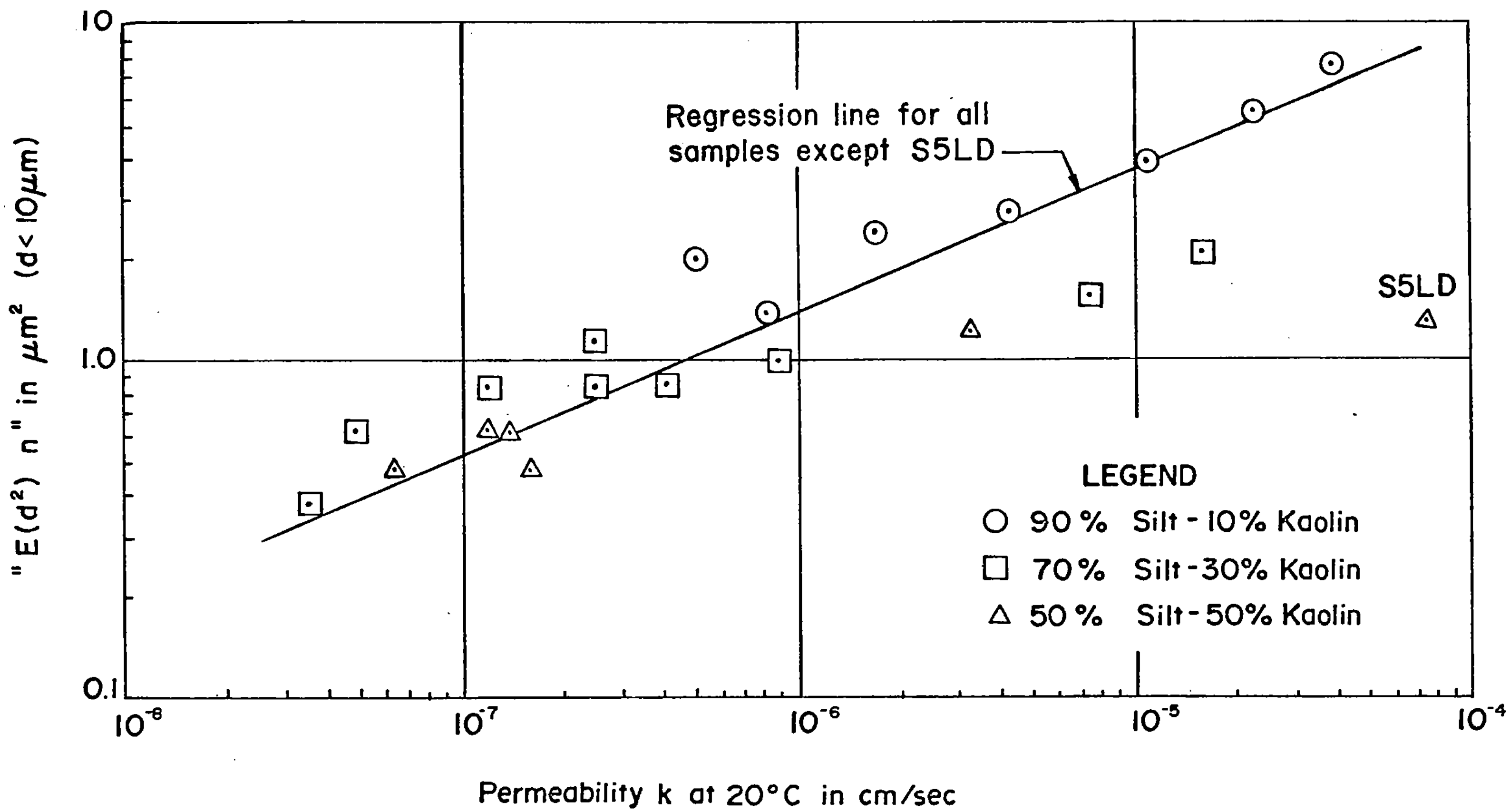

FIGURE 8 PERMEABILITY VS. CAPILLARY MODEL EQUIVALENT PORE SIZE PARAMETER, $E(d^2)$ n (from Garcia-Bengochea, 1978)

Determination of PSD for limestones by means of suction tests is reported in Portugal by de Castro (1974). In Canada, the mercury intrusion approach has been researched at the Ecole Polytechnique in Montreal. Duran (1969) studied the expansion in alkaline concrete of several aphanitic limestones. He found that the reactive samples had a distinctive frequency distribution curve for the pores, with relatively few pores of very small size.

CONCLUSIONS

Impressive as the achievements to date may be, they most certainly represent only a "scratching of the surface". Similar improvements can be expected with respect to predictions of swelling, unstable fabrics and sensitivity to remolding, magnitude and time rate of compressibility, and any other response depending upon the relative retention and transmission of fluids in the porous material. Most of the results cited above are from a modest research effort at Purdue University. The reader is invited to examine applications of PSD measurements to his own practical problems and experiences. The advice to "keep your eye on the donut, not the hole", may often be sound, but in the case of soil fabric, perhaps we should pay more attention to the "hole".

ACKNOWLEDGMENTS

Much of the work summarized herein is taken from completed theses at Purdue University by Ahmed, Bhasin, Garcia-Bengochea, Hodek and Reed. All such work has been financed by the Federal Highway Administration and by the Indiana State Highway Commission, through the Joint Highway Research Project at Purdue University.

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COMPACTION GROUTING METHODS FOR CONTROL OF SETTLEMENTS

DUE TO SOFT GROUND TUNNELING

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COMPACTION GROUTING METHODS FOR CONTROL OF SETTLEMENTS

DUE TO SOFT GROUND TUNNELING

INTRODUCTION

Control of ground settlements is one of the typical problems associated with soft ground tunneling in urban areas. Several studies of the extent and magnitude of ground displacements caused by various tunneling methods and differing soils conditions have been made *(1,2,3). Recent soft ground tunneling construction for the Washington, D.C. and Baltimore Subway Systems has resulted in surface settlements ranging from a few millimeters to over 30 centimeters.

Conventional underpinning, root piles, jacked piles, or other mechanical support systems are often used to protect structures that would be seriously affected by ground settlements associated with nearby tunneling operations. Alternatively, special tunneling procedures are sometimes adopted to reduce surface settlements. These special methods include the use of compressed air within the tunnel, special breasting and other face control methods, mortar grouting behind the tunnel lining, expansion of the tunnel lining immediately behind the shield, articulated shields, and, very recently, the use of slurry tunneling procedures. Yet even these special tunneling methods are sometimes unable to reduce settlements to an acceptable limit. Also, the use of a special tunneling method may not be economically justified where a strict settlement control requirement exists over only a short portion of the tunnel.

A recently developed alternate solution to the settlement problem associated with soft ground tunneling is the use of compaction grouting methods to replace lost ground above the shield and to recompact loosened ground below affected structures. Compaction grouting to solve static settlement problems is a well documented ground modification procedure (4, 5, 6). Compaction grouting involves the injection, under high pressure, of a stiff, low-slump (under 2 inches) mortar grout, to create a grout bulb which displaces and compacts the adjacent soils. The grout has a high internal friction which inhibits the flow of grout away from the grout pipe tip and thus allows the grouting engineer to maintain excellent control over the location of the grouted

*Numbers in parenthesis refer to references in the bibliography.

mass. The nature of the stiff, viscous, compaction grout material requires high pumping pressures to move the material through the grout lines, down the grout pipe and into the grouting area. To avoid the development of sand blockages within the grout lines, low slump grouts, pumped at pressures of up to 500 psi, are required to exhibit very little water loss under high pressure.

Four case histories involving the use of compaction grouting to solve soft ground tunnel-related settlement problems are presented in the following sections. The fourth case is believed to be the first time the procedure has been used on a "dynamic" basis, where grouting was done as settlements occured, rather than after-the-fact.

CASE HISTORY NO. 1 - Pennsylvania Avenue Sewer

The Outbound Tunnel for Section D-6 of the Washington, D.C. Subway was constructed parallel to a five-foot high, horseshoe shaped combined sewer for several hundred feet along Pennsylvania Avenue (Figure 1). The sewer, located well within the settlement trough area of the outbound tunnel, experienced several longitudinal cracks during the Subway tunnel construction. As an alternate to the complete replacement of the sewer along the affected section, as originally required, the District of Columbia Department of Environmental Services agreed to a two-part sewer rehabilitation scheme. It was decided to recompact the sewer support soils loosened by the tunneling operations by using compaction grouting procedures, and then to epoxy grout the longitudinal sewer cracks from inside the sewer.

To avoid problems with traffic interruption, compaction grout pipes were installed from the Pennsylvania Avenue median strip. The grout pipes were placed at nominal seven to ten foot spacings longitudinally along the tunnel. Two grout bulbs were injected from each pipe. A four to six foot long compaction grout bulb was first injected at the lower level and the grout pipe was then pulled up some 8 feet, and a second 4 foot long grout bulb was injected. Experience indicated that little grout could be injected within a clay layer towards the bottom of the first grout bulb, while larger amounts were able to be injected in the sand layer above. Thus subsequent grout bulbs were located primarily within the sand zone. On the average, 52 cubic feet of grout were injected per pipe, distributed approximately evenly between the two grout bulbs. This represented an average of about seven cubic feet of injected grout per linear foot of sewer and tunnel.

In conventional compaction grouting it is considered important to have grout pipes installed vertically in order to obtain a large lateral projection of grout pressures so as to accomplish as much compaction as possible before sufficient total lift force is developed to cause heave of the ground above the grout bulb. However, due to the extreme difficulties encountered with traffic along this heavily travelled section of Pennsylvania Avenue, and due to the depths involved, it became necessary to utilize a more horizontal orientation for the initial grout mass within the previously drilled borehole. It was shown on this project that grout pipe inclinations in excess of 20° generally caused sufficient lift at low pressures so that significantly less grout could be injected per bulb than for vertical pipe orientations.

Elevation monitoring of the sewer was accomplished during grouting by the use of water levels, appropriately distributed within the sewer. Typically, the lower grout bulb was injected until initial movement of about 1/16 inch was observed in the tunnel. Grouting at that location was then suspended, the grout pipe pulled, and the injection of the second grout bulb was commenced. Grouting was thereafter continued until additional slight movement of the sewer was observed. Total movements within the sewer during all phases of the grouting varied between 1/8 and 1/4 inch. A slight closing of the previously developed cracks was observed, although, complete closing was neither expected nor accomplished, due to debris and mortar chips in the cracks. Grouting pressures at the top of the grout pipe typically varied between 100 and 200 psi, although, sometimes rising to 300 psi at the end of an injection sequence.

CASE HISTORY NO. 2 - Pennsylvania Avenue Junction Box

At the intersection of Pennsylvania Avenue and 7th Street, S.E., Washington, D.C. a cave-in occured during the tunneling of the Outbound Subway Tunnel of section D-6, with the chimney eventually daylighting in the Eastbound pavement area (Figure 2). This cave-in damaged a sewer junction box, exposed a 20-inch high pressure water main and various electrical conduits, and resulted in severe settlements with the area (Figure 3).

Subsequently, the chimney area was totally backfilled with concrete and selected fill, and the paving was restored. Some settlements continued to occur in the area causing considerable concern over the possibility of future rupturing of the water main and additional damage to the junction box. Thus it was decided to carry out a

Compaction Grouting program within the cave-in area box. An average of 153 cubic feet of low-slump compaction grout was injected in 14 grout holes in the area, for a total of some 80 cubic yards of grout (Figure 3). Avoidance of additional stresses on the utility pipes was a condition of the grouting operation. No movement of the 20-inch water line was observed during grouting operations, although slight movements, not in excess of 1/4 inch, were observed at the sewer junction box.

Subsequently, additional compaction grouting has been performed directly over the outbound line, where a branch of the sewer crosses over the previously constructed tunnel. Despite the fact that grouting operations were carried out at the bottom of an open pipe trench with little overburden pressure, a total of 264 cubic feet of compaction grout was injected in this rather limited area.

CASE HISTORY NO. 3 - Restoration of Transformer Vaults at 12th and E Streets, N.W.

Three underground electrical transformer chambers, located at 12th and E Streets, N.W., experienced serious settling and tipping during adjacent open-cut tunnel excavation in conjunction with the Washington, D.C. Subway program. A conventional rehabilitation program was ruled out by the Power Company that owned the transformers, since service would be interrupted for the duration of repairs. Consequently a compaction grouting program was initiated and carried out, without the interruption of services.

Grout pipes were drilled through the deep bottoms of the affected structures, and a quick setting (15 second) mortar grout was injected to a depth of 5 to 10 feet. Since there was virtually no reinforcing of the bottom slabs, it was important to avoid exertion of high grout pressures on the structure bottoms. A grout volume equivalent to about 1 foot spread over the affected areas, was injected in the regions indicated (Figure 4). The transformer boxes were raised between 4 to 6 inches, and returned to a more vertical orientation.

The above procedures were carried out after ground settlement problems affecting the adjacent structures were observed during the tunneling procedures. This compaction grouting program was aimed at preventing any future subsidence of the affected structures, but of course, was carried out too late to avoid the initial damage to the affected structures.

CASE HISTORY NO. 4 - Bolton Hill Tunnels

At present (May,1978), Compaction Grouting techniques are being used for the first time on an on-going basis during tunneling operations at the Bolton Hill Section of the Baltimore Subway System, currently under construction by a joint venture of Fruin-Colnon Corporation and Horn Construction, Inc. Approximately 39 buildings are being protected with this technique. Tunneling has already been completed for several of the protected buildings located along the southern section of the Bolton Hill Tunnels towards the Lexington Market Station. The buildings undergoing treatment are mostly one to four story structures, usually with one or more basements, and were selected by the tunnel designers as those most likely to be critically affected by tunneling operations. Typically the tunnels run parallel to the building fronts, which are some 15 feet off the tunnel centerline, and the buildings would normally be expected to undergo about an inch of settlement if left untreated during tunnel construction.

The typical compaction grouting procedure is as follows: Prior to tunneling operations near a protected building, grout pipes are installed and a subsurface ground movement monitoring system is set-up, using a combination of conventional levels and water levels. Grouting is then performed just as the shield passes. By performing compaction grouting as the tunnels pass below, building settlements are being kept with $\pm$ 1/8 of an inch. This contrasts with settlements of up to 3/4 of an inch measured outside the compaction grouting area just before the tunnels arrived at the first two buildings treated. A typical cross-section of the tunnel grout bulb arrangement is shown in Figure 5.

The tunnels are typically located some 35 Ft. to 50 Ft. below the ground surface and compaction grout pipes are drilled to some 10 Ft. above the crown of the tunnels before tunneling operations arrive at a subject building. The compaction grouting begins within a few feet after the shield tail passes a particular grout pipe location. Just before grouting is started at a given pipe, the pipe is withdrawn a distance of three or four feet to provide an initial void for the compaction grout mortar to exert a lateral expansion force against the adjacent subsoils. Grout pressures typically range from 150 to 200 psi. All compaction grouting is stopped when pressures reach a maximum of 500 psi, as recorded at the top of the grout pipes. Since the tunnel is being advanced on a 24-hour basis, it is necessary to have compaction grouting crews available or on a 24-hour basis so that grouting can be initiated at the moment the shield passes. Although the original specifications required that compaction grouting only begin when the building settlement exceeded 1/4 inch, the Compaction Grouting Contractor has adopted the goal of recompacting the loosened soils closest to the tunnel before the problem can be detected by building settlements on the surface. This procedure is proving to be very effective.

A detailed subsurface instrumentation program, sponsored by DOT, is being carried out to measure the ground movements produced by Compaction Grouting operations at the Bolton Hills project. The results of the instrumentation work and the completed compaction grouting program will be reported at the end of the project.

CONCLUSION

Compaction grouting procedures offer a valuable alternate technique to tunnelers who need to control ground settlements in the vicinity of urban structures. The method allows for carefully controlled placement of mortar grout to cause the redensification or replacement of lost ground and to keep the loss of ground from being reflected by surface settlements in the foundation support soils of structures above the tunnel.

It is important to note that to be most effective and to avoid associated costs of repairing structures that have previously settled, compaction grouting should be performed before settlements occur. The opportunity to perform compaction grouting during tunneling operations in those areas where loss of ground or excessive settlement is anticipated, is a new and useful technique which has proven to be a successful and easily applicable solution to this common problem associated with soft ground tunneling.

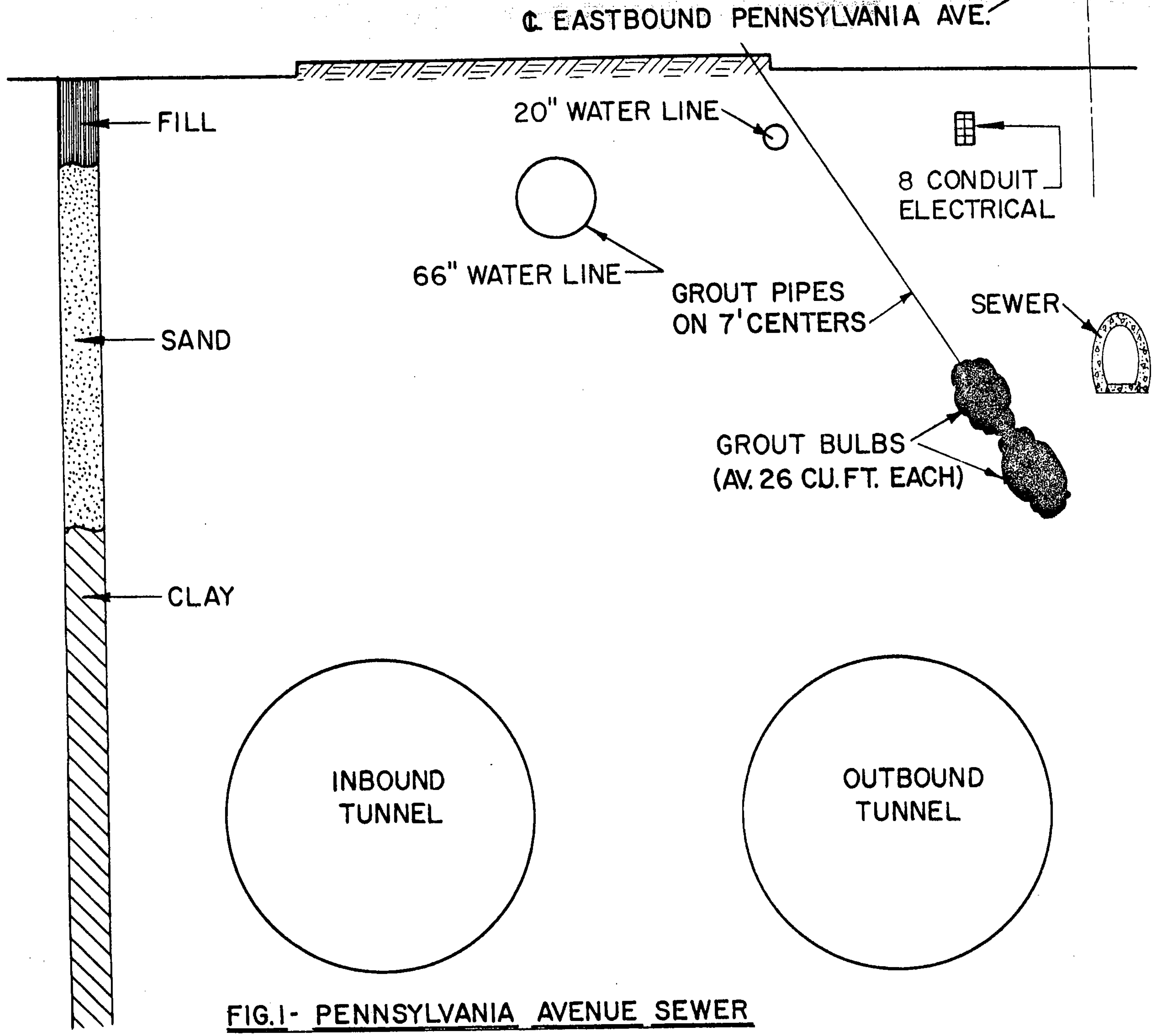

FIG. 1 - PENNSYLVANIA AVENUE SEWER

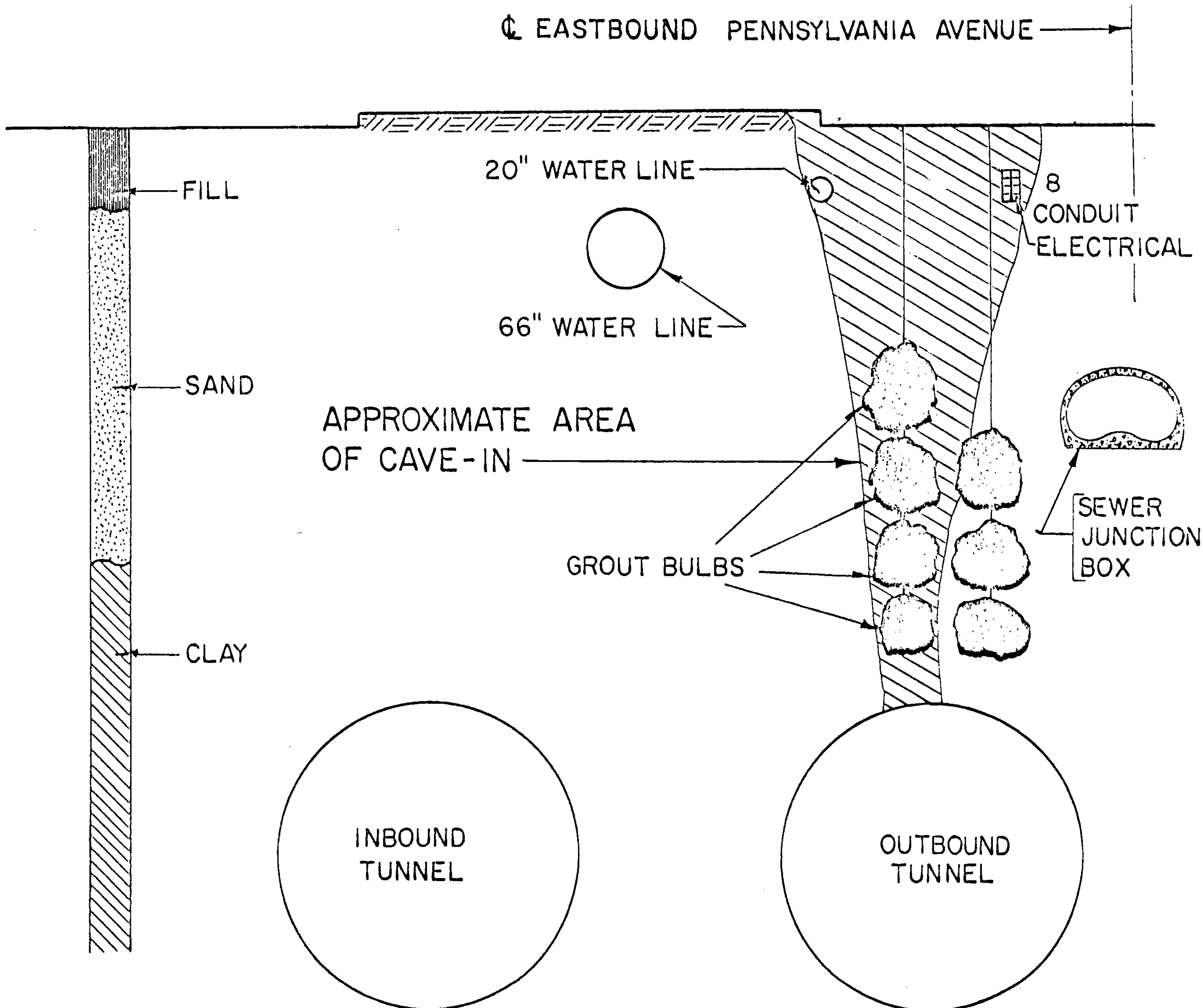

FIG. 2 - 7TH STREET & PENNSYLVANIA AVE. JUNCTION BOX

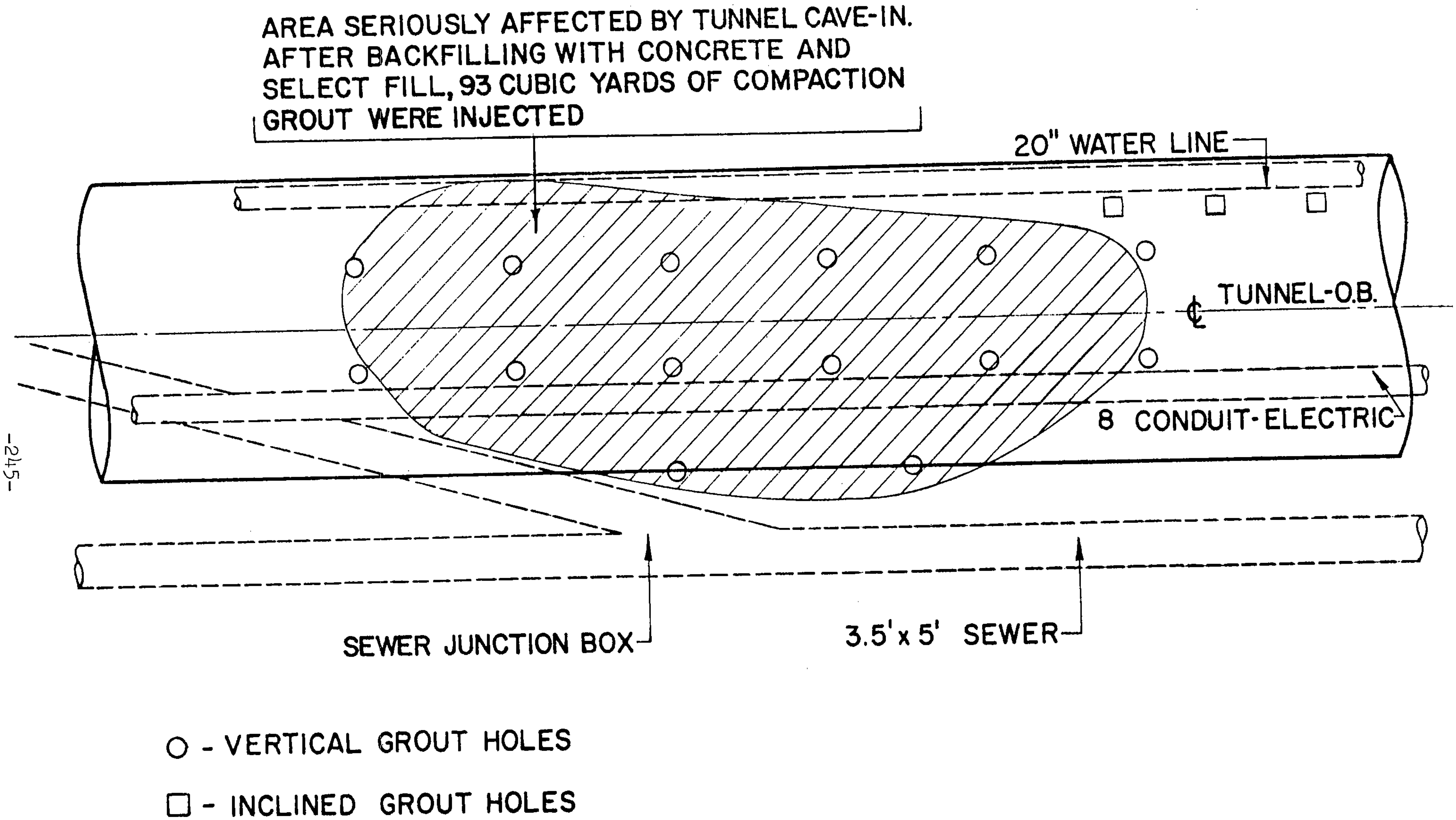

FIG. 3 - PLAN OF CAVE-IN AREA AT JUNCTION BOX

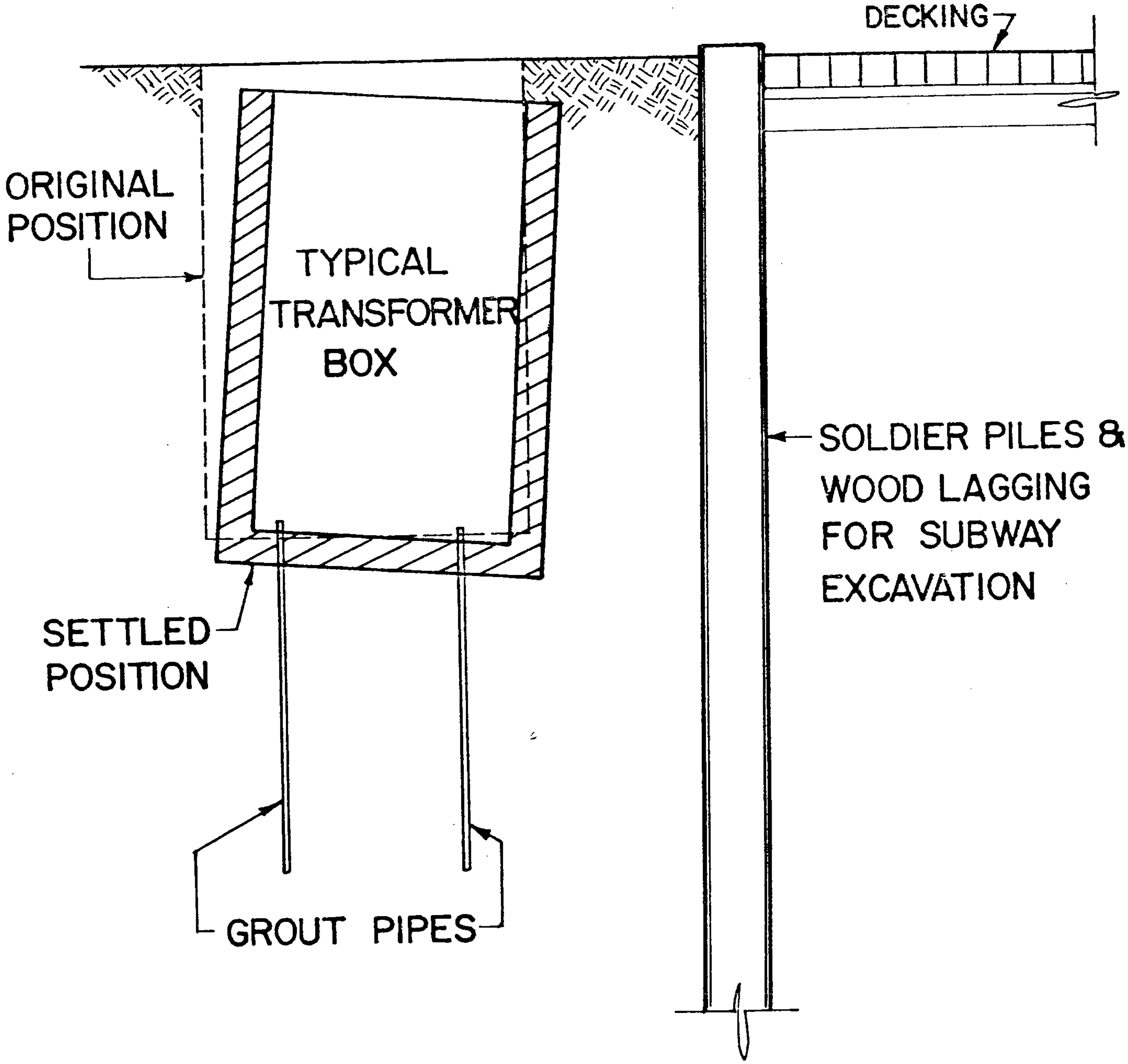

FIG.4- TRANSFORMER BOX, TWELFTH STREET N.W.

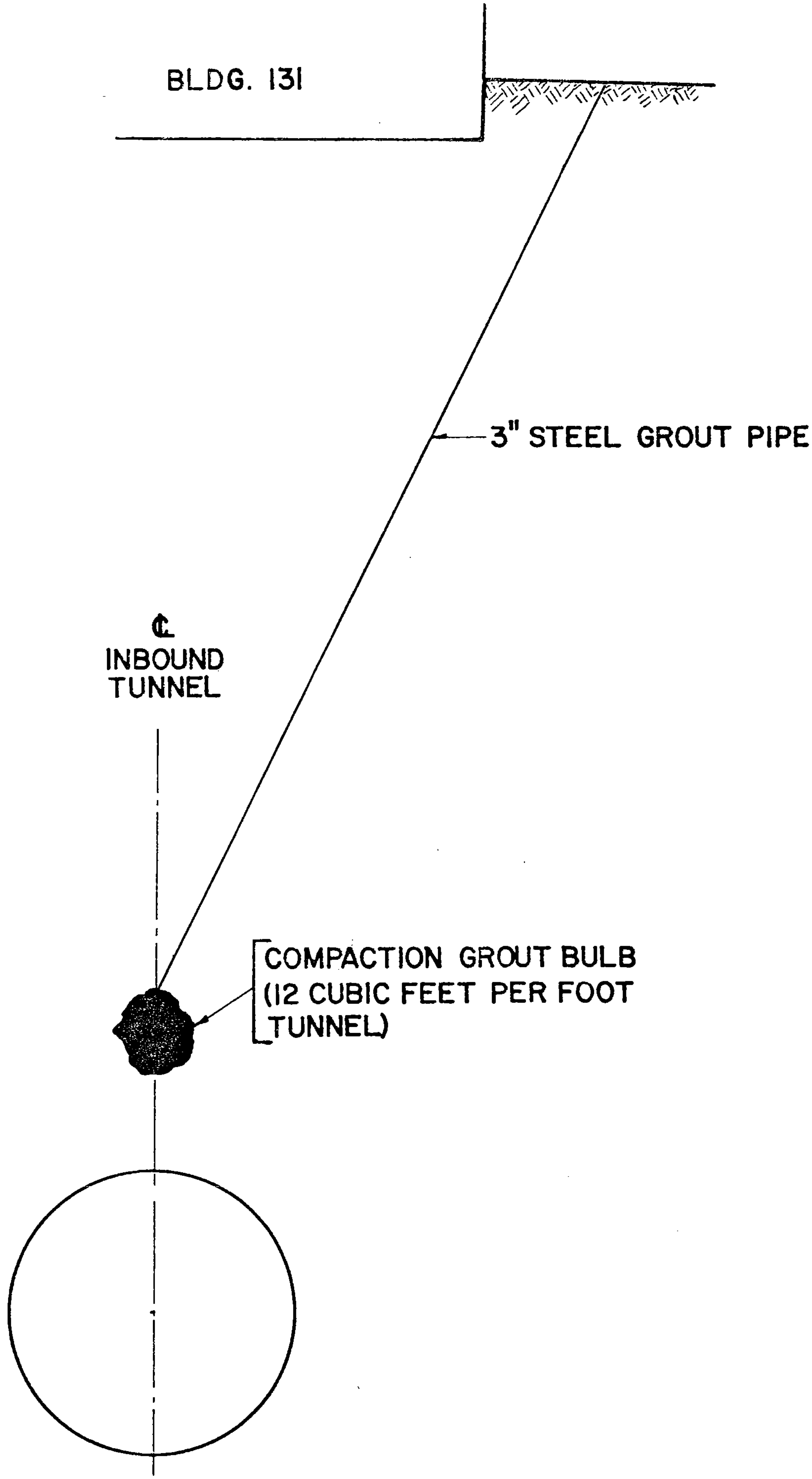

FIG. 5- BOLTON HILL TUNNEL

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SIGNIFICANCE OF GROUNDWATER AND METHODS OF GROUND CONTROL IN TUNNELING

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ABSTRACT

The tunneling characteristics of different soil groups, in general, and non-cohesive soils in particular, are very much influenced by hydraulic conditions present at a site. Several problems, including face and wall instabilities, excessive infiltration of groundwater, loss of ground, significant ground movements, interference with the routine tunneling cycle, inaccessibilities to the face with tunneling machines, difficulties during concreting in the presence of seeping water and in maintaining tunnel grade as well as alignment, and material handling problems with excessively wet muck are associated in one way or another with the presence of water. Furthermore, groundwater would require several special considerations in the design of the shield and lining including long-term distortions, liner reinforcement against differential jacking pressures, and provisions for facilities for rapid breasting and poling plates.

Fundamentally, improvement, whether general or otherwise, of grounds or measures to restrict or eliminate the reduction of stress due to excavation or a combination of both are available for effective ground control. To that end, dewatering, grouting, artificial ground freezing, and use of compressed air and/or fluid pressures, and appropriate combinations therewith not only represent various methods of temporary ground stabilization but have been used successfully as well in tunnels under varying geological conditions as exemplified herein.

INTRODUCTION

In order to successfully prosecute tunneling operations in soft grounds, it is imperative that among the several requirements, the selected method of excavation can safely advance a hole and that the integrity of the opening can be maintained with a minimum possible impact on environment. Furthermore, with the increasing trend towards mechanization, where speed is essential to reduce tunneling costs, the success of a particular tunneling system frequently depends upon a thorough understanding of mutual interactions and compatibilities between different subsystems, such as excavation, ground control, and environmental control, that make up the system. Groundwater probably causes most severe tunnel problems and is perhaps the greatest source of variability of overall project costs. The primary objectives of this paper are to outline at the outset significant problems and/or hazards associated with various activities related to tunneling operations caused directly or otherwise by groundwater and to briefly discuss mechanics and advantages as well as limitations of several methods of ground control with a few examples from tunneling projects to demonstrate their uses in practice so that intelligent appraisals of each method can be made.

TYPICAL PROBLEMS IN TUNNELING BELOW GROUNDWATER

The location of groundwater below the existing ground surface, whether under water table or perched water table conditions, is one of the most significant factors governing working conditions and affecting tunnel design and construction methods. Except for runs in dry cohesionless soils, most catastrophic tunnel problems are related directly or otherwise to groundwater. Granular soils such as sands, silts and gravels displaying little or no cohesion are likely to flow below the groundwater table. For that matter, any ground with an effective grain size in excess of 0.005 millimeter and lying below the water table is potentially flowing ground (Terzaghi, 1950). A mixture of soil and water advances into the tunnel like a viscous fluid through the invert, crown, wall and face. Should the rush of flowing ground be permitted to continue, the opening may be partially, if not completely, filled. Tunneling machines with or without a shield, if used for excavation, may be buried or frozen.

Cohesive granular soils, such as gravels with clay binders, clayey or silty sands, cohesive silts, and residual soils with remnant rock structures, located below the groundwater table are usually fast raveling grounds. Tunneling through such grounds having some cohesion would invariably increase the tendency of the chunks or flakes of the materials in the arch or walls to disintegrate and subsequently ravel into the heading within a few minutes as seepage forces caused by the flow of water toward the heading are developed. Raveling is progressive and may lead to open cavities of increasing size. Adverse seepage pressures toward the tunnel, if and when allowed to continue in fast raveling grounds, may further reduce the stand-up-time of the ground and transform the soil into flowing ground.

Soft cohesive soils such as saturated soft clays, plastic silts or silty clays have very low undrained shear strength below the water table. As the stresses developed about the opening approach the uniaxial compressive strength, these materials exhibit squeezing tendencies and accompanying plastic flow. The hydrostatic pressure about the tunnel contributes to the driving force behind the squeezing of clay. Depending upon the degree of overstressing, movements associated with slow squeezing, fast squeezing or very fast squeezing grounds are materialized (Brahma, 1977).

Geological boundaries between permeable strata underlain by impermeable strata, of clay or silt lenses below perched water in permeable soils, and of waterbearing seams of sands and silts within otherwise impervious formations connected to a source of supply could often lead to difficult heading problems. Channeling and concentration of flow in the vicinity of the interface are likely to increase potential soil erosion and migration of soil particles. Seams and pockets of waterbearing permeable soils may develop instability and a run. When the tunnel is driven through the mixed-face zone, groundwater may be perched above the interface so that as the tunnel approaches the permeable stratum, both face and roof instabilities may result.

Permeable soils below the groundwater table can be the source of large infiltrations of water during tunneling. Quantities of flow in the heading, for example, in well-sorted sands can exceed several thousand gallons per minute. Interception of such high volumes of water may lead to a slow-down or require the excavation to be stopped completely. Construction difficulties associated with the delay and actual handling of heavy inflows would surely result in significant increase in tunneling cost.

Loss of ground due to flow, ravel, soil erosion, migration of soil particles or squeeze under varying geological conditions are likely to result in ground subsidence and displacement. The long-term consolidation of the diaphragm of remolded soft clays around the tunnel and delayed migration of soils with flowing water through tunnel lining may often cause further movements.

Displacements and settlements associated with a major ground loss would most likely severely impair the existing surface as well as near-surface installations within the zone of influence. Catastropic ground losses with a heading collapse in flowing ground, very fast squeezing ground or fast raveling ground would be hazardous for tunnel personnel. Progressive raveling leading to sinkholes at the existing ground surface would create a severe hazard to surface traffic.

Soft soils below the groundwater table requiring complete containment or support at both the tunnel periphery and the face have been shoved blind or semi-blind with bulkheads or hand-mined among others with forepoling, spiling, breasting, heading and bench or pilot drift to combat potential problems with unstable face. Frequently, shields are designed under such circumstances with facilities for breasting and poling plates. Although these endeavors to excavate soft soils with large quantities of water met with varying degrees of success over the years, they are more often than not fraught with reduced overall safety, continued slow progress and increased costs.

Delayed movements may cause stress changes and long-term distortions of tunnel linings. Large cavities or loss of heading in raveling grounds are responsible for serious unbalanced or non-uniform loadings on the linings.

Maintaining the designed alignment and grade may become a problem, for example, in flowing or squeezing grounds due to the tendency of the tunneling machine to settle. To keep the machine on grade and to facilitate the steering of shields, differential pressures may be exerted by the jacks necessitating further strengthening of liners. While activities related to reinforcing of liner may interfere with the routine lining cycle of tunneling, those associated with the placement of lining prior to shoving shields cause further interference of various operations at the heading.

Support systems consisting of prefabricated steel liner plates, precast concrete segments or cast iron liners may experience problems during the erection cycle unless water and muck present at the invert are cleared away. Should the cast-in-place concrete be used for permanent ground support, water seeping into the opening may cause difficulty during concreting. Excavated soils, if they are too wet or sticky, may spill from or stick to conveyor belts. Consequently, such muck may require treatment prior to loading and transporting by conveyor belts or hauling through built-up areas.

GROUND CONTROL METHODS

Excavation of tunnels invariably reduces or removes the in-situ earth as well as hydrostatic pressures within the soil mass causing, among other problems, instabilities, difficulties with large infiltrations or ground movements. Both ground improvements, whether general or otherwise, through drainage, introduction of grouts and slurries, or artificial ground freezing and the elimination or restriction of the reduction of stress due to excavation with slurries or compressed air may effectively be used to control ground temporarily, while recognizing the fact that several other construction details including temporary and permanent supports would also significantly contribute to ground control. This section primarily attempts to briefly review methods of ground control with dewatering, grouting, ground freezing, and compressed air as well as slurry shield tunneling under varying geological conditions without pre-empting the role of specialty contractors.

Dewatering: Dewatering is frequently an economical means of ground stabilization in fast raveling or flowing grounds. Lowering of the ground water table to a level located below the invert of the tunnel would not only intercept seepage, which would otherwise emerge in the heading, and eliminate seepage pressures responsible for potential instabilities but increase the strength and stiffness of the soil. The drainage leaves the voids of porous materials at least partially filled with water which is retained by capillary forces. The surface tension at the boundaries between the air and the water in the adjoining voids would tend to pull the soil particles together such that the frictional resistance produced by the contact pressure acquires temporarily the characteristics of cohesive soils. Consequently, samples of moist clean sands, even if unconfined, are likely to display varying magnitudes of compressive strength depending upon their relative densities. Since the slightest cohesion will usually give the soil a capacity to stand up, cohesionless granular soils, which are otherwise flowing grounds below the water table, may be transformed into raveling or cohesive running grounds. In the absence of critical seepage gradients towards the heading, the cohesive granular soils would likewise behave as slow raveling or even firm grounds due to increase in stand-up time. Not only could excavation in such stabilized grounds be successfully carried out but the increased strength and stiffness of such grounds are most likely to result in moderate or negligible loss of ground and accompanying small ground movements so long as the selected method and workmanship are adequate to prevent the start of raveling or a run.

Depending upon the geologic and soil formation at a site particularly with respect to the stratification, thickness of the aquifer or drainability of the soil and upon the required drawdown as well as rate of pumping, predrainage systems including well points, eductors and deep wells used to facilitate tunnel construction. An eductor system, for instance, may most successfully be used where relatively small volumes of water, because of low permeability of the aquifer, are required to be pumped to depths as much as 100 feet, while deep wells may be particularly suited for dewatering deep tunnels located in highly previous materials requiring high rates of pumping. Difficult grounds such as silts and sandy silts of stratified deposits frequently warrant the use of well points or deep wells in conjunction with a vacuum system. The situation illustrated in figure 1 (Ameral and Frobenius, 1974) is an example where dewatering by vacuum wells is undertaken prior to driving tunnels in free air. Low permeability ($K = 10^{-4}$ to 10^{-6} cm/sec) of fine clayey sands and the presence of many silty clay lenses, above which perched water may be trapped and could cause instabilities or a run, led to the selection of vacuum wells. The addition of a partial vacuum to the well screen and sand filter increased the hydraulic gradient and flow to the deep well and created a vacuum within the surrounding soil that prevented or at least reduced seepage from perched water into the tunnel. Dewatering along with good tunneling procedures was effective in limiting settlements of the overpass to approximately 0.5 inch.

Besides the limitation with permeabilities of fine-grained soils, dewatering by pumping from well points, eductors or deep wells can contribute to ground movements. Ground movements result either from pumping fines through improperly completed wells or from consolidation of soft soils in the area due to increase in effective stresses with the lowering of the water table. Loose granular soils, even when dewatered, may not only undergo significant volume

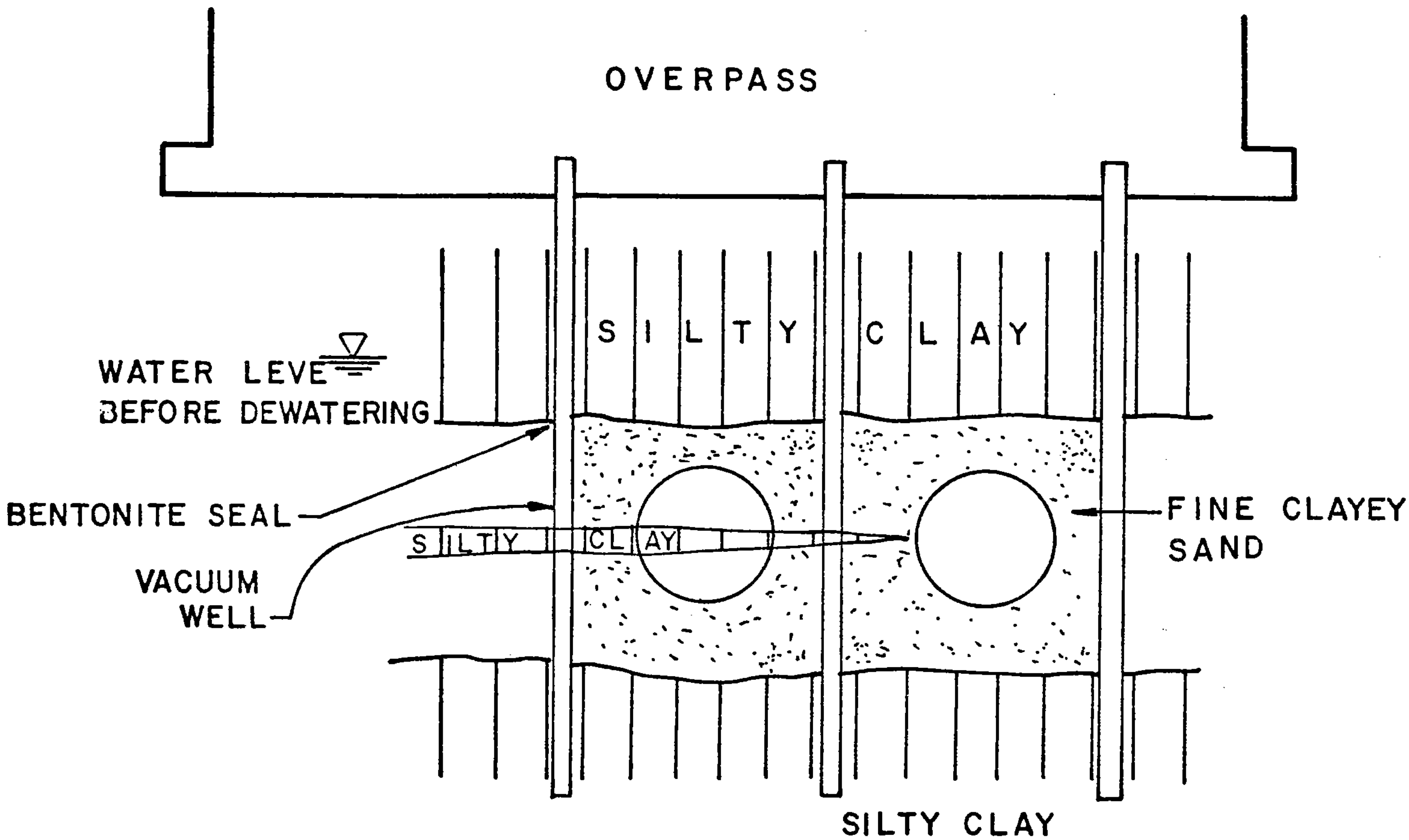

FIGURE I

DEWATERING BY VACUUM WELL SYSTEM

decreases with changes in loading conditions associated with tunneling but relax the beneficial influence of capillarity because of attendant increase in porewater pressure.

Grouting: Grouting often offers potential for selective ground improvement in granular soils below the water table, which are flowing or fast raveling grounds, if its use is limited to short stretches of such grounds. While grouting would stop or greatly reduce seepage by making in-situ soils practically impermeable, the strength as well as stiffness of the soil are likely to increase with concentration of appropriate grouts. The reason for such increase in the two-phase material could be attributed to the development of cohesive bonds between soil grains with attendant increase in tensile strength, to the increase in overall internal friction, and to the stress-strain behavior requiring a large failure stress accompanied by a small strain. The reduction of pore spaces, available initially to accommodate particulate slips, may increase dilatency as the voids are filled with grouts. The grouted soil with considerable cohesion would not only increase critical seepage gradients, but increase unconfined compressive strength and thereby the stand-up time of the medium during construction with contributions from the raised internal friction. Consequently, potentials for piping, instabilities at the heading and loss of ground resulting therefrom are considerably reduced if not prevented altogether. Furthermore, the stiffness of grouted soils which are likely to significantly increase with confining pressures may reduce settlements above tunnels.

Depending upon the primary objectives of grouting, whether water tightening or stregthening of the surrounding soil or both, and upon the existing ground conditions with respect to permeability, porosity, particle size distribution, groundwater flow, or ground chemistry, a number of grouts with varying viscosities may be injected individually or in combination with several grouts by various methods. For instance, a cement or clay grout may fill voids in coarser materials, while the same may be displaced by flowing water and may not be effective in finer materials because of its limited range of penetration. A resin grout (AM-9) has been injected into massive or varved silts without any significant gain in strength. A sodium silicate grout can effectively penetrate medium to fine sands and develop compressive strengths up to several hundred pounds per square inch. In order to gain some insight into the manner in which the grout penetrates or permeates into the soil, the case history of field grouting (Amerel and Froberius, 1974) in difficult tunneling grounds (figure 2) may be particularly instructive. Grout holes were drilled from the surface on a four-foot grid pattern and "tube a manchettes" consisting of plastic tubes perforated with rings of small holes, which are enclosed by rubber sleeves acting as one way valves, at intervals of one foot were lowered into grout holes. The annular space between grout holes and "tubes a manchettes" were filled and sealed with bentonite cement. A small diameter injection pipe fitted with two packers was lowered to the designed elevation of grouting bentonite cement followed by silicate grouts were injected. Sands and silts were split and not permeated by bentonite cement and the sheets of fluid grout under high injection pressures, required to lift the rubber sleeve and crack the annular grout, not only consolidated the intervening material but caused uplift of adjacent buildings as well. The voids in the ground were not filled uniformly or completely as fine sands, adjacent to coarse materials which accepted grout more readily, were difficult to permeate. Although some water pockets were encountered locally, the maximum

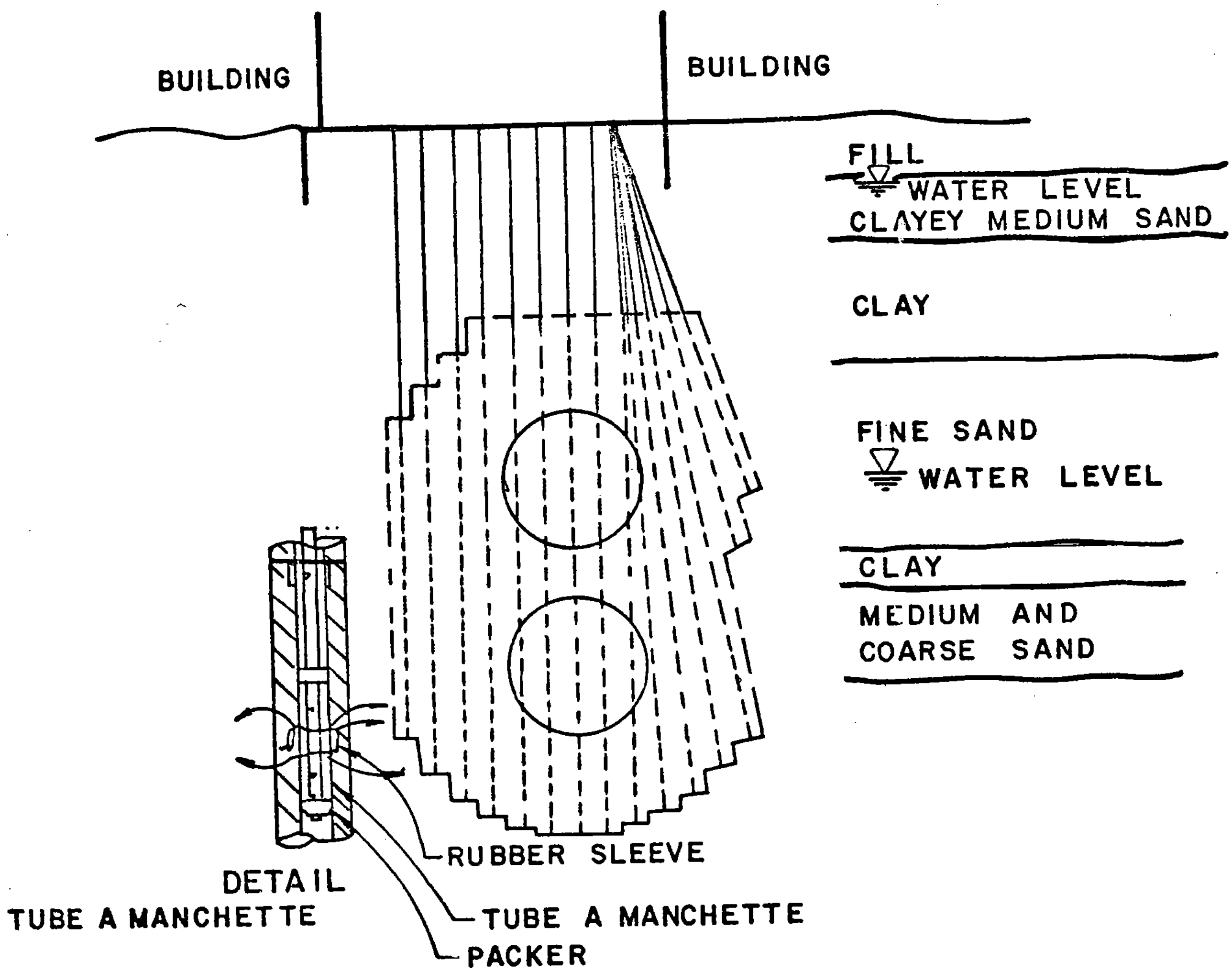

FIGURE 2

CHEMICAL GROUTING SECTION

settlement at the completion of tunneling operations was about 0.5 inch.

For successful grouting, the range in particle size and permeability of the ground approximates closely the limits for adequate dewatering. Groundwater flows may cause difficulties particularly if "windows" having potential for soil erosion are left in the grouted soil. Should the ground conditions include highly stratified coarse and fine materials, it may be difficult to locate in the field materials with different permeabilities with an exactitude. Consequently, grouting may reduce substantially permeability of coarser materials and leave zones of finer material untreated which could cause serious runs into the tunnel.

Artificial Ground Freezing: Stabilization of difficult ground by freezing has ordinarily been regarded as a construction expedient of last resort and used if other remedial measures were deemed not feasible or actually failed to perform adequately. Typically a design pattern of freezing pipes is placed in the zone to be frozen and the coolant is circulated. Freezing proceeds radially outward as a solid cylinder of frozen soil from each pipe. The cylinders finally merge between pipes to form a thick impermeable wall surrounding the area within which the soil is to be subsequently removed. As the temperature is lowered below freezing point, the strength properties of frozen soil, consisting of mineral particles, ice and films of unfrozen water, will significantly increase. The increase in strength could largely be attributed to the increased cohesive strength of the soil. Samples of frozen soil would indicate that at a constant temperature, the lower the applied stress, the greater the time to failure and that under a specified stress conditions, the lower the temperature of the ground, the greater the time to failure. Since the strength properties are influenced by the ice content, duration of applied load or the temperature of the ground, the frozen soil walls are generally designed not only to resist ground pressures consistent with creep displacements after excavation, but to reduce as well the refrigeration plant requirements through efficient freezing schemes.

While the permeability of various soils may range over several orders of magnitude, the frozen thermal conductivity which is indicative of the shape of the frozen zone, in heterogeneous ground shows small variation in comparison. Consequently, the artificial freezing with tunneling may be undertaken in most grounds either from the surface or from below ground parallel to the opening with closed or open systems using various freezing materials. However, lateral groundwater flow, quality of groundwater, sensitivity of adjacent structures to ground movements, nearby utilities, and alignment of the tunnel merit serious consideration. For instance, should the supply of heat from flowing water into the freezing soil exceed the rate, it can be withdrawn by the refrigeration unit, the ground will not freeze; while a ground with a maximum rate of water flow of 3 to 6 feet per day can routinely be frozen by circulating collant at a low temperature through closely-spaced freeze pipes with reduced movement of the frozen wall in the direction of the water flow. Dissolved salts would be likely to affect the performance of frozen wall as the strength of the soil-ice matrix is reduced at any given temperature. Fine silts and sand-silt mixtures may cause the greatest difficulties due to frost expansion if the accompanying pressures exceed the effective confining pressures as the water contained in the void is converted to ice or as ice lenses are formed under capillary action. Subsequent thawing will transform the frozen soil into a zone of highly saturated materials with considerably reduced strengths and increased compressibility. The massage action resulting from ground heave followed by noticeable settlement may prove to be detrimental to the adjacent structures sensitive to ground movements. Excessive deviation of freeze pipes in general, and particularly along curved alignments may develop "windows" of the unfrozen soil causing concentrated groundwater flows.

Figure 3 shows the geological formation consisting of alluvial deposit and powered dolomite through which a tunnel was driven for a distance of about 230 feet in three sections using ground freezing techniques (Braun and Macchi, 1974). The alluvial deposit consisted primarily of sand with varying amounts of coarse gravel and boulders. In addition, several layers of sandy gravel and silt were present. Underlying the alluvial deposit was the powdered dolomite consisting of fine silts with numerous thin fissures. The saturated silts had a low moisture content and a high density and appeared to be quite compact. Two rows of inclined freeze pipes with a distance of approximately 3 feet between individual rows were so arranged that the soil within the entire cross section of about 915 square feet of the tunnel could be kept free of frost. Two additional rows of freeze pipes were also installed in order that the freeze wall would be able to sustain ground loads until a lining had been installed and that continued driving could be maintained without requiring further support of the working face. Calcium chloride brine was circulated in freeze pipes at an average temperature of -20°C. Although blasting was resorted to particularly in the area of the cross freeze walls separating three freezing sections, the tunneling operations were successfully carried out without serious instabilities or detrimental ground movements.

Compressed Air: Tunneling operations under varying geologic conditions have frequently been enhanced by the mandatory use of compressed air. If the tunnel section in cohesionless or cohesive granular soils below the water

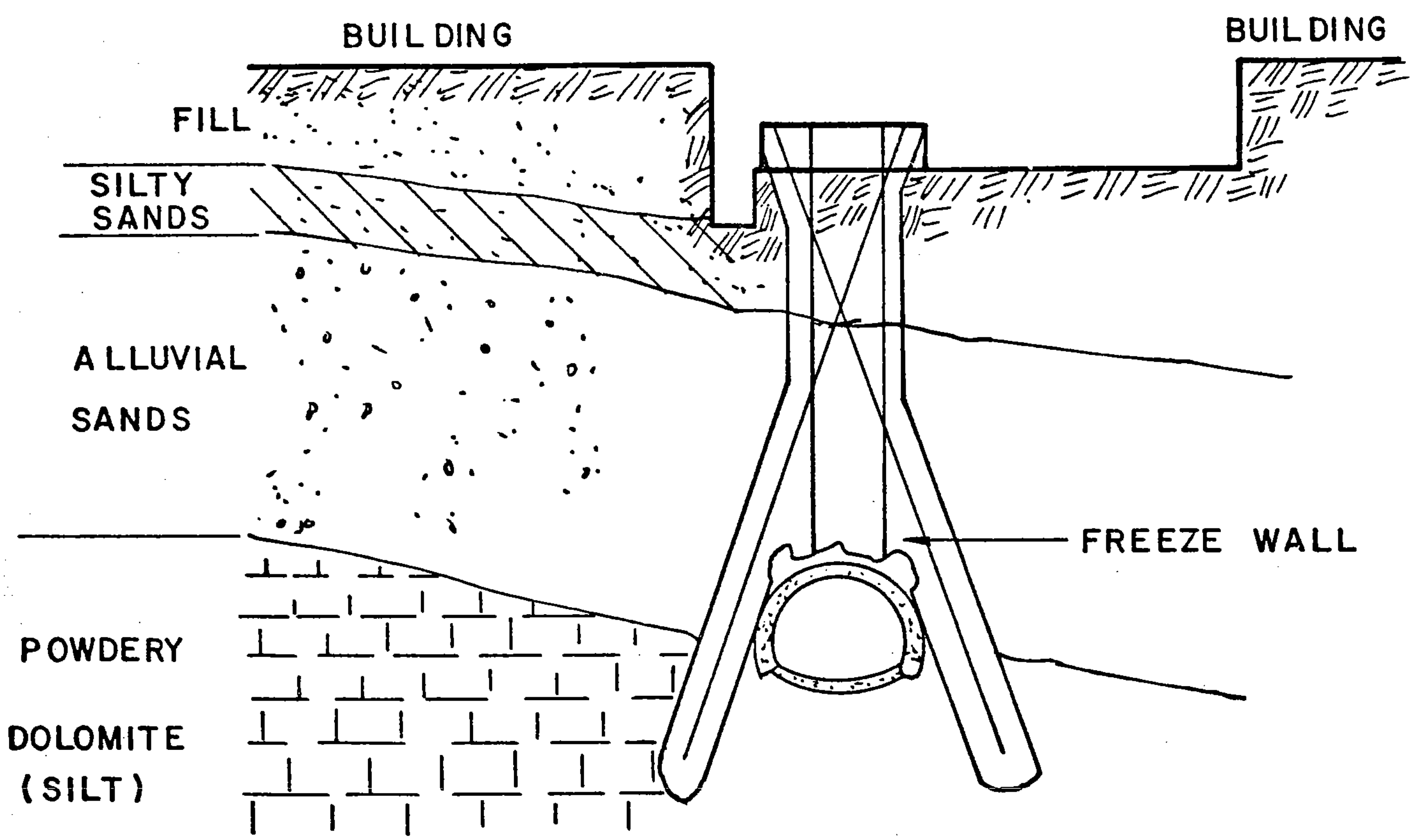

FIGURE 3

ARRANGEMENT OF PIPES WITH GROUND FREEZING table is filled with air under appropriate pressures, the air would not only prevent or reduce groundwater inflows and seepage gradients toward the opening as capillary stresses and positive outward seepage pressures, whether from the seepage of water or the seepage of air, are set up but also impart a cohesion to the soil. In clays, air pressures would lower the degree of overstressing by reducing circumferential stresses which tend to develop at the tunnel wall and also stop the seepage. Consequently, the potential instabilities and associated ground movements with fast raveling, flowing or very fast squeezing grounds are considerably reduced as the stand-up time of ground is increased with air pressures.

For a tunnel built below the groundwater table, compressed air at a pressure approximately equal to the existing porewater pressures has been applied in the tunnel or at the face only depending upon whether the crew within the tunnel work in free air or not. The differential air pressure requirements, because of the difference in hydrostatic pressure between the crown and the invert, would tend to cause granular soils in particular to dry out and run at the crown or become moisture-laden and flow near the invert. The use of high air pressures, for example, under relatively low hydrostatic heads and overburden in unstratified highly permeable soils could even lead to blowouts with devastating consequences. However, the stability of the exposed soils in most situations could greatly be improved by spraying with water, mudding with clays or lowering and raising air pressures so that the crew working at the heading could place breast boards as close as possible

Figure 4 shows the profile of the geological formation ranging from clays and clayey sands to medium sands. Twin tunnels of approximately 20 feet in diameter were driven with compressed air tunneling machines under

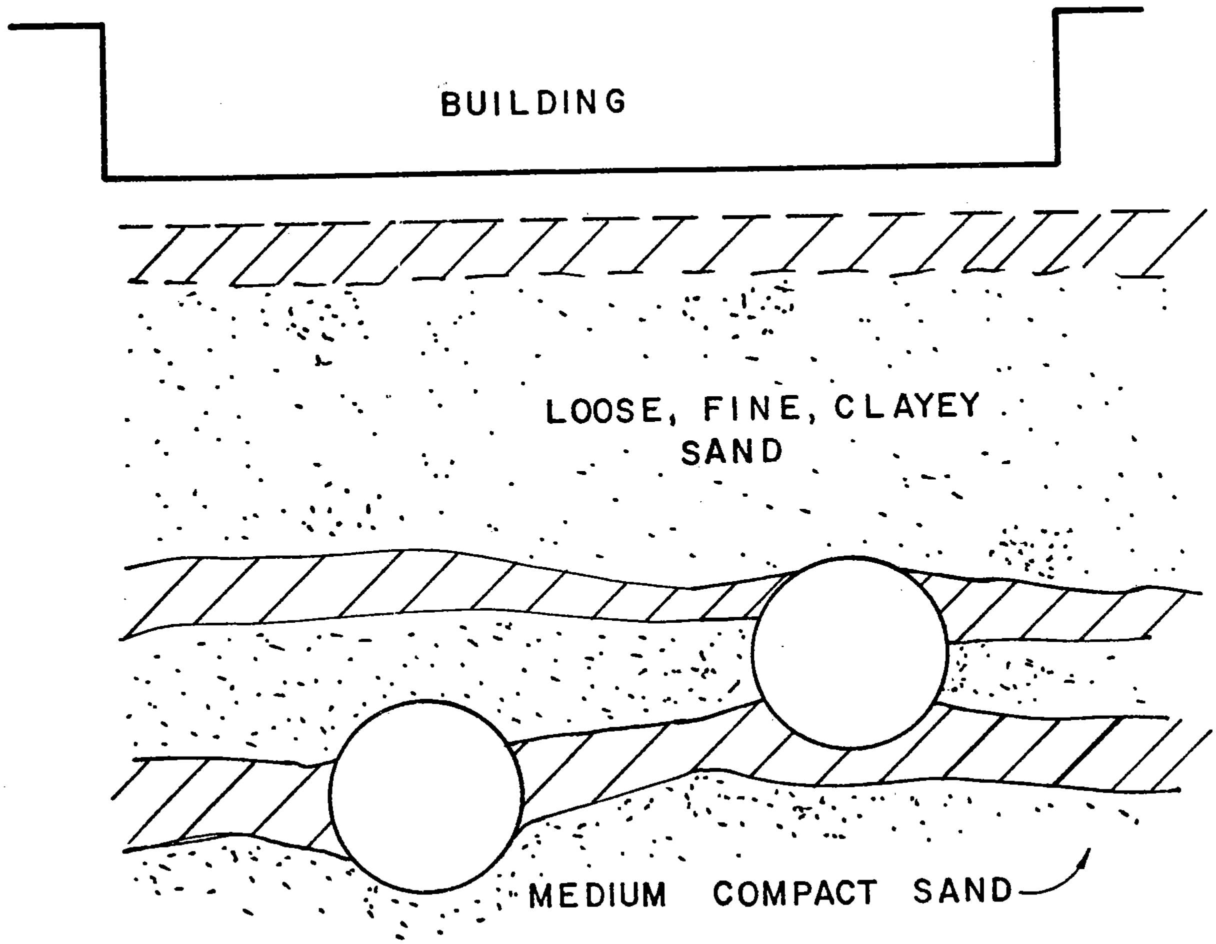

FIGURE 4

SUBSURFACE PROFILES;
COMPRESSED AIR TUNNELING

spread footings of multi-story buildings at a low clearance. Compressed air ranging from 7 to 14 pounds per square inch was effective not only in positively controlling deformation towards the heading in plastic clays and in arresting instabilities in sandy soils, but in reducing the maximum surface settlements to less than one inch causing practically no damage to the existing structures.

Besides the physiological effects of high air pressure on working men, the risk of a fire hazard with compressed air would limit or prohibit the use of combustible materials which are used otherwise to arrest water seeps or instabilities. With compressed air tunneling, it is moreover plausible that deoxygenated air may be forced into working chambers or nearby basements.

Slurry Shield Tunneling: The slurry shield tunneling method maintains a soil and water balance by applying hydraulic pressures on the face. A mud or bentonite slurry is fed in the space between the diaphragm and the cutting face. The slurry is pressurized not only to prevent inflow of groundwater, but also to maintain stability and reduce loss of ground as internal pressures provided by the slurry tend to decrease the reduction of stress due to excavation. The slurry pressure is ordinarily selected to counterbalance groundwater pressure and earth pressure, if any, at the face. The working face in clays and silts having a low permeability, for example, may be supported during excavation with slurries at pressures close to or slightly higher than the existing hydrostatic pressures and with further pressures developed by the cutterhead as it is pressed against the face; while the face in more permeable soils, such as silty sands and medium sands, would invariably require relatively increased slurry pressures. Both instabilities and associated ground movements are further reduced by using a heavier fluid, and by developing increased strength in impermeable films as well as slurry cakes. In highly permeable grounds, the potential leakage of slurry or a run of coarser materials may necessitate the use of slurries with a higher specific gravity. As the pressurized slurry flow into the surrounding permeable medium displacing groundwater, the suspended particles in it will fill the pores leading to formation of slurry cake and finally impermeable film depending upon the speed of the rotating cutterhead and the distance between the structure of the cutterhead with projecting cutting tools and the excavated face. Consequently, a reduction in peremeability and an increase in critical seepage gradient as well as strength would result.

Slurry shield machines with or without compressed air facilities have been used in tunneling through difficult grounds. Compressed air, if used in conjunction with slurry machines, displaces slurry until the cutterhead and the face are exposed for repairing the cutter or for removing boulders or other obstacles which cannot otherwise be cut or pumped out with hydraulic material handling systems. Of severl types of slurry, those containing native muds or bentonite are most widely used. While the bentonite slurry may reduce fluid loss in highly permeable soils and thickness of the impermeable film or may increase viscosity and gel, the difficulties with setting up of elaborate plants for treating excavated muck mixed with a slurry may prohibit its use in urban areas.

As the space between the cutterhead and the face is filled with slurry, there is no way to monitor visually the conditions of ground in the face so far as the amount of muck entering the bulkhead or the behavior of soil against the bulkhead fluid pressure are concerned. Excessive mucking and loss of fluid pressures would cause ground movements unless the excavated volume of soil and the level of bulkhead pressure are constantly measured and adjusted.

Ground stabilization with slurries has the greatest chance of being highly effective at the face, while the beneficial influence of the slurry to such degree is not likely to continue behind the bulkhead of the machine. The soil in the tail voids may cave causing further loss of ground and accompanied ground movements.

The slurry face machines have been utilized to effectively control ground under varying geologic conditions and under relatively shallow overburden. Figure 5 (Takahasi and Yamazaki, 1976) shows the subsurface profile in which a sewer tunnel was bored through sands and silts with a slurry mole. Groundwater was encountered within a few feet below the existing surface. Several considerations including the cost of grouting, potential blow-out with compressed air and possible excessive settlements with dewatering or freezing led to the use of the slurry mole which is equipped with precise control of the muck volume and the bulkhead pressure. Tunneling was successfully completed with a mean settlement of about one inch.

CONCLUSIONS

Some of the typical problems in tunneling below the groundwater table and mechanisms responsible for the same have been considered. Also discussed are several methods of ground control including applicability to different ground conditions, influence on soil properties, basic design considerations, construction procedure, and advantages as well as limitations of each method. Although each method of ground control could be used independently, it must be recognized that a combination of several ground control measures outlined hereinbefore may also be used to bring about the desired improvement of soil behavior.

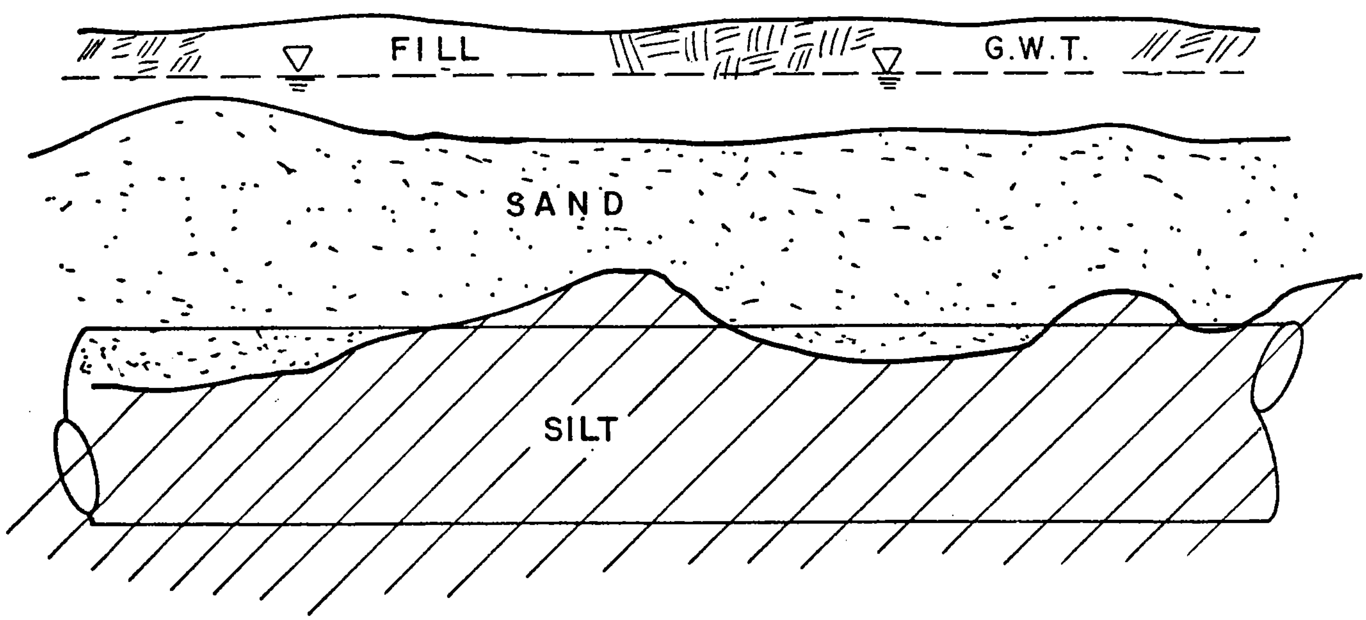

FIGURE 5

GEOLOGIC PROFILE, SEWER MAIN NO. 2

ACKNOWLEDGEMENTS

The author wishes to acknowledge assistance provided by the staff members of Daniel, Mann, Johnson & Mendenhall in Baltimore, Maryland. Thanks are due to Drs. Ralph B. Peck, Geotechnical Consultant, Albuquerque, New Mexico and Shyama P. Brahma, Principal, Progressive Subsurface Surveys India (Private) Limited, Calcutta, India and authors of several technical publications, whether specifically referenced herein or not, who were also of assistance in enriching the author's understanding.

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EXPLORATION AND GEOTECHNICAL REPORTING FOR THE ROCK TUNNELS AND STATION OF THE MONDAWMIN SECTION OF THE BALTIMORE REGION RAPID TRANSIT SYSTEM

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ABSTRACT

Exploration included vertical and inclined core borings using double and triple split tube core barrels, oriented core, and outcrop studies. Unique material definitions were developed for residual materials to describe various degrees of weathering. Standard terms were defined and used in logging and describing core samples. Core logging included comprehensive descriptions of lithology and structural features. Discontinuity studies were performed which accounted for type, orientation, shape, roughness, coatings, and fillings. Final reports were separated into those including factual data and those providing interpretation of the data as applied to the proposed construction.

INTRODUCTION

The Mondawmin Section of the Baltimore Region Rapid Transit System consists of approximately 1.5 miles of twin, horseshoe, tunnels with a mid-line station and two line vent shafts. The tunnels are to be constructed almost entirely in rock by drill and blast methods. The station and line vent shafts are to be constructed in both residual materials and rock by cut-and-cover methods. The tunnel invert will be approximately 100 feet below the ground surface at its deepest point. The Mondawmin Section generally follows an arterial street out of the City, passing through a light commercial and residential area.

The objective in preparing the geotechnical reports for the Mondawmin Section was to develop factual data and present an interpretative assessment of the behavior of the various materials for differing construction methods and conditions. These reports would then be available to the Final Designer, Construction Manager and Contractor. This approach follows the basic philosophy recommended by Standing Committee No. 4, Contracting Practices, of the U. S. National Committee on Tunneling Technology in their paper "Better Contracting for Underground Construction," of November, 1974.

The Mondawmin Section lies within the Baltimore Mafic Complex of the Piedmont Province. The Baltimore Mafic Complex is primarily composed of dark-colored, recrystallized supercrustal or volcanic, and plutonic rocks of Late Precambrian or Early Paleozoic Age. Four major rock types were encountered along the Mondawmin alignment: mica schist, foliated hornblend-plagioclase gneiss, amphibolite, and tremolite gneiss.

EXPLORATION PROGRAM

The exploration program began with a review of available literature regarding the general geology of the area and discussion with representatives of the Maryland Geological Survey familiar with the area. Cored samples from preliminary geotechnical studies of the Transit alignment were also studied. Several field trips were organized to a number of rock outcrops both near and at some distance from the alignment. Sites visited included localities of various rock types which were anticipated to be encountered along the alignment. Effort was directed at becoming familiar with the various lithologies and the manner in which they weathered. In addition, discontinuity shapes and characteristics were observed to note how they may compare with characteristics seen in the cores. Some sites were visited after the final design phase cores were obtained for more detailed comparison of core features to outcrop features.

Subsurface exploration during the final design phase consisted of vertical and inclined core borings using Christensen split inner tube and Longyear triple tube core barrels. Along the tunnel alignment, the vertical holes were spaced approximately 150 feet on center and were staggered from inbound to outbound tunnel. The inclined borings were located to furnish general lithologic data, structural data at important vertical to horizontal intersections of the facilities (such as at the vent shafts and station) and to confirm the regional trend of the direction of foliation. Data was also generated by oriented coring in the mica schist immediately adjacent to the Mondawmin Section. Coring was accomplished with the Christensen-Hügel system consisting of a multishot directional survey instrument, a modified double tube core barrel and bit, and the Hügel scribing shoe.

DEFINITIONS AND DESCRIPTIONS

Each rock type was described in detail in both the geotechnical reports and the contract documents. This description included typical features of the rock as well as known variations in the features. Each rock type was given a letter code which was used in preparation of the boring logs. Thus, repetition of the description was minimized. A brief description of one of the four major rock types is as follows:

AMPHIBOLITE: a fine to coarse grained metamorphic rock composed chiefly of amphiboles (primarily hornblende) and feldspars (primarily plagioclase) with foliation ranging from an indistinct massive texture to pronounced almost gneissic texture. It is frequently crossed with random, thin carbonate and/or other crystalline stringers, and thin pegmatites ranging in thickness from 1 inch to approximately 1 foot. The amphibolite is frequently interlayered with bands of coarser grained gneissic textures and mineralogies ranging from a quartz-plagioclase gneiss to a hornblende rich amphibolite gneiss with a salt-and-pepper appearance. The amphibolite is locally chloritic and contains zones of ultramafic mineralogies characterized by serpentine-chlorite which exhibit slick, smooth serpentine-chlorite joint surfaces.

The ultimate users of the geotechnical reports will have a wide variety of backgrounds; from structural designers to construction management personnel and contractor's personnel. As is usually the case, many of these people are not geotechnical engineers or geologists and they often come from various parts of this country or abroad. Therefore, the terms used in describing the physical features of the core samples were defined in the geotechnical reports and the contract documents. Those terms defined were: joint; parting; shear; shear zone; surface configuration such as planar, curved, and wavy; degree of surface roughness such as smooth, rough, very smooth, very rough, jagged, and stepped; and coatings and fillings. Some sample definitions are as follows:

1. Joint: any non-parting, non-mechanical minor break or fracture within a rock mass along which no apparent movement has occurred.
2. Shear: a discontinuity along which there has been an apparent single planar movement, including those discontinuities where there is very minor disruption along either side of the plane to those discontinuities where brecciated material is present. (There is no finite width measurement applied to define a shear).
3. Rough surface: joint surfaces in which the high and low points along the surfaces are well distributed along a plane, and the distance between the highs and lows is relatively moderate with individual grains or cleavage faces being distinct.

Due to the variable weathering and decomposition characteristics of rock types along the alignment, definitions were developed to distinguish between the degrees of decomposition. Although the tunnels are fairly deep, there are areas where decomposition of the rock mass exists within the profile. Also, at all vertical excavations construction will be carried out through the full range of decomposed materials. The Mondawmin Section is northwest of the Fall Line and there is no sedimentary soil cover. In other segments of the project where the sediments exist, they were also described in detail. The definitions established for the residual materials are based on visual observation and to some degree, their reaction to test boring and sampling. Because each category of material is within itself somewhat variable, no strict test value limits or other quantitative measurements were applied to the definition. Of course, there is a degree of interpretation and judgement which must be applied when categorizing core samples in this manner. In addition, it must be recognized that the transitions between the various residual material zones and rock are not sharp boundaries as may be inferred from the boring logs and that one or more zones may not be present above the rock at all locations. The definitions of the residual material zones used for this project are as follows:

1. Residual Soil (RS): Those materials which have decomposed to the degree where they have lost their visible remnant rock structure or have been reworked following partial decomposition and have characteristics which suggest relation to the underlying materials. The major or total component of this material is soil-like, although it may contain rock fragments, most of which are friable. This material does not usually exhibit remnant rock structure such as schistosity or relict joints. Standard Penetration Tests in this material may have a wide range of results greater or less than 100 blows per foot.
2. Residual Zone #1 (RZ-1): Residual material in this zone is considered transitional between Residual Soil and the less decomposed Residual Zone #2. It consists of material derived from the in-situ decomposition of the parent rock with soil-like components and partially weathered and/or fresh rock components. RZ-1 material usually retains some cohesion of the parent rock and exhibits visible remnant rock structure such as schistosity and relict joints. Materials in this zone can normally be sampled with soil sampling techniques. In most, but not all, cases the Standard Penetration Test

results are greater than 100 blows per foot. After the material is disaggregated by hand, or using motar and pestle, the constituents are described as soil.

3. Residual Zone #2 (RZ-2): Material which is rock-like, being derived by partial decomposition of the parent rock with partially weathered and/or fresh rock components. This material, in-situ, usually retains rock structure and considerable strength of the parent rock. The RZ-2 material is commonly heterogeneous with respect to weathering, ranging from decomposition throughout the entire body to partial decomposition throughout the material. This material cannot usually be disaggregated by hand and it is described with rock descriptions, notation of soil-like matrix or filler when appropriate, and a notation of the RZ-2 designation. Material in this zone usually requires rock sampling techniques to obtain specimens from boreholes.

LABORATORY TESTING

Compared to the amount of effort applied to core logging and discontinuity studies, the amount of laboratory testing was minimal. Tunnel design is, by and large, an art rather than a science. Thus, copious amounts of quantitative data which would probably never be fully utilized were not produced. Those data which were developed were petrographic analyses, unconfined compression tests, unit weight measurements, and X-ray analyses. The X-ray testing was performed primarily to determine the presence and kinds of clays in gouge, suspected gouge, residual materials, and some coating material associated with a number of chlorite segregations.

CORE LOGGING

Core logging was a very important part of the geotechnical study. A great deal of time and effort was applied to this aspect of the work in order to provide very detailed data. However, utilization of rock type codes, abbreviations for residual material zones already defined, and previously defined wording for various physical features eliminated vast, wordy repetition.

Each core was described with the appropriate rock type code with notation made as to its being catagorized as rock or RZ-2, as necessary. Other lithologic features such as intrusions, mineral segregations, stringers, degree and variation of weathering, and foliation development were described. Also, the variation in foliation orientation with respect to the core axis was noted.

To aid in assessing the overall rock quality, core recovery and RQD were measured on a run-by-run basis. These values were measured in the field and, as much as possible, before the core was removed from the core barrel. This procedure was only possible with the split inner tube core barrels.

In addition to the attention given the lithologic features, concentrated effort was placed on describing the physical character of the discontinuities. This included description of discontinuity shape, degree of surface roughness, types of coatings and fillings, and degree of iron staining. Shears are of particular importance and each shear, or feature which may possibly be a shear, was clearly labeled.

Sketches of each core box were drawn in the field. Each sketch indicates discontinuity locations and inclination to the core axis, whether the discontinuity is a joint, shear, parting or mechanical break, location of broken zones, location of shear zones, presence of filler material, rock type, and depth of each core run. Discontinuity inclinations (90 degrees from dip) are shown by letter code with each letter corresponding to an angle range of about $\pm$ 7 degrees.

Each core box was photographed in color and a color print was attached to each log in the geotechnical report. Although this procedure seems relatively expensive, having the photographs available at design meetings was tremendously helpful. In addition, during the pre-bid period, contractor's personnel can study the photographs and pinpoint those cores representing typical conditions and areas of special signifiance. Then, without spending long hours routing through hundreds of core boxes, they can spend their time more efficiently being able to select only those core boxes which they may want to study in detail.

An example of part of a complete boring log, with a copy of the core sketch and photograph is presented as Figures 1 and 2 at the end of this paper.

DISCONTINUITY STUDIES

The purpose and practical value of the rock mass structure studies is to provide information useful in predicting potential problems and assessing conditions which might be expected during tunneling and other excavation. This information is intended to provide a general understanding of the rock when considering such items as rock loading, stability of cuts and tunnel headings, evaluation of tunneling methods, overbreak, and temporary support systems.

Cores selected for the discontinuity studies were generally those with higher values of recovery and RQD. The studies would have been impractical on the more broken cores. Cores were also selected to be representative of the rock types encountered and to provide samples from throughout the alignment.

The first procedure used in studying the vertical cores started with a reassembly of the core in the box as it was in the ground. This meant that each piece of core had to be matched, the general direction of foliation was used to align the core. The core was re-assembled so that the foliation strike was approximately vertical. The discontinuities were then examined and identified as to strike and dip direction.

The second procedure employed for studying vertical cores was to hold each piece of core in the hand and to align the foliation dip direction to zero on a protractor. The azimuth of the discontinuity was then read directly. In each case, the discontinuity dip angle was measured with a hand-held inclinometer. The dip azimuth and dip angles were plotted on equal area polar projections and contoured. After the plots were made, the foliation dip value was aligned at the previously determined (from outcrop data, oriented core and inclined core) preferred orientation so that the plots would be directionally correct.

After orientations were determined they were grouped into sets defined by strike and dip. The strike categories were: foliation, orthogonal (perpendicular to foliation), intermediate east-west, and intermediate north-south. Dip categories were: horizontal and sub-horizontal (0° to 25° dip), midrange (25° to 60° dip), and vertical and sub-vertical (60° to 90° dip). These are shown graphically on Figure 3. Having categorized the discontinuity orientations, major and minor sets were defined as: major, greater than 30 percent of the total discontinuities exluding vertical and sub-vertical discontinuities; and minor, 10 to 30 percent of the total discontinuities excluding vertical and sub-vertical discontinuities. For each category the average distance between discontinuities, as measured along the core axis, was determined and then converted to average spacing between discontinuities. Vertical and sub-vertical data were neglected because of the difficulties in studying nearly vertical features in vertical borings. Typical graphical representations of discontinuity orientations for two of the major rock types are shown on Figure 4.

Some of the cores from inclined borings were studied to assess the spacing of vertical and sub-vertical (steep) discontinuities. The cores studied were primarily those exhibiting a good degree of foliation development. The inclined cores studied included three pairs with one boring from each pair having been drilled approximately perpendicular to the strike and dip of foliation and the other boring having been drilled at the same inclination but at a bearing 45 degrees north of the first. The borings drilled perpendicular to foliation were analyzed by using the simplifying assumptions that variation in foliation perpendicular to the core axis was due to variation in dip, not strike, and that where foliation was not perpendicular to the core axis it was steeper than the estimated dip. These assumptions were based on data obtained from the vertical core studies. With these assumptions it was possible to determine the upper side (of the cylindrical surface) of the cores drilled perpendicular to foliation. The core was then re-assembled in the core boxes and the orientation and spacing of the steep discontinuities determined. Data from core borings drilled at 45 degrees to foliation strike was obtained by using a goniometer. The goniometer was set at the bearing and inclination of the boring and a piece of core inserted. The core was then rotated until the foliation corresponded to the estimated orientation of foliation for that rock type. With the core in this position, the strike and dip values of the discontinuity could be measured. Equal area polar projections were made from the data so that the preferred orientation of the steep discontinuities could be obtained.

REPORTING

The geotechnical information was separated into that representing factual data and that representing interpretation of the data. This information was compiled in a set of reports which were then made available to prospective bidders with the contract documents.

The factual material was presented in a four volume Data report. The report text covered such topics as a brief route and site description, general geology of the area, general subsurface conditions at the site, a description of the methods and procedures used in performing test borings, sampling, field testing, and laboratory testing. All of the laboratory and field test data was presented in both summary and detail form. The report included boring plan and profile drawings along with the detailed boring logs including the core photographs. The Data report also included the descriptions of the various rock types, definitions of the residual material zones, and definitions of terms used in the report and on the boring logs.

The purpose of the interpretative report was to present the results of the geotechnical engineer's analysis and review of the geotechnical data in assigning physical properties and discussing anticipated behavioral characteristics of the materials encountered. The report format was designed with summary information first and increasing detail in later sections. This approach permits rapid review at various levels of detail for varying purposes.

For each rock type the report addressed such topics as possible block and wedge configurations, stability problems, in-situ stress conditions, identification of shears and shear zones, slaking, swelling and ravelling potential. The pros and cons of various methods of tunnel excavation, such as full face tunnel boring machine, boom excavators, and drill and blast, were discussed. The implication of the direction of tunnel driving for each rock type was discussed. The report also covered such topics as overbreak, rock pre-reinforcement, temporary support for tunnels and vertical excavations, tunnel pillar stability, and the effects of groundwater.

The interpretations were based on integrated information and findings from various sources such as published reports, surface outcrop investigations, experience from other projects in the area, and the detailed data collected from the subsurface investigation and laboratory testing programs during the final design phase.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the following persons for their contributions through the design phase of the Baltimore Region Rapid Transit System:

Mr. Frank A. Hoppe, Director of Engineering and Construction for the Mass Transit Administration of the Maryland Department of Transportation, for his foresight in realizing the value of thorough geotechnical engineering studies.

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Mr. Harlan C. Lunn, Associate, Hatch Associates, Inc., for providing tunneling expertise, technical experience and assistance in preparation of the geotechnical reports for this project.

BORING NO.: NWE-20 STATION: 169+99 OFFSET: 66' Lt

DATE START: 5-20-75 DATE COMP: 5-27-75 ELEV: 287.8

BORING METHOD / DIA: DC/4" ID RIG TYPE: 40-C

STRATA DEPTH	SAMPLE DEPTH	CASING BLOWS	SAMPLE NO.	BLOWS/6"		TYPE/SIZE	REC/ATT	DESCRIPTION (Color, Density, Moisture, Proportion, etc.)
								287.8 Greenish brown ROCK FRAGMENTS (RZ-1, -)
2.1								285.7 Begin Coring 2.1'
	2.0		1	100/.1'		DS/3"	2"/2"	

RUN NO.	DEPTH (Ft.) FROM	TO	RECOVERY IN.	%	R.Q.D. IN.	%	DRILLING TIME	REMARKS (Water Level, Core Jam, etc.)
1	2.1	4.5	17	63	0	0	15 Min	RZ-2 MATERIAL
2	4.5	6.5	17	71	0	0	10 "	" "
3	6.5	9.6	31	84	0	0	20 "	" "
4	9.6	14.5	58	98	4	7	45 "	RZ-2 MATERIAL TO 14.0'
5	14.6	19.4	57	97	26	44	60 "	
6	19.4	24.3	55	93	45	76	75 "	
7	24.3	28.0	48	109	34	77	60 "	
8	28.0	32.7	56	100	40	71	60 "	
9	32.7	35.0	25	89	22	79	25 "	
10	35.0	37.5	25	83	25	83	25 "	
11	37.5	41.5	51	106	35	73	25 "	
12	41.5	46.0	54	100	50	93	70 "	
13	46.0	48.5	30	100	21	70	150 Min	
14	48.5	51.7	34	89	21	55	25 Min	
15	51.7	56.0	55	106	27	52	40 Min	
16	56.0	60.5	54	100	36	67	60 Min	
17	60.5	65.2	57	100	34	60	90 Min	
18	65.2	70.0	58	102	52	91	70 Min	
19	70.0	74.8	57	100	46	81	70 Min	
20	74.8	79.5	57	100	19	33	65 Min	
21	79.5	84.5	59	98	34	57	65 Min	
22	84.5	89.4	57	97	51	86	65 Min	
23	89.4	94.3	59	100	41	68	60 Min	
24	94.3	99.0	56	100	50	89	65 Min	

DESCRIPTION OF ROCK TYPE

DEPTH (Ft.) FROM	TO	TYPE CODE	DESCRIPTION (Color, Type, Weathering, Structure, etc.)
2.1	14.0	RZ-2	Dark gray-green decomposed fine grained AMPHIBOLITE
14.0	16.5	A	Dark to light grayish green AMPHIBOLITE interlayered with coarser grained light gray Quartz-Plagioclase GNEISS, with numerous thin carbonate and quartz stringers, and locally chloritic
16.5	50.5	D	Light gray Quartz-Plagioclase GNEISS
50.5	76.5	A	
76.5	77.4	E	Light gray to pink coarse to very coarse grained PEGMATITE
77.4	99.0	A	

DESCRIPTION OF JOINTS

	a	b	c	d	e	f
Inclination to Core Axis	60°	45°	30°	90°	0-20°	70°
Shape and Roughness	Joints are generally rough & planar, frequently very rough & irregular, occasionally smooth & slightly curved or wavy.					
Coatings — Filling Material	Generally iron-staining and associated weathering products to 50'; with carbonates and quartz predominating at depths greater than 20'. Apparent serpentinized-chloritized surfaces, especially within shear zones.					

OTHER DISCONTINUITIES

Possible Shears at: 48.1', 48.6', 54.0', 67.2', 68.5', 74.0', 74.5', 75.0', 79.0', 83.5', 86.0', 86.5', 86.7', 87.1', 89.0', 89.4'.

Possible Shear Zones at: 60.0'-62.0', 64.2'-64.7', 78.0'-78.5'

X = Broken, No Filler ; W = Broken, Soil or soil-like Filler.

Foliation generally indistinct but appears to be approx 30° to Vertical.

Slightly weathered throughout.

CORE SKETCH

LEGEND: A, B, C, ... = Rock Type
a, b, c, ... = Discontinuity
M = Mechanical or handling break
S = Wood spacer block
P = Foliation Parting
a, b, c, ... = Parallel to foliation
* = Possible Shear
** = Possible Shear Zone

GENERAL REMARKS

5-21-75 at 0730: Water at 9.2' measured with casing in ground.

5-22-75 at 0730: " 9.2' " " "

5-23-75 at 0730: " 9.2' " " "

Falling head permeability test performed at 48.5'

Hole grouted upon completion

FIGURE 1

Typical Boring Log as Presented in Geotechnical Report and Contract Documents

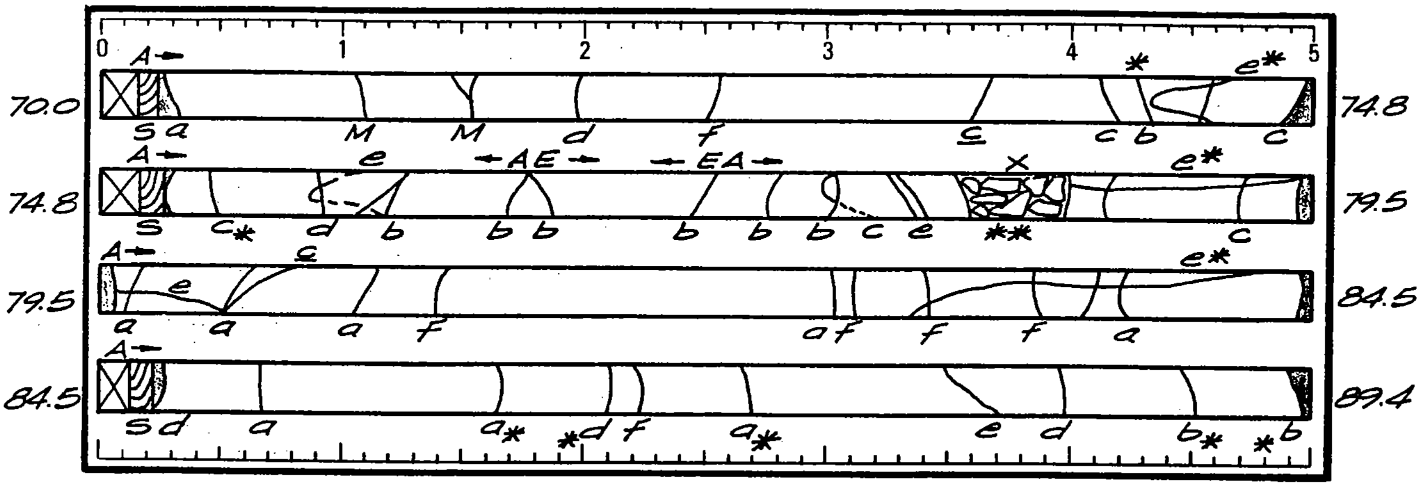
CORE SKETCH AND PHOTOGRAPH

Project: 3900 BALTIMORE REGION RAPID TRANSIT SYSTEM

Design Section: NW-06-01-D MONDAWMIN LINE Date 5-27-75

BORING NO. NWE-20 Box 5 of 6.

CORE SKETCH



LEGEND: A, B, C,.... = Rock Type
a, b, c,.... = Discontinuity
M = Mechanical or handling break
S = Wood spacer block
P = Foliation Parting
a, b, c,..... = Parallel to foliation
* = Possible Shear
** = Possible Shear Zone

FIGURE 2
Typical Core Sketch and Photograph Included With Geotechnical Report

CORE PHOTOGRAPH

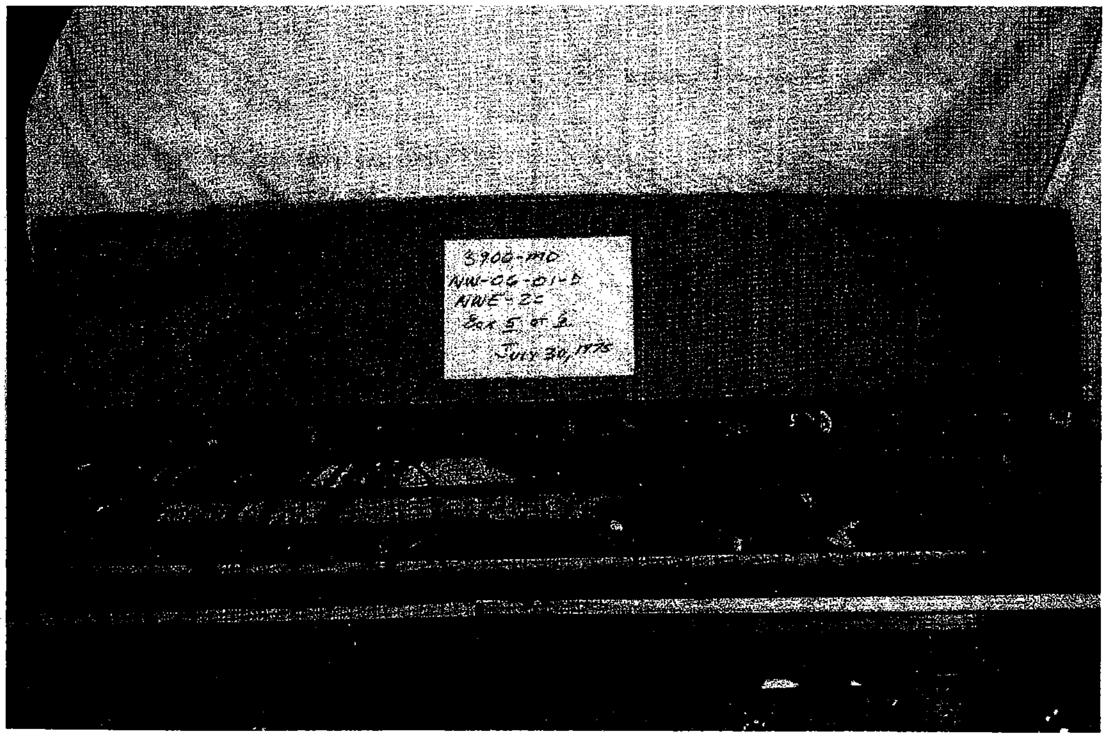
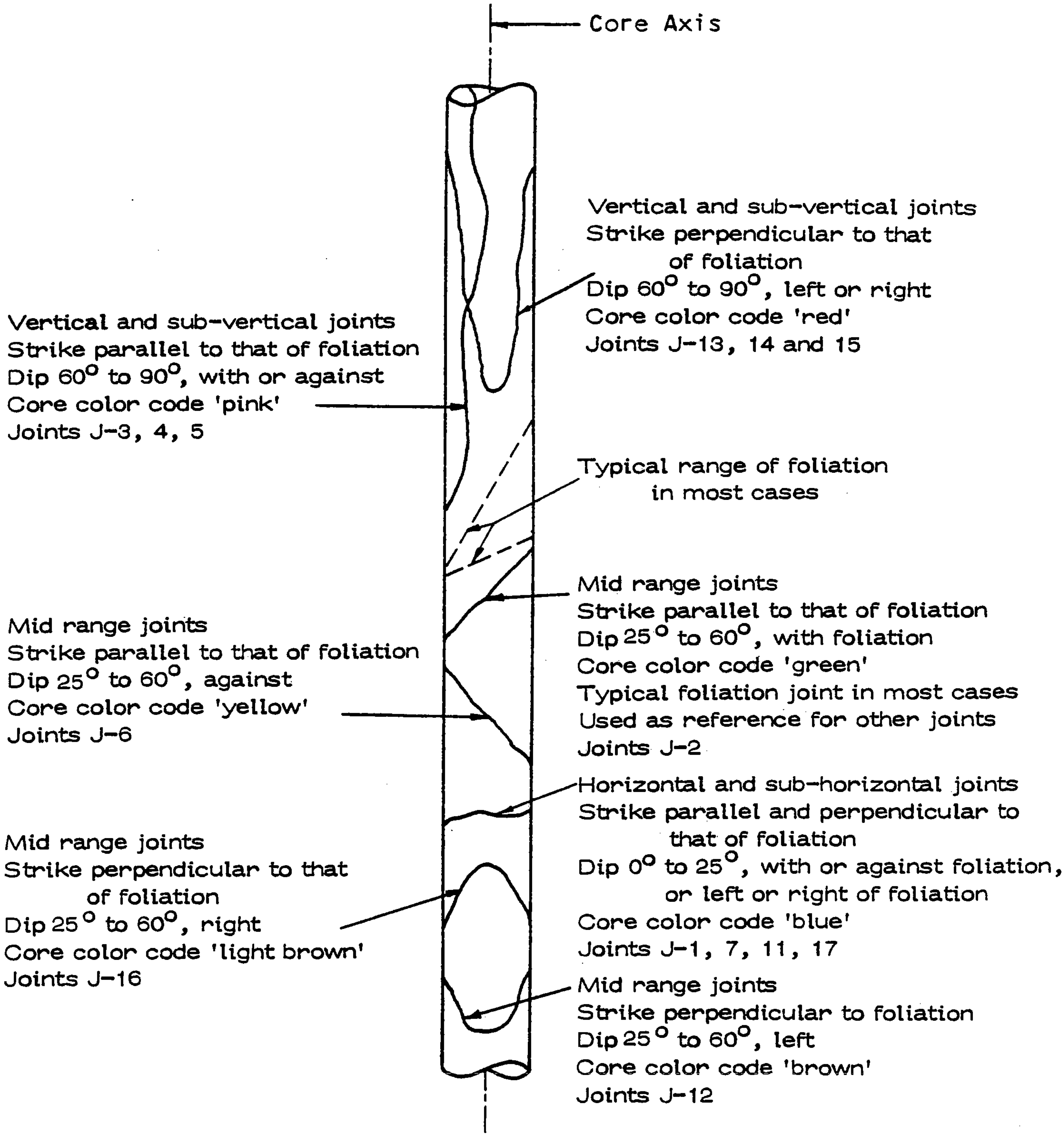


FIGURE 2



Notes: Core color code indicates those colors applied to the core samples when performing the joint analysis.
Joint orientations given as per JOINT AND SHEAR REFERENCE SYSTEM, PLATE III-1
All angle ranges are approximate.

FIGURE 3
Graphical Representation of Discontinuity Reference System

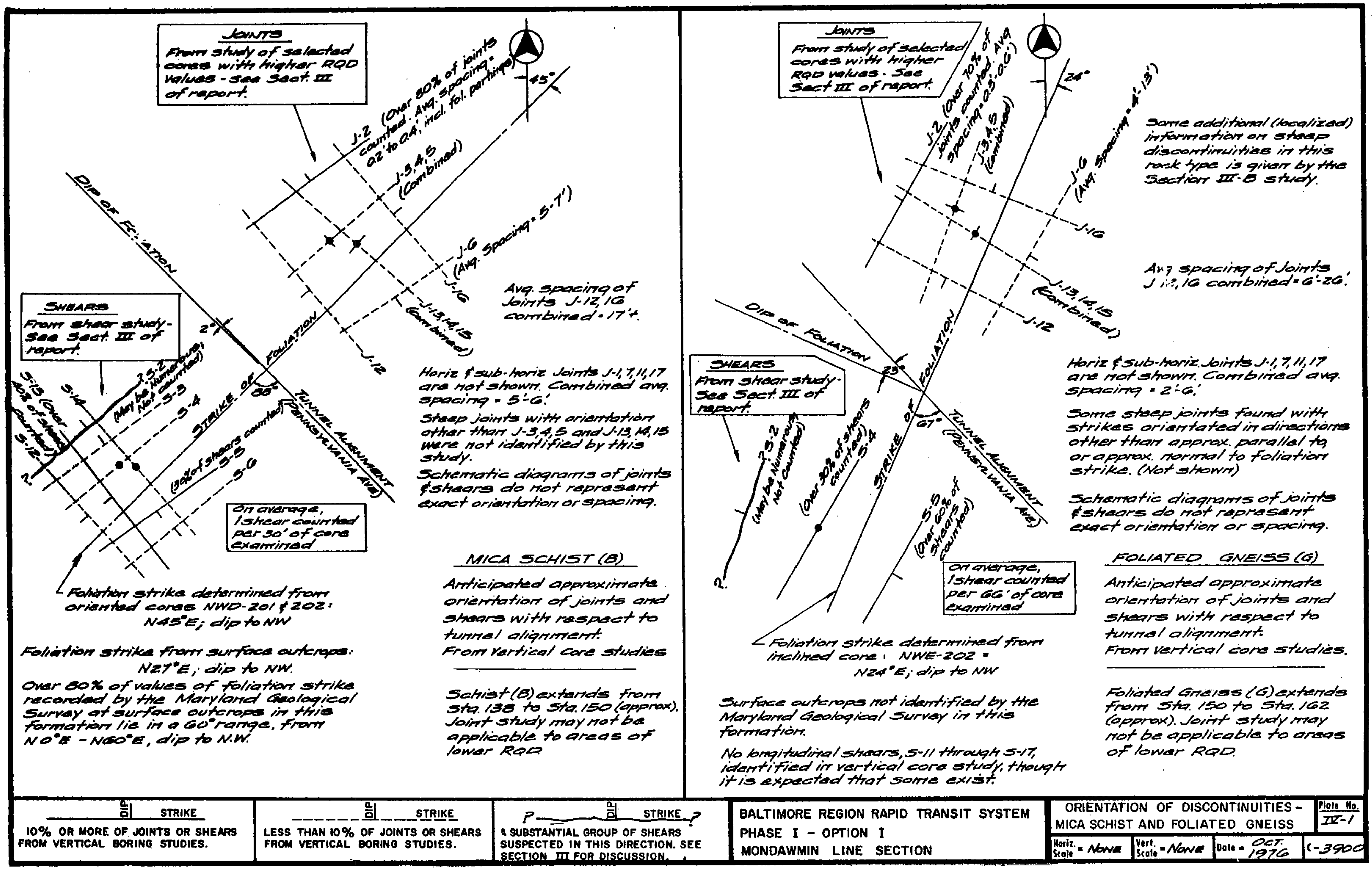


FIGURE 4

Graphical Presentation of Discontinuity Orientations

NEW METHOD OF SHEAR SURFACE GENERATION FOR STABILITY ANALYSIS

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ABSTRACT

Slope instability is a major geologic hazard on both sides of the Fall Line. The slope failure may occur within the bedrock, totally in the soil overburden, or in a contact zone between the two. Assessment of failure risk is commonly achieved through statical analysis of a sliding mass mobilizing cohesion and/or friction. A major uncertainty in the analysis may be the appropriate position and shape of the most likely surface of sliding.

For some time, techniques for analysing many positions of an assumed shape of sliding surface (viz., a circle) have been available. More recently, techniques have been developed to allow analysis for an assumed position of any shape of sliding surface. This paper reports on the development and use of an analytical technique in the computer program STABL, whereby effectively any position of any shape of sliding surface may be quickly and economically assessed for risk of failure. Where the position and shape of the critical sliding surface is poorly known, the program uses a random number function to generate a large number of irregularly shaped failure surfaces. Where a linear zone of weakness seems to control the stability, randomly generated sliding "blocks" may be generated and analysed for risk.

While the major uncertainties in slope stability analysis will probably always be the available strength of the subsurface materials, engineers and engineering geologists can now analyse <u>more realistic sliding surfaces</u>, and hence make more accurate estimates of the risk of stability failure.

INTRODUCTION

To accomplish a slope stability analysis, an engineer or geologist must have a thorough understanding of the mechanics of the failure mechanisms involved. He must be able to reasonably ascertain the insitu shearing stress and resistance of the materials involved, and if water is present, he must be able to define the flow regime and the water's relationship to strength. Knowing these, he can formulate a model which can be used to make a prediction as to whether a slope will perform safely or not.

The primary cause of motion is a component of gravity acting on the sliding mass in the direction of movement along a failure surface. Other forces may also act. Seepage forces, which result from energy lost by ground water flowing through permeable earth, can be and often are significant. In regions of seismic activity, earthquake displacements can impart significant forces which can initiate mass movement in an earth slope.

These forces produce shear stresses within the slope. Unless the strength of the earthen material is sufficient along any possible shear surface, a movement of mass down slope will occur. Except for a few special cases, the position and shape of the surface most likely to fail or most likely to approach failure is not obvious by examination of slope profiles. Therefore an analysis using limiting equilibrium usually requires that a number of trial surfaces be evaluated in order to determine the position and shape of the most likely path of failure.

The method of slices, particularly applicable to heterogeneous soil systems, requires a great amount of computation per trial surface, regardless of the assumptions made for analyses. If performed by hand, time in general will allow only a few trial surfaces to be analyzed economically. However, with the aid of a programmed computer, hundreds of trial surfaces can be analyzed quickly and economically.

To this end, the computer program, STABL, has been developed for performing limiting equilibrium analysis by the method of slices for general slope stability problems. The program can handle slope profiles having multiple slope ground surfaces. Any arrangement of subsurface soil types having different soil properties can be specified. Pore pressures may be related to a steady state flow domain, related to the overburden, or specified within zones. Surcharge boundary loads and psuedo-static earthquake loading are also considered.

The main purpose of this paper is to describe in detail the three new trial shear surface generators that have been developed to search for the more critical sliding either of circular shape, of general irregular shape or of a specified character such as sliding blocks.

SEARCHING TECHNIQUES

In certain cases the more critical sliding surface can be intuitively selected. For example there may be a layer or seam of low strength, through which the critical surface must obviously pass. However, the position and orientation of the critical shear surface within a weak layer may not be obvious unless the layer is thin. Even then, it may be neither obvious where the critical shear surface enters and exits the layer, nor what path it might take beyond these points. Quite a few trial shear surfaces may have to be analysed before the critical surface is located.

To reduce the amount of time required to prepare data defining a large number of trial shear surfaces, three trial shear surface generators have been developed for use with program STABL. The first, named CIRCLE, generates surfaces of circular shape. This generator is good for slopes comprised of roughly homogeneous and isotropic material. The second, named BLOCK, generates trial failure surfaces of a sliding block character, and is most useful when a well defined weak layer exists. Finally, RANDOM generates surfaces of general irregular shape. It is most useful for heterogeneous soil profiles where both the location and shape of the critical surface are uncertain. Each generator uses pseudo-random techniques for generating the surfaces.

RANDOM NUMBERS

Pseudo-random numbers are generated in an exact deterministic process by which a number is generated from its predecessors in a series. The random number generator used for program STABL must be able to generate real numbers having magnitudes within the range between 0 and 1. To be random, the numbers generated must be uniformly distributed within this range. A number generated by such a process is not truly random, but for most practical purposes it may be assumed so.

Most computer systems have a library function for generating random numbers. Control Data Corporation has such a function, named RANF(X), in its 6000 series system library. This function is referenced within program STABL. Relatively simple random number generators can be written for any system, if not readily available (Ralson and Wilf, 1962).

CIRCULAR SURFACE GENERATION

A circular-shaped surface, composed of a series of straight line segments of equal length, is generated from left to right. The length of the line segments is specified. The surface initiates from a point on the ground surface.

To begin, a line segment is projected from the initiation point in a direction chosen between two limits. The direction limits are either specified or automatically calculated by the program and are shown in Fig. 1. In the latter case, the clockwise direction limit, α_2, is -45° with respect to the horizontal, and the counterclockwise direction limit, α_1, is 5° less than the inclination of the ground surface to the right of the initiation point.

The selection of the inclination, θ, at which the initial line segment of a surface projects, is determined randomly. A random number, R, having a value between 0 and 1, is generated and squared. The squared random number and the difference in the inclination of the two direction limits, $\alpha_1 - \alpha_2$, are multiplied. The result is added to the inclination of the clockwise direction limit, α_2, to obtain the angle of inclination of the initial line segment,

$$\theta = \alpha_2 + (\alpha_1 - \alpha_2)\, R^2 \tag{1}$$

Squaring the value of the random number generated, before multiplying it with the absolute value of the difference of inclination of the two direction limits, $\alpha_1 - \alpha_2$, introduces a bias in the random direction selection process. Although selection of any inclination of the initial line segment within the two limits is possible, it is more likely that the initial line segment be located nearer the clockwise limit.

The primary reason for introducing the bias is to obtain a good distribution of completed surfaces. When the initial line segment was given an equal likelihood of being oriented in any direction within the direction limits, poor distributions of completed surfaces were obtained.

After establishing the initial line segment of a trial failure surface, the remaining portion of the circular-shaped surface is randomly located, within some constraints. Although the exact location where the critical

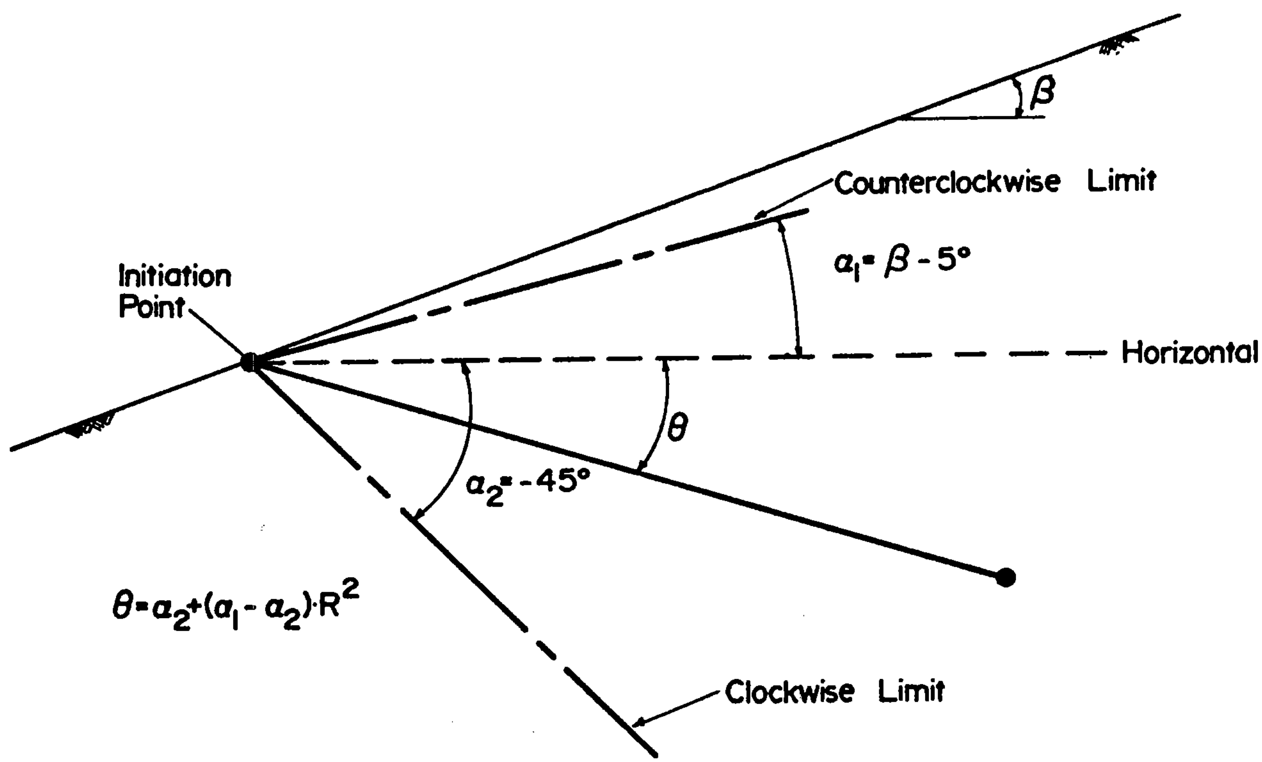


FIGURE 1 - SELECTING THE INITIAL LINE SEGMENT.

shear surface exits the top of the slope is not known, a zone where it probably will exit, can be assumed. Termination limits are established at the ground surface, so that only surfaces of interest are considered.

For a circular-shaped surface composed of straight line segments of equal length, each successive line segment is deflected by a constant angle, $\Delta\theta$, with respect to the line segment preceding it. The two limiting deflection angles, $\Delta\theta_{min}$ and $\Delta\theta_{max}$ (Figure 2), which bound all the circular-shaped surfaces of interest are calculated as shown by Siegel, 1975.

A deflection angle, $\Delta\theta'$, is randomly chosen from within the range defined in a fashion similar to selection of the direction of the initial line segment,

$$\Delta\theta' = (\Delta\theta_{max} - \Delta\theta_{min})\ R^2 \qquad (2)$$

The orientation, θ, of each succeeding line segment is equal to the inclination of the preceding line segment, plus the randomly selected deflection angle,

$$\theta_1 = \theta_{i-1} + \Delta\theta \qquad (3)$$

There are a number of circumstances when the procedure generates unsatisfactory circular-shaped surfaces, and adjustments to the surface generation parameters, α_1, α_2, $\Delta\theta_{max}$, and $\Delta\theta_{min}$, become necessary (Siegel, 1975).

GENERAL IRREGULAR SHEAR SURFACE GENERATION

The inclination of the initial line segment for a general irregular shear surface is selected in the same manner as described for the circular-shaped surface. However, the direction of each subsequent line segment is determined

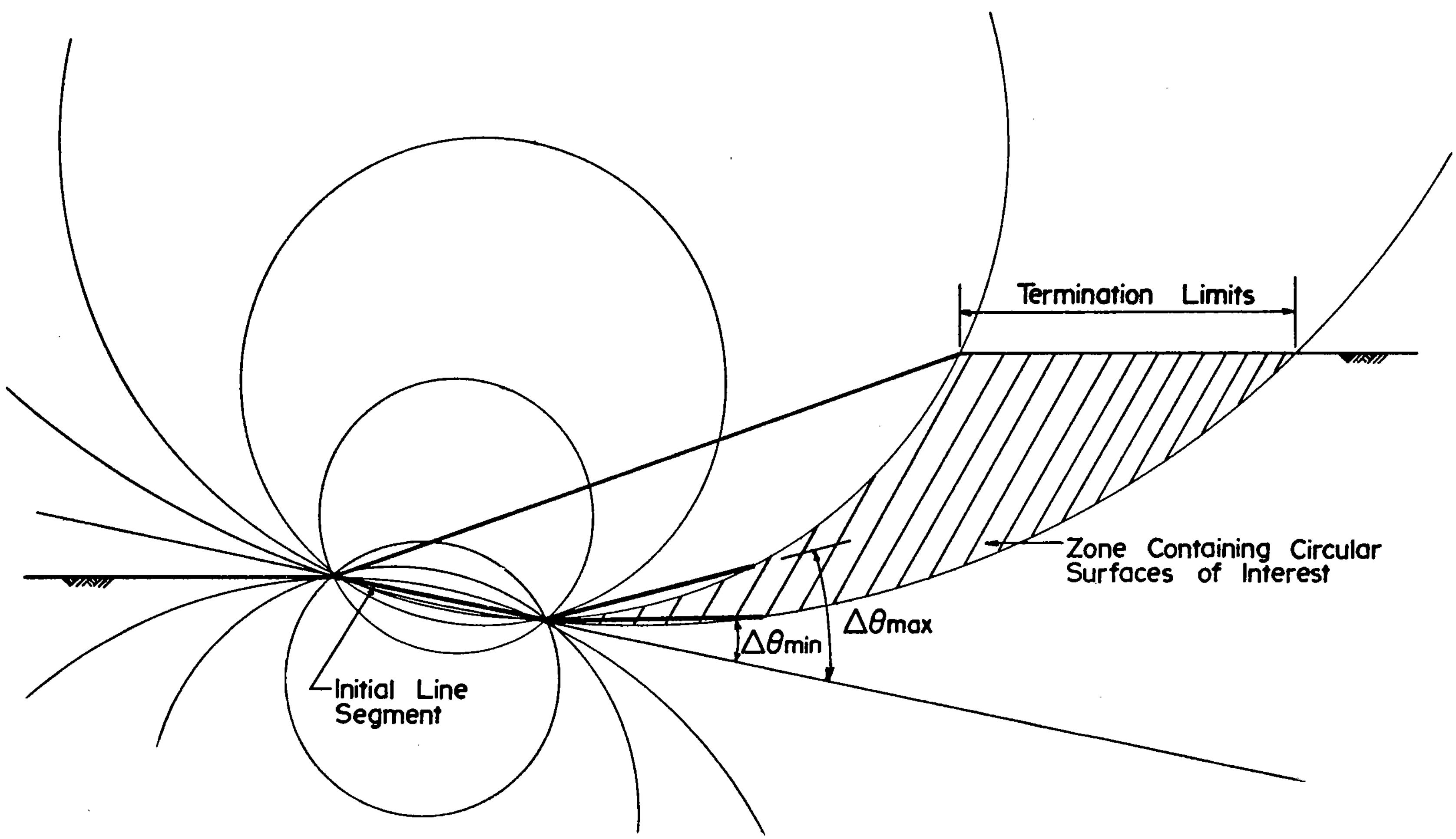


FIGURE 2 -FAMILY OF CIRCLES HAVING INITIAL LINE SEGMENT AS A CHORD.

independently of the line segments preceding it.

The counterclockwise limit for the direction of a line segment, $\Delta\theta_1$, is normally deflected 45^o counterclockwise from the projection of the preceding line segment. This is shown in Figure 3. If for a particular line segment the orientation of the counterclockwise direction limit is greater than 90^o (measured with respect to the horizontal), the inclination of the counterclockwise direction limit, θ_1, is adjusted to 90^o.

The clockwise deflection limit for the direction of a line segment, $\Delta\theta_2$, is randomly selected for each shear surface generated,

$$\Delta\theta_2 = R^2\ 45^o \tag{4}$$

If for a particular line segment the inclination of the clockwise direction limit, θ_2, is less than -45^o, it is set at this value.

The inclination of a line segment is then established by,

$$\theta = \theta_2 + (\theta_1 - \theta_2)\ R^{(1+R)} \tag{5}$$

where the angular extent of permissible directions, $\theta_1 - \theta_2$, is multiplied by a random number raised to a random power between 1 and 2 and added to the inclination of the clockwise direction limit, θ_2.

The above procedure, although somewhat arbitrary, does produce reasonable irregular surfaces of random shape and position. Reverse curvature of a shear surface is possible, but the frequency of occurrence is small. Unless very short line segments are used, "kinkyness" of the resulting generated shear surfaces is not a problem.

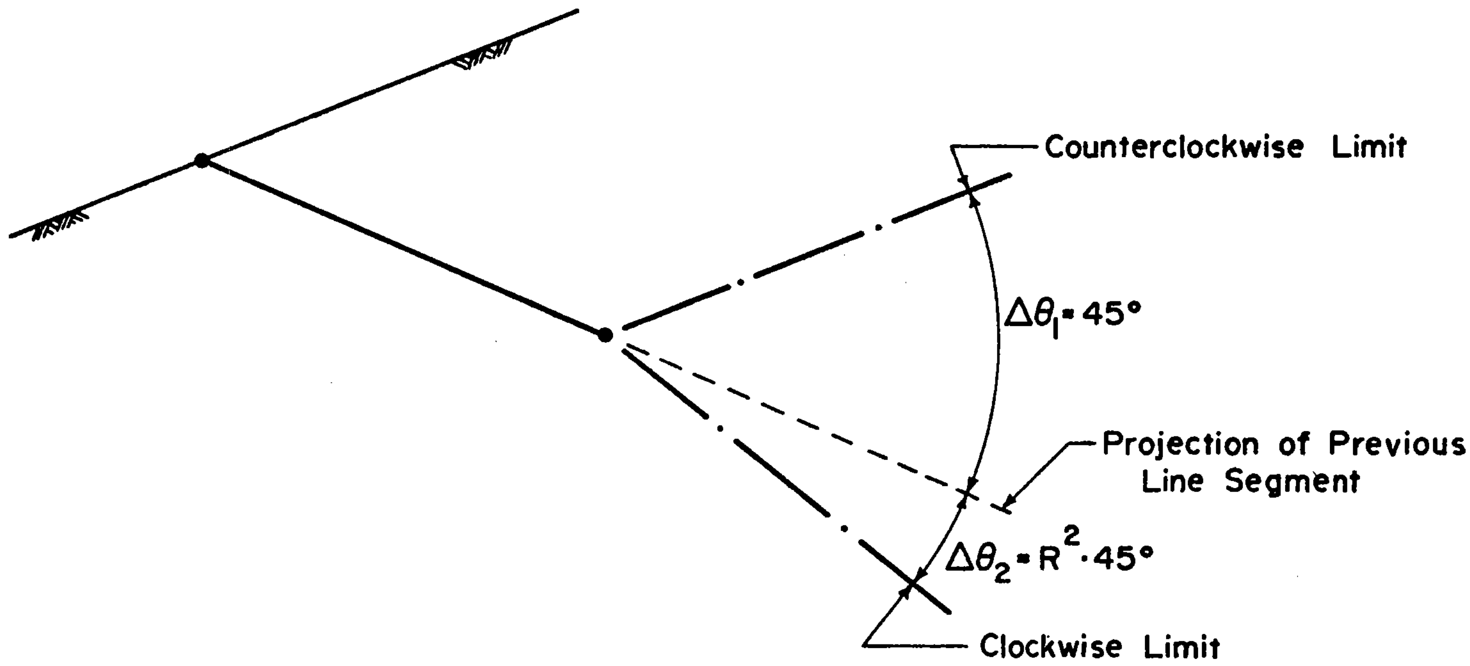


FIGURE 3 - DIRECTION LIMITS FOR SUCCESSIVE LINE SEGMENTS OF IRREGULAR SURFACE.

SEARCHING FOR THE CRITICAL CIRCULAR OR GENERAL SHEAR SURFACES

Trial shear surfaces are generated from a number of initiation points with equal horizontal spacing along the ground surface. For example, Figure 4 shows ten circular-shaped trial shear surfaces generated from each of ten initiation points; a total of one hundred surfaces. As the surfaces are generated, their respective factors of safety are evaluated, and the ten most critical are identified and presented in Figure 5.

The ten most critical surfaces will in most cases form a distinctive critical zone. Occasionally, more than one critical zone can be distinguished. In this case each zone could be considered individually.

The compactness of the critical zone, its location within the zone containing all the surfaces generated, and the magnitude of the range of values for the factors of safety of the ten most critical surfaces, indicate the likelihood that a shear surface exists with a factor of safety significantly lower than any calculated. If the critical zone is narrow, if non-critical surfaces have been generated on both sides of the critical zone, and if the values of the ten most critical surfaces are nearly the same, the defined factor of safety of the most critical surface generated can be assumed, with reasonable confidence. On the other hand, if the critical zone is wide, and/or the more critical surfaces lie along one edge of the zone containing all the surfaces generated, and/or the magnitude of the range of factor of safety values for the critical surfaces is large, the possibility of a shear surface with a significantly smaller value for the factor of safety may require generation of additional surfaces.

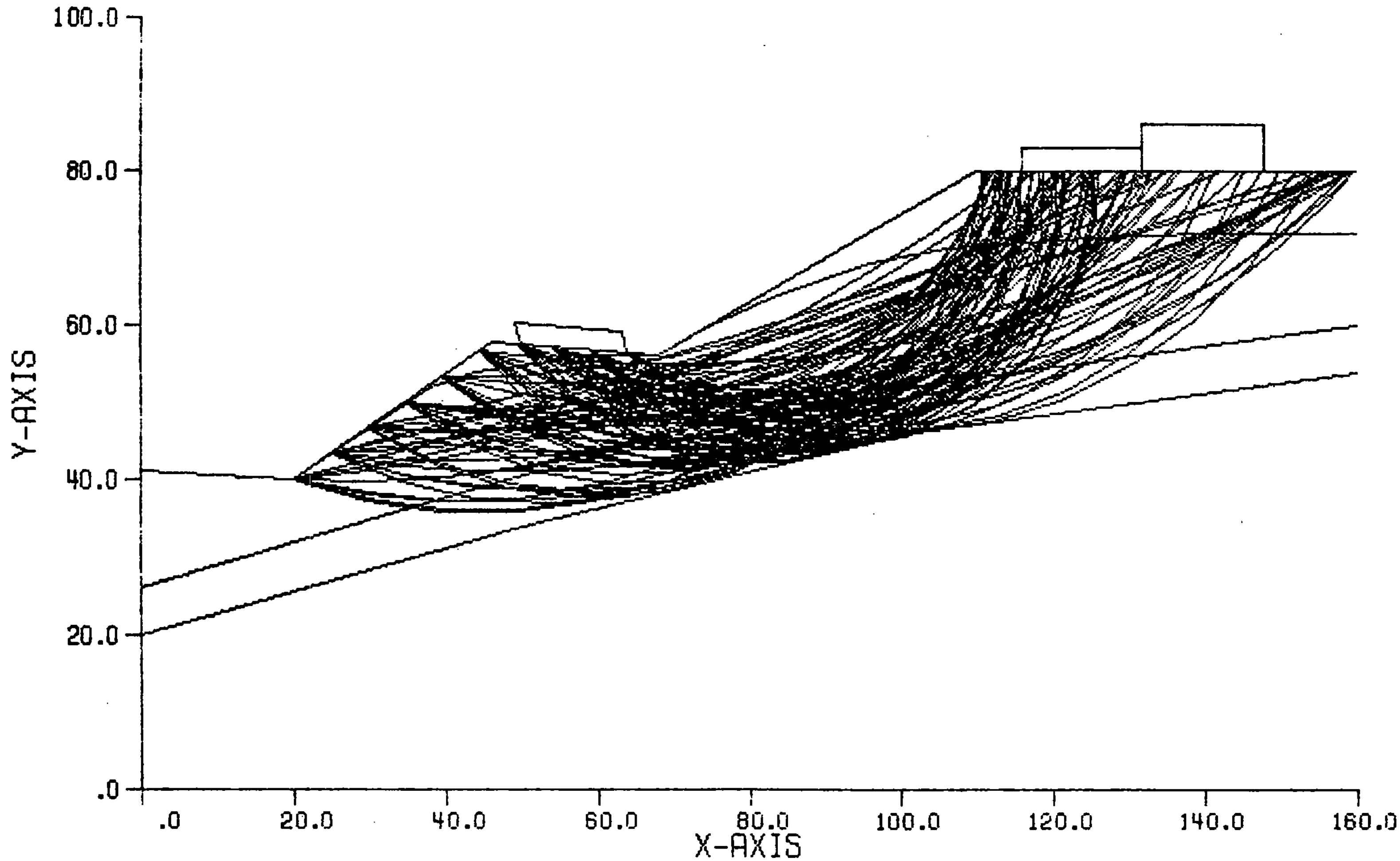


FIGURE 4- EXAMPLE GOULD PLOT WITH 100 CIRCULAR SURFACES.

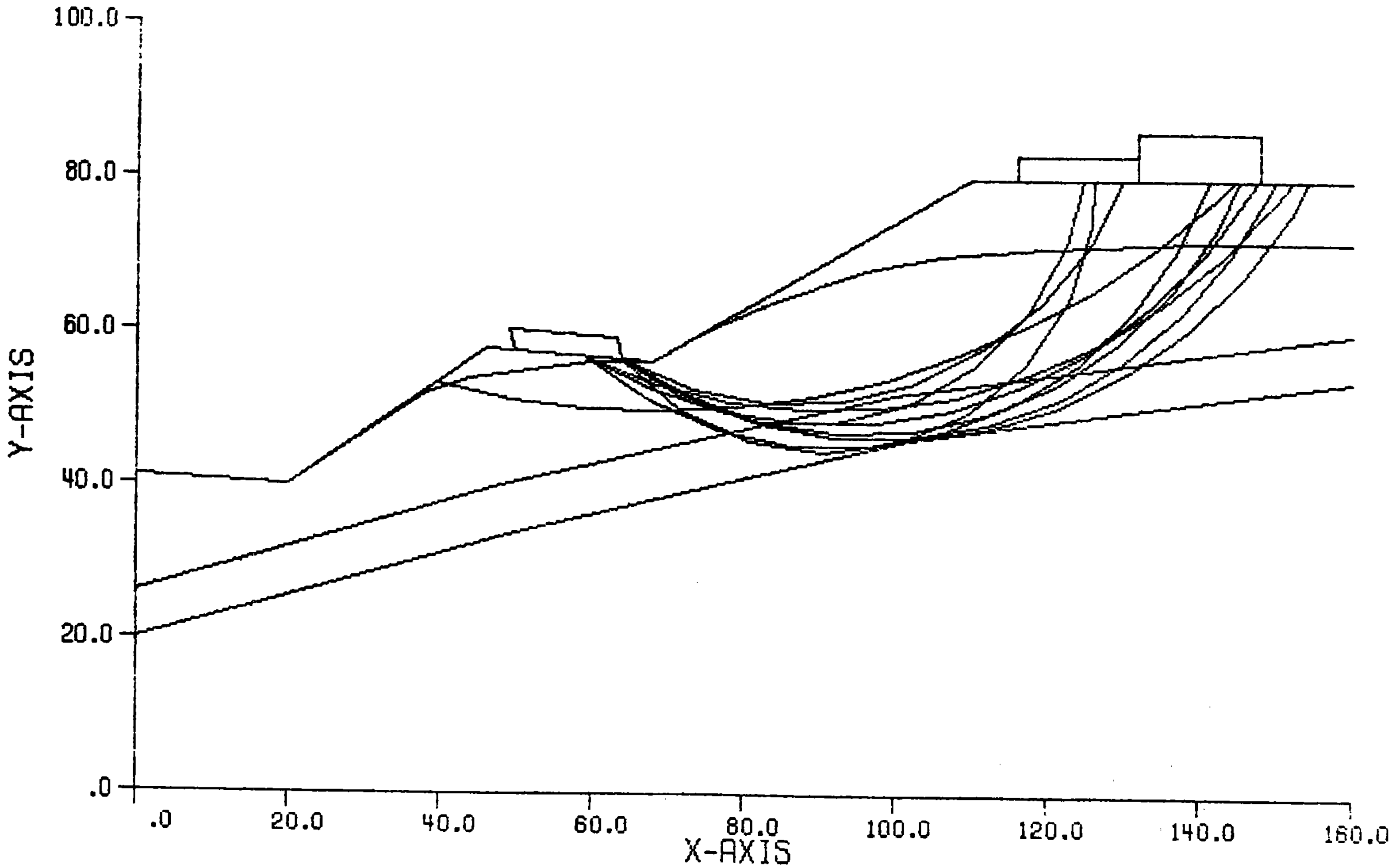


FIGURE 5-TEN MOST CRITICAL SURFACES FROM FIGURE 4.

SLIDING BLOCK SURFACE GENERATION

If a zone of weakness within the mass of a slope is obvious, or if a critical zone has been fairly well defined by previous usage of the circular or irregular surface generators, it is often advantageous to use a third generator.

A series of boxes, a minimum of two, is located along the path of the zone to be investigated. The boxes are parallelograms with vertical sides. They can be specified to bracket the zone to be investigated as illustrated in Figure 6.

A point is randomly selected from within each box. Any point within a particular box has an equal likelihood of being selected. The points are then connected in sequence with straight line segments (Figure 6).

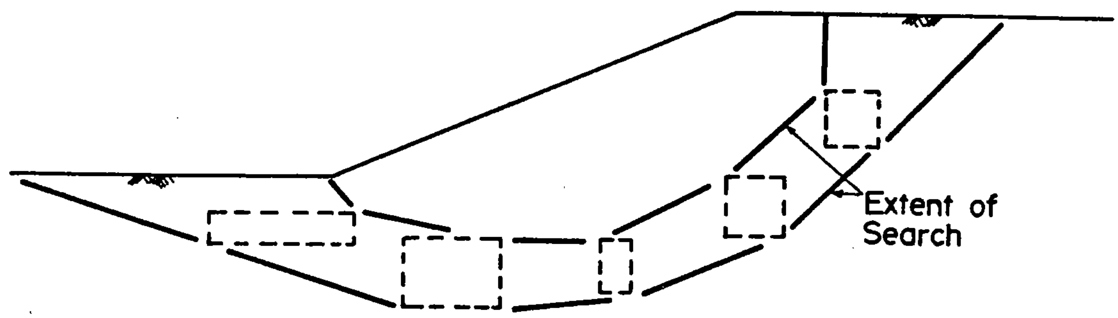
Next, the active and passive portions of the trial shear surface are generated as shown in Figure 7. The active and passive portions are composed of line segments of equal specified length. The inclination of each line segment for the active portion of the shear surface is selected in a biased random fashion between direction limits inclined at 45^o and 90^o,

$$\theta = 45^o + 45^o\ R^2 \qquad (6)$$

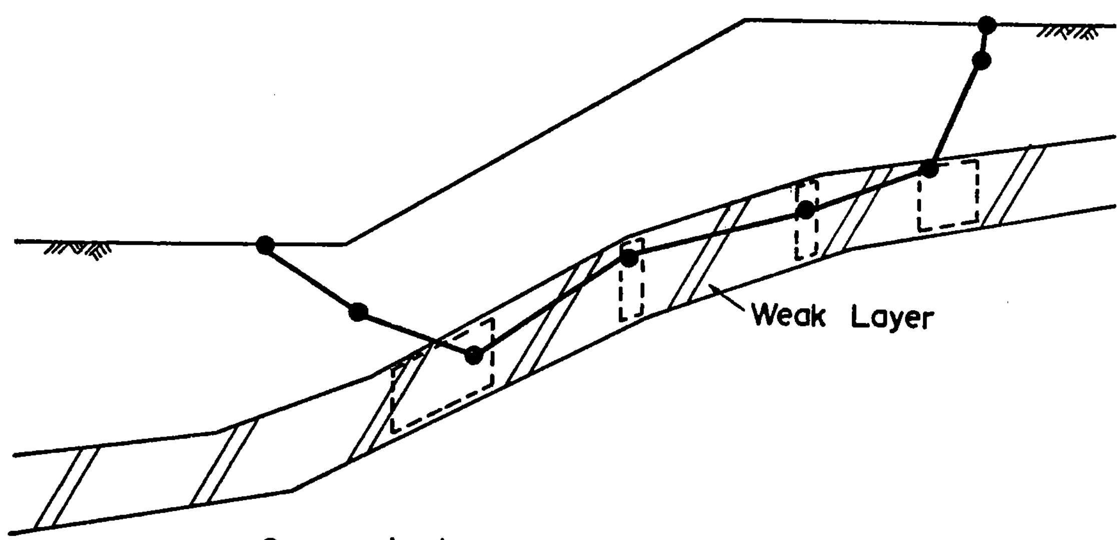
For the passive portion of the shear surface, the inclination of each line segment is selected in a biased random fashion between direction limits inclined at -45^o and 0^o,

$$\theta = -\ 45^o + 45^o\ R^2 \qquad (7)$$

The same judgment, as discussed for critical shear surface searching with the circular and irregular-shaped shear surface generators, can also be applied to this generator, with regard to the probable existence of a significantly more critical shear surface than those generated.



Intensive Search of Critical Zone Previously
Defined by CIRCLE or RANDOM



Search in Irregular Weak Layer

FIGURE 6 - SLIDING BLOCK GENERATOR USING MORE THAN TWO BOXES.

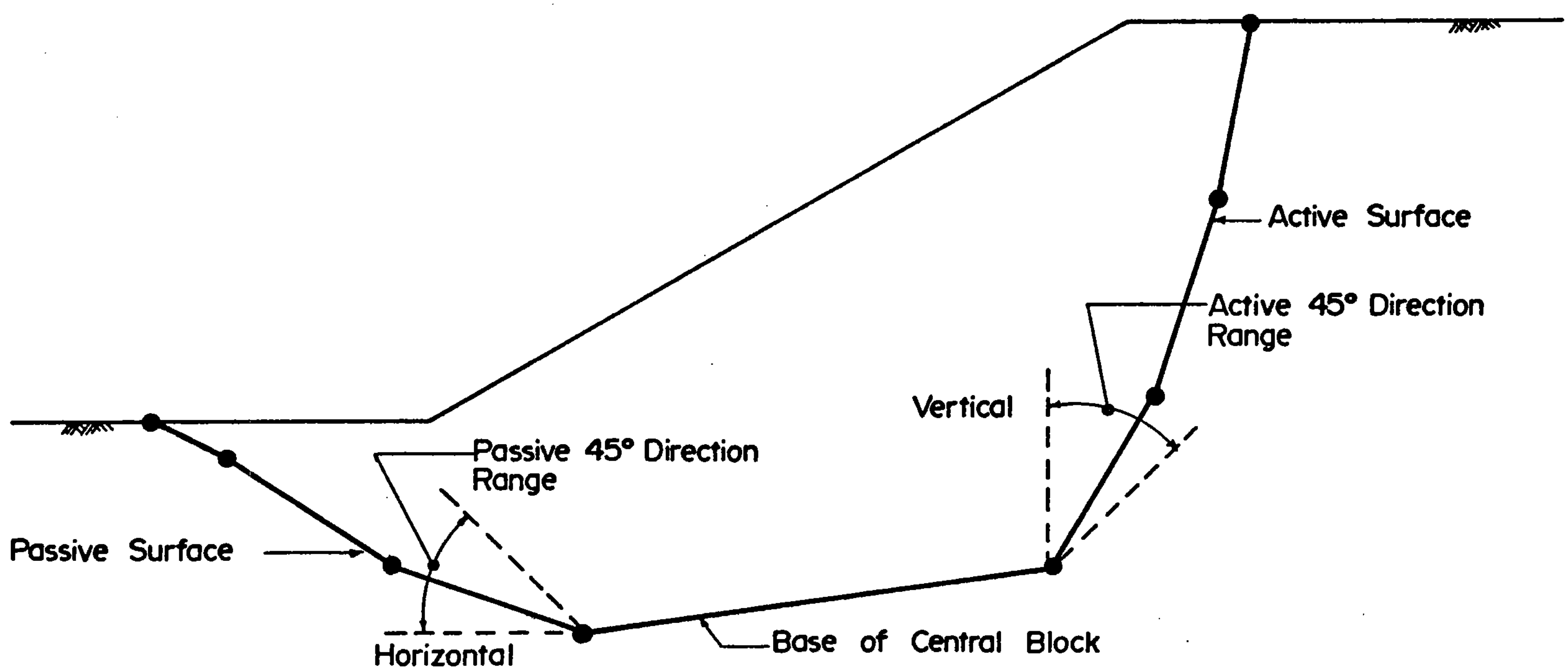


FIGURE 7 - GENERATION OF ACTIVE AND PASSIVE PORTIONS OF SLIDING SURFACE.

SUMMARY

For some time, only searching codes for circular shapes have been available for slope stability analysis. Program STABL, in use by various agencies since 1975, removes these constraints. Circles, sliding blocks and irregular surfaces can be generated in a random manner. Plotting routines show the shapes and positions of generated surfaces and aid in the definition of those with lower factors of safety.

ACKNOWLEDGMENTS

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GEOTECHNICAL PERSPECTIVE ON SLURRY WALL SYSTEM

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And
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ABSTRACT

Construction of the excavation support system may be considered one of the most expensive items in building the transportation tunnels or stations within a built-up area. Until recently, major decisions in underground works as to the methods of construction, temporary support systems, groundwater control, and protection of near-surface installations during the construction were often left to the contractor's choice. However, with the increased understanding of the behavior of excavation support systems nowadays, it is important that both designers and contractors perceive fully the interplay of design and construction of support systems. The purpose of this paper is to outline various rationales with the emphasis on the construction, design and analysis aspects of one support system, namely the slurry wall system. Design and construction are perhaps the most interdependent and inseparable on slurry wall systems as even minor construction details and workmanship frequently have profound impact upon the behavior of the wall. Accordingly, this paper outlines several construction considerations such as stability of slurry trench and guidewalls, soil-slurry interaction, panel dimension, excavation methods and equipment for varying ground conditions was well as the effect of construction details on the ground movements.

Of the various elements of slurrry wall design, wall and support systems, stability of braced and tie-back walls, earth loads on wall systems, water loads, ground movement and the effects of prestress and system stiffness are considered.

INTRODUCTION

The slurry (diaphragm) walls, unlike the soldier pile-lagging walls or the sheet pile walls, have not only been used as a means of temporary support for excavation but have frequently been incorporated into the permanent structure. The displacements experienced by slurry wall support systems are small compared to those associated with other excavation support systems and therefore do not often warrant costly underpinning or other protection measures for the adjacent structures. Furthermore, the seepage of groundwater into the excavation is considerably reduced, if not practically eliminated with such impermeable walls. This paper presents several geotechnical considerations in the design and the construction of the slurry wall system.

DESIGN CONSIDERATIONS

Wall and Support System

The slurry wall, whether cast-in-place or precast concrete, can be given different configuration for increased stiffness, stability or load-bearing capacity. For instance, while a 14-inch thick concrete wall and a PZ-38 sheeting have approximately the same stiffness (EI) of 5.67×10^4 kip-ft^2/ft, the stiffness of slurry wall could be significantly raised by increasing the thickness of the wall, by using a key-patterned wall configuration or by incorporating a soldier pile tremie concrete (SPTC) wall in the design. The soldier piles in SPTC walls or portions of the slurry wall are often extended to a considerable depth below the bottom of the excavation in soft or loose soils to support the expected vertical loads. Transverse walls may be cast below the excavation bottom to increase the safety against bottom heave by providing a larger wall surface for the clay to adhere to and a relatively rigid bracing system at the base as well.

Both struts and tie-backs constitute the support element for the wall. Steel members of either wide-flanged or tubular sections are frequently used in cross-lot strutting in most soils. The use of tie-back systems with grouted prestressed rods and cables is limited to those soils capable of developing adequate anchorage.

Stability

In a braced excavation, external stability is primarily related to the bottom stability. Instability such as boiling under adverse seepage conditions or bottom heave could result due to the lack of sufficient embedment of the diaphragm wall. While boiling can occur in permeable soils only, the mobilization of limited passive resistance to balance the active thrust may cause bottom heave and the accompanied inward wall movement. Furthermore, the soil adjacent to an excavation acting as a surcharge may cause bottom heave due to a bearing capacity failure in soft clayey soils. Though recognizing the fact that bottom instability can also occur with tie-back walls, several other modes of possible failures in such walls are caused by deep-seated rotational movements behind the anchors, anchor pullout or inadequate bearing beneath the wall. Not only should adequate tie-back capacity be assured but the anchor capacity must be developed far enough away from the wall in order that stability of the soil-wall system could be maintained. Inclined tie-backs are likely to increase vertical loads in diaphragm walls which are supported by end bearing. Accordingly, the wall must be found on competent materials to assure that a bearing-capacity failure can not occur. In a case where the slurry wall is socked into the rock, the underlying rock must be stabilized during the construction against the potential rock-wedge fallouts leading to the total loss of bearing and even collapse of the wall system.

Earth Pressure

Earth pressure on semi-rigid diaphragm walls, like other flexible walls such as sheet pile or soldier pile and lagging, are primarily influenced by the type of the soil behind the wall, the prestress load, the stiffness of support system and the interaction between the support system and the surrounding soil. During the initial stage of excavation, an internal strut near the top, if properly installed and wedged with or without preloading, would create a reaction restraining further inward movement at the strut level. Internal bracings, when preloaded, are generally prestressed lightly in order to remove mainly elastic compression in the braces and the existing slacks, if any, in the support system and thereby increase somewhat the stiffness of the system. Subsequent stages of excavation and bracing would invariably result in progressive inward movement below the previously installed struts. The typical inward movement roughly corresponds to translation, flexural bulging, and rotation around a point near the first level struts and therefore resembles the arching active condition. The wall is most likely subjected to parabolic loading conditions. The total thrust against braces derived from these parabolic loading diagrams are considerably higher than the active thrust computed with Rankine or Coulomb theories. Accordingly, the resultant active thrust based on the conventional earth pressure theories is increased and distributed in a manner consistent with the field observations. Of the several semi-empirical loading diagrams (Lambe, 1970), the maximum apparent earth pressure envelopes (Fig. 1) as proposed by Terzaghi and Peck (Terzaghi and Peck, 1967; Peck, 1969) are most commonly used by designers as an expedient for calculating the values of strut loads

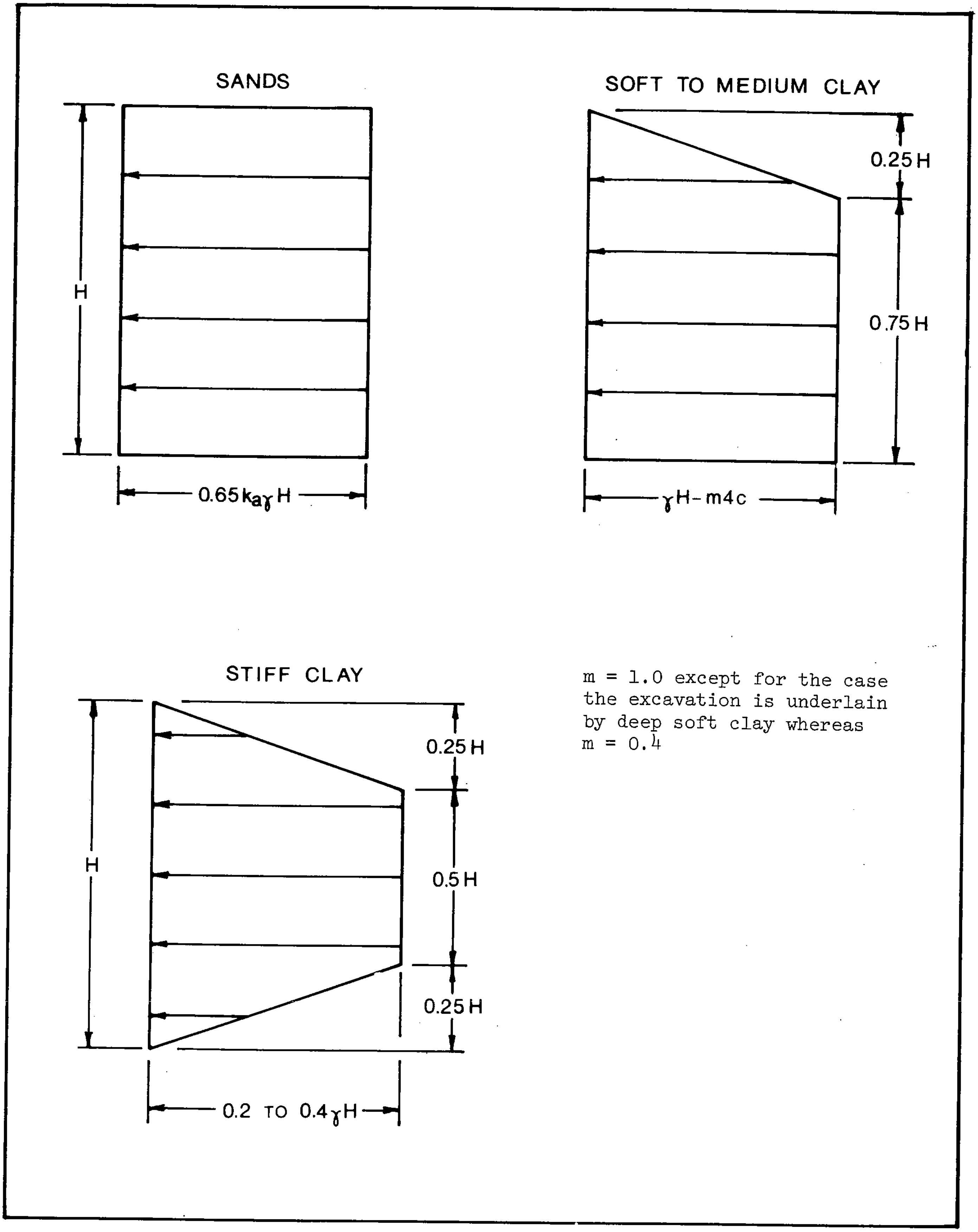

Figure 1. Earth pressures envelopes for design of struts (after Peck, 1969)

with some modifications in the case of cofferdam. A comparison of total thrusts derived from the Terzaghi-Peck pressure envelopes and those based upon Rankine theory shows that the former method yields a load 30% higher in sand and 75% higher in soft to medium clays.

In the case of soft clays extending to a great depth below the cut, where the stability number, N (defined as the ratio of the existing overburden pressure, γH, at the bottom of the cut to the undrained shear strength), is on the order of 6, the predicted earth pressures are considerably increased. The increase in earth pressures is largely attributed to the increased depth of soft clay as well as the extent of the surface of sliding. Consequently, with flexible support, relatively large wall deflection would occur below the excavation whereas earth loads are transferred by arching to the stiffer strutted part of the support system. For excavation in stiff clays with a stability number less than 3 or 4, the theories of plastic equilibrium are not particularly applicable and the earth pressures of trapezoidal distribution are determined empirically based on the field measurements.

Design earth loadings for cofferdams with diaphragm walls are ordinarily developed after the Terzaghi and Peck trapezoids with appropriate modifications to reflect the system stiffness and the extent to which movements are to be restricted. Comprehensive measurements made on a number of instrumented braced diaphragm walls have also shown the trapezoidal distribution to be a reasonable basis for their design (Lambe, 1972; Gould, 1970).

Diaphragm walls supported externally by tie-backs would likely undergo a pattern of movements characteristically different from that associated with braced walls and accordingly, the distribution of lateral earth pressures may be different. Compared to struts, the cables or tie-rods are flexible as only a small area of high strength steel would normally develop the necessary anchor capacity. As the excavation progresses, flexible tie-backs with little or no prestress would likely undergo large displacements. The wall would translate laterally and rotate outward around a point near the bottom of the wall so that the deformation decreases from the top to the bottom of the excavation. Because of system stiffness, the lateral movements may be reduced somewhat so earth pressures between active and at-rest are developed against the wall. The general absence of arching would result in a triangular distribution of such pressures. However, tie-backs are usually prestressed to relatively high levels and are locked off at loads in excess of those computed for struts. Clough and Tsui (1974) has recently indicated that the earth pressure on the wall under such prestressing will reflect the prestress diagram used in the calculation of the prestress loads. Although the movements of the flexible tie-back supports would result in the triangularization of the calculated earth pressures, it is generally conservative to assume the earth pressures are the same as the prestress pressure diagram (Anderson, 1970; Larson et al., 1972).

Groundwater

Groundwater can affect the loading as well as the movement of the diaphragm wall. Static water acts upon the wall if it extends to a impermeable stratum below the base of the cut and therefore provides a complete cut off. In the absence of such a cut off, the downward seepage would develop in permeable soils leading to a reduction in water loading and accompanied increase in the earth pressures and to the settlement of soil surrounding the excavation. The upward seepage beneath the base of the cut, on the other hand, can not only reduce the passive resistance but carry soil into the excavation as well causing further ground movements.

Ground Movement

Trench excavation for diaphragm walls as well as the excavation of the soil between such walls would invariably release the total stress along the sides and the bottom of the cut. The reduction of load would result in not only the inward movement of the soil along the sides but also the upward movement of the soil below the excavation level. Consequently, the ground adjacent to the excavation would settle and display lateral displacement. While the amount and the distribution of movements would primarily depend upon the characteristics of the surrounding soils, such factors as the excavation dimensions, wall and system stiffnesses, prestress loads, surcharge loads from the nearby structures and the construction sequence are likely to exert significant influence on the ground movements.

For excavation in permeable sand and silt, the maximum movements are generally small if the goundwater level is below the bottom of the cut or otherwise completely controlled. While the magnitude of typical movements is on the order of 0.1 to 0.2 percent of the depth of the cut, failure to adequately control the groundwater may lead to the loss of passive resistance below the bottom of the excavation and the accompanied large inward movements. In soft to medium clays, ground would undergo relatively large movements on the order of 0.1 to 0.4 percent of the excavation depth although larger movements in excess of one percent were also reported in a few cases (Cunningham and Fernandez, 1972; Clough, 1975). With increase in undrained shear strength and stiffness, ground movements due to excavation decrease. Stiff to hard clays generally experienced movements less than 0.35 percent of the excavation depth (Goldberg, 1976).

Ground movement surrounding an excavation is influenced by the stiffness of the support system and not by the wall stiffness alone. Since movement occurring below the excavation level is controlled by the combined resistance provided by the embedded portion of the wall and the soil below the bottom of the excavation, the wall stiffness against bending may be of particular significance. In soft or loose soils, the difference between active pressures and the maximum passive pressures which can be developed must be sustained by the wall if the large inelastic movements are to be restricted. While the conventional flexible sheetings will likely be overstressed with the accompaning large lateral movements prior to the development of the necessary passive resistance, a slurry wall with sufficient stiffness may be designed under such conditions to reduce considerably the ground movements (D'Appolonia, 1971; Kuesel, 1967; Lambe, 1972). Effective reduction of ground movements above the excavation level would involve, on the other hand, the elements of the support system such as tie-backs, walls, struts and the connections between the various components. For instance, reduction in ground movements by 50 percent in stiff clays were reported when the stiffnesses of the wall and the tie-backs are increased respectively by a factor of 32 and 10 (Fig. 2). Poor connections between support elements could, to some extent, negate the beneficial effects of the diaphragm wall.

Wall and soil movements above the base of the cut are likely to decrease as struts or tie-backs are prestressed. Although the movements associated with tie-backs where a higher prestressing load is used are reduced with increasing resultant prestress load, the use of such loads far in excess of those proposed by Terzaghi and Peck (1967) for strutted excavations may not prove to be economically justified (Clough and Tsui, 1974). Tie-backs prestressed according to a triangular load diagram are subjected to increasing prestress loads at the lower levels in order to restrain movement of a relatively large mass of soil.

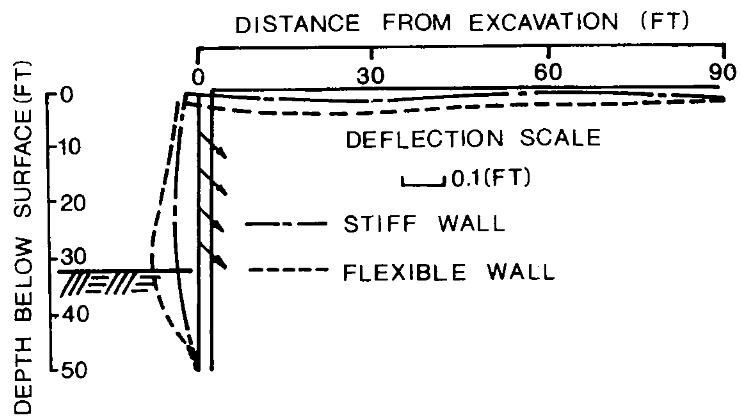

Figure 2A. Effect of wall rigidity on movements

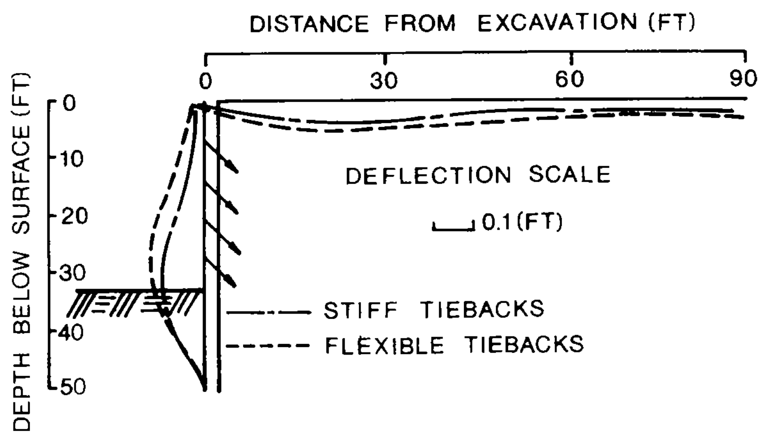

Figure 2B. Effect of tie-back stiffness on movements (after Clough and Tusi, 1974)

However, tie-backs prestressed in such manner would not only tend to redistribute loads to an apparent trapezoidal diagram but create difficulty in maintaining high prestresses in the tie-backs at the lower levels of the excavation. Trapezoidal distributions of design prestress load would be likely to result in less overall movement and considerably reduced movement near the top of the excavation.

Surcharge loads from the adjacent surface and near-surface installations, construction equipment or other sources would contribute to the increase in the lateral pressures and ground movement resulting therefrom. Structures adjacent to an excavation often dictate various details of construction. For example, the existence of a shallow foundation adjacent to the diaphragm wall may necessitate the use of higher prestress loads in the upper levels of tie-backs in order to minimize movement of the top portion of the wall.

CONSTRUCTION CONSIDERATIONS

Stability of Slurry Trench

The primary function of a slurry is to stabilize the trench walls against sliding. The thixotropic slurry must exert hydrostatic pressure exceeding the active earth and water pressures. An increase either in the specific gravity of the slurry or in the fluid level in the trench would enhance the stabilizing force. The driving force causing the slide could be reduced by excavating narrow trenches in short lengths or by lowering temporarily the groundwater. The development of the shearing stresses along a horizontal plane through the sliding wedge would, to some extent, redistribute the lateral pressures and hence increase the stability of the lower portion of the wall. These horizontal shearing resistances are generated at the elevation where the outward slurry pressure equals the inward earth and water counterpressures. Since the trenching is usually excavated in short lengths compared to its depth, arching would develop along the trench walls. The failure of the trench requires the shearing of its end portions, therefore the sliding surface most likely is to be a three-dimensional curved surface. For stability in granular soils, a wedge having the shape of a segment of a vertical parabolic cylinder sliding along an inclined plane is analyzed (Piaskowski and Kowalewski, 1965). For deep trenches in clays, it is assumed that the earth pressures along the perimeter of the two end portions are the same as for cylindrical or square cuts (Meyerhof, 1972). Several researchers (Meyerhof, 1972; Piaskowski and Kowalewski, 1965) have considered the effect of arching in developing simple expressions or nomograms for determining active earth pressures.

Several other factors such as formation of impermeable membrane along the trench wall, penetration of the slurry into the granular soils, shearing resistance of the slurry, and electro-osmotic pressure would also contribute to the stability of the trench. The penetration of the slurry into granular soils increases their cohesion and therefore their shearing resistance. The gelled slurry in "mudcake" or "filter cake" prevents soil particles from collapsing and thus maintains the integrity of the soil structure along the face of the trench. As the slurry penetrates through the soil pores, a layer of solids builds up along the face of the trench forming an impermeable membrane which prevents further loss of the fluid into the surrounding soil and maintains the differential pressure at the soil-slurry interface. However, considerable loss of water from the slurry in highly permeable soils or a large infiltration of groundwater from the adjacent permeable soil into the slurry may cause stability problems. The electrical potential between soil and slurry would cause negatively charged clay particles in the slurry to migrate and deposit on the exposed face contributing to the formation of the impermeable membrane. The inherent shearing strength of the slurry derived from gelation or viscosity would resist, to some extent, inward pressures on the walls.

Excavation Methods For Varying Ground Conditions

Two basic systems of slurry trench excavation are commonly used depending upon existing ground conditions. First, hydraulically or mechanically operated clamshell grabs either suspended by cable or controlled with kelly bars, are used for excavation in soft to medium soils. The buckets are equipped with a square end or a rounded end to allow effective excavation. Of the various other types of equipment, the use of a scooping bucket or a reverse circulation drill is much less widespread. Secondly, heavy chisels, churn drills or reverse circulation systems with percussion tools or rotary bits are employed in excavating hard to very hard soils or in grounds containing boulders or other obstructions. A hydraulically expandable chisel is used to remove soils between two poured circular-end elements. A chisel used in conjunction with a clamshell or a percussion rig can provide a shallow socket in soft to medium rock while a rotary or a churn drill may be required to remove a large mass of hard to very hard rock for deep socketting.

Panel Length

Panel length is dictated by trench stability requirements, the type of and size of available equipment or the movement toleration of the adjacent buildings. Ordinarily panels are 6 to 30 feet long. Short panels are used in loose unstable soils or in ground subjected to high surcharge loads from the adjacent structures while long panels may be used in relatively stable ground. When the slurry wall is used not only as a temporary support but also for the protection of the adjacent buildings, it is highly desirable that trench excavation should not expose completely the foundation. Field test (ICOS) showed that when the foundation was not fully exposed during the trench excavation, the resulting settlement was significantly smaller than when it was completely exposed. Needless to say, the panel length must also be compatible with the bucket length and coordinated with the structural framing or bracing schemes.

Effect of Construction Details On The Ground Movement

Both construction details and contractor's performance affect the ground movements. The actual sequence of excavating soil and installing struts or tie-backs can influence the movements of the surrounding soil. Deflection of the diaphragm wall would increase as the spacing between supports is increased. If the struts are properly installed, there should be little, if any, movement occurring at the strut levels. However, poor wedging and prestressing of struts would result in inward movements at the strut levels and the accompanied deflection of the wall. Over-excavation of in-situ materials more than a few feet below the support level prior to installing the struts has a significant affect upon the lateral movement of the wall (Clough and Tsui, 1974; Lambe, 1970, 1972). Ground movements are likely to increase with the time an excavation remains exposed prior to the installation of the struts. Therefore, over-excavation should be held to a practical minimum. A construction sequence employing excavation and installation of supports in alternate bays could minimize ground movement. The utilization of temporary berm against the wall could have a beneficial affect on the lateral movement of the wall. Drilling holes on flowing or raveling grounds for installation of tie-backs may result in loss of ground.

CONCLUSIONS

In summary, several factors such as the stability of the slurry trench and support system, earth and water loads, the affects of prestress and system stiffness, ground movements, excavation methods as well as construction details are objectively analyzed in order that the interplay between the design and the construction of slurry wall system could be thoroughly perceived by the users. It is imperative that the use of these factors in the design and the construction will improve the chances for successful application of this excavation support system.

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