23rd. Annual

Highway Geology Symposium

"Engineering Geology of Unconsolidated Materials"

April 26-28, 1972

Hampton, Virginia

Sponsors:

Va. Hwy. Research Council
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TWENTY-THIRD ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

Proceedings of a Symposium
held at the
Chamberlin Hotel, Fort Monroe, Virginia
April 27-28, 1972

Local Chairman
Michael A. Ozol

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Editor

Dr. A. C. Munyan

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PREFACE

The 23rd annual highway geology symposium was held in the Chamberlin Hotel at Fort Monroe, Virginia on April 27 and 28, 1972.

The field trip on April 27 dealt with the engineering geology of the second Hampton Roads Bridge-tunnel crossing.

The technical sessions, held the following day, were planned to complement the field trip by reiterating the topic of engineering geology of unconsolidated materials; and the papers as a group dealt mainly with these types of materials.

George S. Meadors, Jr. of the Virginia Department of Highways served on the local arrangements committee in the special capacity of being in charge of the organization of the field trip and the preparation of the guide book.

Our speaker at the banquet on April 27 was Norman F. Williams, State Geologist, Arkansas, and the local committee thanks him very kindly for the illuminating remarks on his topic "The Noble Highway Geologist".

Dr. Arthur C. Munyan of the Department of Geophysical Sciences, Old Dominion University, Norfolk, Virginia, performed the very basic and necessary task of receiving and reviewing the abstracts of the offered papers and making the selections for presentation at the Symposium. Dr. Munyan also presided at the Technical Sessions on April 28, and later received and reviewed the written submissions for publications in these Proceedings. We very gratefully acknowledge the donation of his time and energy on behalf of the Symposium.

As local chairman of the 1972 program committee I want to express my gratitude to those other committee members who helped to do all the necessary things that had to be done, and also for the cooperation extended by the Virginia Highway Research Council.

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EMBANKMENT FAILURES ON THE TIJUANA--ENSENADA TURNPIKE IN THE LOWER CALIFORNIA PENINSULA, MEXICO

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INTRODUCTION

The rapidly growing volume of traffic on the then existing highway, the rising numbers of accidents as well as the attraction of the coastal zone of Lower California (Baja Peninsula) for tourists, were the factors which led the Ministry of Public Works to first consider and finally build a turnpike between the cities of Tijuana and Ensenada in the state of Lower California, Mexico. The construction of the turnpike has solved, on the one hand, the problems of growth and high traffic incidence while on the other, it has helped the development of the area as a tourist attraction, thus contributing to a considerable increase in the amount of foreign exchange and therefore strengthening the economic and social welfare of the country. Photo 1 depicts the route of the turnpike along the western coast of the Baja Peninsula. It has a total distance of 100 kilometers.

Both preliminary and final studies made it clear to authorities that the section of the roadway comprising the southernmost 10 kilometers of the route, positioned next to the Bay of Salsipuedes, was characterized by unstable ground that opened up the distinct possibility of slide failures in the future roadway; the anticipation of failures was due to the presence of existing ancient failures evident in the talus zone which is present in

Photo 1. State of Lower California, Mexico, showing location of turnpike and failure areas.

this area. The decision was made to accept the talus material zone as a calculated risk after a comparative study was made which clarified the advantages and disadvantages of the chosen route over another one farther inland and of greater length, involving greater cost and which, moreover, eliminated the stretches of greatest scenic beauty and thus the whole attraction of the highway.

The purpose of this paper is to show construction and explain design procedures used by contractors and designers for two slide sites (from a total of 12 that had occurred to September 1970), as well as to describe the geology of the northern half of the peninsula and that of the area of the slides.

Geology of the Peninsula

According to geological studies (1) the northern half of the peninsula is generally formed by granite-type intrusive rocks of the Cretaceous. Coastal plains extend on each side and in places the coast is covered by volcanic rocks.

There are sediments upon the surface which vary from coastal and continental Pleistocene to clastic marine rocks of the Miocene, Eocene, Cretaceous and Jurassic. The great delta of the Colorado River is alluvium.

The tectonic aspect of the peninsula is that of a great block which failed longitudinally on both sides and inclined toward the Pacific. Its features are heavily eroded and it is highly accented topographically. Granite batholith occupies 80% of the area in the northern half of the peninsula. There is considerable evidence of volcanic activity. Igneous flows of lava both acidic and basic exist on the surface.

Geology of the Slide Area

In the area of the slide problems there are recognized four distinct geological units. Photo 2 shows the boundaries as they were located in a geo-photo interpretive survey.(2)

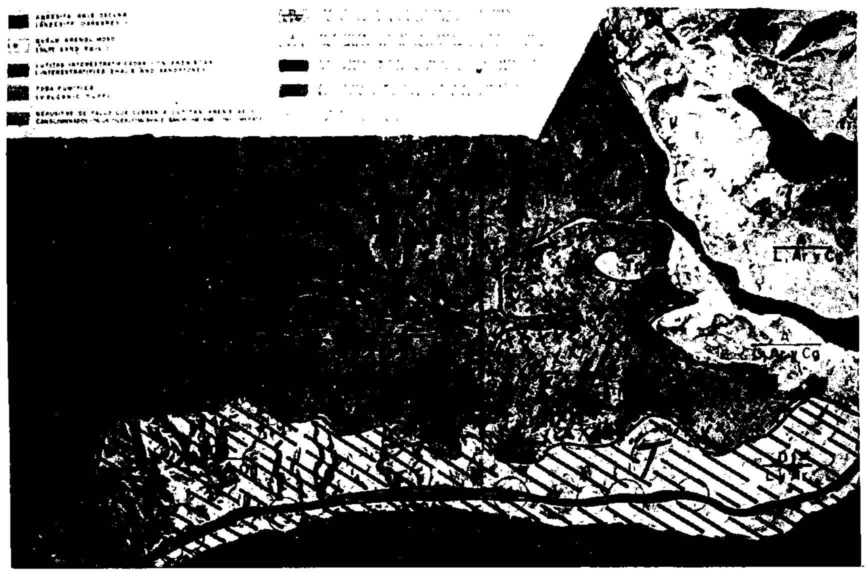

Photo 2. Geo-photo Interpretation of Area Geology.
TP = ash cone, pumice tuff; A = Andesite;
S = sedimentary, shales and sandstones;
T = talus.

Uppermost is a meseta of lava flow of variable thickness which has been identified petrographically as andesitic rock. It probably derives from the ash cone where pumice tuff exists.

The andesite and also an area of fractured material overlie sandstone and shale and the sandy silts which cover them. The fourth unit, which borders the coast, overlies the sedimentary marine rocks and originated by the weathering of the andesitic flow which originally covered the narrow coastal strip. Signs of ancient slides are clearly visible in this unit; the majority coincide with the roadway failures.

The talus material is largely angular fragments of rock, some as large as 2 meters. It may be bound in loose sandy silts and occasionally has no binder at all. Thickness of the unit may be as much as 37 meters. It is highly permeable. Rain water easily infiltrates to the underlying sedimentary rocks, and therein lies the problem: take a normally unstable, unconsolidated mass of highly permeable material, disturb it further with construction procedures, add to it the factor of sea erosion, which relentlessly wears away at the foot of the embankment, and you may expect problems.

Investigation

The investigation of a site was conducted with the purpose of determining the following facts:

A. Localization of the slide mass.

B. Magnitude and direction of surface displacement.

C. Geological characteristics.

D. Form and depth of the slide surface.

E. Ground water conditions.

Construction

Once the slide area has been studied and corrective measures are designed the field construction begins. Procedures observed at two of the slides were as follows:

First they repair the roadway to allow normal traffic movement. They next remove the slide mass above the road grade to stabilize the embankment. An amphitheater is formed where the slide mass has been removed. The ground line is dipped toward the sea for good drainage. A drain gutter is built around the perimeter of the amphitheater to keep water from entering into the talus material, and finally they shoot oil onto the surface of the amphitheater to make it impenetrable to moisture. A guard fence is put up at the front of the amphitheater to keep out traffic. Photo 3 shows a completed amphitheater where the above notations have been carried out.

Photo 3. Completed Amphitheater.

Conclusions

The studies undertaken to 1970 by Mexican Highway authorities lead to the following conclusions:

A. The slides occur at the sites of ancient failures covered by talus upon which embankments have been constructed.

B. The slide surface develops at the line of contact between the talus and the underlying sedimentary rocks. They occasionally develop where weathered shales are in contact with unweathered material.

C. The velocity of displacements reveals a cyclic law where the maximum movement corresponds to the rainy season, with a lag of 1-2 months, and the minimum corresponding to the dry season. Annual rainfall is 4 to 8 inches, falling December to March.

D. There are two main causes for the activation of the slides:

1. The weight of the embankments which increases the actuating forces, and
2. The action of water which infiltrates the highly permeable talus.

FINALLY AUTHORITIES CAUTION:

The methods employed for the correction of the slides (no two of which are alike) have eliminated displacements of the failing mass, damage to the roadway and thus no interference with the normal flow of traffic and, more importantly, there have been no extra maintenance costs attributable to any of the corrected areas for over four years.

HOWEVER:

Given the conditions of the failures and their present state of development, it must be accepted that any solution will involve a degree of risk dependent, naturally enough, upon the desired level of performance which it is desired to attain.

(Numerous color transparencies were shown during the presentation to show scenery, construction details and regional geology.)

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GEOLOGICAL ENGINEERING INVESTIGATION OF TALUS SLOPES IN LEWISTOWN NARROWS, PENNSYLVANIA

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Abstract

In the Ridge and Valley Province of central Pennsylvania, unconsolidated talus and colluvium cover many of the highway corridors through which construction is necessary, presenting a major problem. In solving these problems, the geological engineer must utilize many of the tools of geotechnics, such as rock mechanics, geophysics, field geology and air photo interpretation. The study reported on outlines a method of approach for talus and scree covered slopes.

The area studied lies south of Lewistown, Pennsylvania, along the Juniata River, in the "Lewistown Narrows". It is a steep, narrow synclinal valley with talus covered slopes. An existing road and railroad located in this valley force proposed roadways to cross areas of extensive talus.

A detailed geotechnical investigation was performed using a seismic refraction survey to delinate the extent of talus. Analyses were completed regarding the slope stability of the proposed construction. Upon completion of this investigation the geological engineer was able to present recommendations to the designer regarding the feasibility of locating the highway in this type of area.

Introduction

In the Valley and Ridge Province of central Pennsylvania unconsolidated deposits of talus and colluvium cover most of the mountain slopes. In this area of long parallel ridges water and wind gaps are utilized for economic reasons in the design and location of highways.

The area studied lies south of Lewistown, Pennsylvania along the Juniata River in an area known as the narrows. It is a steep synclinal valley with talus covered slopes. Due to the existing road, railroad and river with little or no flood plain, it was necessary to study a route location consisting of sidehill construction. This location presented gross stability problems to the geological engineer because of the unconsolidated "talus" covering on the slopes.

It was therefore felt necessary to provide the highway designer with a report delineating the area of major concern to the proposed construction and provide recommendations for solutions and/or alternatives for these problem areas.

A detailed geotechnical investigation was undertaken utilizing the tools of the geological engineer. The basic geology of the area was studied in the field and through a search of existing literature. Aerial photographs were utilized in the preparation of the geologic map. A seismic refraction survey was completed to delineate the extent of talus (scree). Rock mechanics studies and drilling were completed in selected areas to complement these other studies.

This overall study was the redesign of U. S. Routes 22 and 322 along the existing highway. The north side of the river was studied by a consultant for the State in 1968. However, in 1969 a request by the Susquehanna Economic Development Dist. to provide access for a proposed industrial park made it necessary to study additional routes along the south side of the Juniata River (Figure 1).

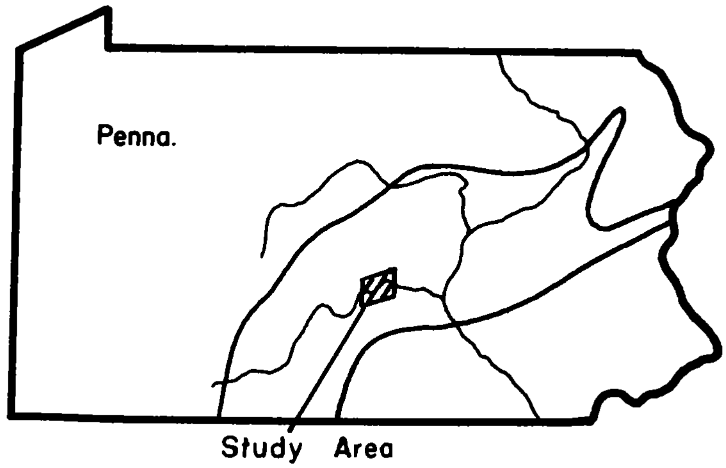
General Geology

The Lewistown Narrows is located in the central part of Pennsylvania, within the Appalachian Mountain Section of the Valley and Ridge Province (Figure 1). The topography of the area is rough in nature, with overall relief of 1500 feet. The Juniata River is the major stream in the area. Its course traverses the regional structure throughout most of the narrows. The minor streams flow perpendicular into the Juniata, forming a trellis drainage pattern. The portion of the valley which is structurally controlled occupies a synclinal basin and is oriented along the strike of the axis of the Lewistown Narrows Syncline.

The major rock types exposed in the area range in age from Upper Ordovician to Middle Silurian and represent a section of several thousands of feet in thickness. Few outcrops are present along the study area, because of the extensive talus fields.

The structure in the area is comprised of en echelon type folds. The regional strike is northeast-southwest. All the folds are plunging, with an average plunge of 5 to 10 degrees.

The most competent bed in the area is the Tuscarora Formation, which does not respond by tight folding but rather by large sym-



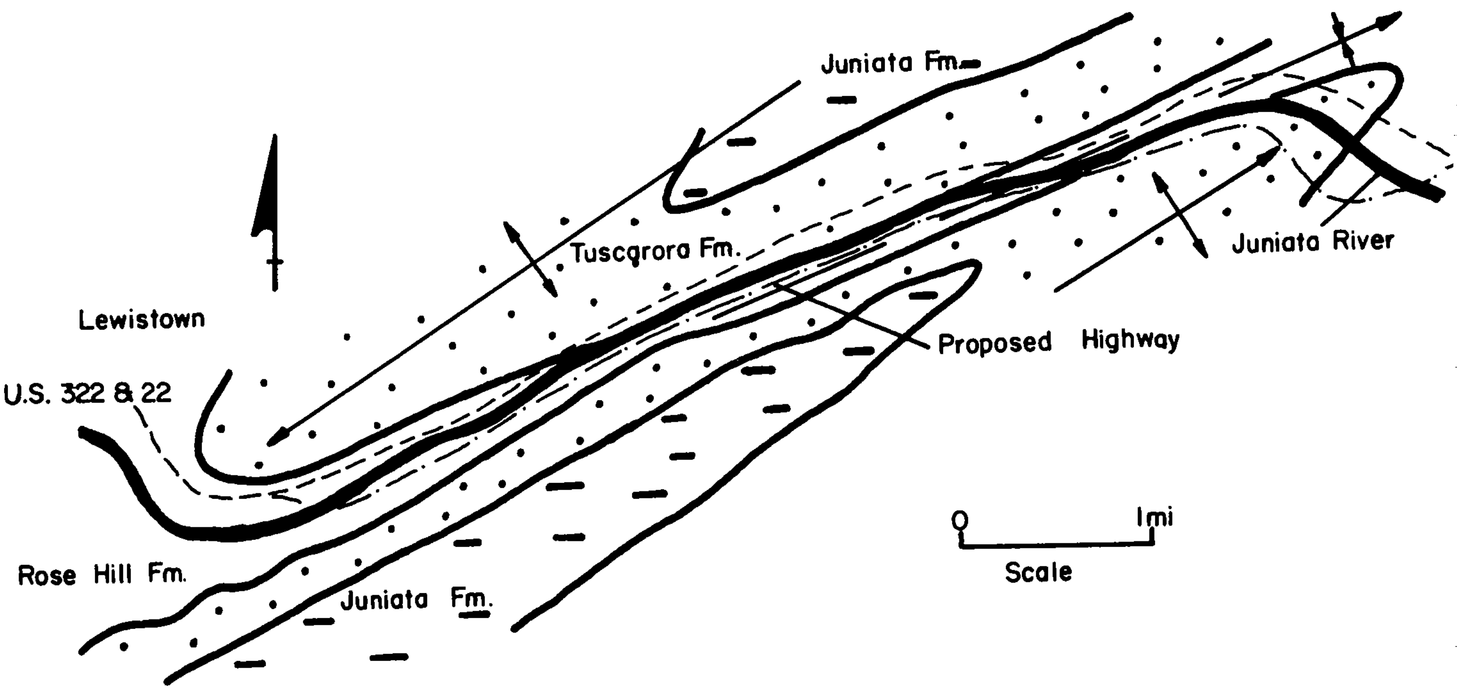


Figure 1. Location map and general geology map showing existing and proposed highways.

metrical folds and small-scale faulting. This formation averages 750' in thickness and is basically a light gray-pink, medium-to-thick-bedded quartzitic sandstone. A majority of the talus blocks are made up of this material. The other formation which is to be considered in this study is the Rose Hill Formation. It represents a unit of intermediate competency. This formation is mostly covered by the talus; it averages between 850 and 900 feet in thickness.

The geologic map was done after the work of Conlin and Hoskins (1962) and U.S.D.A. aerial photographs (scale 1:20,000) to prepare a map on the desired scale.

Field Investigation

This investigation was undertaken during the summer of 1970. The geologic field investigation began with a fairly comprehensive tabulation of the strike and dip of the regional as well as the local systems. This was completed for utilization in the rock mechanics or geomechanical studies to be performed on the intact rock.

As this phase of the work progressed a seismic refraction survey was undertaken to help to delineate the extent and determine the thickness of the talus along this alternate route.

The seismic survey was carried out by personnel of the Bureau during June and July 1970. A twelve trace RS-4 seismograph was used with energy supplied by 40% 1¼:8 special gelatin. A standard spread length of 200 feet (20 foot geophone spacing) was used, and shot into from each end, the center, and 200 foot offsets.

The data was interpreted by using standard critical distance and intercept time techniques after plotting time-distance curves and measuring velocities. The Bureau's desk top computers were used for the calculations. Upon completion of the interpretation the results were compared to the core borings for correlation and validation.

Examples of seismic profile, sections and drill hole ties can be seen in Figure 2. The drill hole ties were in all cases within acceptable ranges, thus validating the survey. The velocity ranges and material correlations are as follows:

1000 - 1500 fps	-	Near-surface colluvium and overburden.
1500 - 3000 fps	-	Overburden; talus to the east, fine colluvium and residual soil to the west.
3000 - 4000 fps	-	Overburden and/or weathered rock, mostly talus to the east, highly weathered rock and residual soil to the west.
4000 - 8000 fps	-	Weathered bedrock. Mostly quartzitic sandstone east and sands and shales to west.
8000 - 14,000 fps	-	Unweathered bedrock. The higher

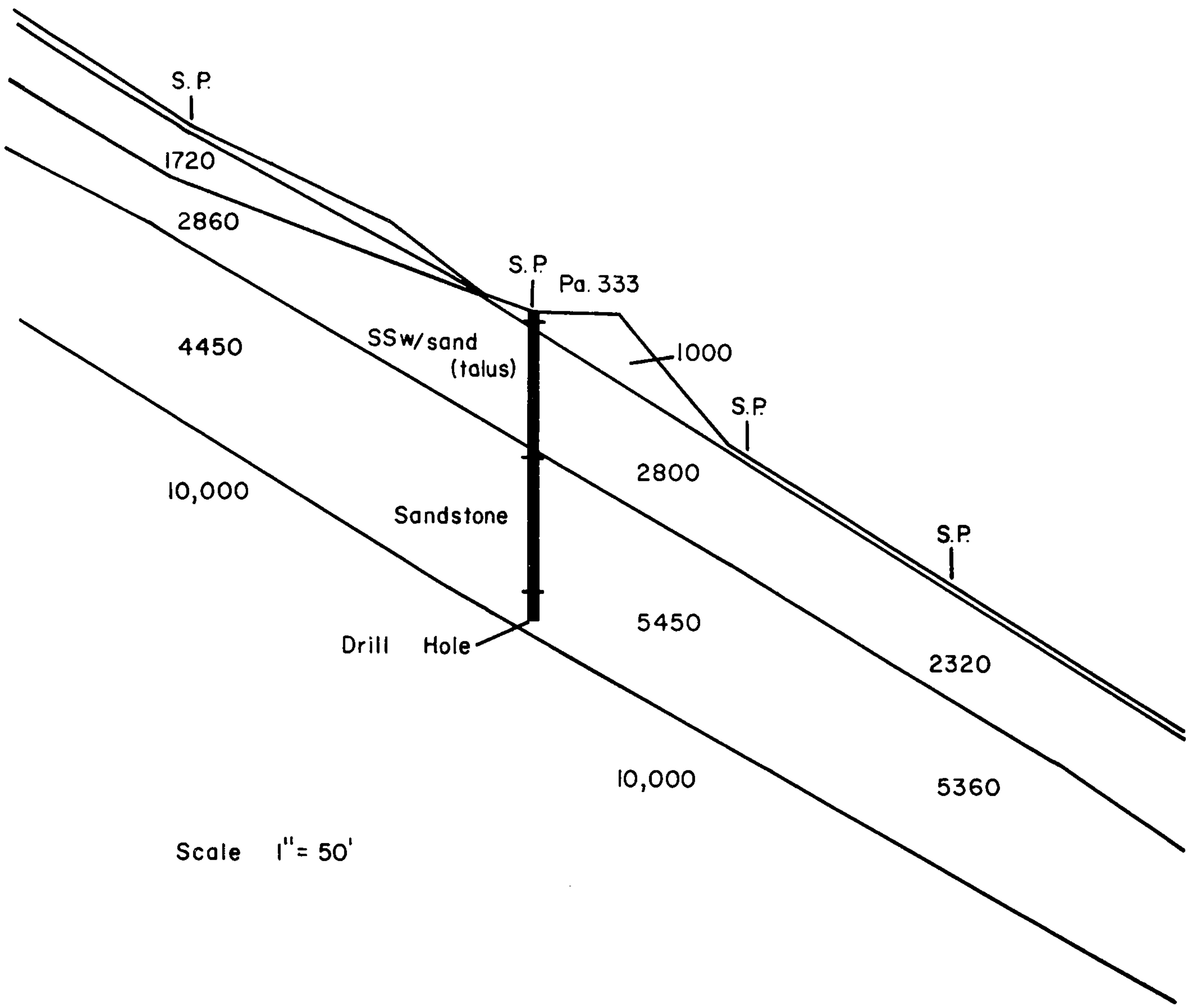


Figure 2. Typical section showing seismic and drill information.

velocities indicate sandstone and some quartzite.

The talus deposits become quite deep, ranging in depth from 40 to 50 feet in certain areas as defined in this study. This **seismic** information formed the basis for later stability analyses of the talus fields (Figure 3).

In order to aid both the geological and seismic investigation, a total of ten core borings were drilled. Due to the extreme roughness of terrain, these holes were located along the existing secondary roadway, PA. Route 333. A total of 472 lineal feet of BX core were obtained, using an Acker B-60 skid mounted drill. Due to the type of rock encountered, mainly that of the Tuscarora Formation, extremely difficult drilling was encountered. Problems of poor recovery and loss of drill bits due to drilling in talus composed of such hard rock (quartzitic sandstone) made a more extensive drilling program economically unfeasible. Enough information was obtained to validate the seismic survey and establish what materials made up the talus deposits in different sections of the project area.

Stability-Hillside and Cuts

From field observation and geophysical data the most extensive stability problem areas exist along the eastern end of the project (Figure 1). The stability of the existing hillside and the reaction to the proposed construction by the materials present is of primary impor-

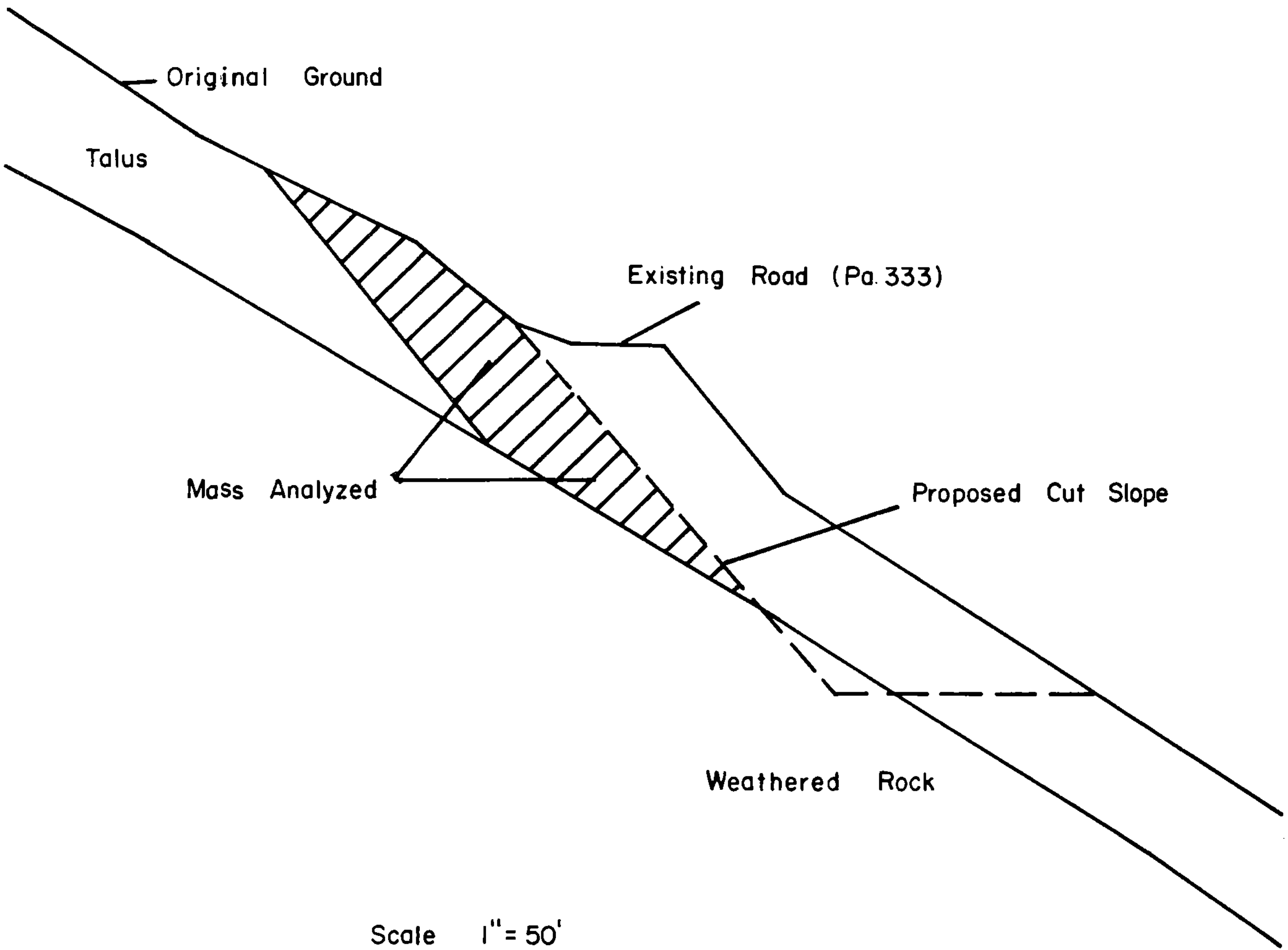


Figure 3. Typical section showing mass analyzed for stability problem.

tance in this and future studies and designs. Three conditions were evaluated: the condition and stability of the talus; the mass stability of the hillside; and the stability of joint systems in the bedrock.

Talus Stability

The first phase is the condition and stability of talus, which consists of large boulders ranging in size from less than one foot in diameter to as large as tens of feet in diameter. A large portion of these boulders are rectangular in shape, lying so that the longest dimension parallels the slope. Trees are present over a large portion of the hillside, with several large barren areas occurring. These barren areas are generally associated with deeper accumulations of talus and little if any fine material.

The talus deposits under consideration are at or near static equilibrium. It may be assumed that at the time of deposition, each rock in a talus slope has theoretically found its assigned place according to the laws of statics and dynamics. The conditions required for this situation are the size of the rock, the shape of the rock, its momentum, the slope angle and the roughness of the slope over which it has traveled. When rock comes to rest it is usually in static equilibrium and further movement can occur as a result of very minor disturbance. If for some reason a large boulder comes to rest on the slope higher than would be expected mechanically, a deposition of small rocks can occur around it. Such a collection of large rock debris at a higher elevation than would normally be expected will cause a decrease in the overall stability of the slope at the time of deposition.

Portions of talus deposits which do not move after this deposition can increase in stability over a period of time. Factors which control this are weathering and gravitational re-arrangement of the boulders.

An example of the minimal stability can be seen along existing Route 333. The small road cuts in the talus show signs of constant boulder and rock movement even though the slopes are often near the same angle of repose as the natural talus slopes.

In cuts made in the open graded talus zones, where the boulder stability depends on rock to rock contact, rockfall will occur. Such rockfall will be more severe immediately after the cut is open, and this rockfall will continue for a long period of time. Surface weathering, erosion, and local rearrangement of boulders under gravity forces bring some stability, through time, at or near present natural slope angles.

Boulders and rocks buried in talus are "floating" in finer soils and when exposed in cut slopes represent a long-term hazard as the finer materials are weathered and eroded away from the rocks. This action is responsible for loss of support and the rockfalls which occur. As in the case of the zones with rock to rock contact, these areas, through surface weathering and local rearrangement, can stabilize in time, near natural slope angles.

Thus, in a talus slope, the material tends to continually migrate down slope via gravity until static equilibrium is reached. Disruption of the equilibrium condition, through natural causes such as toe erosion by a stream or man-made works, such as an excavation cut, results in downward movement until equilibrium is again reached, usually at the previous slope angle. Continued removal of rock blocks near the toe of a slope (i.e., maintenance on a cut) usually results in a permanent state of inequilibrium and perpetuates itself in the form of continual downward movement of rock debris.

Mass Stability

The second consideration in evaluation of the proposed alignment is the effect the construction would have on the steep talus slopes which exist. The major stability problem along this section is related to the potential for a mass failure of the talus overlying the bedrock.

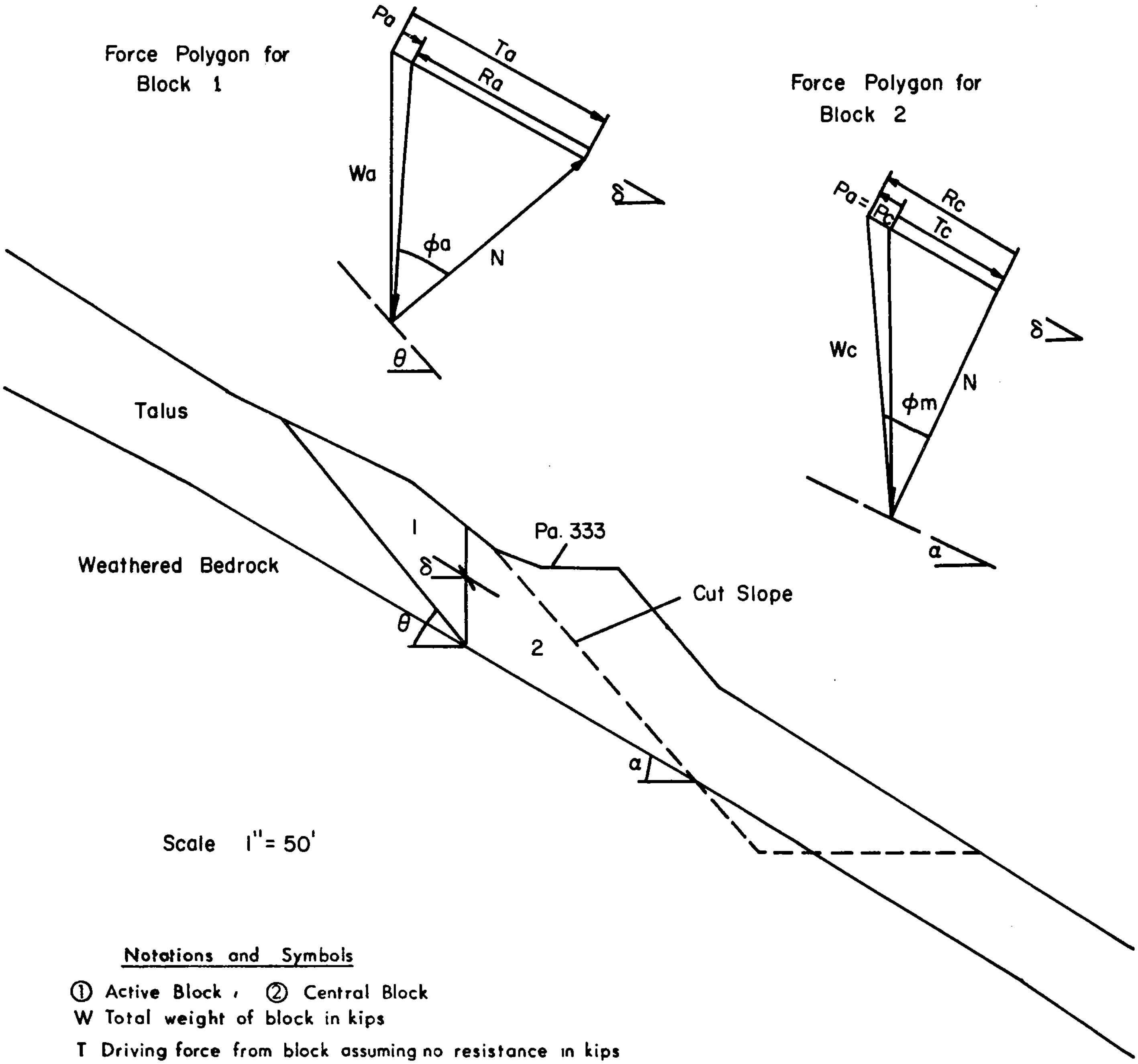
The evaluation of mass stability of the slopes in this area involved several steps. The surface and subsurface profiles were developed for each section using the seismic and drill hole data (Figure 2). The basic nature of the soil and rock material was determined by field investigation.

The stability analyses were performed to establish the parameters necessary to achieve equilibrium, as well as a safety factor of 1.25. Analyses were made for the in situ condition with the proposed cut as an integral part of the problem (Figure 3).

The actual stability calculations were made using the wedge or sliding block method (Figure 4), incorporating the simplified method of blocks with side forces, as developed by Richards (1969), Seed (1967) and incorporating the developments of Sherard (1963).

The analyses completed for this represent a typical example of the entire east end in the Narrows Section. From the results, it may be assumed that gross instability would result from the proposed cuts. The equilibrium curves show (Figure 5) that friction angles higher than can normally be expected in this type of material would be required for equilibrium. When the factor of safety of 1.25 is added into the problem, the situation deteriorates in all cases, and requires ϕa's (developed friction angles) of 35° or more, and ϕm's (mobilized friction angles), in excess of 35°. These requirements to meet stability cannot be reasonably expected everywhere for the slope conditions existing on this project. The typical equilibrium curves seen in this slide using a simple two block and a three block analysis can be seen in Figure 5.

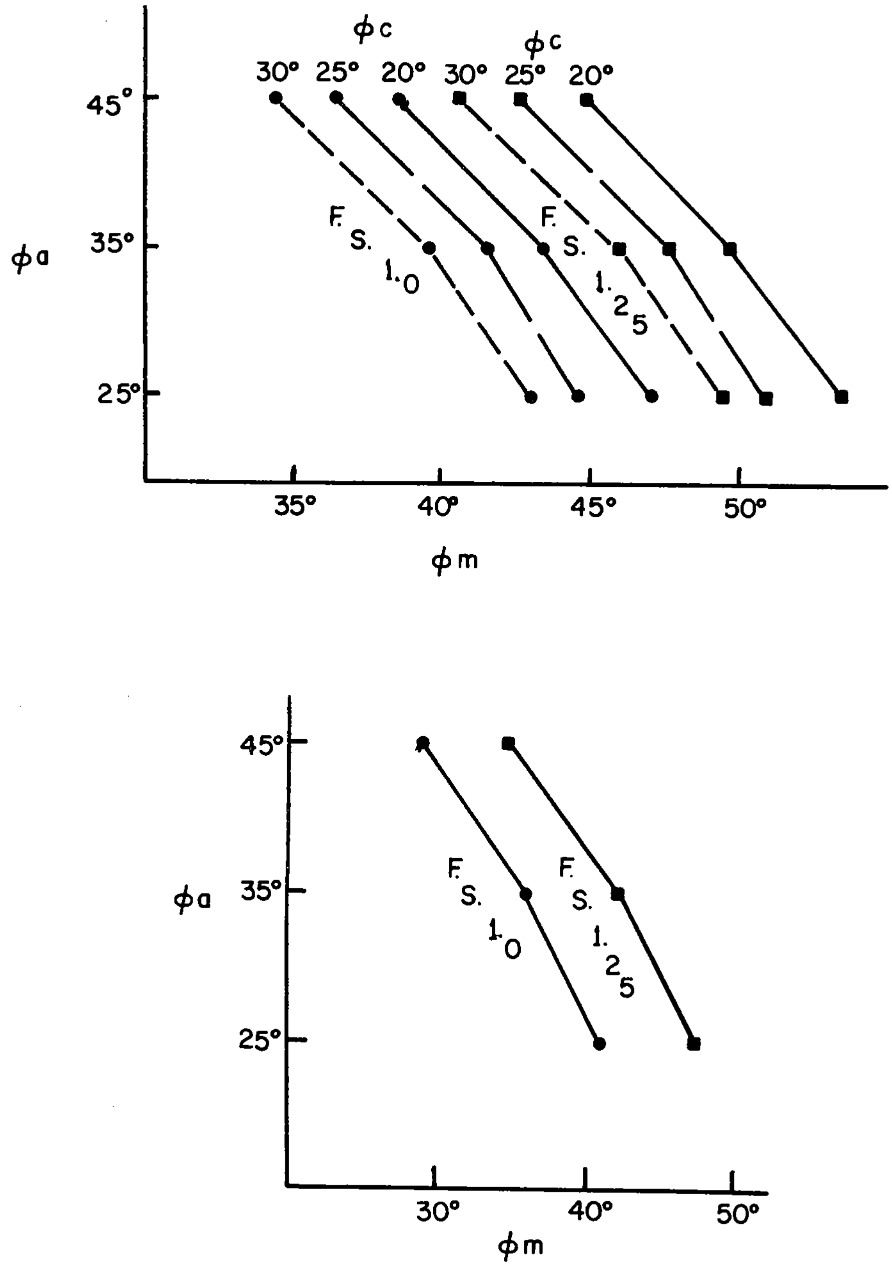
The drainage of the steep hillside areas is an important key to the problem of mass stability. All of the information available, including observations of similar cuts, indicates that the hillsides are relatively free-draining and that the natural ground water level is located in bedrock. There are indications that during periods of sustained precipitation there is ground water flow along the contact between the overburden and the bedrock.



Notations and Symbols

① Active Block , ② Central Block
W Total weight of block in kips
T Driving force from block assuming no resistance in kips
R Resisting force from block assuming no resistance in kips
P Net resultant force in direction of δ
ϕ_a Developed friction angle assumed on backface
ϕ_m Mobilized friction angle required for equilibrium on basal plane
δ Dip of interblock forces
θ Dip of backface, active block
α Dip of basal plane, central block
γ Unit weight of material in block

Figure 4. Typical section showing sliding block analysis.



ϕa - developed friction angle required on the backface

ϕm - mobilized angle of internal friction required for equilibrium along the contact between talus and bedrock

ϕc - developed friction angle in the talus

F.S. - Factor of Safety

Figure 5. Equilibrium curves showing simple 2-block case and more complex 3-block case.

This drainage is considered responsible for the development of zones of highly weathered shales and a clay layer at the base of the talus which appear in drill holes. It can also be reasonably assumed that the effective strength of these zones is reduced by the prolonged presence of water and by the elevated water levels in the talus slopes.

In the analysis, water in the talus was not considered because none was found in the drill holes, which were placed during the dry part of the year. Its presence in the analysis would raise the weight of the block and reduce the shear strength along the overburden-bedrock contact. These two conditions would help to reduce the overall poor stability already present in the hillside and therefore contribute to the already unstable mass of material, and would prevail in the wet part of the year, particularly during spring thaws.

Rock Stability

The third phase of the investigation was to study the joint systems which would affect the proposed cuts. Information from detailed studies in the general area indicate that the joints maintain a fairly constant orientation to the fold axes. At some localities, two sets of transverse joints are present. In the competent beds, both the longitudinal and transverse sets are oriented nearly perpendicular to bedding. Thus the dip of the transverse joints varies only slightly from steep northeast to steep southwest, while the longitudinal joints form a pronounced fan-shaped pattern in the cross section of a fold.

In considering the stability of the rock formations, only failures along jointing and bedding plans were considered. The information was compiled and then combined to consider the two major formations; the Tuscarora and the Rose Hill. The joints in each formation and their relationship to one another and the bedding were analyzed graphically, with the use of stereonets according to John (1968).

The stability of each formation was evaluated with the available joint information. The Tuscarora Formation was evaluated and careful study indicated that slopes of 1:1 or flatter would be necessary to minimize rockfall in this formation. The Rose Hill Formation was evaluated in a similar manner. In this formation a slope of 1:1 would also be necessary to minimize rockfall.

In addition, water in the joints, although not considered in this study, could cause excessive pressures and reduce the shear strength in the joints. This would decrease the overall stability and increase the chance for failures along the planes.

Recommendations

Based on the discussion and analyses presented in this study, it is concluded that construction along the east end was not feasible. It appears from the information available that excessive excavation costs would be involved in the talus and the rock slopes, in addition to the instability which would result in the following three conditions; talus slope, mass stability and rock slope stability.

Mass stability is questionable even at flat slopes of 2:1, which would not be feasible to construct in this area. Consideration of talus slopes shows that when the mass is stable, rockfall and movement can occur as the talus moves downward to establish new equilibrium. Slopes of 1:1 in the existing rock formations may be excessive in excavation costs since the rock formations are so massive. Excessive blasting could also be anticipated for large blocks of talus in order to make them an acceptable size for embankment use.

Based on field observations, drill information, and seismic data the west end appears to be more stable. Cut slopes of $1\frac{1}{2}$:1 and 2:1 can be expected to be stable since the overburden material is residual and colluvial rather than talus.

All of the analyses, conclusions and recommendations contained in this report are based on field observations, seismic information and assumed data. Final design investigations for this area, if chosen, would require the use of more detailed geophysical surveys, drilling and possibly field or lab tests analyses such as used in this report. Final design investigations should include the establishment of strength parameters for all questionable areas such as cut slopes, embankment foundations, and structural foundations.

Conclusions

As can be seen in this example a great deal of usable information may be obtained during a preliminary investigation. In this particular case, some of the most expensive and extensive problems are directly related to the geology of the area. This type of study enables the geological engineer to establish and define problem areas for the designer to incorporate in the basic design and location of the highway. By such study and cooperative undertaking it may be possible to avoid these problems and utilize the alternative alignments available such as was done in this case. The geological engineer through a thorough investigation established that a particular alternate was not feasible from the point of stable slope construction.

It must be always foremost in the mind of the geologist and the designer that a design such as this, in an area of extensive talus and colluvial deposits, is a serious undertaking and requires a thorough investigation in order to develop a safe, well-engineered design. With the tools of geotechnics available to the geological engineer this objective can be obtained in the preliminary stages of route location and design.

Acknowledgements

The author is indebted to the following persons who made this manuscript possible; Leo D. Sandvig, P.E., Director, Bureau of Materials Testing and Research; Donald L. Keller, P.E., Chief Soils Engineer; B. E. Speegle, P.E., District Engineer, Engineering District 2-0. A special thanks is extended to James W. Richards, P.E., for the use of his seismic survey report which is incorporated in this report, for the guidance and direction of the original report and for critical reading of this manuscript.

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Cut Slope Failure in Residual Soil

by

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ABSTRACT

Construction of a portion of Interstate Route 81 in the southwestern corner of Virginia required a small cut into the residuum blanketing the shale bedrock forming the southern slope of Tinker Mountain. During excavation for this cut in 1961 a small slide developed which grew to such an extent that considerable corrective work was required during the following year. The remedial measures did not fully stabilize the sliding mass and periodic maintenance involving an expenditure of up to $35,000 per year was required. By 1971 the headward progression of the slide had necessitated considerable increased right-of-way taking and a continued high maintenance cost in order to protect the roadway. A mutual decision was made to seek a permanent repair to the slide. This paper describes the subsurface exploration and instrumentation which was performed and the final remedial design proposed to stabilize the hillside.

1. Introduction

The construction of a portion of Interstate Route 81 in the southwestern corner of Virginia (Figure 1) has provided a vivid case history of the stability problems common to cut slopes constructed in residual soils. This example of a cut slope failure also substantiates the following general conclusions found in the literature:

a. Slides in residual soil, if not arrested in the early stages of development, can and do develop into massive movements.

b. There is a direct relationship between transient water tables developed by seasonal rainfall and the development and acceleration of slope movements.

c. The inherent nature of failures in residual soils prevents the application of conventional analytical procedures. Reliance must be placed on adequate subsurface exploration and field instrumentation tempered by previous experience with similar materials.

2. Project History

About 5 miles north of Roanoke, Virginia, I-81 traverses the southern slope of Tinker Mountain. A small cut of about 30 feet maximum depth with two horizontal to one vertical side slopes was made into colluvium and residual soil blanketing a shale bedrock. During construction (1961) a small slipout occurred.

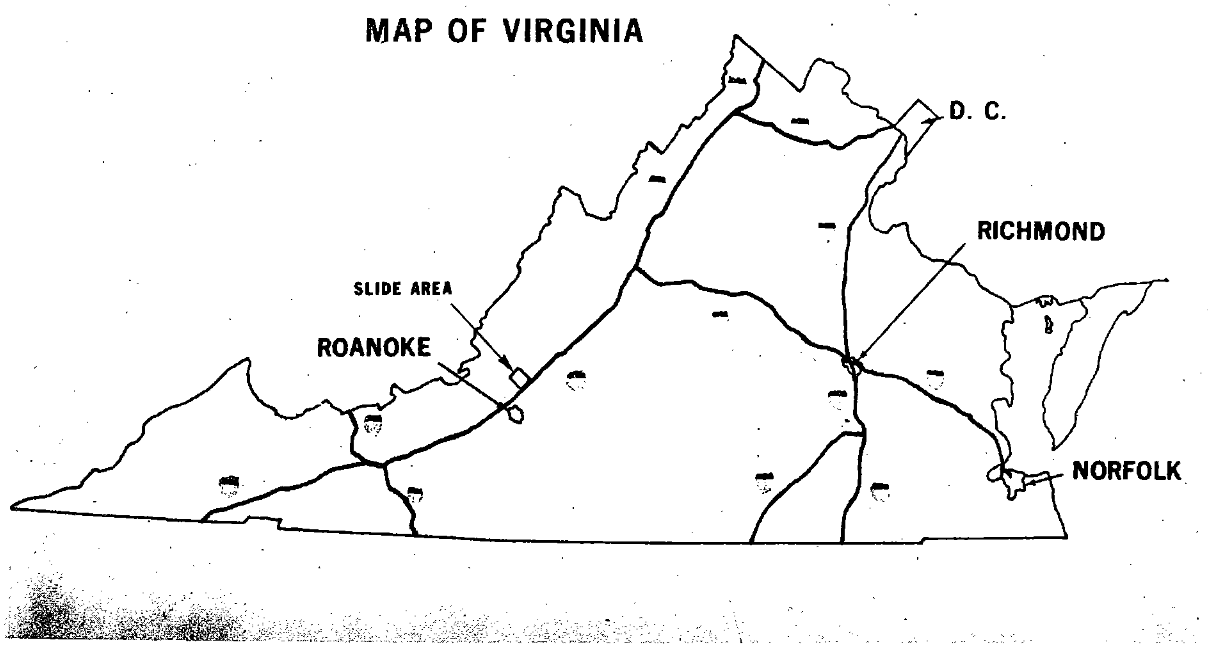

Figure 1. Location of failure.

First attempts toward stabilization involved slope flattening to a three horizontal on one vertical. The regraded slopes remained stable through the seasonal changes until the following spring when a second, much larger slide occurred. It was obvious that a relationship between rainfall intensity and slope movement existed, yet stability analyses with the water table assumed at ground surface indicated a more than adequate factor of safety against movement.

After much consideration of many possible alternatives, a remedial scheme was selected which included (1) a trench drain around the slide scarp and up the face of the slide more or less at right angles to the roadway; (2) flattening the slope to four horizontal to one vertical; and (3) an intermediate bench at approximately midheight of the slope. Construction of this remedial scheme was completed in 1962. (See Figure 2 for a typical cross-section of the completed slope.)

Unfortunately, the movement was not completely arrested by this treatment and periodic maintenance involving an expenditure of about $35,000 per year was required to prevent slide debris from encroaching upon the roadway. Figures 3 and 4 are aerial and plan views, respectively, of the failure after 9 years of this maintenance activity. By 1971 the headward progression of the slide had necessitated considerable increased right-of-way taking and an unsightly scar had developed on the side of the mountain. A mutual decision was made to seek a permanent repair to the slide. The steps taken to arrive at a satisfactory solution are discussed in the following sections.

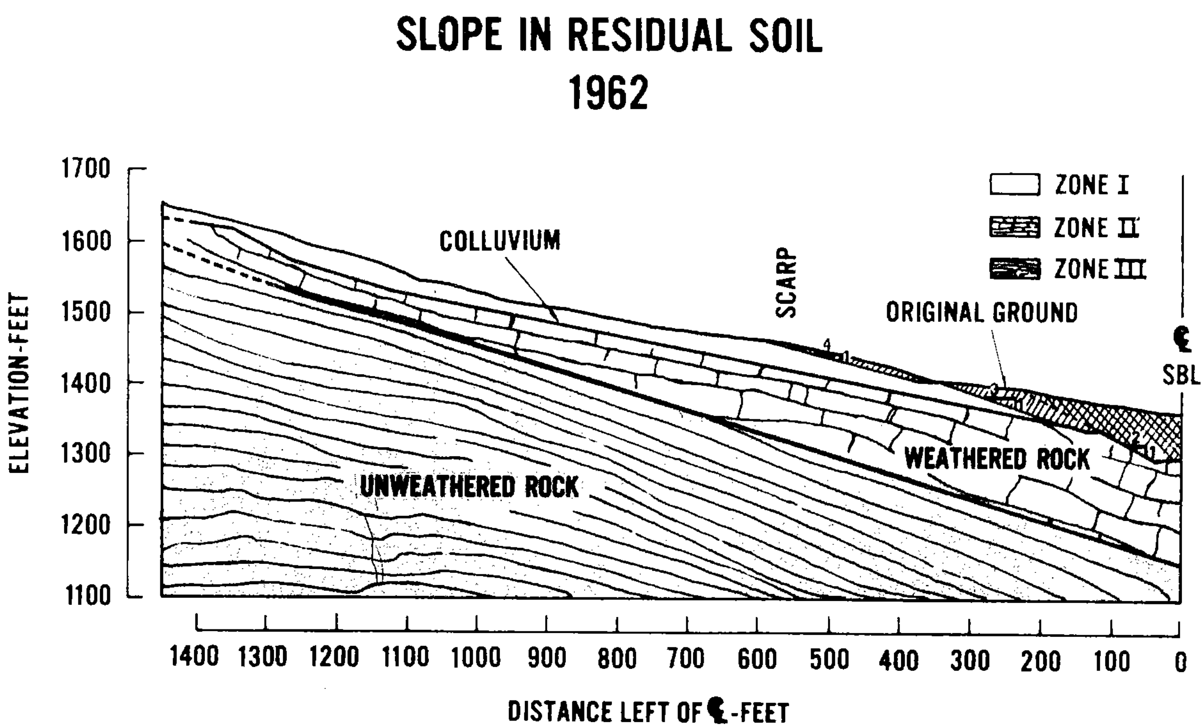

Figure 2. Typical X-Section- 1962.

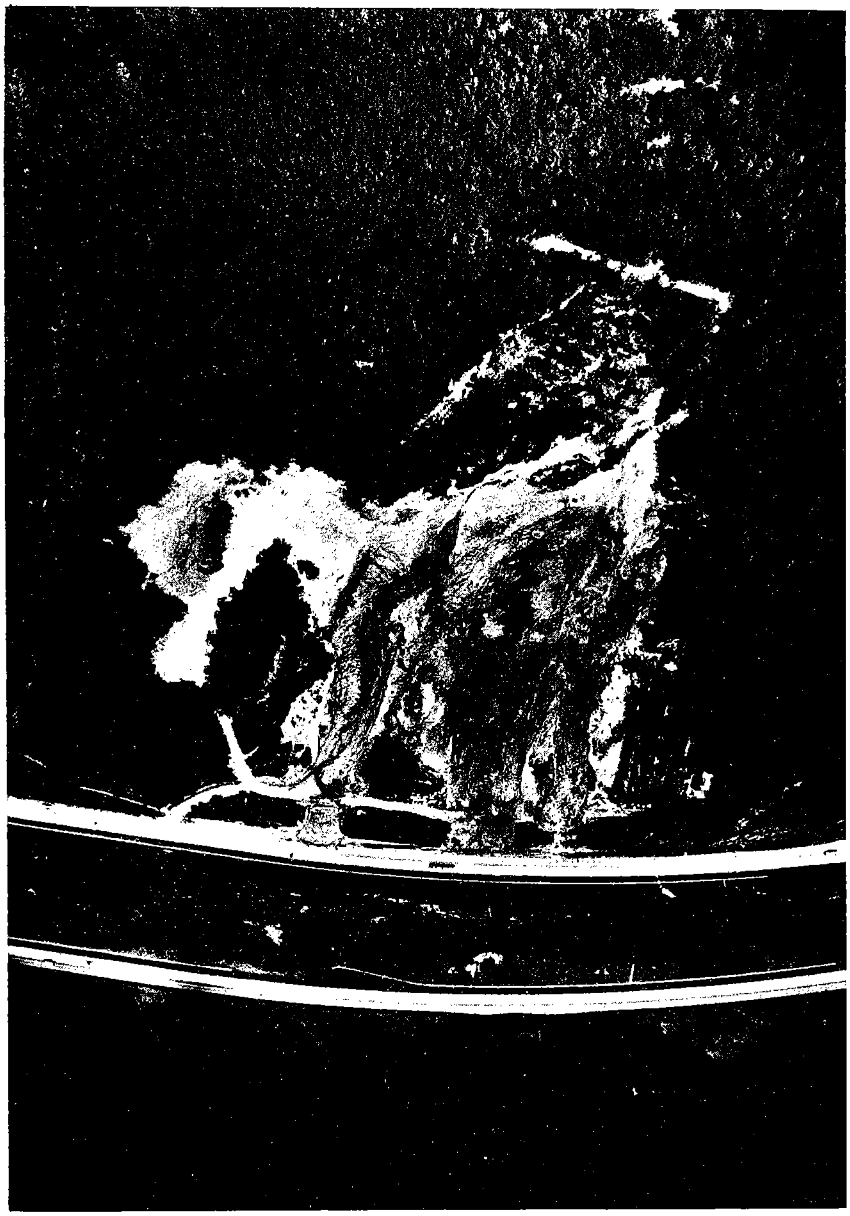

Figure 3. Aerial View-1971.

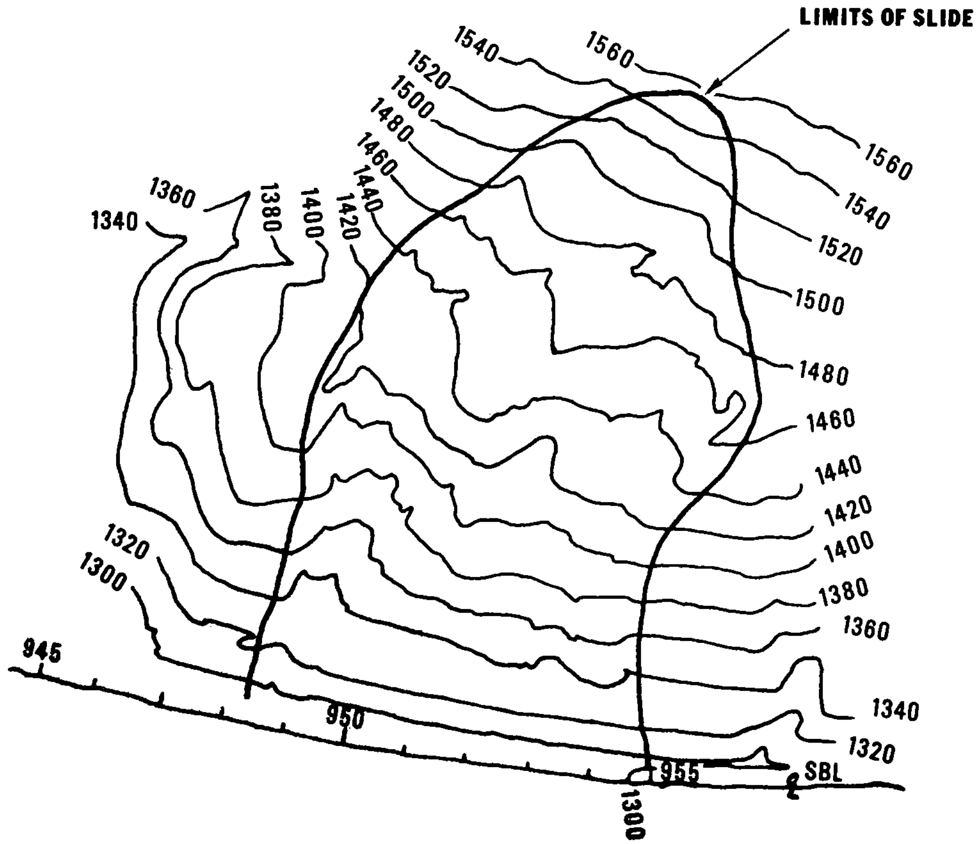

Figure 4 . Plan view-1971 .

3. Geologic Aspects

As quoted from geologic information obtained for the site (3)*: "Tinker Mountain is the northeast extremity of a large folded area of the Roanoke Valley division of the great Valley of Virginia known as the Catawba syncline.

The southeast slope of the mountain where Route 81 is located is covered with a varying thickness of colluvium that consists of material ranging from a light gray montmorillonitic clay to sandstone boulders 10 or more feet in largest dimension. These large sandstone boulders are derived from the Clinch sandstone of Silurian age that forms the crest of the mountain. Attitudes taken at various locations on top of the mountain overlooking the slide area vary from N55°E (strike), 15°NW (dip) to N70°E, 15°NW which corresponds to the trend of the Catawba syncline.

Underlying the Clinch sandstone on Tinker Mountain is the Martinsburg shale of upper Ordovician age. This material where it is exposed in gullies that have eroded through the topsoil in the cut for Route 81 is a light brown to maroon, slightly calcareous shale with some scattered thin calcite seams. The material exposed in these gullies is stable but quite weathered. The Martinsburg formation in its unweathered state underlying the problem area, as determined from drill holes, is a light to dark gray calcareous shale and gray argillaceous limestone. This formation is weathered quite deeply in this location especially at the west end of the problem area where the depth is approximately 100 feet to fresh shale.

*Numbers in parentheses refer to the references listed at the end of the text.

It was assumed during previous field explorations that the shale dipped slightly into the road. This was for the most part because the simplest way to measure a dip direction was from small gullies eroded through the topsoil mantle. Due to a considerable amount of rainfall during the summer of 1971, the topsoil has washed off areas large enough to be able to measure the attitude of the shale more precisely. As determined from the east portion of the problem area, the attitude of the shale is N50° -60°E, 10°-15°NW which corresponds to the attitudes taken in the Clinch formation. There are still areas in some of the gullies where, due to either false bedding or very localized deformation, the bedding could be assumed to dip 10°-15° into the road as was the previous assumption."

Aerial photographs have been taken at periodic intervals since the initial slide movement and these were of value in determining the nature of the failure. They also present a graphic history of the slides progressive development.

4. Subsurface Investigation and Instrumentation

Although borings had been made at various times since the initial failure, the continued movement of the slide over the years had significantly altered the topography and it was deemed necessary to fully reinvestigate the subsurface soils. In early 1970 a series of four borings was made which were located slightly above and paralleling the uppermost scarp. Two of these borings were instrumented with inclinometers to a depth of about 75 feet. After several months of monitoring the inclinometers, the data was somewhat erratic and seemed to indicate that the casing had not been installed to a sufficient depth and was moving throughout its length. Therefore, in the spring of 1971 four additional borings were made in a line perpendicular to the upper scarp and extending up the mountain behind the scarp. The uppermost boring was located about 600 feet behind the scarp. Inclinometers were installed in three of these borings to an average depth of 145 feet. To facilitate design of the proposed remedial scheme a third series of 16 borings was subsequently made in late summer of 1971. The subsurface information obtained from these borings and instrumentation and its relationship to the failure mechanisms involved are discussed in the following section.

5. Discussion

The subsurface profile defined by the various borings is illustrated by Figure 5. The division of the profile into zones follows the sequence suggested by Deere and Patton (1).

Zone I consists of (1) colluvium derived from the higher steeper slopes and (2) the residual soil product of weathering of the underlying parent rock. Information obtained from the borings was too limited to seperately define these two strata. In the main body of the slide the disturbance caused by past movement has obliterated any definable contact surface that may have existed between the colluvium and residual soil. Generally, the soil of Zone I is a silty clay containing numerous "floaters." These "floating" sandstone boulders have as their source the sandstone capping the top of the mountain and have been transported by gravity to their present positions. Their maximum dimension does not usually exceed 10 feet. Zone I varies in thickness from 10 to 50 feet.

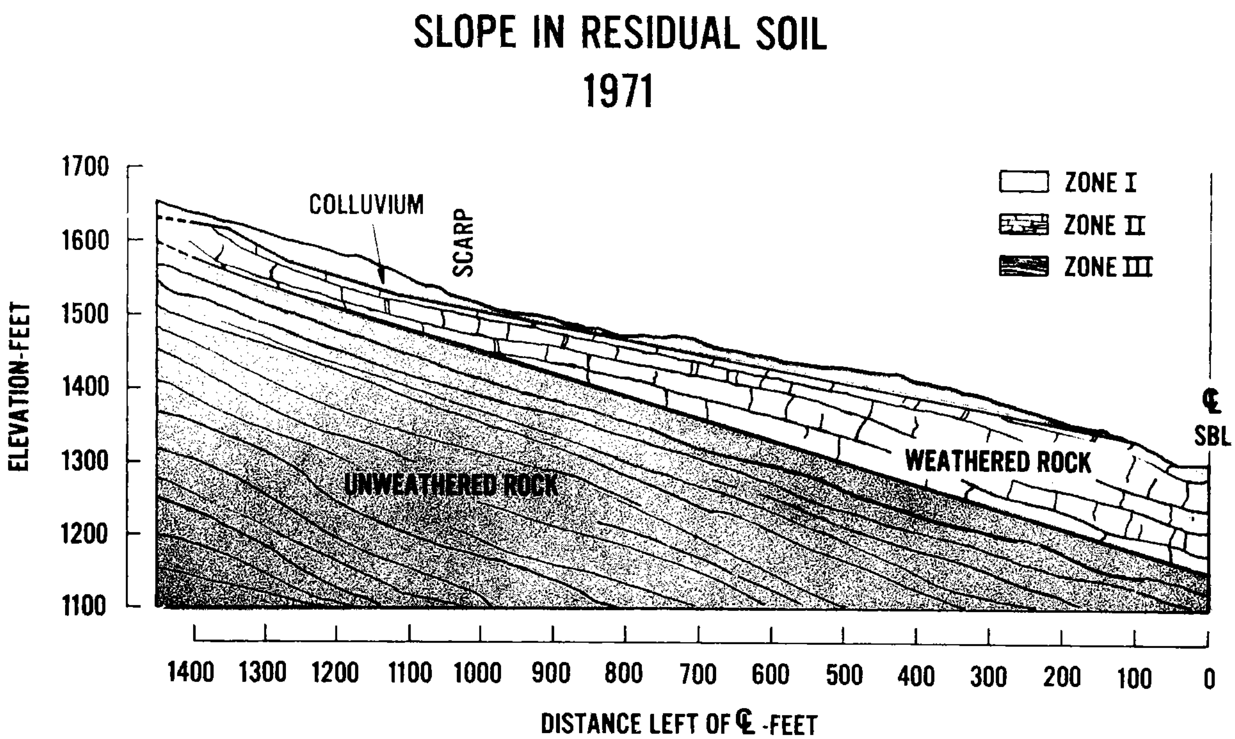

Figure 5. Typical X-Section-1971.

The boundary between Zone I and Zone II was usually easy to detect because of a marked decrease in rate of penetration. Occasionally, boulders hampered the detection of the transition from Zone I to Zone II. The weathered shale of Zone II contained only a few clay seams near the top and became progressively harder with depth.

The top of Zone III was not sharply defined in any of the borings. Rather, a smooth transition, detected only by a gradual decrease in the rate of penetration was observed. Occasionally small calcite filled seams were encountered as the borings progressed through the unweathered shale. The depth to Zone III is not believed to exceed 100 feet.

Although no concentrated effort was made to monitor water levels within the sliding mass, the borings did provide some valuable information regarding the phreatic levels within the slope. Generallv, the borings encountered water near the transition from Zone I to Zone II. The water bearing stratum was usually only a few feet thick and below it the nearly impervious shale was always found to be completelv dry. After periods of heavy rainfall, water could be seen seeping from the scarp at several levels in the Zone I soil. The soil within the slide itself remains completely saturated throughout the year.

Observation of the inclinometers over the past year indicate that no new movement has occurred behind the existing scarp. The scarp face, which does not exceed 15 feet in height, has also remained stable during this time period. The apparent dormancy of the slide for such an extended period of time is difficult to explain in view of the much greater than normal rainfall that has occurred during the same time span.

Two separate failure mechanisms can be distinguished within the unstable slope. The first is a mudflow condition which exists in the lower portions of the slide. This mudflow activity is, of course, directly related to rainfall intensity. As the debris in the heart of the slide turns to mud and flows down to the roadway, the second failure mechanism is triggered. The transient stability in the scarp area is slowly reduced by this loss of support and in due time a failure occurs and the scarp continues its headward progression up the mountain. For these reasons a two-step solution, as discussed in the following section, was developed and analyzed.

6. Proposed Remedial Scheme

It was considered of major importance to limit any further enlargement of the slide. For this purpose a rock buttress with dimensions determined by wedge-type stability analyses was designed to restrain the slope above the existing scarp. A conservative value (ϕ = 10 degrees) was estimated for the shear strength against sliding of the weathered shale upon which the buttress will be founded. Paved ditches above the buttress, a filter blanket on the upslope side and underdrain within the buttress were included to prevent surface and subsurface water from entering the sliding mass from adjacent areas.

Upon completion of the buttress construction, the slide debris in front of the scarp will be treated. Large granular drainage trenches have been proposed for this purpose. They will be located in the two major channels developed by the mud flowing down the slope. This will not only effectively drain the remaining soil mass but will result in excavation and removal of a significant portion of the existing saturated soil and its replacement with granular material of greater strength. Regrading of the failed slope,intermediate small benches with paved ditches, and seeding to establish growth of ground cover are also proposed.

Figure 6 is a plan view of the slide illustrating the proposed location for the buttresses and drainage trenches. Typical sections for the buttress and drainage trench are shown on Figures 7 and 8, respectively.

As part of the correction, long term instrumentation will be installed to assist in early detection of any future movements. Inclimometers and piezometers will be read periodically. If localized water pockets develop and exceed allowable levels within the slide mass, pumped wells will be installed to provide relief to the area involved. Such measures will be necessary in order to assure long term stability of the total mountainside.

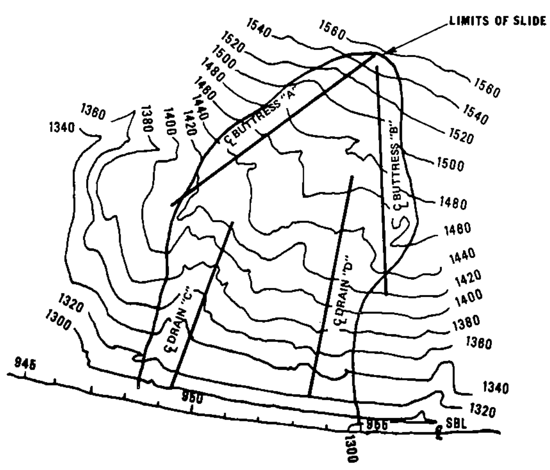

Figure 6. Plan of proposed remedial scheme.

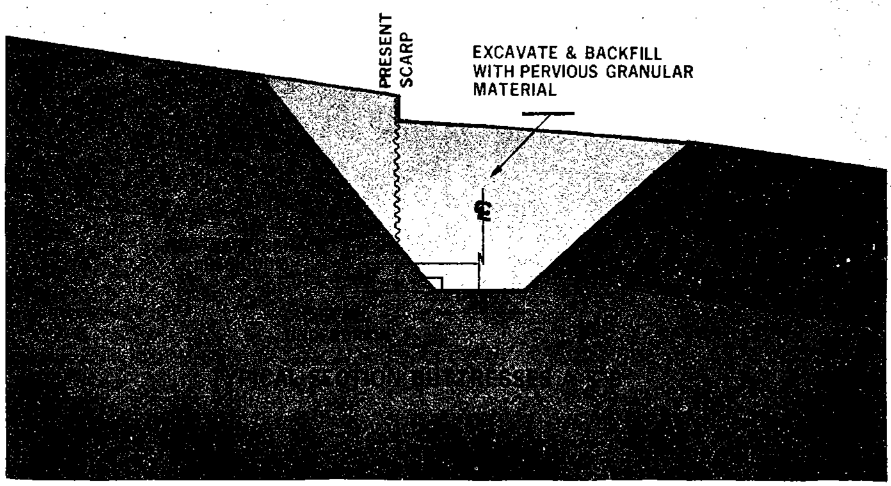

Figure 7. Typical x-section of buttress.

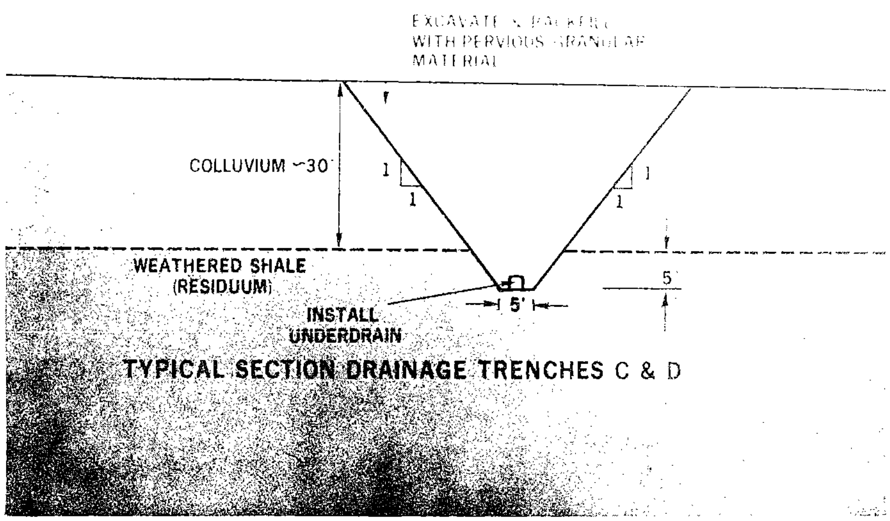

Figure 8. Typical x-section of drainage trench.

7. Summary and Conclusions

Several important conclusions may be drawn from the performance of this cut slope in residual soil. The first, and most obvious, is that small failures can and do develop into massive movements if not arrested in the early stages of their development.

Secondly, there was an obvious discrepancy between design conditions and actual field performance as regards water table elevation at failure. This is evidenced by the fact that although these levels were assumed to be at ground surface in the original remedial design, the failures which occurred were directly influenced by hydrostatic forces. Aside from a possibly unseen structural weakness of a buried relict feature, the only explanation is that proposed by Terzaghi and Peck (4) "that slides are preceded by a sudden, but temporary and local, increase in porewater pressure in the zone of sliding." This sudden increase in pore pressure would be most likely to occur during periods of intense rainfall.

Finally, the value of adequate subsurface exploration and field instrumentation is demonstrated. Only by having a knowledge of subsurface conditions as they actually exist can prior experience with similar materials be utilized in the correction of an instability. The performance of the completed remedial scheme can also be monitored by appropriate instrumentation so that the experience gained can be applied to future problems.

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3. KUHN, KARL, Personal Communication, November 18, 1971

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ENGINEERING GEOLOGY OF THE SEMI-INDURATED STRATA OF THE VIRGINIA COASTAL PLAINS

by

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In working with the engineering geology of unconsolidated material, we usually are working with non indurated materials normally classified as soils for construction purposes. Normal engineering practice is to classify as soils any material that can be removed by power equipment. Defining the minimum degree of induration that constitutes rock as such for construction purposes can be quite difficult. Oftentimes experience has shown that it should be defined in specific terms or excavation methods as well as by the type of equipment used. Gross rock properties in situ on a construction site may sometimes be approximated by the ease or difficulty of excavation according to the specific equipment used. Rock for excavation purposes may be one thing on an open site where a large tractor may be used for excavation and something quite different in a narrow trench excavation where only hand tools can be used. As can readily be imagined, then judgment and experience enter largely into even the apparently simple decision of whether it is soil or rock for excavation purposes.

To start with we thus have to establish accepted criteria for what is soil and what is rock.

The numerical values given to indicate rock conditions are as follows:

Type 1 The Saprolites — Disintegrated Rock — Has the appearance of rock structure but is readily excavated by pick or shovel.

Type 2 Soft Rock — Has the appearance of firm rock but can be drilled with non diamond drill bits — core recovery slight.

Type 3 Firm Rock — Frequent joint planes moderately deep penetration and discoloration in both phases of the joint planes — Required diamond drilling — Edges of the cores rounded at joint planes.

Type 4 Hard Sound Rock — Occasional joint planes — Disintegration at joint planes very shallow — cores are clean and edges separate sharply, requires diamond drilling.

The R.Q.D. rock quality description criteria for logging these cores were developed under the federal grant by Dr. D. W. Deer of the University of Illinois. "Engineering Classification and Identification for Intact Rock", U. S. Dept. of Commerce and the National Bureau of Standards. The basic procedure involved is to eliminate from your core recovery percent calculations all natural core lengths less than 4" long. Cores with machine breaks are fitted together and are included if over 4" long. Using the R.Q.D. method the following correlation of rock quality has been established.

R.Q.D.	—	0 to 25%	Very Poor
		25 to 50%	Poor
		50 to 75%	Fair
		75 to 90%	Good
		90 to 100%	Excellent

The usual range of unconfined compressive strength in unilateral testing found for these categories of R.Q.D. determinations in the Richmond area are as follows:

Very Poor	0 to 1500 psi
Poor	1500 to 2500 psi

Fair	2500 to 5000 psi
Good	5000 to 8500 psi
Excellent	8500 + psi

Type	R.Q.D.	Classification	Strength Unconfined psi	Suggested Bearing Value Range in Kips
1 - 2	0 - 25%	Very Poor	0 - 1500	10 - 30
2 - 3	25 - 50%	Poor	1500 - 2500	30 - 50
3	50 - 75%	Fair	2500 - 5000	50 - 70
3 - 4	75 - 90%	Good	5000 - 8500	70 - 90
4	90 - 100%	Excellent	8500 +	90 +

R.Q.D. values based on core recovery using a diamond bit and double tube core barrel current design practice based on unconfined compressive strength call for use of a Factor of Safety of 5-8. These factors are used to make due allowance for natural defects in rock such as jointing and give conservative values. The values are based on level or near level rock surfaces and where the bearing area has ample lateral support.

No allowance is shown for imbedding in bearing strata. — Design Manual DM-7 U. S. Navy and some building codes allow a 10% increase per ft. of embedment in firm sound rock ignoring the first ft. Igneous and massive sedimentary and non foliated metamorphic rock only maximum increase 25% usually.

Defining the minimum degree of induration that constitutes rock suitable for construction purposes can be very difficult. In many cases, it should be specified in terms of the excavation methods as well as by means of the excavation equipment used.

Gross rock properties within a construction site may sometimes be approximated by the ease or difficulty of excavation according to the equipment used, such as a backhoe or as a drill. For most uses, however, Type 2 Rock would seem to be a satisfactory lower limit.

In using this suggested procedure, we have a starting point for what we shall consider soils. This, of course, is an exception usually only needed in the fall line zone where this transition is a problem. The usual formation in which preloading occurs in this area is of marine origin. These marine deltaic fluvial deposits feather out over the fall line and increase in thickness a few degrees per mile to the southeast.

Unconsolidated coastal plain formations, depending on their age and geological history, are considered as either normally consolidated formations or pre-loaded on preconsolidated formations. It is the preconsolidated formations that we are going to try to outline the construction characteristics of in this paper. Since construction activities are usually centered in larger towns so are detailed data thus concentrated in large urban areas. We are going to limit this discussion to one formation and to the major urban area where sufficient soil parameters have been established from large quantities of test data and valid conclusions can be reached.

The formations we are going to discuss have been heavily pre-loaded in their geological history. The source of this pre-load can be attributed to 3 major causes.

1. <u>Changes in soil structure due to secondary compression (aging).</u>

This is natural aging and consolidation of a geological formation due to normal consolidation in effect producing a minor pre-load by chemical changes in environment such as pH, temperature, salt concentration, precipitation, etc.

2. <u>Dessication and Related Oxidation</u> .

Dessication and oxidation caused in older coastal plain formations by the lowering of the water table during pleistocene glaciation and consequential increase in effective stress in the soil mass due to the lowering of the water table are some of the major causes of preloading.

Total stress in a soil consists of two parts, the effective stress transmitted from soil grain to soil grain and neutral stress, that part of the load supported by interstitial water below the water table. Neutral stress can be either positive or negative. Consolidation or consequently preloading of a formation causes a positive neutral stress and for a period of time, depending on the permeability of the soil, part of the total stress is supported by pore water. Since water is essentially non-compressible, the effect is to create a head on the water causing it sooner or later to migrate thus increasing the effective stress on the soil particles when the water migrates. Each increase in pressure results in an increase in head 1 psi of pressure created 2.313' of head. So as the water table is lowered, the neutral stress gradually becomes effective stress causing the soils to consolidate.

Dessication and oxidation and the resultant capillary tension are capable of producing pre-loads of considerable magnitude through the gradual drying of cohesive formation from the surface down. This withdrawal is retarded by surface tension at the top of the water column by the surface of the water adheringto the side of the tube. The result is more curvature in the surface of the meniscus as it becomes more u-shaped and produces a tension pull on the walls of the tube. The end result is the consolidation of the formation as the water table is lowered by capillary tension. This is normally a nonreversible process. In the soils where the process has acted to produce preloading, oxidation that would normally accompany this process changes the color of the soil so that the extent of the area thus affected can usually be determined by the color changes in the formations. Also as the water table is lowered the effective stress in the soil increases as soil changes from a buoyant weight of 50-70 lbs. per cubic ft. ± to a unit weight of 90-110 lbs. per cubic ft. ± bringing about a nearly 100% increase in effective overburden stress.

3. Removal of Overburden by Erosion Produced by Changes in Stream Gradient Due to Pleistocene Lowering of Sea Levels or Crustal Warping.

A study of the boring logs associated with large engineering projects show that major flucuation in sea level has occurred that would have caused greatly accelerated erosion due to stress on stream gradients.

Extrapolation of borings from the Bay Bridge crossing at the mouth of the Chesapeake may indicate the depth of the buried pleistocene Susquehannah River to be at -350'. This same channel exists at a reported depth of -130' at the bay crossing between Newport News and Norfolk at the Willoughby Spit.

Other depths as noted from borings are as follows:

Bay Bridge at Annapolis, Md.	-170'
James River at Newport News on Rt. 17	- 80' -90'
Williamsburg Sewerage Out-Fall	- 30'
Newport News Shipbuilding & Dry Dock	- 85'

Route 295 across James River 5 miles below Richmond	- 45'
Sever crossing on James River at Southern Materials Plant	- 26'
James River — Richmond-Petersburg Toll Road Bridge	- 21'
Rappahannock River at Grey's Pt., Va.	-145'
York River at Yorktown, Va.	-125'
Potomac River at Alexandria	-100'
Potomac River at Georgetown	- 70'
Pamunkey River at West Point Va.	- 80'

Conversely test borings in the Richmond, Virginia area on major streams at or near the fall line zone show no signs of a major buried stream channel.

The evidence then is fairly conclusive that a considerable gradient has existed in area streams and that accelerated erosion could have removed considerable soil overburden.

The excessive depth for the buried Susquehannah River Valley at the mouth of the Chesapeake could also reflect the effects of headward erosion of the Norfolk Trench. The stream channel depths reported for the Miocene may, possibly, be the result of an anastomosing network of former channels created near the end of Miocene time on a then-existing flood plain or tidal flat.

Part of the overburden removed by erosion should have been the oxidized portion of the Miocene Marl. This does, however, not seem likely since the oxidized portion varies greatly in elevation between elevation 160' - 170' to elevation 70' and is always approximately 5' in thickness. The uniformity of the oxidized zone is hard to reconcile with current evaluations.

The preload in existence was undoubtedly produced by a combination of the above methods with overburden subsequently removed being the apparent major factor.

The recognition of the geological history of a formation is probably the most important criterion for anticipating the existence of a preload.

In many instances, standard penetration test borings are the only means authorized by the owner for the investigation of a potential construction site and soil conditions on the site may be such that this method of investigation is insufficient for recognition of a preloaded condition. Where the more advanced laboratory tests are authorized, it is not difficult to detect the preload.

The prestressing of soils in their geological history can be determined by means of these other laboratory tests on cohesive soils:

Time Consolidation Test — An undisturbed sample of soil from the strata and elevation in question is subjected to a confined incrementally increasing load test under laboratory conditions, and the resultant "E Log P" Curve may be used to determine the range of preloading.

Triaxial Shear Test — This test when used is similar to a time consolidation test, with the major variation being with the capacity while running the test to duplicate confining pressures that exist in an in situ condition. (Primarily Shear Test).

Unconfined Compressive Strength — An undisturbed sample of soil is tested to determine its shear strength in an unconfined position by gradually applying a load to a cylinder of soil at right angle to its normal bedded position.

These soil parameters as developed in these tests are valuable as a basis for design data on engineering projects. For example, as the prestressing increases are shown up by Time Consolidation Tests, a similar increase in the shear strength in cohesive soils must occur. The shear strength (resistance to horizontal displacement) is taken as 50% of the unconfined compressive strength and is considered to be equal to the cohesive strength in the triaxial test. In a time consolidation test the extent of the preload is determined by graph from the "E Log P" Curve (Void Ratio vs. Load & Time). Two other methods are also used using rebounding for each load increment. The simplest method of checking to see if penetration tests are misleading is the use of in-place moisture contents and the Atterberg Limits Test. These are fast and relatively inexpensive tests and under some conditions can be run in the field while other field work is in progress.

The cohesive soils at the time of deposition have their moisture content near the liquid limit value for the soils. Where in-place moisture and liquid limit value are in the same order of magnitude, no appreciable preloading has occurred. As the various means of consolidation take effect, the moisture content of the soil is reduced to at or near the value of the plastic limit or the plastic index depending on the degree of consolidation of the soil.

The liquid limit depends on the mineral constituents of the soil, the intensity of the surface charge of the molecule and the thickness of the attached water and ratio of surface area to volume or shape. Usually the harder the surface charge, the thinner and plattier the clay molecule. The stronger the charge, the thinner the plate, the greater the attached portion of viscous water and the higher the compressibility of the soil.

SOIL PARAMETERS

A. — Index Properties

Index Properties — Atterberg Limits are useful primarily for comparing the results of laboratory tests on clayey soils of the same geologic origin. The ones normally used in Atterberg Limits tests today are as follows:

Liquid Limit (LL)	—	Lower Limit viscous flow expressed in percent moisture, above which the soil water mixture flows as a viscous liquid, and below which the mixture is plastic.
Plastic Limit (PL)	—	Lower Limit of the plastic range expressed as percent moisture, below which the soil will start to crumble when rolled into a thread under the palm of the hand.
Plastic Index	—	LL - PL = PI = Measure range of Plasticity of clays.

The moisture content of inorganic clays usually has the following ranges:

Kaolinitic clays — up to 50%

Illite Marine Clays — 50% to 100%

Montmorillonite — Volcanic clays — 100% +

Inclusion of other soil admixtures or organic content can effect considerable change in these ranges.

Experience has shown that the liquid limit is indicative of the compressibility of a clayey soil strata. Consolidated clays usually have a water content in excess of the liquid limit. Heavily compressed or preloaded water content at the plastic limit, the presence of a pre-load in a soil can be detected and the existence of pre-load verified in strata of the same geological history. Liquid index $\frac{\text{Nat. W.} - \text{Pl}}{\text{Pl}}$ Liquidity Index of 1+ means soil is at liquid limit — very soft — Liquidity Index of 0 or below at plastic limit — stiff. The use of these tests then gives us a rapid, inexpensive method to determine if a cohesive soils formation has been preloaded or normally consolidated. Care must be used to air-dry these soils prior to testing as oven-drying of the soils in question can produce a large decrease in plasticity. The effect of oven drying is to change irreversibly the organic colloidal content so that it acts as a cementing agent between inorganic colloidal particles. The tertiary marine formations that we are discussing are sensitive, that is the ratio of their initial shear strength to the shear strength of the remolded soils at the same water content ranges above 1 (unit).

Soils are rated as to sensitivity as follows:

Insensitive clays	—	1
Low Sensitive clay	—	1 - 2
Medium Sensitive clay	—	2 - 4
Sensitive clay	—	4 - 8
Extra Sensitive clays	—	8
Quick clays	—	16

(After Skempton — page 257) (Physical Properties of Soils — Means and Parcher)

The sensitivities of these cohesives in the areas under discussion range between 10 - 20 for the clays and 4 - 8 for the sandy clay.

These soils then are sensitive or thixotropic when subjected to dynamic shock shearing such as produced by the standard penetration test. This is usually a reversible action. The end effect is that low penetration test results are produced that are in no way representative of the <u>in situ</u> strength of the formation. This is why a method of quick field or laboratory test is required to evaluate properly the competency of this soil.

A word of caution is in order here, since for a considerable period of time the marl formation represented the erosional surface, a good deal of reworking of the marl has occurred. Where the marl has been removed by erosion and redeposited in the marl area, the effect of preloading will have been lost. Thus, just to recognize the marl formation as such is no insurance that a preload exists.

ENGINEERING GEOLOGY CHARACTERISTICS OF THE MIOCENE SOILS

A layer of gray green marine deposits of Miocene age exists from elevation 70 to 180' above sea level in most of the area covered by the USGS Richmond Quadrangle sheet. This deposit of the Miocene Epoch is of marine deposition and is identified as the Calvert Formation. This formation extends westward nearly to the north-south belt line railroad track and follows mainly to the east of the Richmond-Petersburg Turnpike north of the belt line. The formation has been eroded to form the cliff and hills along the flood plain of Upham Brook and the bluffs along the Chickahominy River in the western section of the Richmond Quadrangle and lies covered by a thin veneer of recent alluvium along these flood plains. The formation is effectively masked in the bluffs by Colluvial Soils from the terrace formations.

The Marl formation consists of two distinct facies, the lower layer extending from elev. approximately 70' to elev. 110' is a highly plastic clay, dark gray in color with a small collodial organic content. Some marine shell fragments may also be included. Soil indicies for the facies of the strata show an average liquid limit ranging from a low of 55% to a high of 110%. The natural water content usually falls between the liquid limit and plastic limit, usually 45 to 75%. The upper facies of this formation is a sandy clay. The sand content is usually poorly graded and fine grained causing the soil to appear as a slightly cemented clay. The color of the soil is slightly lighter than the lower part of the strata. Marine fossils are also common in this part of the formation and some connate water may occasionally be found in these fossils. The sandy clay ranges in elev. from 110' to 180'. Liquid limits range from 50% to 70% and the natural moisture content falls around 20% to 60%.

This formation has been heavily preloaded in its geological history and is very competent in its undisturbed state, but it is also very sensitive and thus can create many problems in its engineering use.

Working the soil by conventional means changes it from a hard to very brittle consistency in the natural state to a soft consistency. A backhoe or dragline can satisfactorily remove this marl formation while wheel or tractor equipment working in it will produce a quagmire. Because of this characteristic it is useless as a fill material.

Displacement piles in the Richmond area are not recommended for use in the Miocene strata due to the sensitive nature of the soil. Two adverse effects can arise: considerable soil heave or lateral displacement can occur, and negative skin friction due to remolding of the Miocene soils can occur to an objectionable degree.

Due to its preload, the maximum stress to which it was subjected in its geologic history, the formation can support a considerable load in the undisturbed condition. Test results showing the magnitude of this load as well as supporting laboratory data are presented in the following tables and charts. By subtracting the current overburden pressure above and below the water table from the results of the laboratory test on the soil the usable portion of the preload can be determined. That is the preload minus the current overburden effective stress is the soil pressure that will have to be exceeded to produce any appreciable compression or settlement under load in the strata.

The low results obtained in some of the tests can readily be explained: Soils that have been subjected to considerable prestressing exist in an intermediate position, more or less a transitional state on the way toward becoming hard indurate stratas (rock). This is brought about by the compression and subsequent partial desiccation of the strata. This also produced shrinkage and the beginning of a preliminary joints system. This then brings about the existence of many planes along which failure can occur. These are usually invisible to the naked eye, and have little to no effect in the undisturbed mass of soil, but exist as planes of weakness that may adversely affect some test results.

The results of the Liquid Limit — Plastic Limit Test should serve only as a guide to determine a preload. As the chart shows, a random picture can emerge. Regular, more sophisticated lab tests should be used for design purposes.

Some of the results described here are regional in application and could be applied or hold true for many other marine or flubial deltaic deposits of this general region. For example, the average preload in the undisturbed Yorktown formation along the Virginia Beach Toll Road is 3 - 4 tons per square foot.

		Del.		Md.		Va.	
QUATERNARY	PLEIST-OCENE	COLUMBIA GROUP		COLUMBIA GROUP		LOWER TERRACE SAND & GRAVEL	
TERTIARY	PLIOCENE	BRYN MAWR		UPLAND GRAVELS		UPLAND GRAVEL and SAND	
TERTIARY	MIOCENE	CHESAPEAKE GROUP (undifferentiated)		CHESAPEAKE	YORKTOWN	CHESAPEAKE	YORKTOWN
					ST. MARYS		ST. MARYS
					CHOPTANK		CHOPTANK
					CALVERT		CALVERT
	OLIGOCENE	a b		s e		n t	
	EOCENE	PINEY POINT C RANCOCAS A		PINEY POINT CHICKAHOMINY		CHICKAHOMINY	
				PAMUNKEY	NANJEMOY	PAMUNKEY	NANJEMOY
					MARLBORO		MARLBORO
					AQUIA		AQUIA
	PALEOCENE	B			BRIGHTSEAT	MATTAPONI	
CRETACEOUS	UPPER CRETACEOUS	MONMOUTH	RED BANK	MONMOUTH			
			MT. LAUREL - - NAVESINK				
		MATAWAN	WENONAH	MATAWAN			
			MERCHANTVILLE				
		MAGOTHY		MAGOTHY			
		POTOMAC		POTOMAC	RARITAN		
	LOWER CRETACEOUS				PATAPSCO	POTOMAC	PATAPSCO
					ARUNDEL		
					PATUXENT		PATUXENT

CONVENTIONAL GEOLOGIC CORRELATION
of the
CRETACEOUS, TERTIARY and QUATERNARY

FIG. 1

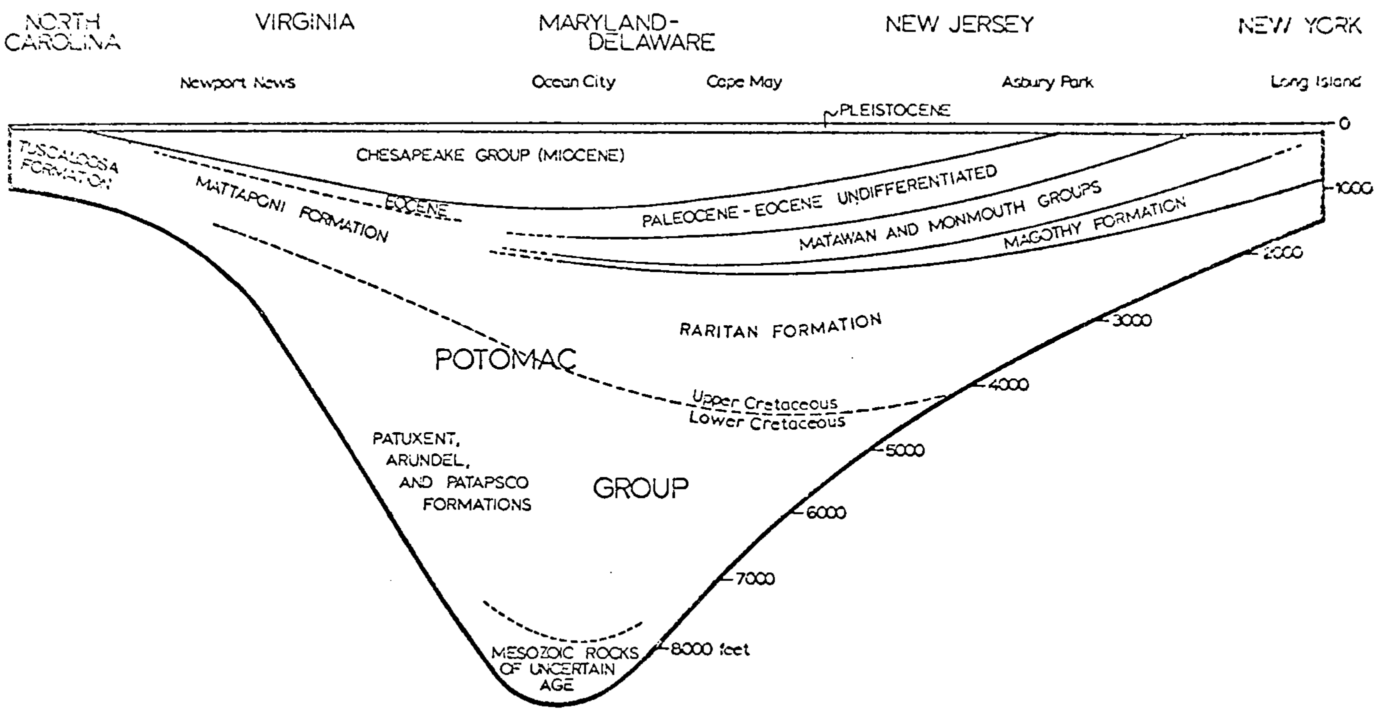

Figure 2. North-south diagrammatic cross-section of the Chesapeake-Delaware Embayment showing probable startigraphic arrangement of basin fill (modified from Glaser, 1967, fig. 15).

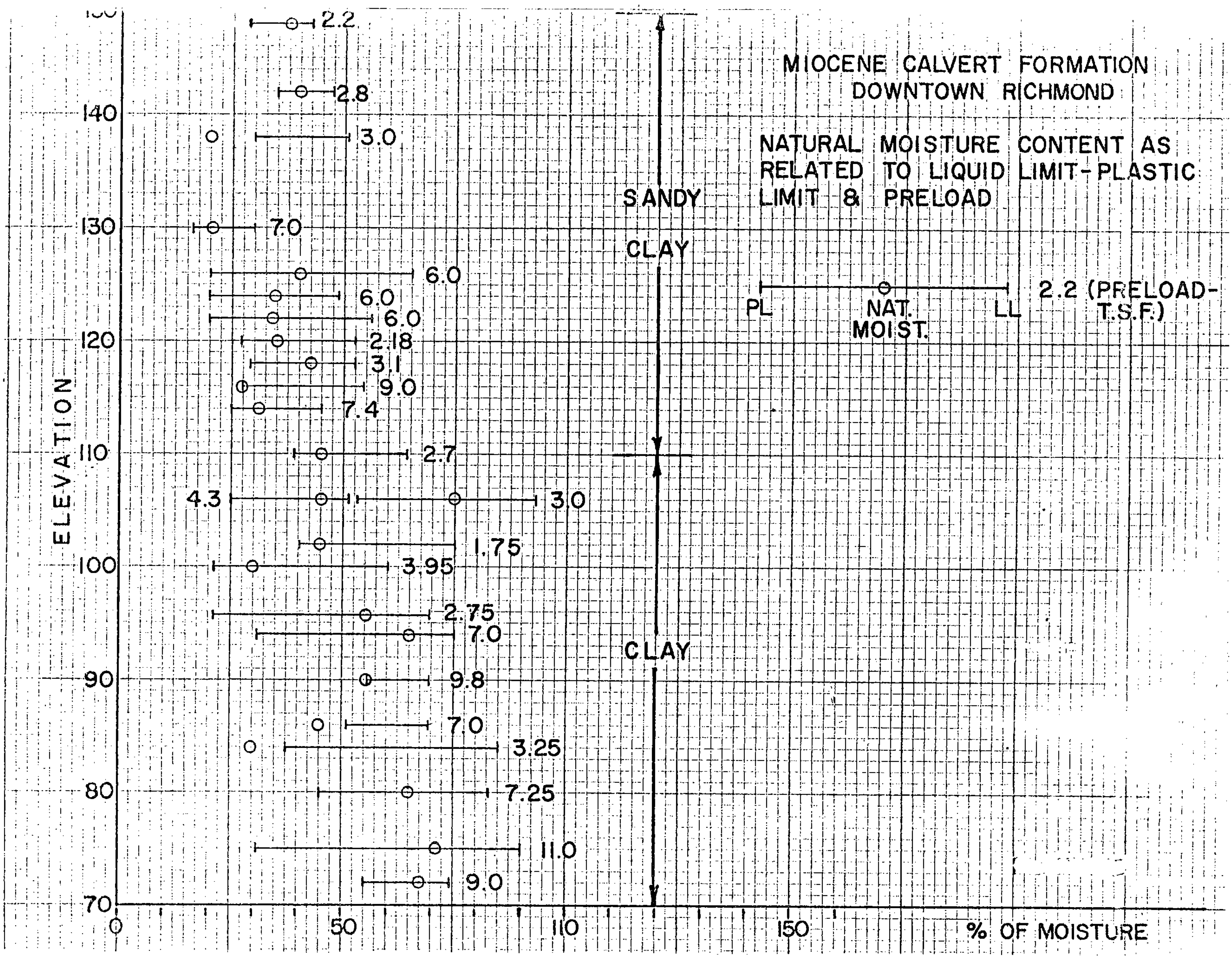

Figure 3. Miocene Calvert Formation (downtown Richmond) — natural moisture content as related to liquid limit-plastic limit and preload.

REMOTE SENSING APPLICATIONS TO NEAR SURFACE GEOLOGY

by

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Environmental Systems Corporation
Knoxville, Tennessee

INTRODUCTION

Remote sensing is a very loosely defined term covering a great variety of techniques by which data are collected through observations at some distance from the object being observed. Some examples, illustrating the diversity of methods, include traditional aerial photogrammetry, magnetic surveys, all types of specialized photography, line scanning techniques, optical radiometry, radar, and even visual observations by the human eye. Certain types of laser techniques as well as acoustic measurements also would fit the definition.

This paper deals with recent advances in the "state of the art" in remote sensing as a tool for the analysis of surface and near surface geological characteristics. Near surface geological information so obtained has obvious value in problem areas such as highway corridor location and design, industrial site development, foundation studies, land fill sites, reservoir or sewage lagoon sites, dam sites (including leakage analysis on existing dams), and more general large area terrain analysis.

For highway corridor studies, the remote sensing technique is of greatest utility for early reconnaissance and analysis. Well designed reconnaissance surveys over terrain amenable to remote sensing analysis can provide information which will assist in a more useful choice of drilling or seismic locations, minimize the chances of missing potentially troublesome areas, provide a complete synoptic coverage of the area, and thereby reduce the overall costs of the engineering geology studies.

If extensive drilling has already been completed or if site preparation is already underway, the chances of discovering much "new" information are greatly reduced. Also, the present "state of the art" makes the technique more useful for reasonably open terrain. Reliable data analysis over mountainous or heavily wooded terrain at present is quite difficult, but techniques are improving rapidly.

The remote sensing techniques should not, at present, be thought of as a replacement for traditional ground based methods such as drilling, seismic tests, etc. Rather, they are supplemental tools which make the overall analysis more effective and thereby less expensive.

This paper deals with the use of photographic techniques in the visible and near infrared region, and thermal mapping techniques in the thermal infrared region. The reason is that equipment and operational costs now place these in a cost range such that they are economically acceptable for current applications. Side-looking radar also is quite useful but at present is not economically attractive for smaller sites such as a few miles of highway corridor or a few hundred acres of an industrial site.

TECHNICAL EQUIPMENT AND OPERATIONAL METHODS

Figure 1 is a chart illustrating the regions of the electro-magnetic spectrum which can be dealt with by various forms of remote sensing. For the purposes of this paper, the photographic region will be defined as the visible spectrum and the near infrared out to approximately 0.9-1 microns. Camera and film combinations now available do not allow for photography out into the "thermal infrared" regions at longer wavelengths, and there are fundamental reasons why these probably will not be available for practical operations in the field in the future.

The photographic data are useful in two ways: (1) pattern recognition and analysis, including qualitative topographic information since the photography is generally flown with 60% forward lap between frames; and (2) color composition analysis from both the visible and the "false color infrared." Since most users already are familiar with category (1), the discussion will be oriented primarily towards category (2).

In order to "sort out" the color content information from the terrain, multiple cameras and films and filter types generally are used simultaneously in combination. Several special camera designs with from four to nine lenses are available. By a proper choice of filters, various portions of the photographic spectrum can be imaged as separate frames on the film. For reasons of cost and operational flexibility, we have chosen to use individual 70mm Hasselblad cameras, which are bore-sighted and synchronized and operated from a standard aerial photographic ring mount in the aircraft. The camera generally used is the Hasselblad 500EL electric camera. Interchangeable lens

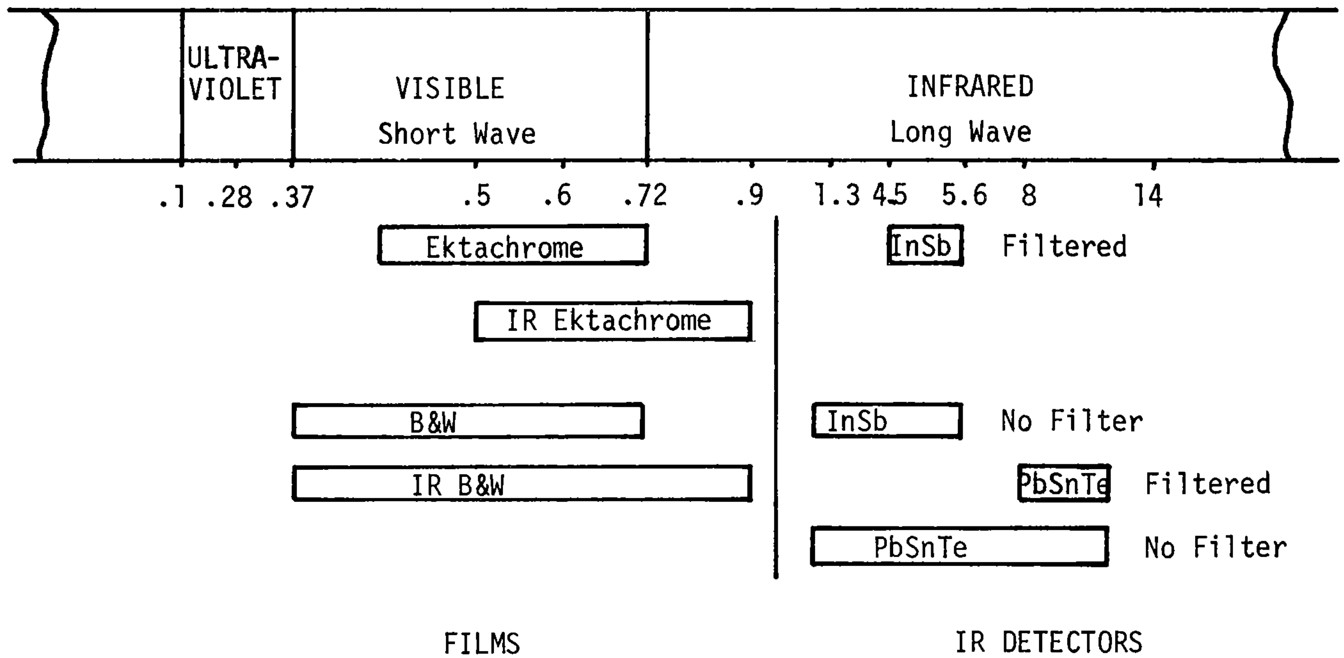

Figure 1. Useful Spectral Regions for Photography and Thermal Mapping.

focal lengths from 80mm to 250mm have been used. We also utilize the Hasselblad SWC camera with a 38mm lens for wider angle coverage. For a given survey altitude, the 38mm lens provides approximately the same area coverage as does a standard 9" mapping camera with 6" lens but with a different film scale, of course. We have not found the different film scale to be a handicap in data analysis. On the other hand, the use of the 38mm wide angle lens does allow a reduction in the overall amount of flying and thereby the overall costs of the remote sensing operation. Motorized SWC cameras are not now commercially available so special operational procedures are required.

For a four-camera operation , the most frequently used combination is as follows:

- Ektachrome (false color) infrared
- Black and white infrared
- Aerial Ektachrome
- Aerocolor (color negative)

The black and white infrared and the Aerocolor are flown to provide an economical source for subsequent prints if needed. For direct interpretation from the original film, the Ektachrome infrared and the aerial Ektachrome usually are preferred.

The thermal mapping (infrared line scanning) is conducted with electronic thermal mapping equipment. By means of rotating optics in the mapper in the aircraft, sequential lines are scanned on the terrain in a direction perpendicular to the flight path of the aircraft. The scanning rate is high enough so that contiguous coverage along the flight path is obtained.

The optical resolution obtainable is dependent upon both the optics and the electronics of the system used. Typical figures range from about 1.5 to 10-15 milliradians, i.e. 1.5 to 10-15 feet on the terrain per 1,000 feet of survey altitude above terrain. Differential temperature sensitivity is dependent primarily upon the detector type used as well as the overall electronic characteristics of the system. Typical differential sensitivities are approximately 0.5°F.

The signal (thermal energy emitted from the terrain as a function of both temperature and emissivity) is detected in the thermal mapper and converted to an electrical signal. The signal can then be presented directly to black and white film in a camera cassette (direct record) as part of the thermal mapper. Alternatively, it may be recorded on magnetic tape and subsequently processed in the laboratory to produce the black and white thermal imagery on film.

We believe that the magnetic tape record method has major advantages in that it allows for a variety of special data processing methods with minimum cost and effort. These special data processing methods are quite important for useful terrain analysis.

It should be clearly understood that the analysis of the thermal imagery of the terrain is based upon thermal pattern analysis and not upon true surface temperature analysis. (There may be occasional very special applications where true temperatures are required.) The acquisition of "true" temperature data by thermal mapping of terrain is quite difficult, as well as generally unnecessary. The difficulties lie in the varying emissivity of the terrain and its ground cover, and also in the atmospheric effects between the terrain and the aircraft. The latter can be substantial, amount of several °F on typical humid days in the eastern United States.

Figure 2 illustrates thermal imagery obtained in the spring of 1970 at a survey altitude of 2,000 feet over karst terrain in upper East Tennessee. The mottled area is one containing sub-surface cavities, as confirmed by core sample data. In this particular area, the depth of the overburden varies from 0 to approximately 75 feet. The thermal mapping technique does not, of course, "penetrate" the surface. The surface temperature effects seen here are a by-product of the differential moisture and drainage characteristics which in turn are related to the sub-surface cavities.

Vegetative stress as well as variations in soil moisture (the latter seen only in open terrain) can be observed in the camera photography and correlated to the thermal imagery. In addition, the stereo coverage with cameras allows one to "sort out" the topographic features, such as depressions. This sorting out cannot be done from the thermal imagery since it does not provide stereo coverage.

In addition, the metric quality of thermal imagery generally is not as good as that of photography. Consequently, the photography provides a more accurate means for tying in the overall remote sensing data to base maps.

It must be understood that infrared photography and infrared thermal mapping are two completely dissimilar techniques, they provide basically different types of information, and there are differences in interpretive approach. The two methods used together, however, provide a powerful tool for surface and near surface geological analyses.

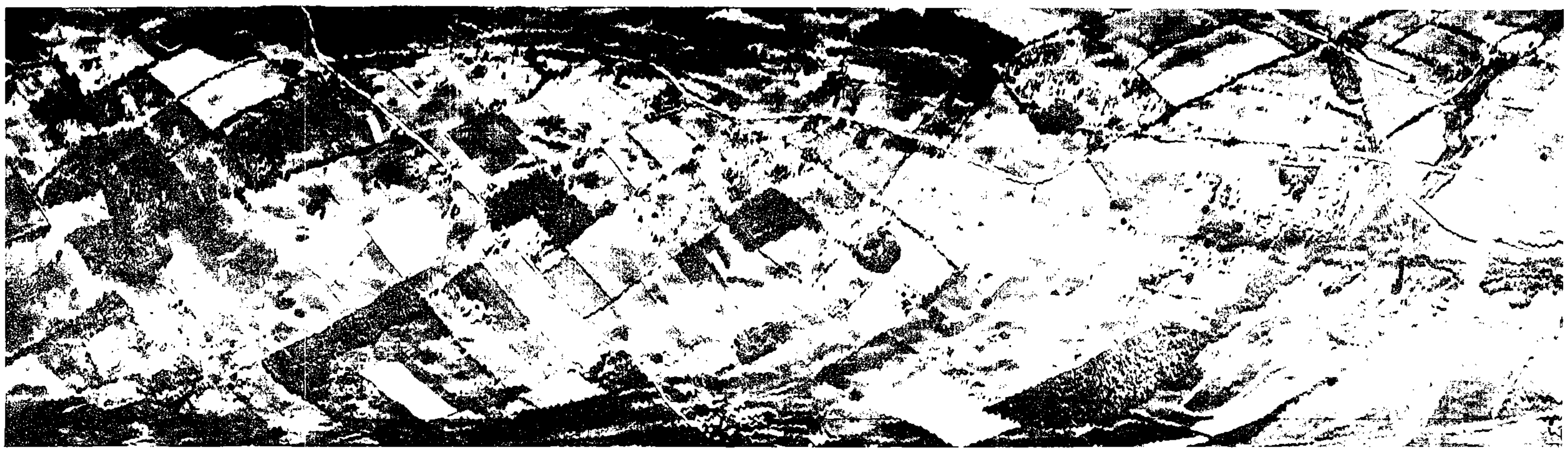

Figure 2. Thermal Imagery Showing Effects of Sub-Surface Cavities in Karst Terrain.

ILLUSTRATIVE RESULTS

Several illustrative applications will be briefly summarized here. Figure 3 is a sample of thermal imagery obtained over the southwestern end of Tobago in March 1971 as part of a large area resort development study[1] conducted by the Environmental Systems Corporation. The terrain is generally low lying coral limestone, partially covered with older cocoanut plantations. The area shown is approximately 4 miles in the horizontal direction. In the area immediately below the "cocoanut plantation label" there are darker (cooler) areas which are indicators of retained soil moisture and poorer drainage. A similar situation exists in the western area around the perimeters of the marsh. The thermal mapping can thus be used as an indicator of soil moisture and drainage by virtue of the higher heat capacity of wet soil as well as evaporative cooling which occurs at the surface of wet soil. Depending upon the nature of the geology, these thermal indicators can be used to infer information regarding sub-surface cavities, fault lines, and other features which have a bearing upon drainage.

Figure 4 illustrates a special approach to thermal mapping data processing in which the thermal imagery has been "contoured." In normal thermal imagery, there are continuous gradations of the gray tones which are indicators

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1 "CASE STUDY - REMOTE SENSING APPLICATIONS IN RESORT DEVELOPMENT," by Carl O. Thomas, Presented at the Seminar on "Planning and Engineering in the Coastal Zone," Charleston, South Carolina, May 19, 1972.

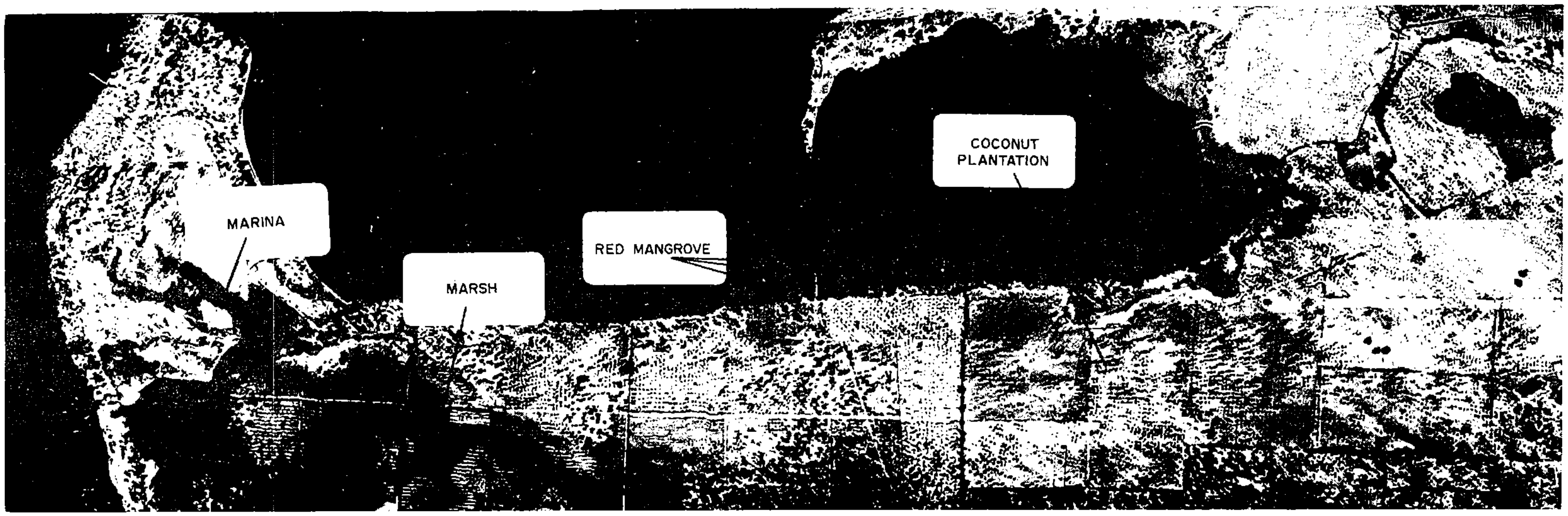

Figure 3. Thermal Imagery From Southwestern Tobago.

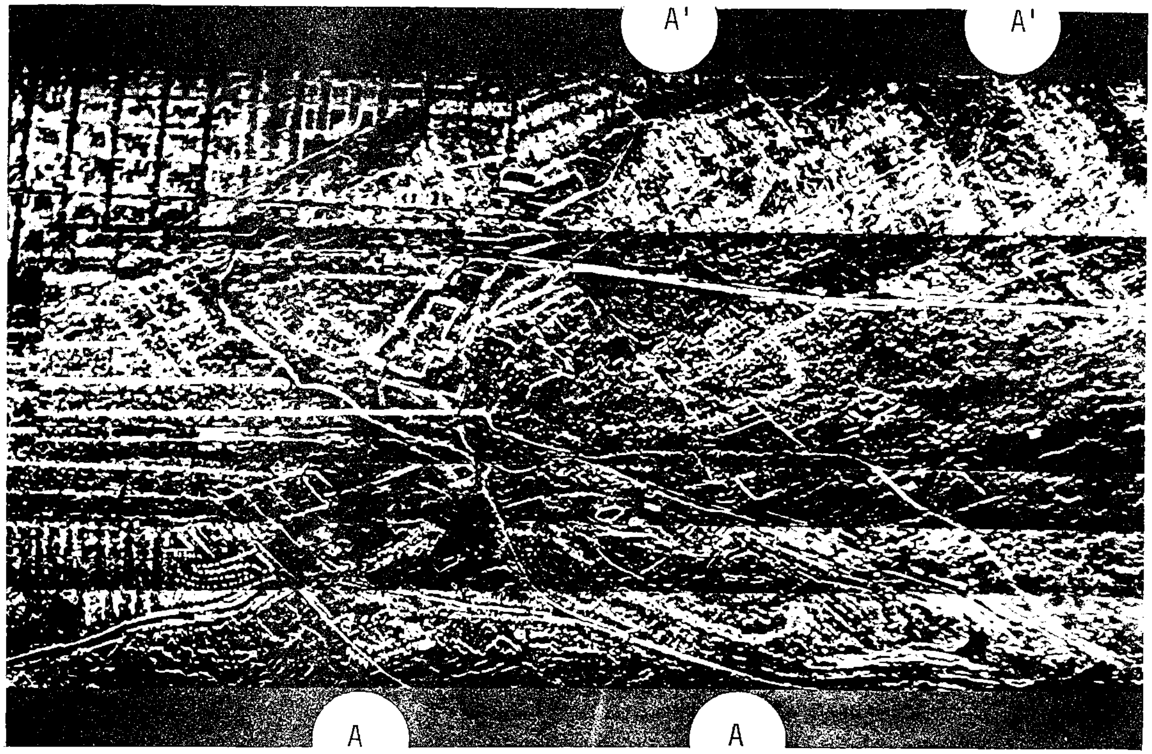

Figure 4. Contoured Thermal Imagery
Showing Fault Line AA'.

of temperature gradations. In the contoured imagery, a digital processing technique is used to indicate various discrete temperature levels. Thus, there is a partial analogy to photographic contour mapping but in this case the contours are related to temperature rather than to vertical relief. In this contoured imagery, one sees a speckled background resulting from localized temperature variations caused by trees, etc. A pattern analysis of contoured imagery can, however, give clues to the presence of fault lines and other subsurface features. For example, such a fault line is shown at AA' in Figure 4.

The ability to carry out special data processing operations such as this illustrates one of the advantages of magnetic tape recorded data in thermal mapping.

Another example of an application in the southeastern United States was reported by Hensey and Barr.[2] In that study, data interpretation from remote sensing operations by Environmental Systems Corporation was compared against detailed ground based information obtained independently. The authors reported: "Approximately 85% of the inferred solution zones were confirmed by borings. The cost of the remote sensing and interpretation was about 10% of the total cost of all site investigations and provided data of value well in excess of its proportionate cost." These results clearly indicate the potential technical and economic values of a properly designed airborne remote sensing study program.

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2 "PRELIMINARY EVALUATION OF AN INDUSTRIAL PLANT SITE USING REMOTE SENSING TECHNIQUES," by Dr. David J. Barr and Mr. Melville D. Hensey, Presented at the 38th Annual Meeting of the American Society of Photogrammetry, Washington, D. C., March 12-17, 1972.

SUMMARY

Airborne remote sensing techniques, particularly thermal mapping and specialized forms of photography, are now within the economic reach of study programs covering only a few square miles of area. The data acquisition and analysis techniques are sufficiently advanced to make these a useful tool for preliminary studies of the surface and near surface geology in highway corridors, industrial sites, and similar locations. Data interpretation is quite difficult in mountainous or heavily wooded areas, but for a substantial portion of the geography where development is underway, the techniques have present value.

ECOLOGICAL IMPACT ON HYDRAULIC CONSTRUCTION METHODS IN GEORGIA

By
David A. Mitchell, Soils Engineer
And
Craig C. Brack, CE II

Interstate 95 was located through Georgia to service coastal cities with a minimum of connector route miles. The basic location was established between 1955 and 1957 prior to the Nation's present concern over the environmental impact of highway locations. If the location had to be selected under today's ground rules the route would undoubtedly be shifted inland five miles or more for there just is not any way to build a highway through 60 plus miles of virgin marshland and wooded swamps without seriously altering that environment. Due to public opinion, we,like many of our sister states, are in constant pursuit of better "cleaner" methods for designing and constructing hydraulic dredging projects.

In the fifties, several hydraulic highway embankments were constructed along Georgia's coast without much concern for the environment. Damage to adjacent streams and marshes was heavy; however, construction prices were relatively cheap. The estimated cost ratio between trestle type bridge construction and hydraulic embankment construction in the fifties was approximately 10 to 1. Designing and constructing these early hydraulic projects was a snap compared to today's involved procedures.

Present Procedures

The Laboratory submits a request for the material to the Georgia State Mineral Leasing Commission, who has the authority to grant the material from State owned rivers and streams, if it feels that it is in the best interest of the state. Before the request is granted, the Georgia Marshlands Protection Agency is consulted. The Highway Department must convince this agency that the proposed method of constucting the project will result in the least damage to the marshlands.

If the proposed borrow areas are located in the marshes, the Highway Department deals directly with the individual property owner. Borrow areas in marshes are set-up only as a last resort since this would not only destroy the actual marsh area of the pit, but the unsuitable material usually overlying these marsh pits will require additional areas for its deposition, also.

After the borrow areas are secured, the Laboratory applies to the Georgia Water Quality Control Board for a certification stating that it has received reasonable assurance that the proposed activity will not violate Georgia Water Quality Control Standards. This certification is required by Public Law (91-224), which states that a certification such as this will be included in any dredging permit application made to the Corps of Engineers.

This board does not attempt to design the pollution control devices, but puts the burden of design on the Highway Department and only checks the Highway Department's design to satisfy its engineers that the proposed pollution control devices are adequate.

As a sidelight, it may be interesting to note that this board's engineers also act as consultants to the Highway Department in pollution problems. This agency has been very cooperative and has helped the Highway Department solve many problems dealing with pollution control.

Usually at this point, the Highway Department arranges a meeting with Ecologists from the University of Georgia. Before this meeting, the Ecologists are flown over the proposed construction area for a firsthand look at the the proposed roadway, borrow pits, and waste disposal areas. Afterwards they give their opinions as to the relative values of the land the Highway Department proposes to use. If at all possible, the Department will relocate any areas that the Ecologists find objectionable to less productive areas.

After we secure our Ecologists' approval, we submit an application to the Corps of Engineers for permission to construct the proposed project. The Corps of Engineers advertises the request and solicits objections to the proposed work. The advertisements are sent to U. S. Government Regional and State Agencies, individuals and organizations who have expressed interest in similar projects. After all of the objections are reviewed, the Corps of Engineers evaluates the probable impact of the proposed activity and either grants a dredging permit or rejects the application.

This concludes the permission and permit phase of the project and it is now ready to be let.

Before construction begins on a project, samples are taken at various points in the vicinity of the project and a background level of certain pollution is determined.

We attempt to establish background levels on all pollutants listed in the Corps of Engineers Permit attachment entitled Water Quality Considerations for Construction and Dredging Operations. Specifically these are:

(a) Volatile Solids

(b) Chemical Oxygen Demand

(c) Nitrogen

(d) Oil-Grease

(e) Mercury

(f) Lead

(g) Zinc

We also establish background levels on turbidity and suspended solids. Since most of these tests require expensive laboratory apparatus, all preliminary samples are tested in the Central Laboratory. After the project is let, the dredging inspector tests for turbidity in the field.

The remainder of the samples are tested in the Laboratory and results are reported back to the dredging inspector.

We feel that with the turbidimeters located on the coast that the dredging inspector has the most needed piece of equipment to control the job at his disposal. It has been our experience that turbidity or suspended solids not settling out of the water used in dredging before it is returned to the stream has been our greatest pollution problem. Next in frequency is oil and grease pollution. Most dredgers will pump out the oil that accumulates in the bilge of the dredges directly into open water. Property owners in the vicinity recognize and associate this with dredging quicker than a silted stream.

If any of the parameters exceed those listed in the C.O.E. Permit, the dredging inspector will shut the contractor down until he feels that the contractor has improved his system in such a way that the effluent will meet the standards.

One of the most promising is the side-cast method of removing unsuitable material from beneath the roadway. The contractor uses a large dragline mounted on a barge to remove this material and side-cast it beside the removal trench. This method eliminates large settlement ponds necessary in hydraulic removal methods. Usually the contractor will dig into the area and build a plug behind him so that he is isolated from the natural stream.

In all respects, we have found this method to be better than any other previously used, and hope that we will be able to require it on future projects.

These improved methods of hydraulic embankment construction do not come cheap. The increased cost in unit prices for unstable material excavation and in-place-embankment reflects the rising cost of hydraulic construction. Graphs prepared show this increase for Interstate 95 projects let since 1966.

Cost estimates for trestle type bridge construction versus hydraulic embankment construction for one project showed bridges would cost only $3\frac{1}{2}$ times as much. Still, even this is a significant savings of tax dollars on just the initial cost of the project. Hydraulic embankments constructed inside the barrier islands along Georgia's coast have given good service and have cost considerably less to maintain than trestle bridges. In many instances bridges constructed at the same time have already been replaced.

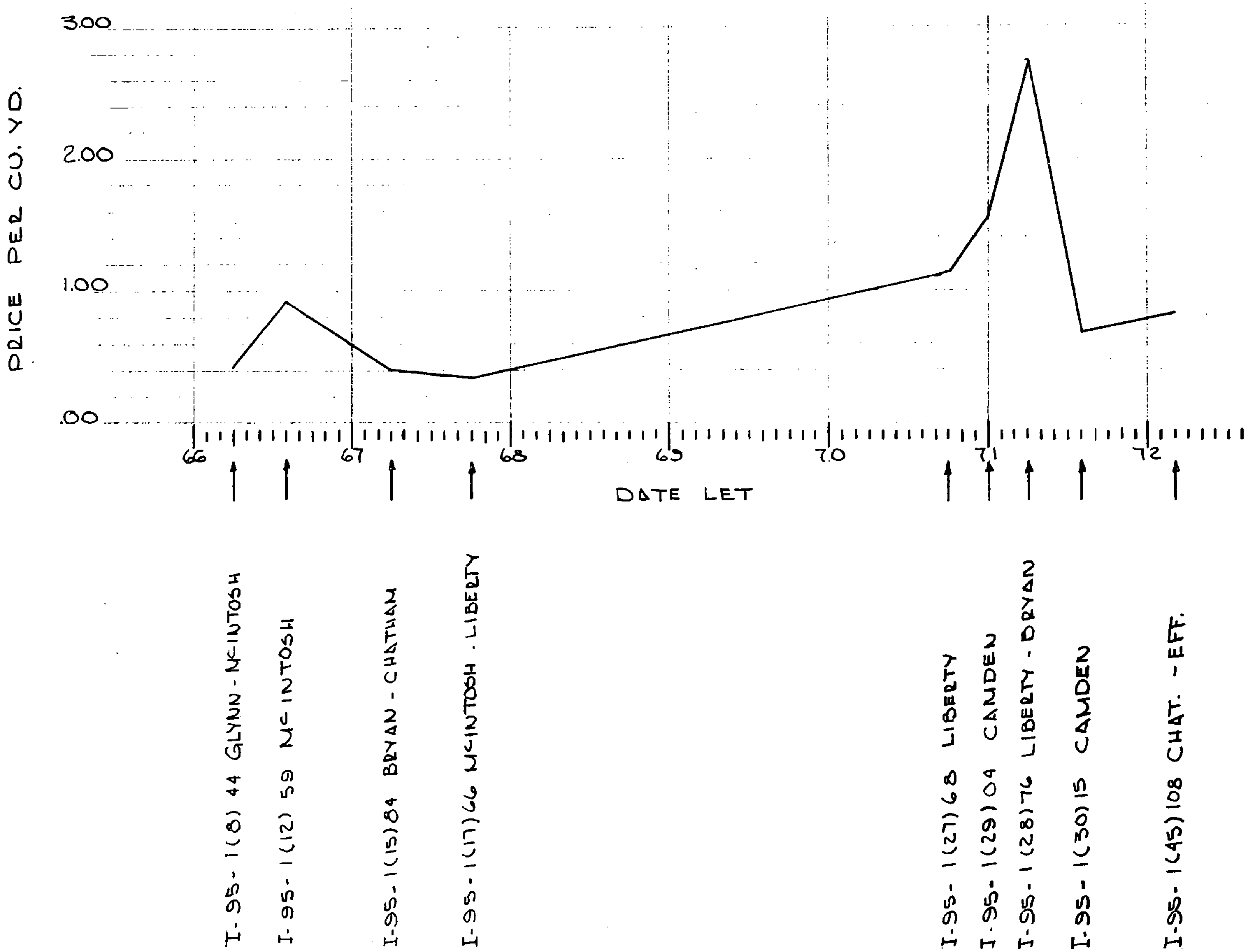
In our opinion, hydraulic embankment methods of construction are here to stay and it is our job as engineers and geologists to work increasingly closer with ecologists to design these projects as nondamaging to the environment as possible.

From past experience, we have learned several factors which seem to improve the economics of our dredging projects. Some of these that may be worth mentioning are as follows:

(1) We usually obtain better bid prices when letting the dredging portions of our projects in a separate contract. This allows dredging contractors to be the prime contractor and bid direct.

(2) We usually obtain better bid prices when our borrow pits are located in the rivers rather than in marsh areas which require stripping. In some cases dredging contractors would rather pump three times as far to avoid stripping marsh pits.

Figure 1
UNSTABLE MATERIAL EXCAVATION



(3) We usually obtain better bid prices when we are liberal with the size of waste disposal areas. Contractors are of the opinion that the larger the area, the less problem they will have producing a clear affluent.

(4) We usually obtain better bid prices when we have borrow areas that can be worked with dredges having ladders either 35 or 60 feet deep. Many times our pits are capable of producing nearly twice the required amount if worked by deep ladder dredges.

(5) We find that it is best to show unstable material as it occurs and not set removal grades. This allows the contractor to use his own innovations.

(6) To avoid overruns in unstable material or "muck" excavation we try to maintain close field supervision. It is difficult for inexperienced field personnel to determine precise limits for removal. We have trained an expert in this field and he is available to check questionable removal areas within 24 hours notice.

(7) We avoid removing unstable material below 25 feet whenever possible. Partial removal and surcharges are usually more economical in areas underlain by deep, soft, compressible strata.

In summation, I would like to say that it is not our intention to try to sell hydraulic construction methods. The engineering risk involved is such that each individual highway department should decide which construction method is best for their particular project. We can say that hydraulic construction methods have been economically justified along Interstate 95 in Georgia and millions of tax dollars have already been saved by utilizing this construction method.

INVESTIGATION OF DETERIORATION IN CONCRETE ROADWAY SLAB OF THE ROBERT E. LEE BRIDGE RICHMOND, VIRGINIA

Burrell S. Whitlow*
Geologist
Geotechnics, Inc.
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ABSTRACT

The investigation to evaluate the degree, extent and probable future development of deterioration of the concrete in the Robert E. Lee Bridge deck was authorized in late August, 1967. Field work was started in early October, 1967 and the final report delivered in May,1968. The investigation included the taking of core borings in the deck and selected piers, microscopic examination of polished surfaces and thin sections, chemical analyses, x-ray diffraction tests, expansion test of the cores and measurement of sidewalk expansion joints. Record photographs were taken of both temporary and permanent deck surface patching, significant features of railings and piers, and each drilled core immediately after its removal from the bridge deck.

It was concluded that alkali-silica reaction was the initial cause leading to the present state of deterioration of the bridge deck. Cracks observed in 1942 probably resulted from expansion caused by alkali-silica reaction. Processes contributing to the distress were freeze-thaw action and loading and impact incidental to service. The degree of responsibility attrib-

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utable to each process could not be determined.

INTRODUCTION

Objective

The objective of the investigation was to evaluate the degree, extent, and probable future development of deterioration of the concrete in the bridge deck. Conclusions reached were based largely upon judgement, supported by observations and tests, and upon the ability of the investigators to reconstruct the causes of abnormal behavior from available records and presently observable features.

Procedure

Two hundred and six concrete cores were drilled for examination, analyses, or special testing. Petrographic examination of polished and thin sections was performed in addition to physical, chemical and other special tests to provide supporting data.

Expansion joint openings in the sidewalk portion of the bridge deck were measured and compared to the original openings as shown on the "As Built" drawings.

Photographs were taken of significant features and a record photograph of each core was made.

Description of the Bridge

The bridge spans James River and has a total length of 3,710 feet. It was constructed during 1933-34 and has a total width of 50 feet 4 inches. The structure consists of a south girder span; a south abutment span; 16 twin ribbed, open spandrel, reinforced concrete arches having a total length of 3,209 feet; and a series of reinforced concrete beams and girder spans totaling 421 feet. The slab between arch ribs is about 7 inches thick; the cantilevered portion about 9 inches thick.

CONDITION AT INITIATION OF INVESTIGATION

At the initiation of the investigation, the condition of the deck may be summarized as follows:

A coal tar epoxy coating had been in place about 2 years. The coating was missing in a number of areas, having failed to adhere to the concrete deck. However, the epoxy coating seemed to be intact over most of the deck.

City maintenance personnel made the following comments regarding repair work observations.

> "The condition of the concrete below the bottom of the patches varies. Some is sound; some is easily removed below the lower layer of reinforcing steel and removal was stopped to keep from breaking through the deck. The paste matrix is sometimes powdery; aggregates are loose or easily removed from their sockets; and an air hose (80 psi) will erode the matrix and permit the aggregate to be removed. The steel displayed only minor rusting, very little flaking, and deformations on the bars were still visible."

At the start of core boring operations, extensive repairs were being made using gunite with an epoxy coating. Several of the patches extended entirely through the concrete deck. Horizontal cracks up to $\frac{1}{4}$ inch wide were disclosed along the sawed edge of some patches. Numerous small cracks were in evidence along the edge of nearly all patch excavations. Horizontal cracking, in general, occurred above the top steel. Generally, the depth of cracking seemed related to the depth of steel. This observation was substantiated in numerous core borings throughout the deck.

INVESTIGATIVE METHODS

General

The basic investigative approach was a random pattern of test borings covering the approaches and arch spans. The average number of core borings taken per arch span was ten.

Each core hole in the bridge deck was logged and the recovered core photographed as a permanent record of its condition at the time of recovery. Cores were stored in the field laboratory awaiting immersion testing and microscopic examination.

Core Boring Operations

Drilling equipment was truck mounted and self-contained. The heavy traffic flow required that the entire unit take up a minimum of bridge space and that all equipment be sufficiently mobile to permit drilling in short increments, if necessary, between rush-hour traffic flows. No equipment was permitted on the bridge overnight.

Logging of Core Holes

Each core hole was logged immediately after completion of the boring. This was done to secure an accurate deck thickness at the boring location since the deck varied in design thickness as well as workmen's tolerance. The depth of steel, depth and description of cracks, unusual features (e.g. rust stains, presence of fractured zones, loose aggregate, or dirt along fractures) were noted on the log.

Examination of Cores and Immersion Testing

Immersed cores were carefully examined stereoscopically. Aggregates exuding gel were marked and recorded; the descriptive term "many" was applied where the number of exuding particles exceeded 20. The purpose of the immersion testing was to determine the abundance of reactive particles exposed on the drilled surface of individual cores and the location and number of affected cores in the deck.

Sample Preparation

Thirty-eight cores were sawed and finely ground for detailed microscopic study. Samples were cut with a Frantom slab saw and fine-grinding accomplished with a Sommer and Maca lapping machine.

Petrographic Examination of Polished Surfaces

Nineteen samples were selected representing the best (highest recovery) cores, one from each arch span, south abutment span, girder span, and north approach. These were sawed parallel to the long axis of the core and the surfaces were finely ground. Features relating to the aggregate, paste, microfractures, secondary deposits, evidence of alkali-silica reaction, and reactive particles were noted and recorded during the petrographic examination.

Subsequently, 19 additional core samples were selected and prepared for routine petrographic examination. These cores represented the poorest (least recovery) cores and came from the same spans as the previous group.

Petrographic Examination of Microscopic Thin Sections

Thin sections were made from portions of three cores, the polished surfaces of which had been previously examined in detail using a stereoscopic microscope. Features of interest, such as air voids containing secondary deposits and aggregate particles displaying evidence of alkali-silica reaction, were noted. The examination revealed the characteristics of the cement paste matrix, the composition of the fine and coarse aggregate, evidence of alkali-silica reaction, occurrence of microfracturing in the concrete, and the presence of secondary chemical deposits in voids and microfractures.

Expansion Test

A length comparator was fabricated for measurement of expansion in the bridge deck and pier cores.

Due to imperfections in cutting the cores, high and low readings occurred when the core samples were rotated in the comparator. All readings were made by recording the high's and low's and averaging the result thereof.

SUMMARY OF DATA

General

Data obtained during the investigation are presented in this section to indicate their significance and permit a clearer understanding of the cause and effects of the deterioration.

Horizontal Breaks

Seventy-two percent of the cores showed the presence of two or more breaks. The location of breaks was primarily above the top steel and below the bottom steel of the deck. Occasionally, the entire zone surrounding the steel was weakened and recovery consisted of a mass of concrete fragments and the steel.

Presence of rust

Each core and/or fragments thereof was examined closely for evidence of rust, rust stains, or migration of iron compounds. Pitting of the reinforcing steel, discoloration of the enclosing concrete or the disposition of iron oxide along small cracks or fractures was considered positive evidence. Rust coatings formed on the bare surface of scratched, drilled, or abraded steel were not counted since the steel probably rusted as a result of exposure to air and drill water. Migration of iron was observed in a few cores as deposits along fractures extending outward from the steel from a few millimeters to an inch or more. Thirty-nine (39) percent of the deck cores exhibited some rusting.

Badly broken core

Cores were considered badly broken if the recovered core consisted entirely of broken fragments or if one or more zones of badly broken material occurred between zones of coherent material. Cores considered badly broken were usually accompanied by significant core losses. Sixteen percent of the cores recovered were badly broken.

Epoxy surface coating loose

The condition of the epoxy surface coating was observed at all boring locations and its adherence to the cores noted. Since the epoxy had been applied as a sealant and protective coating, it was thought that its condition might show some correlation with the present condition of the bridge deck.

No such correlation was found. Only slightly more than 6 percent of the cores had loose epoxy surfacing. On a per span basis the number of cores having loose coating ranged from none to 2; they were evenly distributed throughout the bridge length and showed no correlation with the other significant features.

Core recovery, less than average

The average recovery for all cores in the bridge deck was 93.6 percent. Slightly more than 35 percent of the cores showed losses exceeding the above average.

Exudations

Immersed cores were examined for exudations of alkalic-silica gel. Sixty-three percent of the cores showed exudations. The number of exuding particles ranged from zero to more than 20 on core samples from 3 to 5 inches in length.

The presence of exudations was considered as a reliable indicator of remaining expansion potential in the cores themselves and the concrete from which they were taken. The absence of gel was not interpreted as a lack of potential, nor was the abundance of gel construed as indicative of the relative expansion potential remaining.

Expansion Test Results

All cores subjected to moist storage developed significant expansion. No correlation was found between the number of exudations present and degree of expansion developed. The range of expansion of samples was 0.04 - 0.09%. All figures are expressed as a percent of the core length at the end of 10 weeks in moist storage.

The most significant finding of the expansion tests was that all samples expanded appreciably and to a damaging degree. As a group, pier samples expanded more and in less time than the deck samples.

Expansion of Deck Slab

Measurements of sidewalk expansion joint openings in arch spans were made to gather information as to the probability of overall expansion of individual deck spans. The sidewalk joint openings were measured in lieu of the roadway joint openings because the roadway joints had previously been repaired and may not have been reset to original construction tolerances. It was fully realized that the original sidewalk joint openings may not have been set to plan dimensions but it was felt that errors were more likely to cancel than to accumulate.

The sum of measured openings at both ends of each arch span was less than the sum of the plan dimensions. This strongly suggests overall expansion has occurred, since it is unlikely that every pair of joints could have been erroneously constructed to less than the specified opening.

With regard to magnitude, the expansion based on joint measurements compares closely with the expansion measured in test cores after moist storage.

Petrographic Examination

The presence of alkalic-silica gel is usually a reliable criterion for establishing the occurrence of alkali-silica reaction, but thin-section examination is required to prove that the reaction has seriously damaged the enclosing concrete. Accordingly, thin sections were prepared and examined and photomicrographs were made of significant features.

Distinguishing features of the reaction observed in thin sections were clarified reaction rims, microfractures, the presence in voids of high-sulfate calcium sulfoaluminate (commonly called, ettringite), and evidence of internal carbonation.

Conclusion

It is felt the data and evidence herein submitted prove that alkali-silica reaction was the initial cause of a series of events resulting in the presently advanced state of deterioration of the bridge deck. Cracks observed as early as 1942 probably resulted from expansion caused by alkali-silica reaction.

The available data and related information regarding the history of development of the distress in the deck are consistent with the following interpretation: (1) Alkali-silica reaction involving chalcedonic cherts in the aggregate caused expansion of the concrete; the alkalies were supplied in part from the cement, in part from the hydraulic lime admixture, and in part from sodium chloride presently used as a de-icing agent on the deck. (2) Expansion of the concrete, being restrained, in part by internal reinforcement and at the edges of the roadway by the spandrel beams, caused several types of cracking; namely, pattern cracks in the surface of the deck, horizontal cracks related to the distribution of the reinforcement, and vertical cracks related to warping and flexure. Vertical cracks were noted in the spandrel beams in our previous investigation which excluded the deck. (3) The cracking of the slabs promoted important subsidiary phenomena of deterioration. Exposure of reinforcing steel along fractures permitted rusting to occur because the steel was no longer protected by the highly alkaline environment of the portland cement matrix. Water penetrated along cracks in the concrete and, with freezing, widened and extended the cracks. The cracking partially destroyed the bond between the steel and the enclosing concrete and thereby reduced the stiffness and load-carrying capacity of the slabs. This allowed differential movement between the concrete and the steel with consequent shattering of the concrete adjacent to the rods. The continual increase in the volume and weight of traffic over the years has aggravated the situation causing further damage to the deck in its already weakened condition.

While the degree of responsibility attributable to each factor could not be determined, it is presumed that freeze-thaw probably accounts for the disintegration of the near-surface mortar and for the raveling and cracking of the sidewalk and curbings at edges and corners. The fact that surface depressions presently develop in greater numbers during the winter months implies that they also may be more closely related to freeze-thaw.

It was concluded that the present condition of the structure is due to the interaction of several deleterious or damaging processes: increased loading and impact, freezing and thawing, and alkali-silica reaction; and, to a much lesser degree, carbonation of cement paste matrix and rusting of the reinforcement. Although alkali-silica reaction was the initial cause of deterioration, it cannot be stated unequivocally that it has been the major cause of damage. Once the concrete had expanded and become weakened by microfracturing, either freezing and thawing or heavy loading and impact could have become equally as important in the development of the present condition.

The deck concrete shows no evidence of significant deterioration caused by leaching or percolating waters, rusting of the embedded steel, attack by aggressive environment or the presence of iron sulfides in the aggregates.

The writer expresses his appreciation to Hayes, Seay, Mattern and Mattern and the City of Richmond for permission to use the data contained herein.

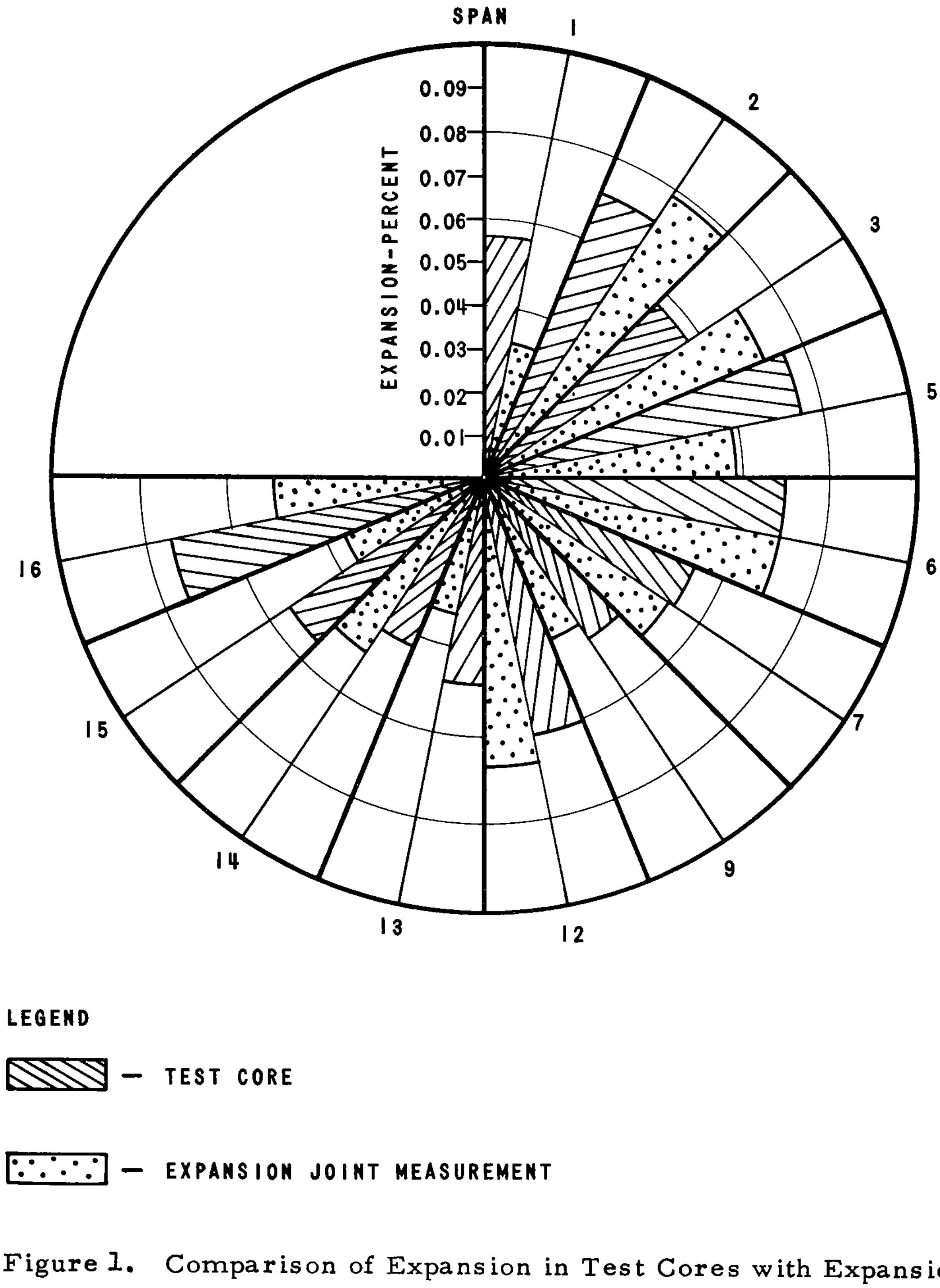
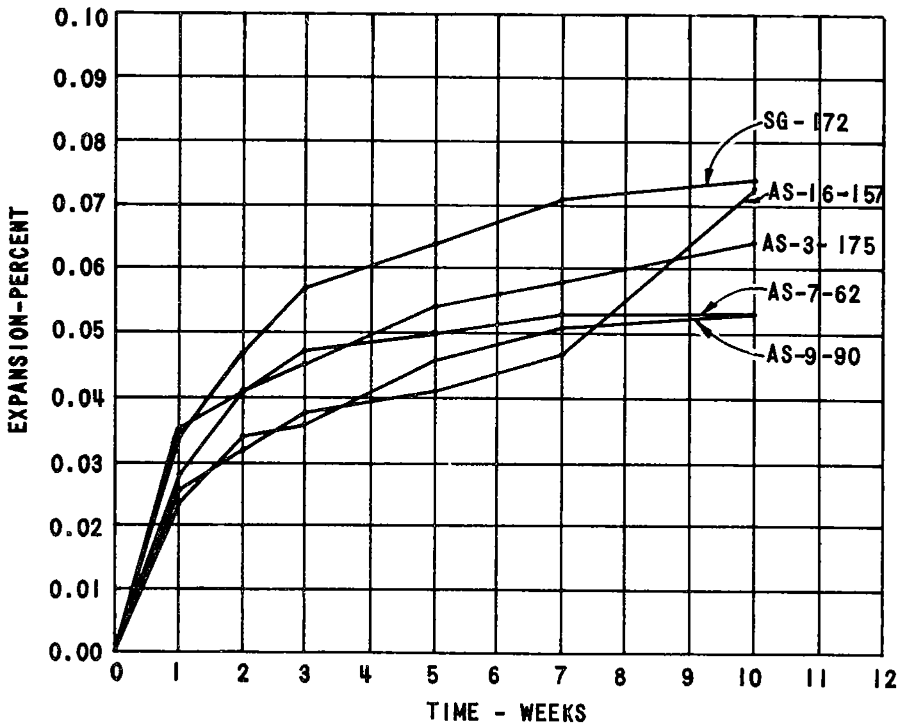
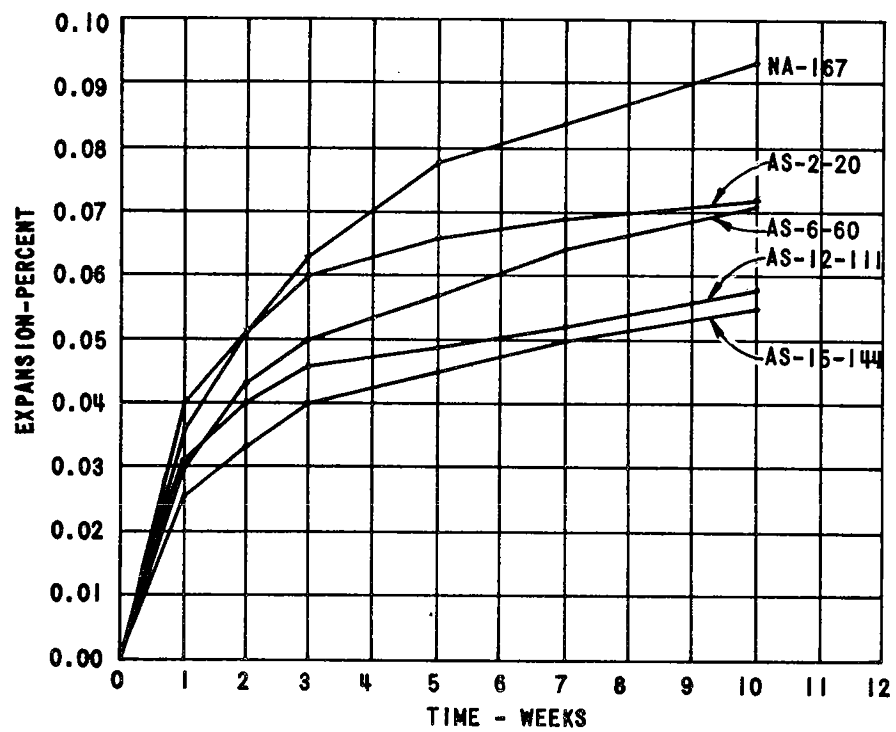


Figure 1. Comparison of Expansion in Test Cores with Expansion as Determined by Expansion Joint Measurements.



GROUP C



GROUP D

Figure 2. Expansion of Samples in Moist Storage.

Plate 1. Typical depression failure in concrete bridge deck.

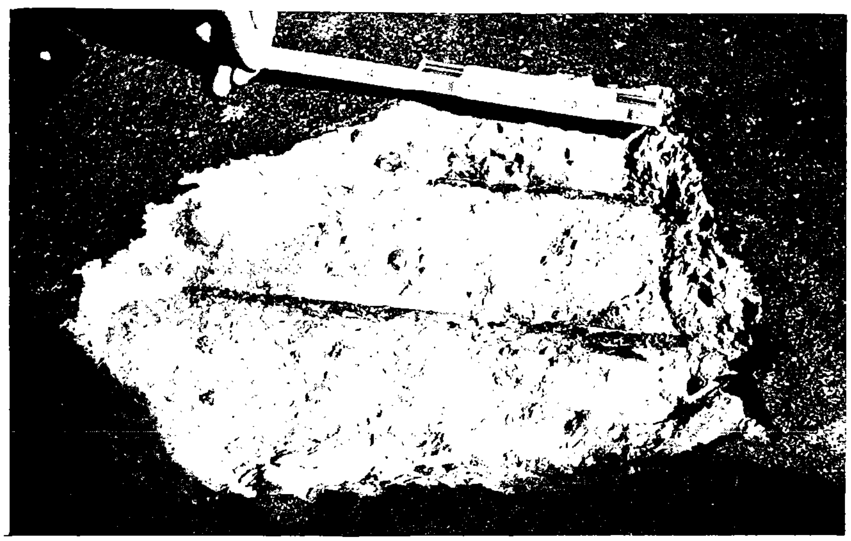


Plate 2. Typical depression shown in Plate 1 after excavation with air hammer. Note darker damp spots in bottom of excavation.

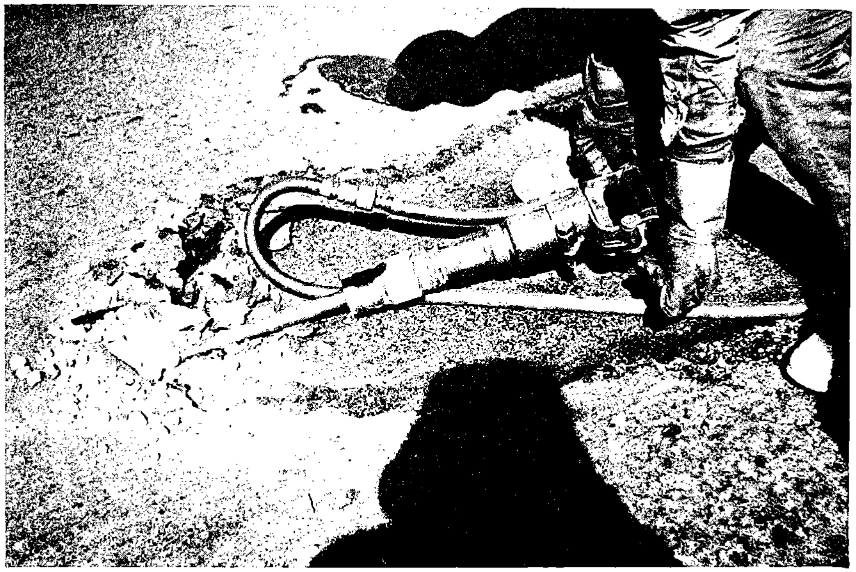


Plate 3. Excavation operation for repair of depression failure. Note failure occurred at periphery of existing patch. Two earlier patches are visible; an original patch and a previous peripheral patch.

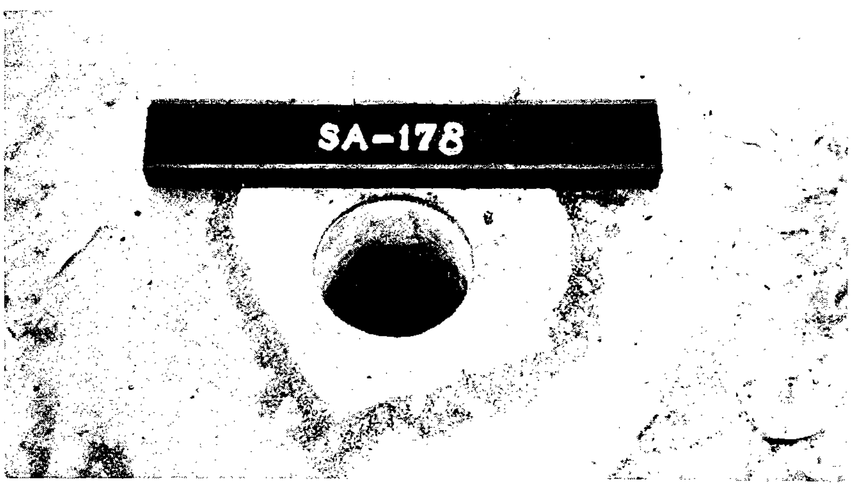


Plate 4. View of bore hole SA-178. Note two open fractures in wall of hole.

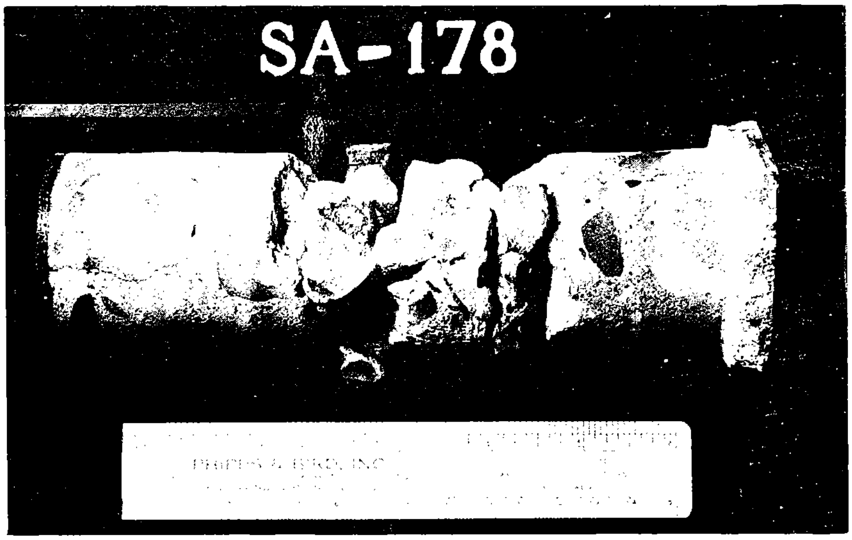


Plate 5. Vertical crack below epoxy surfacing and broken condition at steel. Plate 4 shows the hole from which this core was removed displaying horizontal fractures in the wall.

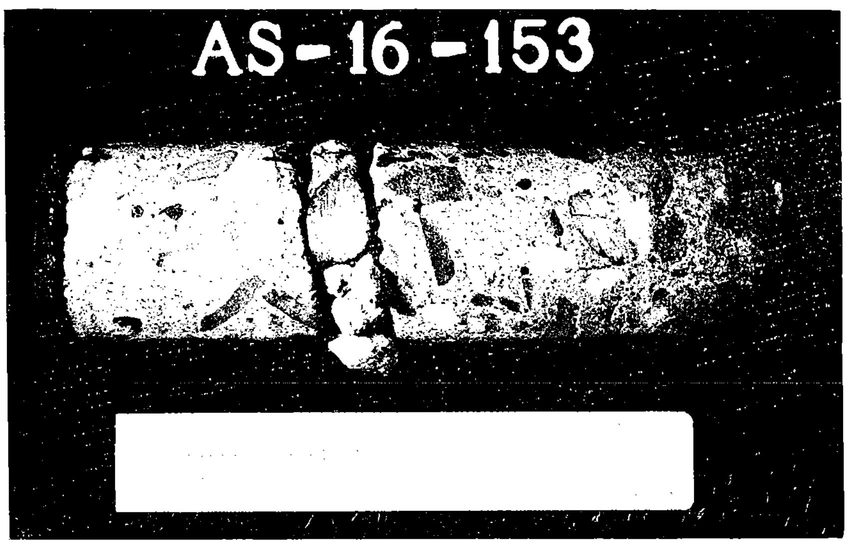


Plate 6. Broken condition at steel, vertical crack extending upward from bottom of core.

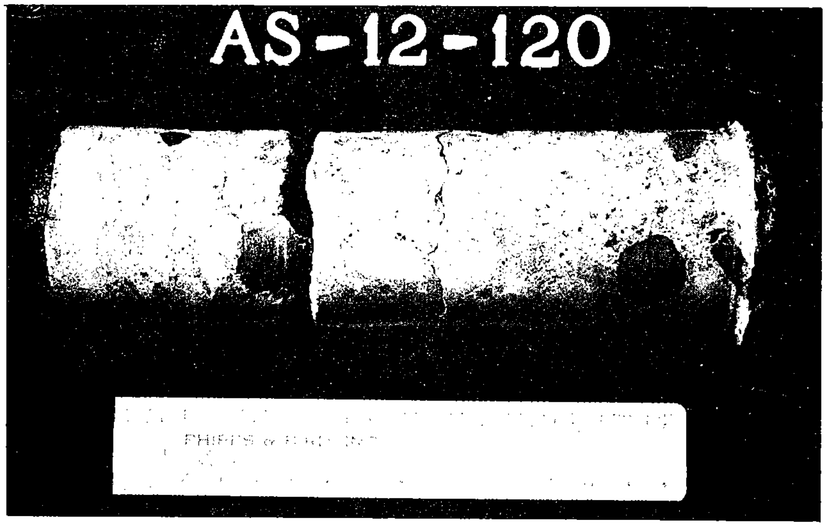


Plate 7. Crack above steel, weak zone at steel; break near middle of core passes around aggregate particles.

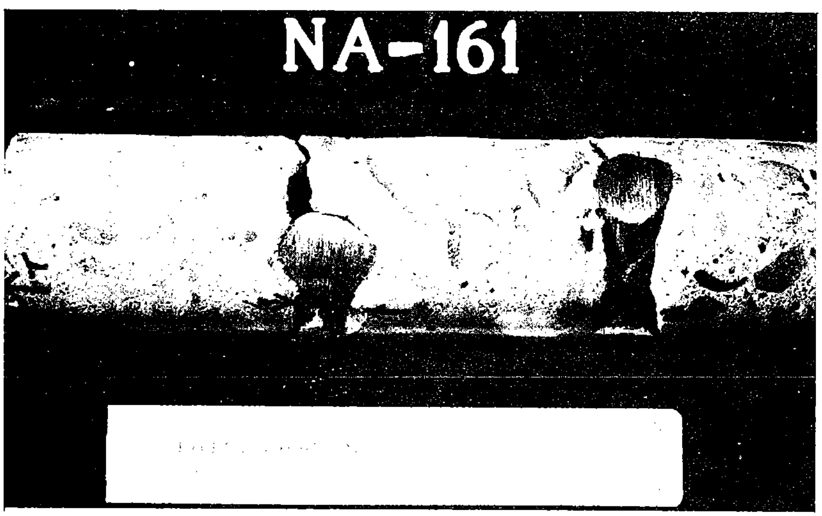


Plate 8. Weak zones at steel; note that the steel did not rotate or chew the surrounding concrete.

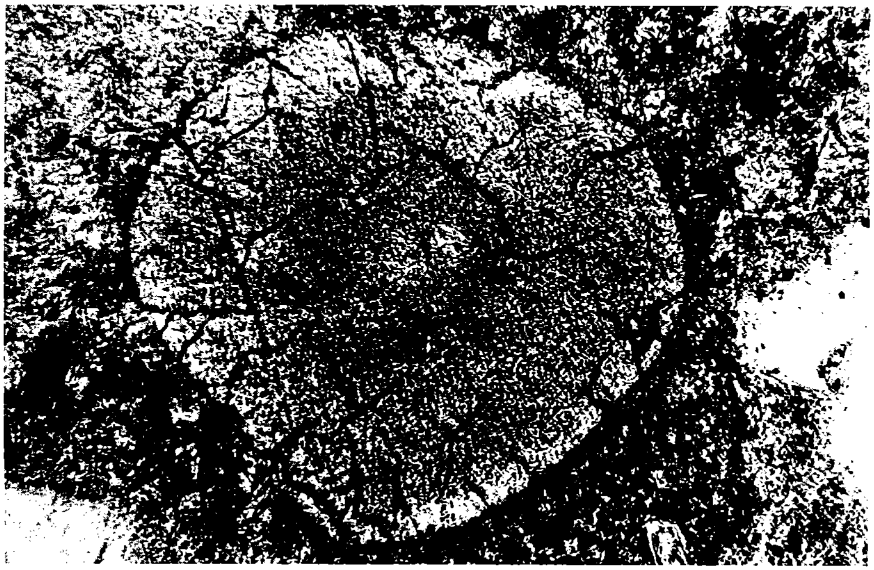


Plate 9. Photomicrograph of a sand grain of chalcedonic chert that has been altered to siliceous gel throughout its diameter. Note the internal fractures that extend into the adjacent cement paste matrix of the concrete. Top portion of core AS-16-153. Magnification X 240.

EFFECTS OF ANGULAR SANDS ON PORTLAND CEMENT CONCRETE

John S. Baldwin
James W. Dawson

INTRODUCTION

Angular sand, when used in Portland Cement Concrete, has been proven in the past to be a significant factor in the characteristics of the hardened material. The direct effect has been an increase in air content, while indirect effects include increased amounts of water to achieve desired slump or flow and lower compressive strength values.

This paper originates from a study of routine test results of concrete cores from a highway project in southern West Virginia. Most all of the cores tested revealed relatively low compressive strength values and, consequently, a study was initiated to determine the cause.

The only factor which appeared to deviate from ordinary concrete was that sand from the James River Basin was used as the fine aggregate. Analysis of the James River Sand revealed that it is considerably more angular than the sand of the Ohio River Basin which supplies the bulk of fine aggregate to the West Virginia Department of Highways.

A comparative study using three different sands was initiated to determine what effects the property of angularity of fine aggregate had on Portland Cement Concrete. For ease of preparation and testing, it was decided that standard 2 inch mortar cubes would be used instead of concrete cylinders, since both would exhibit the effects of angularity equally well. Ottawa Sand, Ohio River Sand and James River Sand represented a well-rounded, a semi-rounded and an angular sand, respectively. The Ottawa Sand was chosen as a standard reference since it is specified as such by the American Society for Testing and Materials (ASTM) for mortar testing.

PROCEDURE

In order to assure that testing results were relatively accurate and consistent, two separate sets of tests were conducted for each of the three sands. Tests conducted include sieve analysis, specific gravity, deleterious material, sodium sulfate soundness, (See Table I-A), sphericity and roundness (See Table I-B).

To better correlate the effects of angular sands, two different tests were conducted and replicated concerning the actual mortar mixes. The first test was designated as "fixed flow", in that a variable amount of sand was added to a constant amount of cement (1200gm) and water (720ml) resulting in a constant flow which was achieved at 100 ± 5. In the second test, a constant amount of sand equal to the amount used in the Ottawa mix above (3480gm), was added to a constant weight of cement (1200gm) and a constant volume of water (720ml). This second test was designed to vary the flow and was achieved due to the different natures of the three sands used. In both tests, air content was measured, and differences in the mixes noted (See Table II).

Compressive strength tests were conducted in replicate on all mortar cubes containing the three sample sands, at one, three, seven and twenty-eight days. A graphic representation of those results is shown in Figure I, while a summary of the results may be found in Table II.

RESULTS

As can be seen from Table IA, there is a significant difference in the sieve analysis between the Ottawa Sand and the Ohio River and James River

QUALITY TEST RESULTS

TESTS	SAMPLES					
SIEVE ANALYSIS:	OS-A	OS-B	JRS-A	JRS-B	ORS-A	ORS-B
3/8	100 %	100 %	100 %	100 %	100 %	100 %
NO. 4	100 %	100 %	100 %	100 %	98 %	98 %
NO. 8	100 %	100 %	94 %	95 %	93 %	92 %
NO. 16	100 %	100 %	76 %	78 %	86 %	84 %
NO. 30	51 %	52 %	53 %	55 %	70 %	68 %
NO. 50	16 %	16 %	22 %	24 %	38 %	36 %
NO. 100	1 %	1 %	4 %	5 %	13 %	12 %
T-11	0.1 %	0.1 %	2.6 %	2.9 %	6.1 %	6.6 %
BULK SPECIFIC GRAVITY (SSD)	2.635	2.646	2.555	2.564	2.541	2.550
SODIUM SULFATE SOUNDNESS	5.0 %	6.0 %	4.3 %	27 %	3.2 %	5.0 %
DELETERIOUS MATERIAL						
SHALE	0	0	0	0	TRACE	TRACE
COAL	0	0	TRACE	0	0.025 %	0.15 %
CLAY LUMPS	0	0	0	0	TRACE	0.647 %
LIGHTWEIGHT PIECES (T-113)	0	0	0	0	0.75 %	0.35 %
OTHER FRIABLE PARTICLES	0	0	TRACE	0	0	0
COLORMETRIC PLATE	1	1	1	1	1	3

TABLE 1A

TABULATION OF SPHERICITY AND ROUNDNESS VALUES

SAMPLE	SIZE	SPHERICITY	ROUNDNESS
OTTAWA SAND-A	#8 → #50	0.776	0.661
	#50 → #100	0.662	0.567
OTTAWA SAND-B	#8 → #50	0.764	0.640
	#50 → #100	0.670	0.503
JAMES RIVER SAND-A	#4 → #8	0.652	0.267
	#8 → #16	0.584	0.232
	#16 → #50	0.580	0.204
	#50 → #100	0.612	0.276
JAMES RIVER SAND-B	#4 → #8	0.591	0.279
	#8 → #16	0.582	0.235
	#16 → #50	0.628	0.2585
	#50 → #100	0.584	0.282
OHIO RIVER SAND-A	#4 → #8	0.618	0.534
	#8 → #16	0.678	0.5685
	#16 → #50	0.646	0.5285
	#50 → #100	0.688	0.472
OHIO RIVER SAND-B	#4 → #8	0.694	0.559
	#8 → #16	0.690	0.5775
	#16 → #50	0.644	0.478
	#50 → #100	0.666	0.5885

AVERAGE SPHERICITY AND ROUNDNESS

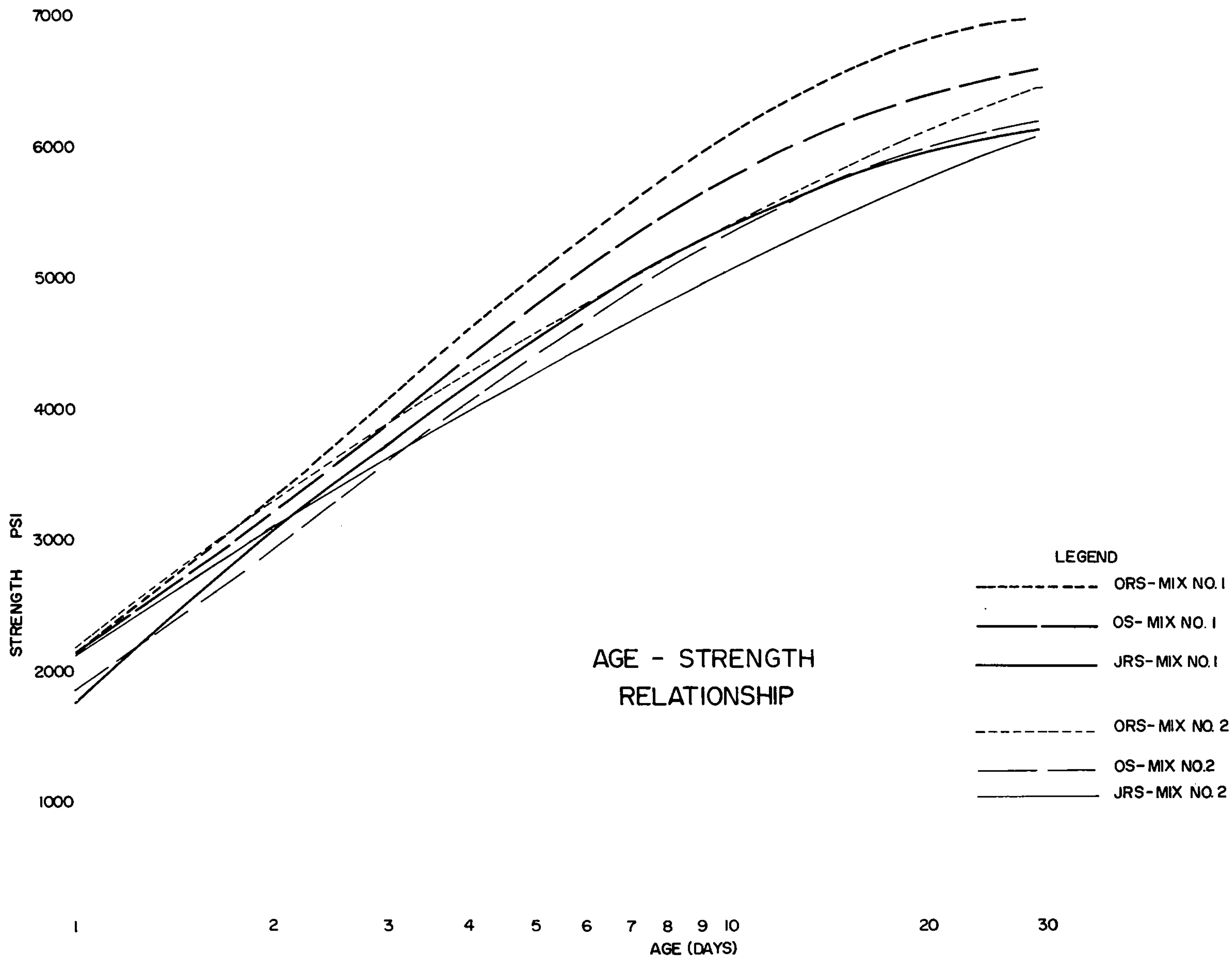
SAND	SPHERICITY	ROUNDNESS
OTTAWA SAND	0.718 ≈ 0.7	0.593 ≈ 0.59
JAMES RIVER SAND	0.602 ≈ 0.6	0.254 ≈ 0.25
OHIO RIVER SAND	0.674 ≈ 0.7	0.551 ≈ 0.55

TABLE I B

MIX CHARACTERISTICS AND COMPRESSIVE STRENGTH VALUES

	AGGREGATE TYPE	WATER / AGGREGATE (GM/GM)	WATER / CEMENT (GM/GM)	AIR CONTENT (MORTAR)	STRENGTH (AVERAGE) PSI				FLUIDITY INDEX
					1 DAY	3 DAY	7 DAY	28 DAY	
MIX NO. 1: CONSTANT CEMENT, CONSTANT WATER, VARY AGGREGATE TO PRODUCE CONSTANT FLOW	OS	0.21	0.6	2.90%	2150	3890	5280	6510	1.00
	JRS	0.24	0.6	3.02%	1740	3690	4970	6060	1.00
	ORS	0.28	0.6	0.54%	2125	4055	5525	6945	1.01
MIX NO. 2: CONSTANT CEMENT, CONSTANT WATER, CONSTANT AGGREGATE PRODUCES VARIABLE FLOW	OS	0.21	0.6	2.20%	1820	3590	4870	6110	1.04
	JRS	0.21	0.6	4.95%	2100	3590	4630	6000	0.60
	ORS	0.21	0.6	0.48%	2150	3880	4970	6390	0.82

TABLE II


7000
6000
5000
4000
3000
2000
1000
STRENGTH PSI
1
2
3
4
5
6
7
8
9
10
20
30
AGE (DAYS)
AGE - STRENGTH RELATIONSHIP
LEGEND
ORS-MIX NO. 1
OS-MIX NO. 1
JRS-MIX NO. 1
ORS-MIX NO. 2
OS-MIX NO.2
JRS-MIX NO. 2

(FIG. 1)

Sands. The critical size for air entrapping in fine aggregate appears to be in the 20 - 70 sieve size range. Ottawa Sand has a fine particle size while the other two sands contain particles that have a large size range. Since the particle size does affect the air content to some degree, air contents were taken of mixes made from all three sands with the particle size being in the 20 - 70 sieve size range. From the original hypothesis, one would expect to have low, medium and high air contents from Ottawa, Ohio River and James River Sands respectively. As is evident (See Table II) this relation is not present. The air contents from the 20 - 70 sieve size range (See Table III) however, do exhibit this relationship for mix No. 2 and, consequently, it appears that it is the small particle size of the Ottawa Sand which makes its air content above that of the Ohio River Sand.

The Ottawa and James River Sands were very clean and essentially free from deleterious material, while the Ohio River Sand contained deleterious material. Sample ORS-B had a high clay lump content which might have effected the compressive strength value somewhat but not significantly. The other quality tests summarized in Table IA have results which are not exact but the differences are not great enough to significantly affect the compressive strength values of the different mortar mixes.

Table IB is a summary of the sphericity and roundness values for the different sands. The values were derived from comparison of the grains to Pettijohn's sphericity and roundness scales (See Figure 2) for each grain determined. The average of these values are listed in Table IB for each size. The average sphericity and roundness values for each sand are listed at the

MIX CHARACTERISTICS USING MINUS 20 SIEVE SIZE MATERIAL

MIX NO. I	Aggregate Type	Water/Aggregate (GM/GM)	Water/Cement (GM/GM)	Air Content (MORTAR)	Fluidity Index
Constant cement Constant water	OS	0.19	0.74	6.2 %	0.83
Vary aggregate	JRS	0.27	1.07	5.8 %	0.84
to produce constant flow	ORS	0.20	0.81	5.9 %	0.87
MIX NO. 2	OS	0.24	0.94	0%	1.2
Constant cement Constant water	JRS	0.24	0.94	8.8%	0.61
Constant aggregate produces variable flow	ORS	0.24	0.94	2.7 %	1.15

TABLE III

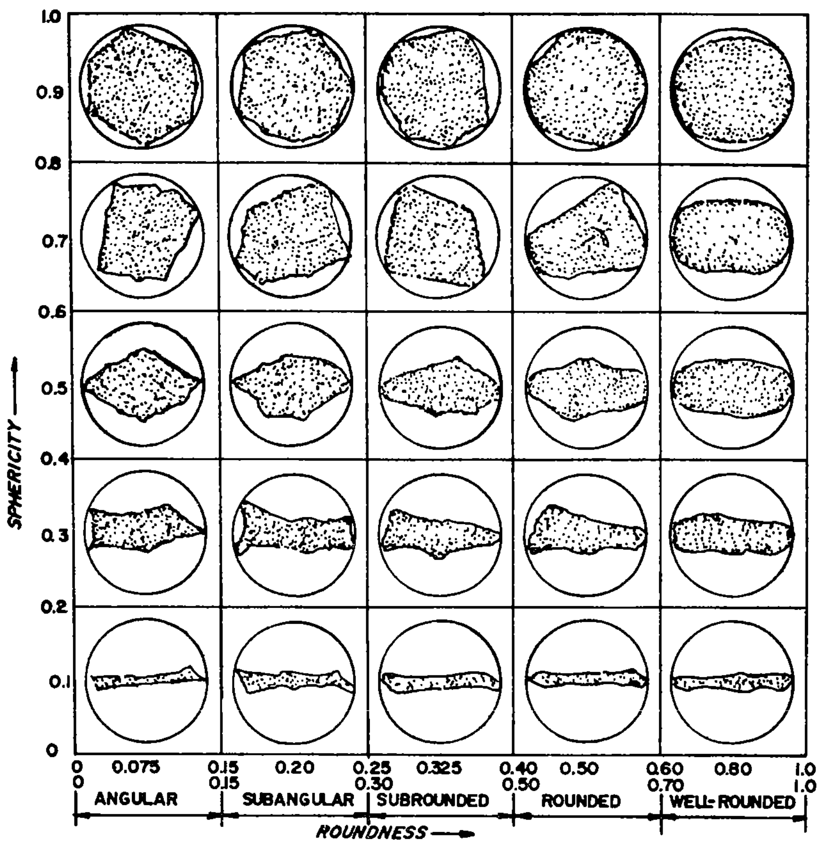


FIGURE 2

bottom of Table IB. Although there is little difference between the sphericity of the sands, (OS - 0.7, ORS - 0.7, JRS - 0.6) there is a marked difference in roundness (OS - 0.59, ORS - 0.55, JRS - 0.25).

The two mortar mixes were designed so that differences in the aggregates could be determined. In Mix No. 1, the water/aggregate ratio indicates that less aggregate was used in the Ohio River mixes than in the Ottawa Sand mix. The Fluidity Index (FI) for Mix No. 1 indicates that a constant flow of 100 ± 5 was achieved. The more angular sand produces a drier mix (with water and cement constant) and one would expect that there would be less James River Sand used than the other two sands to achieve the specified flow. As is evident in Table II, the water/aggregate ratio's in Mix No. 1 exhibit an irregularity, which must be explained by the sieve analysis. This irregularity (that there was less Ohio River Sand used than James River Sand to achieve the same flow), is probably due to the difference in gradation. Since Ohio River Sand is generally finer than the James River Sand, the smaller particles, with their larger total surface area, will tend to absorb more water than will the larger size material. The Ottawa Sand is fairly pure silica and well-rounded (with smaller total surface area) and subsequently, a greater amount of this sand was used.

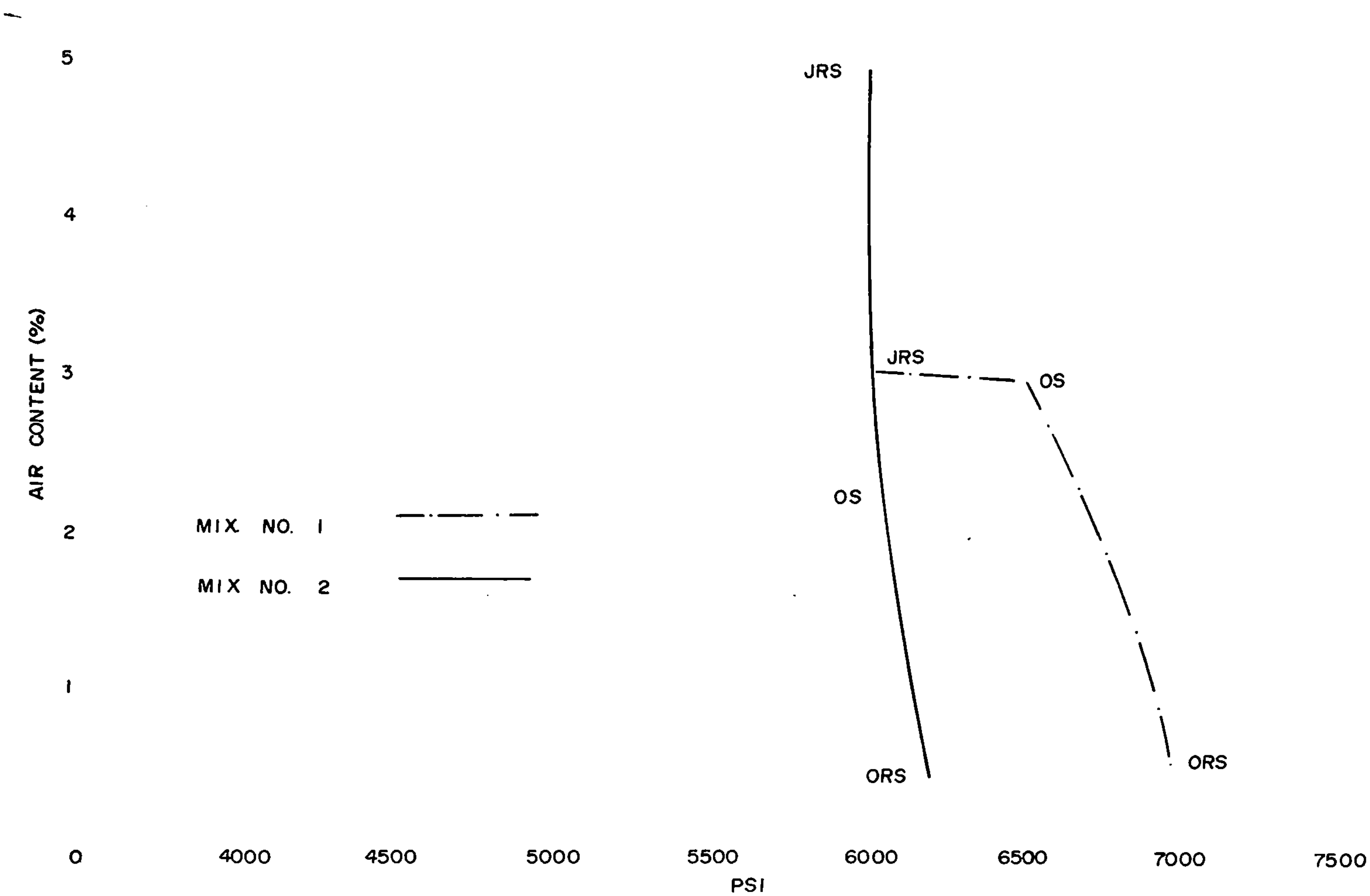
In Mix No. 2, the water/aggregate ratio was held constant with that achieved with the Ottawa Sand in Mix No. 1. The Fluidity Index for this mix indicates that the Ohio River Sand mix is drier than the Ottawa Sand mix, while the James River Sand mix is drier than the other two mixes. These mixes indicate that: (1) in order to achieve constant flow, either less amounts of aggregate must be used while cement and mortar are held constant, or (2) that with equal amounts of aggregate and cement, a larger amount of water must be used for the James River Sand Mix than for the Ohio River Sand Mix, which requires more water than the Ottawa Sand Mix.

Compressive strength testing was conducted on all mixes with replication. Figure 1 shows the normal age versus strength curves, while Figure 3 shows the air content versus 28 day strength relationship. As can be seen in Figure 3, there is a normal air content versus strength relationship for Mix No. 1. Mix No. 2, however, exhibits a highly abnormal curve, which is probably due to the extreme dryness of the James River Sand Mix. The Ohio River Sand Mix always exhibits compressive strength values greater than the Ottawa Sand Mix, except for Mix No. 1. The James River Sand Mix always exhibits compressive strength values less than the Ottawa Sand Mix, except for day one values for Mix No. 2. This apparent irregularity is probably due to packing of the angular grains. In all cases, the Ohio River Sand exhibits higher compressive strength values ranging from a minimum of 2.38% to a maximum of 22.13% above the James River Sand Mix.

CONCLUSIONS

Study of the test data reveals that there are no extreme differences in the sands, except for gradation, the clay lump content of ORS-B and the angularity value for James River Sand. The high clay lump content of ORS-B is probably insignificant in that similar compressive strength and air content values were derived for ORS-A. Certain chemical compounds in an aggregate, such as aluminum (Al_2O_3), which is gas producing, can prove to be detrimental

AIR CONTENT VS. 28 DAY STRENGTH



(FIG. 3)

to the hardened material. Chemical analysis reveals that James River Sands have an Alumina content of 19.4%, Ohio River Sand has 16.3% while Ottawa Sand is essentially free of Alumina. The difference between James River and Ohio River Sand is not great enough to explain the large differences in air content, although it might account for some of the differences. Differential Thermal Analysis indicates that there is nothing else in the sands that would detrimentally react with the cement.

Both Mix No. 1 and No. 2 exhibit an irregularity between air content and angularity in most cases. Ohio River Sand has a lowest air content, Ottawa Sand has a middle value, while James River Sand always exhibits the highest air content of the three. As mentioned before, the difference in sieve analysis is the probable cause of this irregularity in air contents. As Table III shows, (for Mix No. 2) Ottawa, Ohio River and James River Sands exhibit low, medium and high air contents, respectively.The irregularity for Mix No. 1 (Table III) is probably due to differing amounts of the various sizes of material passing the No. 20 U.S. Standard Sieve. Compressive strength values verify these differences since Ohio River Sand had the highest value, Ottawa Sand was the middle one, and James River Sand was the lowest at the end of the 28 day break.

From the test data, the following conclusions may be realized:

1. The angularity of the grains affects the air content of the plastic mix. The more angular the fine aggregate, the higher the air content. The gradation of the sand affects the air content to some degree, but not enough to explain the higher

air contents entirely.

2. The air content, in turn, has a direct relationship with the compressive strength of a specimen. The higher the air content, the lower the compressive strength values.

3. The angularity of the grains affects the workability of a mix. This is evidenced by the Fluidity Index. (See Table II). With all ingredients being held constant, the more angular sand will produce a drier mix, which is more difficult to work.

4. The degree of angularity is a factor which must be considered. Highly angular sands, such as the James River Sand, will produce effects as evidenced in this report. Mixes with well-rounded sands, such as the Ottawa Sand will have lower compressive strength values, when compared to semi-rounded sand mixes, but higher values when compared to angular sand mixes. While the air contents will be low, the low compressive strength values are probably caused by poor cement-aggregate bonding. Semi-rounded sands, such as the Ohio River Sand, produce mixes which have the best characteristics, with all properties considered. Whereas very angular sands have a large surface area and tend to entrap more air than is beneficial, well-rounded sands have poor cement-aggregate bonding characteristics due to the small surface area. The workability of a semi-rounded sand is between that of the angular sand mix and the well-rounded sand mix. The semi-rounded sand, therefore, offers a compromise between the two, exhibiting enough surface area for good cement-aggregate bonding, while not

producing very high air contents.

From studying the test data, it should be noted that some of the relationships which should occur according to the original hypothesis do not appear. For example, in Mix No. 1, Ohio River Sand has the intermediate air content between James River (high) and Ottawa Sand (low). This irregularity is probably caused by a combination of gradation, deleterious material content, and minor mineralogical differences. Petrographic examination revealed that there was a minor difference in constituents between Ohio River and James River Sands. The Ottawa Sand, of course, is almost entirely pure silica sand. Although these differences exist, they alone cannot explain the large differences in the air contents. The only major physical characteristic which exhibits a significant variance is the angularity. Although the different test results do not correlate exactly, the degree of similarity is fine enough to conclude that the angularity of the fine aggregate does sufficiently affect the air content of a mortar mix which, in turn, affects other properties of the mix. The physical and chemical characteristics of the aggregate are some of the variables which will affect the strength of the hardened mortar mix. Other variables include temperature, mixing time, and type of cement used. For this study, as many of these variables were eliminated as possible, while replicate tests were conducted at different times achieving a fairly good degree of reproducibility. The task of eliminating all the variables which exist in the sands used would have been monumental and,since natural sands were used, virtually impossible. The alternative to eliminating the variables was to identify them and determine how, and to what degree, they would affect the characteristics of the mortar mixes. Except for the angularity, the other variables did not differ enough to significantly change the characteristics of the different mixes. The differences in the aggregate properties might explain part of the difference but they cannot be responsible for all of the differences evidenced in the mixes. The original hypothesis was that angularity did affect Portland Cement Concrete. Although mortar mixes were used in this research, the same relationships found in these mixes are applicable to Portland Cement Concrete, perhaps the magnitude of the effects are different but the effects are present in both cases.

ENGINEERING GEOLOGY

ENVIRONMENTAL GEOLOGY AND LAND USE

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Today, engineering geologists recognize the extent that environmental geology and land use are a part of their work. The definition of environmental geology by Peter Flawn, State Geologist of Texas, indicates a close relationship of the terms previously mentioned. He states, "Environmental geology is a branch of the ecology in that it deals with relationship between man and his geologic habitat; it is concerned with the problems that people have in using the earth and the reaction of the earth to that use." Engineering geology, when defined, complements this relationship. Engineering geology is the field where geologic science is applied to engineering practice for the purpose of assuring that geological features affecting the location, design and construction of engineering works are recognized and adequately provided for.

Geologic factors that directly involve the engineering geologist are natural geologic processes and man-made geologic problems. Landslides, erosion, subsidence, floods, swelling soils, ground water and pollution are examples of these two factors. The origin of these factors are different but the results are essentially the same. Some geologic problems may become a menace to man, property or the environment. At such time they are called geologic hazards. Many examples of such problems can be given and many remedies to these problems can be detailed. However, it is not the purpose of this paper to burden the reader with endless examples and solutions but to point out some of the primary engineering geology problems encountered in Colorado and describe two case histories involving unconsolidated sediments.

Experience has shown that there is one basic rule to follow when practicing engineering geology; fit the land use project to the geology, not the geology to the project. Following this rule means leaving certain areas undeveloped, while utilizing other areas only through proper planning and engineering.

Colorado is experiencing a large population increase and has become a developer's bonanza. What happened in California a few years ago is happening in Colorado now. Land use and development have reached such proportions that the state legislature is trying to control and, in some cases, curb development. Needless to say their efforts are meeting with considerable opposition from special interest groups.

In 1971 the state legislature approved formation of a Land Use Commission; delineated local government responsibilities concerning land use; funded local planning groups; and required counties to have zoning, planning and subdivision regulations in force by July of 1972.

The present state legislature is working to enact amendments and necessary new laws to more adequately implement land use legislation. The primary features of the bills being presented are:

1. Land Sales (Senate Bill 36) - Developers and subdividers will be required prior to sale of any land to show there is an adequate water supply available, adequate sewage disposal areas and provide a report delineating any geologic, soils, or topographic hazards.

2. Subdivision Regulations (Senate Bill 35) - These will require that a developer or subdivider must comply with all local and state regulations for planning, zoning and subdivisions. These will include detailed soils, geology and geologic hazards investigations.

3. Registration of Subdivisions (Senate Bill 75) - This will require subdivisions of 10 lots or more to be registered with the State of Colorado. In addition, the seller would be required to furnish the buyer with full disclosure of the financial aspects of the sale.

4. Water Well Regulation (House Bill 1042) - This regulation will require that any one drilling a water well for domestic use must prove that he is not using water already appropriated by someone else.

These bills have not been passed as of this writing and may be changed considerably before they become law. However, the brief outlines given point out that engineering geologists are deeply involved in all phases of the environmental and land use picture.

During the uproar created by the introduction and processing of this legislation there have been regulations quietly put into effect to control the development of solid waste disposal areas, also.

In Colorado, geologic problems that concern development and land use are similar to those of other states but in some cases are more complex because of mountainous topography and other local conditions. These geologically related problems are:

1. Landslides
2. Swelling Soils
3. Ground Water (too much or too little)
4. Subsidence
5. Flood conditions
6. Pollution

The rapidly expanding ski industry and supporting developments are covering many mountain valleys in Colorado. This means a rapid rise in land values. This in turn creates changes in economic and engineering geologic considerations for particular parcels of property. There are some areas where severe geologic hazards exist and the cost of remedial measures will be a critical factor in the land use decision. If development is the decision, will the costs be justified and will the development be safe from geologic hazards.

Breckenridge, Colorado lies in the valley of the Blue River in central Colorado. This is a glacial valley and contains the usual remains of alpine glaciation. The ski area here lies high on the flanks of the mountains and the village of Breckenridge lies on the valley floor. Theoretically, the area in between would be a choice location for development of condominiums and apartments. Much of this "in between" land is of glacial origin. Most of it is stable and suitable for development, however, there are some areas where the glacial material has been over-steepened, saturated and now exists as landslides or potential landslides (see Plate 1). A recent mudslide, the scarp of which is shown in Plate 1, cuts across a 23 acre parcel of land. This parcel, a patented mining claim, was considered for the development of condominium units. The limits of the parcel are approximately those of the topography shown in Plate 2. This parcel is topographically rough and geologically surprising. Prior to November of 1969 it contained a large marsh area, two small landslides and two incipient landslides along with some stable areas of glacial moraine material and alluvial terrace material (see Plate 2)

Plate 1 - Breckenridge, Colorado ski area shown in upper right corner. Scarp of recent mudslide is shown in center. An additional ski area is being developed to the left of the mudslide area and out of the limits of this picture.

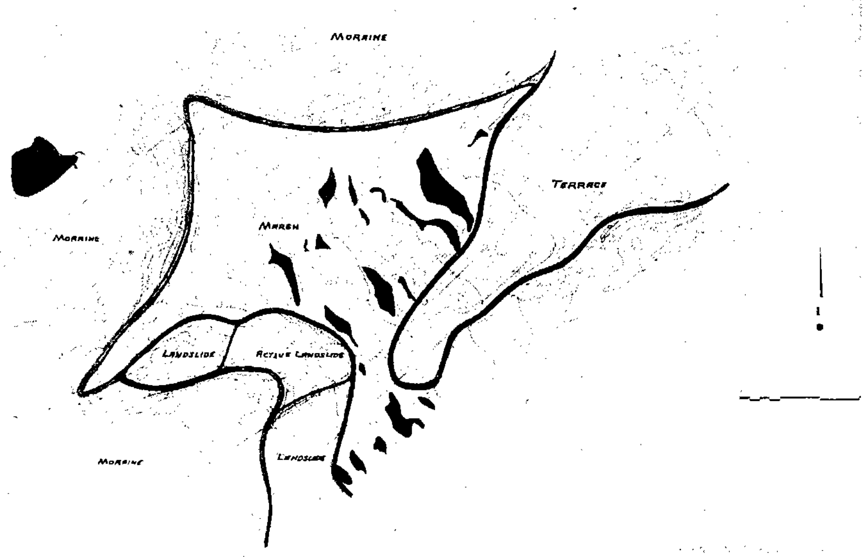

Plate 2 - Surficial geology on proposed condominium development site. Note large pond in upper left corner of photograph.

In November of 1969 one of the incipient slides, saturated by water from a pond, moved as a mudflow containing 80,000 to 100,000 cubic yards of material. The mudflow location is shown in Plate 3. Plates 4, 5 and 6 show other views of the November 1969 mudflow. The smaller incipient slide shown in Plates 2 and 3 remains to this day as a potential threat to the area below.

This 23 acre parcel of land was stratigically located for development. It was under the geologic conditions shown in Plate 3 that an engineering geologic investigation was made for a developer. It was determined that only 6 acres out of 23 were satisfactory for the construction of condominiums. The other 17 acres would require extensive preliminary work prior to any construction. This preliminary work would require the draining and stabilization of the marsh area, recent mudflow and landslides, and the removal of the geologic hazard created by the remaining incipient slide.

The geologic problems, construction problems and the economics involved forced this developer to abandon his idea for developing the land at this time. This decision was conservatively based on the theory that development of this geologic hazardous area should not be attempted because of existing economic problems and possible future liabilities. This problem is not limited to Colorado but will be present in any state where alpine glacial moraine material and the desire for developing exist.

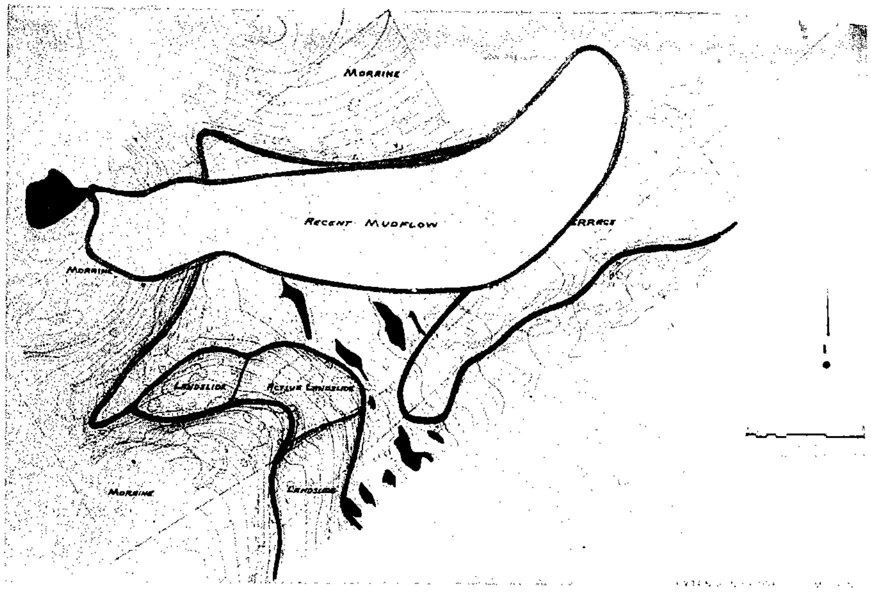

Plate 3 - Surficial geology on property with the most recent mudflow superimposed. This mudslide is approximately 1400 feet long and 200 feet wide. Water from the pond saturated the moraine material and created the mudslide.

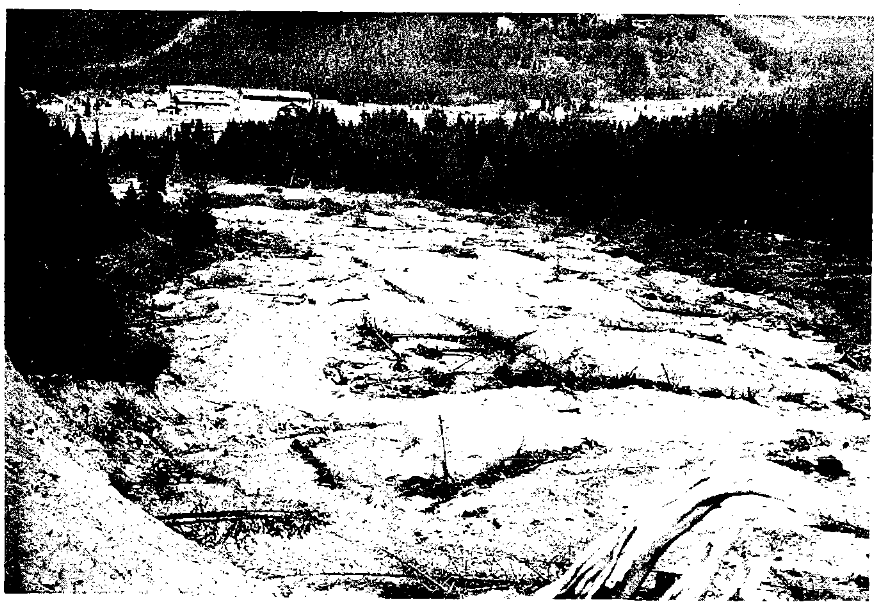

Plate 4 - Mudslide viewed from the scarp.

Plate 5 - Mudslide viewed from near the toe.

Plate 6 - End of the road, toe of the mudslide in front of truck.

The second case history is again not unique to Colorado but is an ever increasing problem everywhere. This case is concerned with the disposal of solid waste, namely: household and demolition trash.

The primary drainage ways in the Denver area are Clear Creek, running across the northern section of the metro area, and the Platte River running essentially through the center of the metro area. The alluvial deposits associated with these streams have been a long standing source of sand and gravel aggregate. Therefore, during the growth of the metro area vast quantities of this aggregate have been removed leaving large water filled abandoned pit sites. Some of these have been cleaned up and made into recreation areas while others have been used as trash disposal sites. After these pits have been filled with trash and the ground water table is allowed to return to normal there is a potential source of ground water pollution as decomposition of the trash occurs (see Plate 7).

Plate 7 - Land fill in abandoned gravel pit along Clear Creek north of Denver, Colorado. This shows poor, but common practices before new state regulations were enacted, i.e., no impermeable barrier to reduce pollution from trash being dumped in a water filled pit.

To reduce this pollution potential there are two alternatives available. One is not to locate a trash disposal area in the pit and the other is to design and construct impermeable barriers to surround the trash to minimize or eliminate the pollution potential.

The first alternative suggests locating trash disposal sites in more acceptable geologic environments. However, these are generally more expensive methods inasmuch as the hole is already excavated from the gravel operation and is, therefore, ready to be filled with trash.

The second alternative is to design a barrier or dike of impermeable material surrounding the disposal area. This dike system should be constructed to minimize or eliminate ground water pollution caused by decomposing trash. Plate 8 shows the entrance to the first disposal site constructed in Colorado to utilize the dike system.

Plate 8 - Entrance to Property Improvements, Inc., Public Waste Disposal Area, 725 West 62nd Avenue, Denver, Colorado.

Recent legislation in Colorado requires site evaluation and approval prior to designation of an area as a disposal site. This site was constructed prior to the enactment of the land fill legislation but is in accordance with that legislation. The area is ideally suited for this type of land fill operation for the material for use in the impermeable barrier is located in the bottom of the pit.

The following information was taken from the report prepared by F. M. Fox and Associates, Inc. for the land fill development.

> This report will present the results of a study made to obtain information pertaining to the safe development of a land fill trash dump in the worked out portions of the Houston Gravel Pit. This land fill will be constructed by Property Improvements, Inc. located at 725 West 62nd Street, Denver, Colorado (see Plate 8). The pit area to be used for the land fill is irregular in shape but is generally bounded by 62nd and 64th Avenues on the south and north, and by Huron Street on the west and by Cherokee Street, if extended, on the east. The proposed land fill shall be so designed and constructed that at no time will it present a pollution hazard to the ground waters of Clear Creek alluvium (see Plate 9 for location).

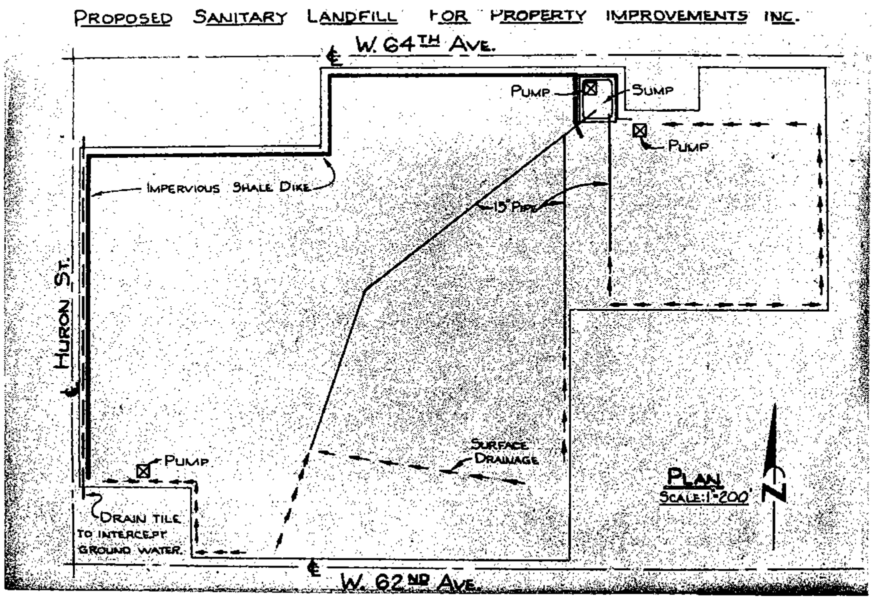

Plate 9 - Plan view of pit limits showing drains, sumps and pumps for de-watering the pit. The impervious shale dike will continue around pit perimeter as gravel is removed and additional trash disposal areas are needed.

The area of development lies along the south edge of the Clear Creek alluvium. Alluvium in this area is generally 20 to 25 feet in depth and lies on shale bedrock of the Denver formation. The alluvium, composed of sand and gravel found in this particular area of Clear Creek is of high quality and is used as construction aggregate. In this area the sand and gravel have been removed to their full depth exposing the shale bedrock underneath (see Plate 10). The water table at the time of the investigation in these gravel producing areas was very erratic and generally low, however, the normal water table is probably high with water moving towards Clear Creek to the north.

The proposal is to place compacted trash in the excavated and abandoned portions of the Houston Gravel Pit located in the Clear Creek Valley in Adams County. To prevent pollution of the ground water in the adjacent Clear Creek alluvium when portions of the trash decompose, an enclosing barrier with a very low coefficient of permeability must be constructed around the trash (see Plate 11). To restrict or eliminate the flow of water through the trash, this barrier must be constructed in such a manner that the trash will be totally surrounded with the impermeable material. To provide the impermeable barrier the shale bedrock found in the bottom of and adjacent to the south and east sides of the existing pit can be used. If placed and compacted under controlled conditions, this shale will provide adequate resistance to the flow of water through the compacted trash (see Plates 12, 13 and 14).

Plate 10 - View of northeast corner of pit with removal of the sand and gravel completed. The bedrock is exposed throughout. Impervious dike construction will take place prior to placing of trash.

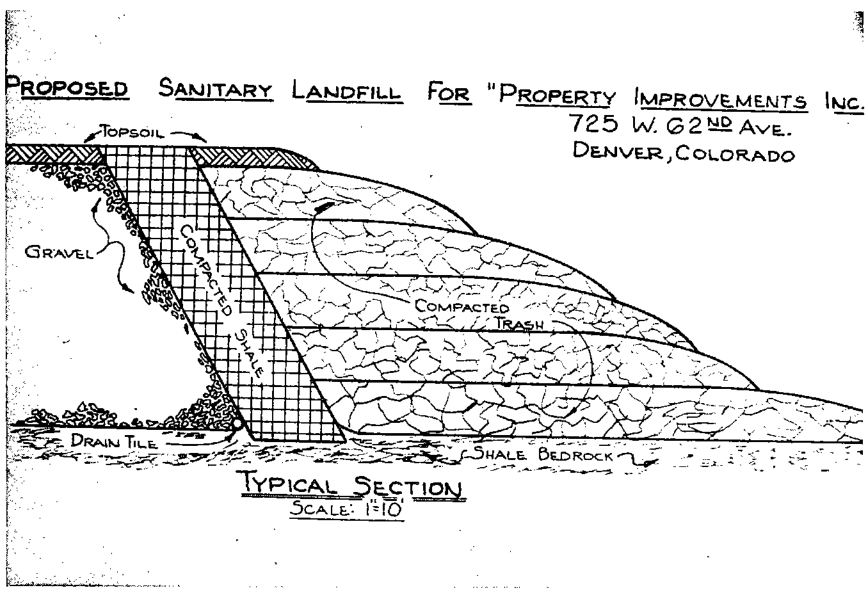

Plate 11 - Typical cross section in trash disposal development. Drain tile as shown was installed only on upstream side of pit area to divert as much ground water as possible around the disposal site.

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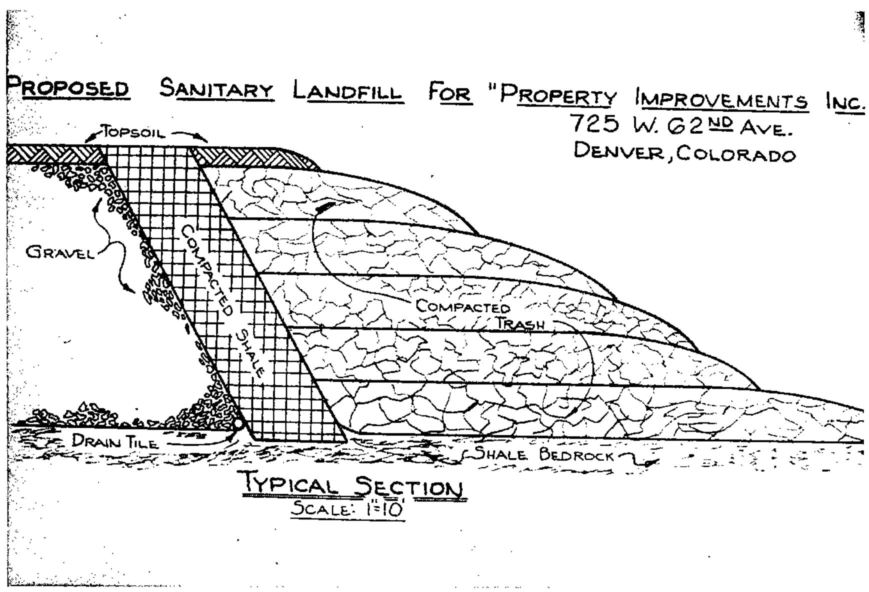

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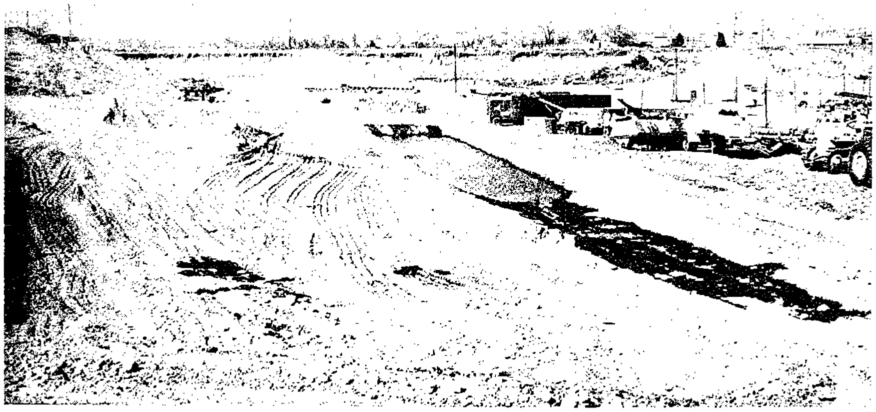

Plate 12 - Borrow area for impervious dike material. Gravel operation is still in progress, left center of photograph.

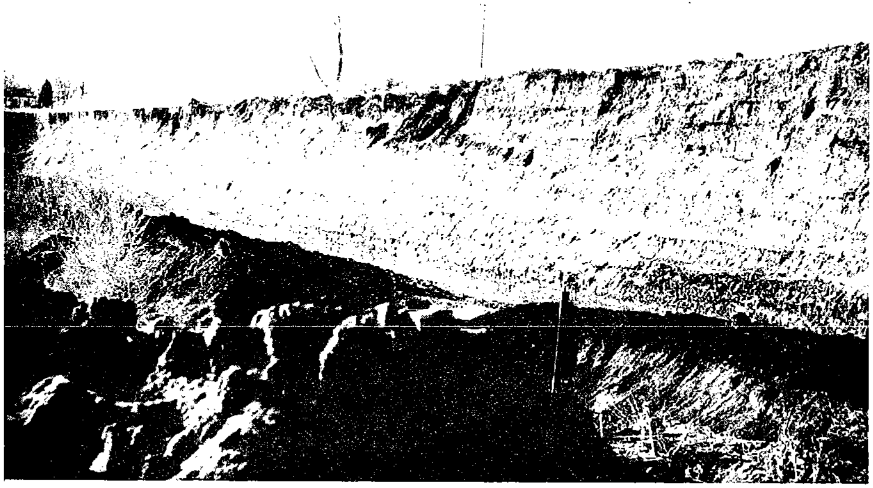

Plate 13 - Upstream limit of pit area near gravel wall has been stripped to shale bedrock as first step in construction of impervious dike.

Plate 14 - Impervious dike under construction in same area as shown in Plate 13. For ease of construction the dike is over 12 feet in width.

During placement of the trash, the area surrounding the pit should be so maintained that little or no water can run into the trash from surface or subsurface sources. Sumps will be required for proper water control (see Plate 9). The compacted trash will be covered periodically with the compacted shale to minimize the exposure of the trash to weather conditions. The surface of this compacted shale must be graded so that most surface water can drain away from the trash to a sump and be pumped out of the pit.

Proper construction of the earth barriers surrounding the trash is the key to lowest possible permeability and no ground water pollution problems. The compaction, gradation, Atterberg Limits and permeability test results indicate that the recompacted shale bedrock is adequate for the barrier.

Prior to placing any trash, the dike surrounding the site should be keyed into the shale bedrock, and the bottom of the proposed dump should be dewatered and cleared to bedrock. The exposed shale should then be scarified, leveled and thoroughly compacted.

The Controlled Earth Work Specifications should be carefully followed to provide the best possible compaction of the fill. Field compaction testing and construction inspections should be provided during fill placement.

As the pit is filled to the top with trash, it should be covered with enough compacted shale and topsoil to tie into the surrounding dikes and seal in the trash (see Plate 11). This should also be done in accordance with the Controlled Earth Work Specifications.

Two check wells were installed at the locations selected by Property Improvements, Inc. The function of these check wells is to provide a point where water elevations can be measured, and water test samples can be obtained. These wells are composed of $2\frac{1}{2}$ inch I.D. plastic pipe slotted every 6 to 12 inches to provide access for the water. The portion of the pipe at the surface is steel so that a pipe can be easily placed on the well.

Well Number 1 is in an upstream position relative to the land fill and Well Number 2 is downstream. The upstream check well was placed just west of the gravel pit entrance along West 62nd Avenue. The downstream well was difficult to locate because of another operating gravel pit and two parcels of private land. However, the well was placed in Delaware Street about 600 feet north of the disposal site. In addition, there are two domestic wells adjacent to and downstream from the pit area that can be used to check for possible pollution.

To determine the permeability characteristics of the shale borrow material, a falling head type permeameter test was performed on the material. The sample used was compacted at optimum moisture content to 93% of the maximum dry density obtained from the Modified Proctor test run for the same material. The resulting coefficient of permeability, 0.0234 feet/year, indicates that the shale borrow, when properly compacted, is essentially impermeable.

It is advisable that for any compacted fill construction, a soils engineering firm be retained to provide field testing and field supervision to insure proper construction of the impermeable barrier. This will require constant field density testing and additional laboratory testing if the borrow material changes as the borrow area is extended.

Plates 15, 16 and 17 show the trash disposal operation. These are included to provide a complete picture of the operation.

Plate 15 - Demolition type trash is placed and compacted with a bulldozer.