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THE GEOLOGICAL DISTRIBUTION OF HIGHWAY BASE COURSE MATERIAL AND AGGREGATE IN FLORIDA

By

Robert O. Vernon[1]

INTRODUCTION

The Florida State Road Department's philosophy of road building includes the purpose of using any rock materials in the area through which the road passes, provided these materials will meet or can be made to meet the specifications of use. This general philosophy also includes planning to have the cuts balance the fills with hauls being made as short as possible.

Failure of the road bed as a result of freezing and thawing is not a particularly distressing problem in Florida, but, because of our heavy rainfall over short periods of time the saturation by water and resulting failures by flow does present some difficulty. Soft clays and organic muds, mucks and peats occupy shallow sinkhole depressions through the State, and are present in large numbers along the coastal lowlands. These soft sediment-filled sinks are excavated where they are intersected by roads and are refilled by sand, which is compacted. If the sinks are excessively deep, piling or bridging may be used.

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For state roads, a base course of several accepted materials is placed upon the grade and sections cut into the native sediments. The thickness of the base course varies from 6 to 10 inches when compacted, depending upon the amount of the road traffic. Upon this base course, pavements of concrete with silica gravel and sand, and with stone and sand as aggregates can be constructed. Bituminous mixes require a prime and a tack coat upon the base course followed by the bituminous pavement, which may consist of 1) various sand-asphalt mixes, both hot and cold, using silica sand, stone screening, slag screening, or shell; 2) asphaltic concrete binder course followed by a surface course, using both stone or slag and the fine aggregates above.

BASE COURSE MATERIALS

Several mineral materials have been used for base-course materials in Florida. These must be porous and permeable, uniform or easily made so by mixing with other sediments, and must be easily worked into a smooth, firm surface free of pits and pockets.

Base-course material should show no tendency to air slake or undergo rapid chemical change when weathered. Limestone must be uniformly graded from the largest pieces down to dust, and of a stated chemical content. Hard, lumpy, and flinty pieces are allowed up to a percentage where their presence prevents the forming of a smooth surface. The presence of trash and other foreign matter is not tolerated.

The plasticity index, the volume of water in percent necessary to change the rock from its plastic limit to where the crushed rock matrix will deform and flow, is closely controlled and is not allowed to exceed 10 percent.

Materials near the road right-of-way are desirable not only for the ease of handling, but also because of low cost. The widespread occurrence oi soft, granular limestones in Florida (frontispiece) provide convenience pits along many roads. Centers of production near Ocala, Williston and Miami can ship these materials at low cost to most of the State. Only in the Panhandle area is a sand-clay base commonly used, elsewhere limestone, marl or shell is specified.

Limestone, the primary material for road-base courses, is sold for about a dollar per ton F. O. B. at the plant. More than 4 million tons are used annually for this purpose. Sand-clay base course is similar in price, but the deposits are limited and becoming increasingly hard to find.

Limestone, sand-clay mixes, shell, marl and, where the traffic load permits, local rock bases may be used. These are listed approximately in their order of preference. Of the limestones, the Ocala group is the best available material. It is, as mined, more than 95 percent carbonate and is clean, uniform, smooth working and easily handled. Dampness rarely disturbs the road bed because the Ocala has an excellent porosity and permeability. The granularity of the limestone allows the disking of the base course, and the soluble organic-calcite forms a smooth compacted upper surface with little wetting. The stone is clean, easily dressed and pavement materials can be easily and firmly placed upon it.

A group of limestones of upper Eocene age called the Ocala group and including the Crystal River, the Williston and Inglis formations, crops out along the crest of the Ocala uplift along the tier of counties of the western peninsula from Suwannee southward to Hernando. These limestones are extensively mined in the Ocala and Williston areas and elsewhere in adjacent counties, where paved roads cross the outcrop. These limestones also crop out in Holmes, Washington and Jackson counties, but no attempt to utilize these as base course has been successful. A reserve of some 3 trillion cubic yards is present in the Ocala-Williston district and 15 billion yards in Holmes, Washington and Jackson counties.

Younger Oligocene and Miocene sediments, the Suwannee limestone and the Tampa formation, have been utilized under the term "Ocala limerock," but the plasticity index is usually too high for these to be used except when mined and combined with the limestones of the Ocala group. The Oligocene and Miocene limestones lie generally around the outcrop of the Ocala group in counties that adjoin the Ocala area. A recent test of dolomitic limestone of the Tampa formation or Suwannee limestone, exposed in Jefferson County, indicates that this rock might be utilized for base course.

Miami limestone as used by the State Road Department includes all limestone mined below the 28th degree parallel, but for the most part is Miami oolite of Pleistocene age. These limestones are allowed to have a minimum of carbonate of 85 percent, a plasticity index not greater than 6 percent and a liquid limit below 35 percent. Hard pieces offer some difficulty in obtaining a smooth upper surface. About 16 billion cubic yards of Miami limestone is present in the southern peninsula of which only a portion is available for mining. Because of large land developments the area available for mining is becoming increasingly smaller.

Shell base course must be clam or oyster shell, but steamed shell cannot be used. Admixtures of other mineral materials having a bearing weight of 30 pounds per square inch is permitted as a shell stabilized base. Oyster shells are dredged from deposits accumulated in bays, bars and occasionally from large Indian shell mounds. One difficulty in using shells for base course is the requirement to wet heavily, and more often than limestone, to secure a smooth, firm surface.

Broken and whole shells make up 80 to 90 percent of the beach ridge along the Atlantic coast and to a limited degree the beach and ridge sediments along the Manatee, Sarasota, and Charlotte County coast. These shells are loosely to firmly indurated and except for a limited local use as a base course or road surface, the material is employed for other more valuable structural uses.

Marl base must be either a shell marl that is uncemented or so indurated that it can be blasted in the pit and crushed by rolling on the road. The plasticity index must not exceed eight percent. Marl occurs rather commonly along the coastal areas of the Peninsula at elevations generally below 25 feet and rather generally along the deep stream-cuts of Holmes, Washington, Walton and Okaloosa counties.

AGGREGATE

Course aggregate is a scarce item in Florida. The State Road Department permits the use of silica gravel, stone or furnace slag. For use in Florida roads, silica gravel must be clean, tough, durable, quartz. The loss in a Los Angeles abrasion test cannot exceed 45 percent and the dry-rodded weight per cubic foot must exceed 95 pounds. Stone must be clean, sound, durable rock that, when subjected to the Los Angeles abrasion test, the loss shall not exceed 40 percent. Slag must be clean, tough, durable pieces of air-cooled, blast furnace slag, reasonably uniform in density and quality, containing not more than 1.5 percent of sulphur. The dry-rodded weight shall exceed 70 pounds per cubic foot and abrasion loss shall not exceed 40 percent. Slag cannot be used for portland cement concrete.

Fine aggregate for concrete shall be hard, strong, durable, reasonably well-graded, uncoated grains of quartz, reasonably free of extraneous substances. However, natural sand, stone screenings, slag screenings, or combinations of these, provided they meet the abrasion requirements and are clean, tough, angular grains free from clay, loam and other foreign matter, may be used in asphaltic concrete or binder course.

Hard, dense limestone is the only coarse aggregate produced in any abundance in Florida. Indurated thin-bedded limestone of the Miocene-Tampa formation and Oligocene-Suwannee limestone is mined in Suwannee, Hernando and Lee counties and more than 3 million tons are used annually at a cost of $1.00 to $3.25 per ton F. O. B. cars at the plant. The harder seams and beds of limestone mined along the outcrop in Broward and Dade counties are screened from that sold for base course and are used as aggregate. To a very minor percent this separation is also made from the Ocala limerock.

Slag is shipped into Florida from the Birmingham district at a cost of $1.75 to $1.95 per ton F. O. B. cars at the plant. The freight rate makes Florida stone competitive. About 1.2 million tons of silica gravel is produced from dredgings in the Flint River. Minor deposits are known to occur in northern Escambia and Santa Rosa counties and in southwest Jackson County, but most of the silica gravel used in portland cement concrete is produced from these limited resources.

Insofar as fine aggregates are concerned, fine to coarse quartz sand is widely distributed over the State. Almost any road built in

Florida crosses suitable quantities of clear, sharp sand. However, because stone and slag are required as coarse aggregates it is sometimes more convenient and cheaper to use slag and stone screenings as fine aggregate, or to combine these with locally produced quartz.

PETROLOGY OF CONCRETE AGGREGATE

By

Bryant Mather[1]

It was my pleasure to participate in the program of the 1956 Symposium. On that occasion I remarked that the fact that it was then the 7th Symposium on Geology as Applied to Highway Engineering should signify that there was no longer any need to advocate as new the use of geological knowledge and techniques in connection with highway engineering. Much of what I had to say in 1956 concerned aggregates for portland-cement concrete and some of the things I said then will be repeated here.

For many years aggregates were defined as "the inert materials such as sand, crushed stone, and similar particles," which together with cement and water compose concrete. If one examines the still valuable Treatise on Concrete, Plain and Reinforced by F. W. Taylor and Sanford E. Thompson, which was in its third edition in 1916, or the fifth edition of Johnson's Materials of Construction, which appeared in 1919, one finds no reference either to the concept that aggregate is, can be, or should be other than inert, or to the suggestion that samples

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of materials proposed for use as aggregate should be examined by a geologist or petrographer. The first record that we have been able to locate of the employment of a petrographer in describing and classifying concrete aggregates is a report published in May 1915 in the Bulletin of the AIME by C. W. Tomlinson entitled "Method of Making Mineralogical Analysis of Sand." This report describes a method that he developed under the direction of Prof. M. O. Withey and Prof. A. N. Winchell at the University of Wisconsin. Tomlinson's procedure consisted of separating a sand sample weighing about 300 g into four size ranges by the use of sieves, separating each sieve fraction into four specific gravity classes by the use of heavy liquids, and then examining and counting the particles of each class in each size range using a hand lens or microscope and computing weighted average composition. We do not have information as to what, if any, specific engineering utilization was made of the information developed from Tomlinson's work.

The literature contains relatively few references prior to 1930 relating to specific geological and petrographic information on aggregates that was used directly in engineering. Taylor and Thompson's treatise cited the work by Feret published in France in 1897 on the effect of the presence of mica in sand on the tensile and compressive strengths of mortar. They also cite additional work by W. N. Willis in 1907 where weighed quantities of mica were added to standard Ottawa sands that were then used in making mortars from which tensile-strength test specimens were molded. It was indicated that the tensile strength dropped off as the mica content increased but it was also shown that this was caused almost entirely by an increase in the amount of water required for mixing rather than due to any specific physical or chemical characteristic of the mica particles as such. The discussion concludes with the interesting and cryptic statement, "Black mica with a different crystalline form is not injurious to mortar." So far as I know, this question has not been subsequently examined.

In the 1920's G. F. Laughlin was called upon to participate in an investigation of the failure of certain concrete in service that was suspected of having resulted from the use as aggregate of material having constituents with undesirable properties. He found that the aggregate was altered anorthosite and found that it contained considerable quantities of the zeolite, laumontite, which characteristically loses water and disintegrates when exposed to air. He was able to establish a correlation between deterioration of concrete and the use therein of aggregates containing significant quantities of laumontite. He continued his interest in concrete aggregates and wrote on various aspects of the question. This work stimulated several engineers connected with highway departments and other concrete-using agencies to send to him samples of aggregates associated with unsatisfactory service from concrete or associated with suspicious test results. His study of these samples, which consisted almost entirely of limestones, involved assistance from C. S. Ross of the U. S. Geological Survey in determining the nature of the clay minerals present. All of the specimens contained clay of the montmorillonite group, according to the optical criteria then in use.

Beginning in about 1940 with the discovery by T. E. Stanton of the California Division of Highways of the kind of concrete deterioration now known as "alkali-aggregate reaction" there was a considerable increase in the degree to which the services of geologists and petrographers were employed in the study of aggregates for concrete. The California Division of Highways and other highway departments, particularly in the West, obtained geological and petrographic assistance, as did the U. S. Bureau of Reclamation, the Corps of Engineers, the National Bureau of Standards, and other agencies. In a few instances, particularly the PCA, the NBS, and the Corps of Engineers, a petrographer was already available in the concrete research establishment as a result of a previously recognized need for the employment of such talent in the examination of portland-cement clinker by reflected light microscopy techniques. The California experience was first directly correlated with aggregates described as "siliceous magnesian limestones." The Bureau of Reclamation's experience was correlated with aggregates consisting of rhyolites, andesites, and dacites. The initial, and entirely logical, reaction of engineers to these correlations was to specify that aggregates shall not consist of or include such materials.

When it became apparent, however, that it was clearly uneconomic to exclude from use as aggregate limestones, volcanic rocks, and siliceous gravels, research was carried on at an accelerated rate in an effort, on the one hand, to pinpoint more precisely the specific reactive constituents and, on the other, to develop alternative means of dealing with the problem. In the course of these investigations, primarily instigated because of concern with alkali-aggregate reaction, many other aspects of the nature and properties of concrete aggregate material have been investigated, classified, and correlated with performance of concrete. As a result, we can no longer speak of aggregates as inert material. A recent treatise, The Technology of Cement and Concrete, published in 1955 goes to the other extreme in stating "Most aggregates when bound into concrete by a cementing medium are highly reactive rather than inert." It is then pointed out that the activity may involve any one or a combination of chemical, physical, and thermal forces. This treatise discusses concrete aggregates in five chapters, the first of which is an introduction and the second is entitled "Geology and Petrography of Concrete Aggregates."

Geological activity relating to concrete aggregates is now technologically and economically important not only with regard to laboratory investigations but also as it relates to exploration and production. The sand and gravel industry has grown during the 20th Century from very small proportions to its present size, in which it exceeds all other non-fuel mineral industries in tonnage and is among the highest in value of product. Ninety per cent of its production enters the construction industry. A somewhat comparable history has characterized the crushed-stone industry. Many deposits of sand and gravel and stone have been exhausted or have been rendered unavailable due to urban development. Similarly, the demand has increased absolutely and developed in localities where it did not previously exist. This has resulted in the need for locating new sources of raw materials in places where their existence is not immediately apparent. Modern production equipment and processing plants for the aggregate industry represent large investments of capital which are justified only when ample resources of raw material are available. It is, therefore, increasingly necessary that the geological situation be thoroughly explored before a site for a sand and gravel or crushed stone operation is brought into economic production.

The policy of the Corps of Engineers of the U. S. Army regarding aggregates for portland-cement concrete to be used in Civil Works construction differs from that of most concrete-using agencies in a number of respects. In the first place, quality is not controlled by specifying conventional limits on the results of standard physical and chemical tests. This deliberate change from conventional practice was adopted because of a belief that such limits are both unreliable and impractical. Control of the factors that influence quality remains with the Contracting Officer. The Corps' policy is to determine the suitability of all sources locally and economically available for a given project, in advance of advertising for bids. In the determination of suitability, all appropriate available testing and examination procedures are employed. In the planning and design stages of a project, an important phase is selection of suitable aggregate sources. This investigation is directed to be made by "engineer-geologist teams working together" and should have as its objective, in the case of a large project, the location of a source which can be owned or controlled by the Government, or, in the case of a smaller project, the location of all sources determined to be economically competitive and acceptable from a quality standpoint. Once potential sources have been sampled and the samples delivered to the laboratory, it is directed that preliminary testing should consist of petrographic examination and determination of elementary properties such as specific gravity and absorption. Subsequent laboratory studies are directed to include more complete petrographic study, determination of additional physical properties, and tests for concrete making properties. At the conclusion of these studies a report will be prepared, giving the results, and recommending which of the sources investigated should be listed as approved. When several sources are approved they are listed in the specifications and the contractor is given the option of supplying from anyone of them or from another source not listed. If he elects to propose a source not listed as approved, it will be sampled, the samples tested, and if satisfactory, it will be approved; if not satisfactory, it will not be approved and he must then furnish from one of the approved sources. The specifications clearly specify that "Approval of a source is not to be construed as approval of all material from that source. The right is reserved to reject materials from certain localized areas when such materials are unsuitable as determined by the contracting office." Further it is required that "material from an approved source shall meet all of the grading, uniformity, and particle shape requirements of the specifications."

I mention the Corps of Engineers' approach to aggregate specifications because it is unique, because it involves a more routine and larger degree of participation by geologists, and because it, to some extent, I believe, points to a possible solution to what is described as "one of the most difficult problems" confronting the aggregate industry, namely, "the complexity and inflexibility of specifications for concrete aggregate." (USBM Mineral Facts and Problems).

As long as it was assumed as a premise that aggregate particles in concrete were inert, there was nothing that could have been properly discussed as "the petrology of concrete aggregates." Now that it is recognized that aggregate particles are rarely, if ever, inert, there is much to discuss. However, in the present state of knowledge, I am not prepared to give final answers regarding the interrelation of the many properties of aggregates with the many aspects of their environment in concrete. Any solid particle - of any size such that it will pass through the larger and be retained on the smaller of any two sieves that may be used in processing materials for use as aggregate may - and usually will - become an aggregate particle. Such a particle will be unique; no other particle in the world will have exactly the same shape, surface texture, mineral composition, pore structure, strength, hardness, resistance to abrasion, thermal coefficient, modulus of elasticity, and so on. Also no other particle will have precisely the same environment, temperature history, loading history, and the like. Hence all evaluations must be statistical and in terms of similarities and differences on a statistical basis. This is why it is impossible - and dangerous - to state categorically that granite is "good" or that shale is "bad" - as aggregate.

In the alkali-aggregate reaction we now know that for deleterious expansion and cracking to occur it is necessary that a variety of circumstances coexist: (1) The aggregate must be reactive. (2) Alkali must be present. (3) Moisture must be present. We also know that, unless there is an external source of alkali - sodium or potassium or both - from sea water, ground water, soil, or industrial contamination of the environment of the concrete - it must be contributed by the cement or the aggregate or both in an amount at least equal to 0.6 per cent by weight of the cement expressed as sodium oxide equivalent; less than this is too little to cause trouble in concrete in service, so far as we now know. With respect to the aggregate the situation is even more complex: the reactive materials are those that are composed of or include silica that is rather readily soluble in alkali - such materials are: opal, chalcedony, acid-volcanic glass, artificial glass, tridymite, cristobalite. However, no deleterious expansion will occur unless the quantity of such material and the ratio of surface area to mass of the particles is within certain limits such that the reaction with alkalies in concrete will result in the production of an unlimited swelling gel as the reaction product. So long as the reaction of alkalies in solution in the concrete with the soluble silica in the aggregates takes place in a solution that is saturated with calcium hydroxide, the product formed is a limited swelling calcium alkali silica gel, which does not cause expansive deterioration of the concrete. Only when the reaction occurs in an environment in which the alkali solution is not saturated with calcium hydroxide does the unlimited swelling alkali silica gel form. Therefore, if the ratio of the quantity of available alkali to the surface area of the reactive aggregate particles is such that all the alkali is used up in reactions that occur in the normal calcium-hydroxide saturated solution of the concrete, no harmful reaction product will be formed. Such a non-harmful situation occurs either when (1) the quantity of alkali is small, (2) the quantity of reactive aggregate is large, or (3) the particle size of the reactive aggregate is small. Only when the alkali content is high, the quantity of reactive aggregate is intermediate, and the size of the aggregate particles is relatively large can harmful alkali-aggregate reaction occur - only in this "pessimum" situation will there be available alkali and unreacted reactive silica in the interior of aggregate particles after the reaction with the aggregate particle surfaces has been completed. In this specific situation, due to the greater ionic mobility of the hydrated sodium and potassium ions, as compared with the lower mobility of the larger hydrated calcium ions, the sodium or potassium, or both, can, and will react with the silica beneath the already formed calcium alkali silica gel and produce the unlimited swelling alkali silica gel. This then is the petrology of concrete aggregate in deleterious alkali-aggregate reaction. It tells us why it is that we may prevent deterioration of concrete caused by alkali-aggregate reaction expansion by any one of a variety of measures: (1) Use of low-alkali cement, (2) use of non-reactive aggregate, or (3) use of an admixture of finely divided reactive aggregate, i.e., pozzolan.

Many other aspects of the petrology of concrete aggregates could be discussed; few, however, are as yet as well understood. The behavior of aggregate particles during freezing and thawing, heating and cooling, wetting and drying, loading and unloading of the concrete depend on the interrelation of the properties of the aggregate and those of the matrix in which they are embedded and on the degree and kind of bond developed between the particle and the cement paste. All of

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ever, we should not carry this approach too far. Engineering for 11 December 1959 noted that 10 technicians had been indicted in Madrid for responsibility in connection with the failure of a dam in Spain. The account did not indicate whether or not there were any geologists among those indicted.

THE AASHO ROAD TEST: A PROGRESS REPORT

By

W. N. Carey, Jr.,[1] and W. J. Schmidt[2]

The AASHO Road Test at Ottawa, Illinois, is a statistically designed research project with the major objective of determining significant relationships between the performance of highway pavements and the loads applied to them.

The project is sponsored by the American Association of State Highway Officials, and administered and directed by the Highway Research Board of the National Academy of Sciences--National Research Council. Financial support is provided by the States, District of Columbia, Puerto Rico, the Bureau of Public Roads, the Automobile Manufacturers Association, the American Petroleum Institute, and The American Institute of Steel Construction, with the cooperation and assistance of the Department of Defense. The estimated total cost of the project is 25 million dollars.

The test facilities are contained in six loops of highway pavement laid out on an eight mile right-of-way which is the location of a future four-lane, divided expressway in the Interstate Highway System.

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[1] Chief engineer for research, AASHO Road Test, Highway Research Board, Ottawa, Ill.
[2] Chief, public information section, AASHO Road Test, Highway Research Board, Ottawa, Ill.

Each test loop has two long tangents, or straight-aways--one of flexible-type pavement and one of rigid-type pavement. Each tangent was constructed as a series of short pavement sections of varied structural design, and was divided into two traffic lanes so that each structural section is, in effect, two test sections.

The six test loops were constructed in 418 structural sections, or 836 test sections. Of these, 368 were rigid-type pavement and 468 were flexible-type pavements. In addition, 16 short span bridges of various designs of steel, reinforced concrete and prestressed concrete were built at four locations in two of the test loops.

Five of the six test loops are being subjected to controlled traffic consisting of commercial trucks and tractor-semitrailer combinations. The sixth loop is used for special tests and to evaluate the effect of weather on the pavement.

The test vehicles represent 10 different axle loads and arrangements, one for each lane in the five traffic loops. All vehicles operated in any one lane apply identical loads to the pavement. Thus, any one test section receives repetitions of one load, and no other, and the performance of the pavement is attributable to the effects of that load.

The pavements under test, both flexible and rigid, represent a wide range of designs. Within each test loop there were pavements presumed to be underdesigned, adequately designed, and overdesigned for the axle loads applied to them.

For example, one of the test loops carries vehicles with 18,000-pound single axle loads in one lane and 32,000-pound tandem axle loads in the other lane. These loads are recommended by the American Association of State Highway Officials as legal limits, and are used as such by a majority of the States.

In this test loop, plain and reinforced rigid pavement slabs were constructed 5, $6\frac{1}{2}$, 8 and $9\frac{1}{2}$ inches thick. A sand-gravel subbase material occurs in thicknesses of three, six and nine inches. All possible combinations of these thicknesses occur in this loop. In other words, a plain 5-inch slab was placed on all three thicknesses of subbase; a reinforced 5-inch slab also was placed on all three thicknesses; and the same is true of the $6\frac{1}{2}$, 8 and $9\frac{1}{2}$-inch slabs.

This use of all possible combinations of the variable is known as a complete factorial experiment. It is an extremely "strong" experiment, and one which lends itself to sound analyses. In this particular case, the experiment is a 4x3x2 factorial--four slab thicknesses, three subbase thicknesses, and two types of pavement, reinforced or non-reinforced. Obviously, it required the construction of 4 times 3 times 2, or 24, different structural sections.

The same type of experiment design was used also in the flexible pavement tangent of each loop. In the test loop mentioned in the previous example, bituminous concrete surfacing was constructed three, four and five inches thick; a crushed stone base course occurs in thicknesses of zero, three and six inches; and a sand-gravel subbase in thicknesses of four, eight and twelve inches. When all possible combinations of these thicknesses of material are constructed there is a 3x3x3 factorial. necessitating 27 different structural sections.

Each tangent of the five traffic loops contains identical or similar factorial experiments. Of course, the range of thicknesses in each loop differs according to the loads being applied. For example, rigid slab thicknesses are $2\frac{1}{2}$, $3\frac{1}{2}$ and 5 inches on zero, three and six inches of subbase on the loop where vehicles have 2,000 and 6,000-pound single axle loads. In contrast, slab thicknesses are 8, $9\frac{1}{2}$, 11 and $12\frac{1}{2}$ inches on three, six and nine inches of subbase in the loop where vehicles have 30,000 single and 48,000-pound tandem axle loads.

Flexible pavements cover a similarly wide range--from a mere surface treatment on embankment soil up to a section with six inches of bituminous concrete surfacing on nine inches of crushed stone base and 16 inches of sand-gravel subbase for a total thickness of 31 inches.

In addition to the factorial experiment design, two basic principles of statistics were employed in laying out the test sections. These are randomization and replication.

Randomization is best explained by comparing it to shuffling a deck of cards. Any card has an equal chance of being anywhere in the deck. In the test loops any structural section had an equal chance of being anywhere within its loop tangent, and the loops were laid out along the right-of-way in random order. Although randomization of the various sections made construction more difficult, it was necessary to assure that the research findings would be unbiased.

Replication merely means repeating. In a research project, it results in performing the same experiment twice in order to determine experimental error and the reliability of the findings. A limited, but sufficient, number of structural sections was replicated or repeated in each of the test loop tangents.

There are only seven variables in the AASHO Road Test experiment. Those connected with the pavements have been described. They are: pavement type (portland cement concrete or asphaltic concrete), surfacing thickness, base thickness, subbase thickness, and reinforcing or non-reinforcing in rigid-type pavement. The remaining two variables are connected with the traffic. They are axle load and axle arrangement. In this case, the term "axle arrangement" merely means single or tandem.

The axle loads are 2,000 - 6,000 - 12,000 - 18,000 - 22,400 and 30,000 pounds on single axles, and 24,000 - 32,000 - 40,000 and 48,000 pounds on tandem axles. The 2,000 and 6,000 pound single axle loads are applied by small trucks operating in separate lanes of one test loop. In the other four loops, loads are applied by tractor-semitrailer type vehicles with single-axle vehicles operating in the inside lane and tandems in the outside lane of the loop.

The test traffic began full operation in November 1958. At that time, the project had a fleet of 70 vehicles--60 to operate on the loops and 10 for stand-by service. In the four main loops, carrying the tractor-semitrailer vehicles, full operation meant six vehicles per lane. In the light truck loop, full operation meant four vehicles in one lane and eight in the other.

Traffic was scheduled to operate about $18\frac{1}{2}$ hours a day, six days a week. Two shifts of drivers were furnished by a special U. S. Army Transportation Corps unit stationed at the test site. Operations were carried on through the winter and as much as possible through the spring "break-up" period.

Some difficulties were encountered in maintaining a full complement of operational vehicles on all test loops, and an additional eight tractor units were added to the vehicle fleet in the fall of 1958. By January 1, 1960, this fleet had amassed a record of approximately six million miles of operation and 400,000 applications of a specific axle load on all pavement sections remaining in test.

During the first part of January, 48 more vehicles were purchased and put into operation on the test loops. This has allowed an increase on the main test loops from six to ten vehicles per traffic lane; and, on the smaller loop, from four to six vehicles in one lane and eight to twelve in the other. Operations are being carried out on a seven-day-a-week schedule instead of six.

This increase is in accordance with the wishes of the sponsor to obtain as many axle load applications as possible before the scheduled end of test traffic next July 1. It is estimated that approximately 650,000 load applications will result.

The effects of the controlled traffic on pavements and bridges are being measured and recorded by systems of electronic and mechanical instruments, some of which were developed specifically for the project. Measurements are generally of two types: (1) transient effects of loads such as strains, deflections and curvatures; (2) permanent effects such as roughness, rutting and cracking of the pavement surface.

Measurements of the permanent effects of traffic will enable the research engineers to satisfy the primary objective of the Road Test. That objective asks for significant relationships between loading and the performance of pavements of different structural designs.

Soon after this objective was stated formally, it became apparent that there was no widely-accepted definition of performance. and thus no satisfactory method of rating pavements in terms of performance. The performance of any product is a subjective matter; or, in other words, it is the opinion of the user as to the degree of satisfactory service the product was given. Thus, the performance of a given piece of highway pavement is some overall appraisal of its serviceability over a period of time.

Therefore, some system was required whereby periodic serviceability ratings of the test pavements could be made. The record of serviceability against time, or number of load applications, would then be the basis for evaluating the performance of the various test sections.

Two questions arise immediately. Who is to make the periodic serviceability ratings? What characteristics of the pavement are to be considered?

The Road Test research staff based its answers to these questions on the following assumptions.

(1) The onlv valid reason for any road or highway is to serve all highway users, and the serviceability of a highway could be expressed by the mean evaluation, or opinion, of all highway users.

(2) Since the AASHO Road Test is a study of the pavement rather than the entire highway, only those characteristics relating to the pavement should be considered. Thus, such features as grade, alignment, shoulder condition, etc., should be excluded.

Obviously, the Road Test pavements could not be evaluated by all highway users. However, an easy solution would seem to lie in the utilization of a representative sample of all highway users, a small group which conceivably could make periodic ratings of the test pavements. This solution was ruled out for two reasons. It was not practical to assemble a highway-user rating panel on the project at the frequent intervals required. Also, many of the test sections were only 100 feet long, too short to allow a satisfactory evaluation of their ability to serve traffic.

The system which was finally developed by the research staff, with the aid of an advisory panel, utilizes objective measurements of certain characteristics of the pavement surface. These measurements are mathematically combined to produce a serviceability index number which satisfactorily estimates the serviceability rating of a representative group of highway users. The necessary measurements can be made at will and, in fact, are made on each test section every two weeks. The resulting index numbers, when plotted against time, will be the basis for evaluating each test section in terms of performance.

The development of this system began with the selection of a 12-man Performance Rating Panel. The members of this panel were selected to represent not only the highway engineering field, but also highway user groups, construction materials suppliers, and vehicle manufacturers. Theoretically, of course, the panel represents all highway users.

After this panel had been oriented as to the task it was to perform and had agreed on specific definitions of the terms involved, it began to rate 1200-foot sections of pavements in service on the highway system. In all, 100 sections of pavement in three midwestern states were rated. The pavements were selected to represent a wide range of serviceability, from sections obviously in very good condition to sections in very poor condition.

These field ratings were made individually; that is, the raters did not discuss the pavement conditions with other raters and did not disclose the ratings they had made. Raters were allowed to ride over the pavements, inspect them, and watch other vehicles travel over them. Their ratings were made on the basis of "present serviceability" which had been defined previously as "the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic." The raters were cautioned to base their judgments strictly on the basis of present serviceability and not on their opinions as to how the pavement might perform in the future.

All ratings were made on a scale of zero to five marked with adjective terms ranging from very poor to very good. The rater merely placed a mark on a simple scale to indicate his opinion of the pavement section. He was also asked to indicate whether or not the pavement section was, in his opinion, satisfactory for service on a primary highway, and to note what characteristics of the pavement had most influenced his opinion.

The panel was asked also to rate some pavement sections twice, with the second rating after a short period of time sufficient to allow the raters to forget their initial rating of the section. This was necessary to determine the ability of the panel to be consistent.

Since the panel was intended to represent all highway users, it was necessary to test its ability to do so. To a limited extent, the Road Test panel was validated by having other groups of highway users rate some of the same pavements and comparing the results.

When the panel members had completed their individual ratings of the selected pavement sections, the mean rating of the panel was designated as the "present serviceability rating" for each section. Meanwhile, the Road Test staff had been making objective measurements on these same pavement sections. Generally speaking, the measurements involved those pavement characteristics which the raters had indicated as influencing their opinions of the various sections. They included longitudinal profile, transverse profile, and the area of cracking and patching.

These objective measurements were then combined into a formula which would reproduce the subjective "present serviceability rating" which the panel had given each pavement section. Mathematical analysis yielded the simplest formula which would reproduce the panel rating, at least within the range of the panel's ability to repeat ratings of any section of pavement. The values obtained from this formula were designated the "present serviceability index."

Once the mathematical formula was derived it was no longer necessary to have a panel of raters for the pavement sections at the Road Test. Despite the relatively short test sections, it was now possible to make periodic objective measurements and combine these to produce a present serviceability index for each section.

As mentioned previously, the present serviceability index number is obtained for each test section every two weeks. When test traffic began in the fall of 1958, all test sections had relatively high index numbers. Under traffic, some of the underdesigned sections have deteriorated and some have failed. As this has occurred, the serviceability indices on these sections, plotted against time, have given an excellent "picture" of the relative performance of each section. Thus, the research staff is enabled to satisfy the primary objective: to find significant relationships between loading and performance of pavements of various structural designs.

The pavement serviceability-performance concept is the basic concept of the entire project. Anyone who hopes to utilize the results of the test in an effective manner should understand the fundamentals of this system. Further, such a system has a great potential for use by the state highway departments in the areas of sufficiency ratings, evaluation of design systems, and evaluation of paving materials and construction techniques.

PRELIMINARY SUB-SURFACE INVESTIGATION OF THE CHESAPEAKE BAY CROSSING

By

P. Z. Michener[1]

The project consists of a bridge-tunnel crossing of the lower Chesapeake Bay between Chesapeake Beach, near Norfolk, Virginia, and the southern tip of the peninsula on the Eastern Shore of Virginia (Cape Charles) and includes approach roads to connect the structure with the existing roadnet at both ends (Fig. 1). The project construction cost is estimated to be approximately $144,000,000 and approximately 3-1/2 years will be required for construction.

The total length of the project, including the approach roads, is approximately 24 miles, of which approximately 18 miles is over open water having the same characteristics as the adjacent Atlantic Ocean.

The individual component structures included in the project are not the longest or the largest ever built, but the project is unique in the number of different major structures included in one crossing.

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[1]Project engineer, Sverdrup and Parcel, Consulting Engineers, St. Louis, Mo.

These components consist of:

		Approximately
1.	Low Level Trestle..................	$11\frac{3}{4}$ mi.
2.	Thimble Shoals Tunnel & Islands.....	$1\frac{3}{4}$ mi.
3.	Baltimore Channel Tunnel & Islands..	$1\frac{1}{2}$ mi.
4.	North Channel Bridge & Approaches..	$\frac{3}{4}$ mi.
5.	Fisherman Inlet Bridge & Approaches	$\frac{1}{4}$ mi.
6.	Fisherman Island Causeway...........	$1\frac{3}{4}$ mi.
	Total Length of Project........	$17\frac{3}{4}$ mi.
7.	Approach Roads (approximately).....	5 mi.
	Total Length of Project (approximately)	$22\frac{3}{4}$ mi.

The vertical and horizontal clearances were determined by National Defense and local marine traffic requirements.

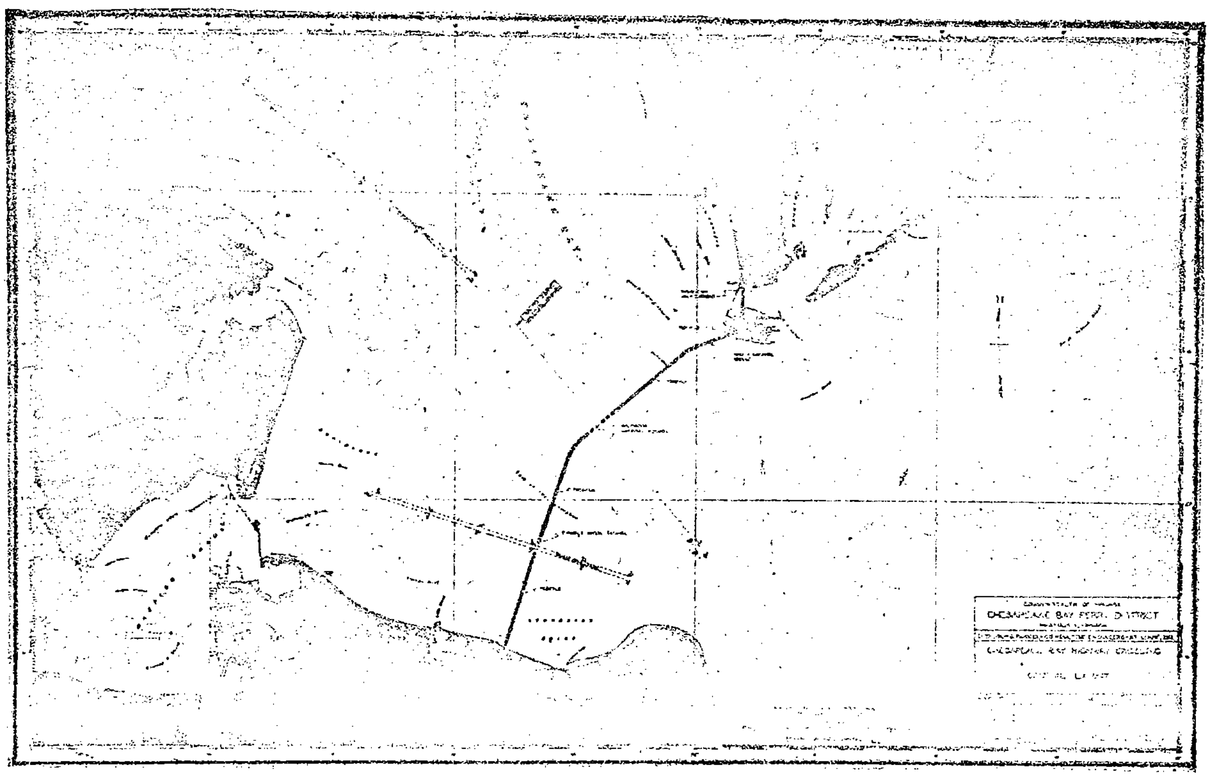
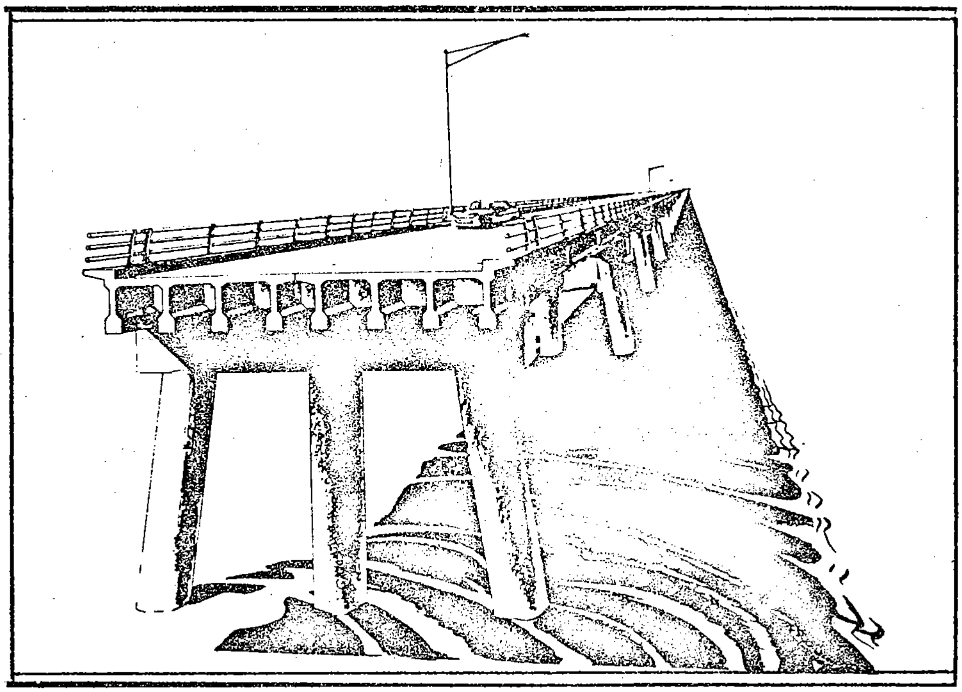


Figure 1

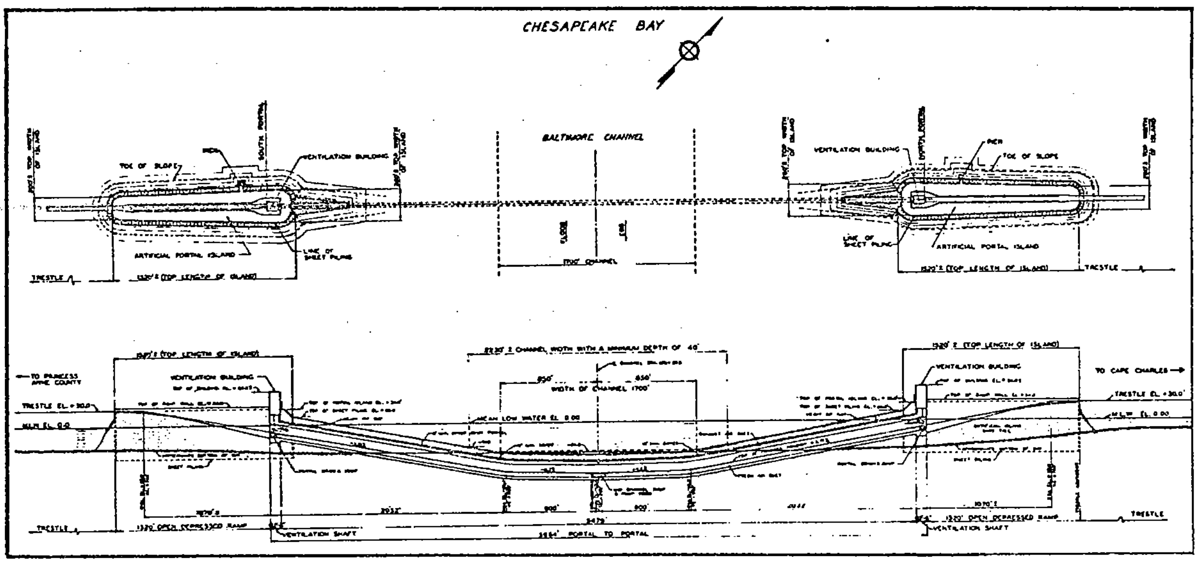
The design loading for all structures is AASHO, H20-S16.

Figure 2 shows a portion of the low level trestle structure.



The major portion of the project length, 61,725 feet, will consist of this type trestle over relatively shallow water, approximately 20 feet to 30 feet in depth, in areas where there are no established navigation requirements. This structure will consist of 825 precast, prestressed concrete spans of 75 feet each and precast concrete bent caps supported on 54-inch diameter, hollow, precast, prestressed cylindrical concrete piles. The structure will have a 28-foot roadway with an 18-inch safety walk on each side. This construction will be similar to that used on the Lake Pontchartrain Bridge near New Orleans, Louisiana. The roadway surface is to be placed on a level grade at an elevation 30 feet above mean high water to accommodate small boat traffic and to keep the superstructure above any wave action. You will note that ours is considerably heavier than Lake Pontchartrain in that we provide 3 pile bents with the outside piles battered. The piles will be filled with sand to enable them to withstand better any shock of collision by small boats or ice floes, which sometimes occur in this portion of Chesapeake Bay.

Tunnels are to be constructed under each of the two major ship channels. The construction details will be identical for both tunnels. Fig. 3 shows a plan & profile of the Thimble Shoal Tunnel. The approaches are on a 3.5% grade.



Thimble Shoal Tunnel will have a portal-to-portal length of 6,200 feet. Baltimore Channel Tunnel will be 5,664 feet portal-to-portal.

The tunnel structures will consist of prefabricated concrete tubes 37 feet in diameter and 300 feet long sunk into place and joined in a prepared trench and covered with selected backfill material. The tunnels and open approaches will have a roadway width of 24 feet between curbs and an overhead clearance above the roadway surface of 14 feet.

The ends of the tunnels will be founded on artificial islands. Each of these islands will be approximately 1,600 feet long and an average of 230 feet wide at the top, with the general surface of the islands at an elevation of 30 feet above mean low water. The islands will provide support for the depressed open approaches to the tunnels and an elevated area for the tunnel ventilation buildings and garages for emergency equipment. The core of these islands will be built of sand dredged from the sea bottom of adjacent waters. A solid row of prestressed concrete sheet piling, driven around the periphery is designed to prevent washing out and undermining of the sand core. Quarry-run stone, protected by heavy derrick-size stone and precast concrete shore protection units, will be used as added protection outside the sheet piling. The estimated cost of protecting <u>each</u> artificial island against severe wave action and high water is approximately \$4,500,000.

The third major structure is the North Channel Bridge which will provide a single navigation opening of 300 feet horizontal clearance and 75 feet vertical clearance above mean high water to accommodate the local fishing fleets. The approach grades are 3%. A

28-foot roadway with 18-inch emergency sidewalks on each side is provided.

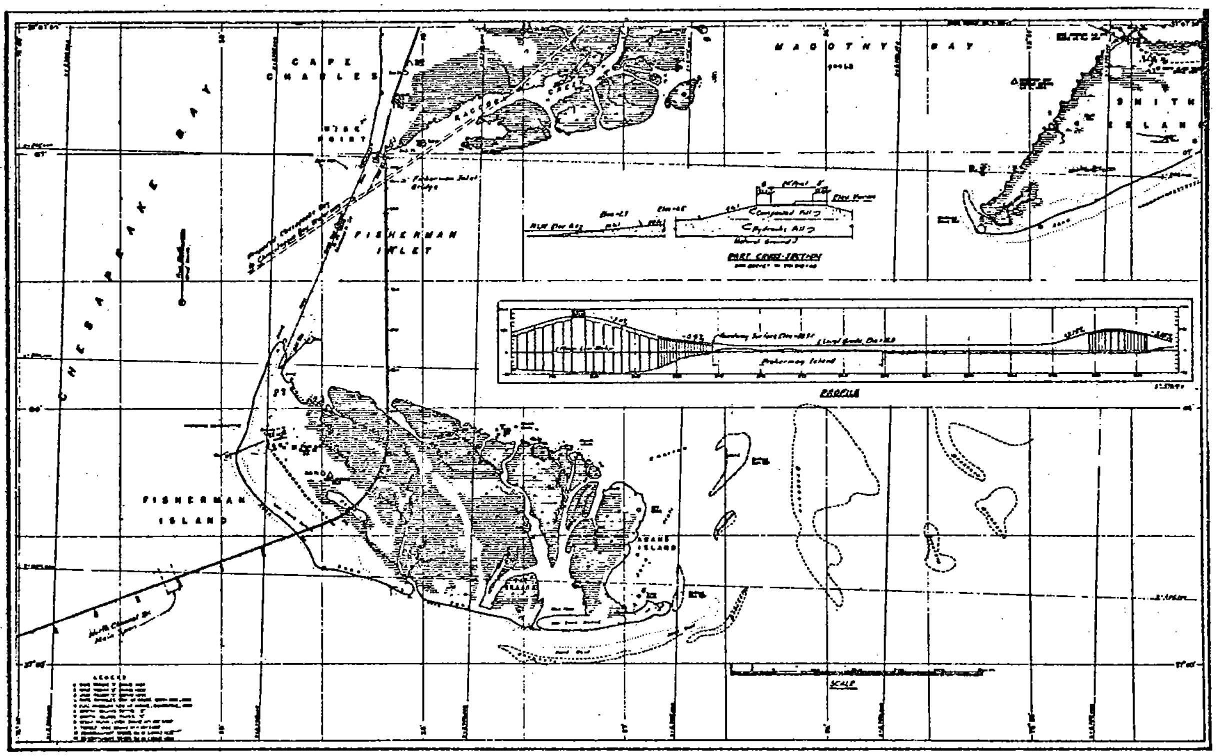
The main channel span will be a curved chord simple truss which combines economy with esthetics. The approaches to the main span will consist of four-span continuous fabricated steel deck plate girder units with reinforced concrete roadway slabs.

On the substructure, we propose to use what is sometimes called a bell-bottom pier. On this type of construction the Bay bottom is excavated to the elevation of the bottom of the pier. A precast concrete template is placed in the excavation, and steel bearing piles are driven thru the template. Then the precast pier shell is placed on top of the template and filled with tremie concrete. This method eliminates the cost of cofferdams.

The Fisherman Inlet Bridge is over one of the inland waterway dredged channels, and is a very simple structure. The center portion of this structure will consist of a three-span, continuous steel deck plate girder with reinforced concrete roadway slab. The center span of 175 feet will provide an opening of 110 feet horizontal clearance and 40 feet vertical clearance above mean high water.

The approaches to the main span of this structure will be of the low level trestle type construction. The main superstructure spans will be supported on reinforced concrete piers founded on steel H piles. These will be constructed using the standard open cofferdam method.

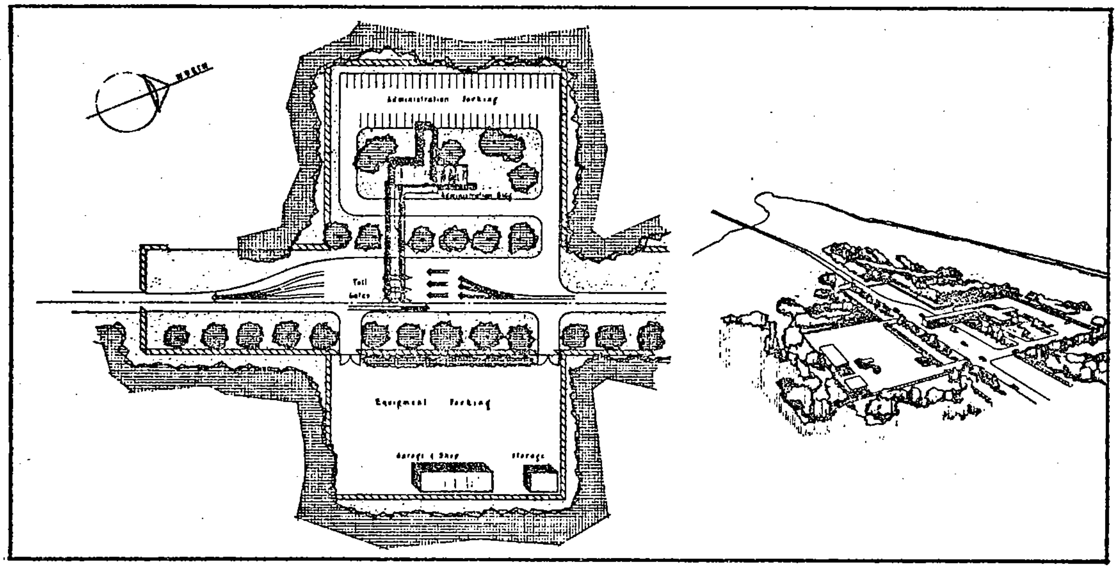
Between the North Channel Bridge and the Fisherman Inlet Bridge, our roadway across Fisherman Island will be carried on an earth-fill embankment (Fig. 4). The existence of defense installations



on Fisherman Island (Defense Radar, Loran, and special experimental projects) influences the alignment of the route across the island and the elevation of the roadway surface. To meet the requirements of the U. S. Navy, the elevation of the roadway surface is placed at 15 feet above mean low water. The sideslopes will be protected from wave action and erosion where necessary by a blanket of stone riprap.

Figure 5 shows a plan and delineation of the Eastern Shore terminal toll plaza and administration facilities.

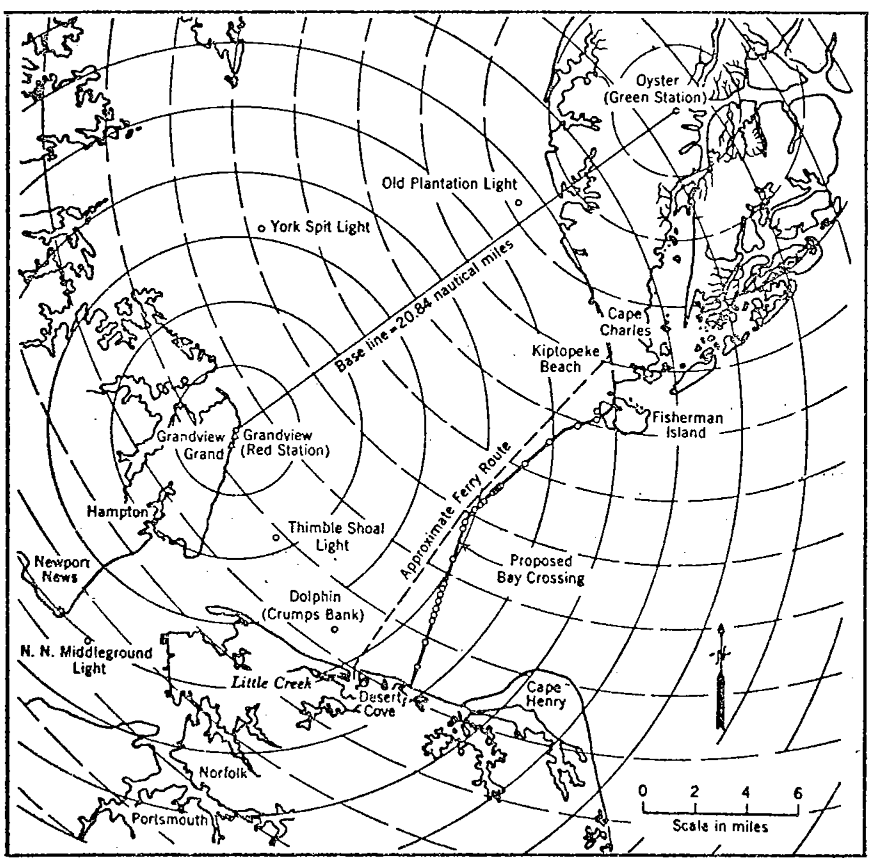
On this project, as on most major projects, one of our first problems was to secure reliable foundation information.



Our proposed project is located in the Virginia Coastal Plain which is part of what is geologically referred to as the Atlantic Coastal Plain, extending from Massachusetts to Florida. The Virginia Coastal Plain, which includes all of Virginia east of the Piedmont Plateau, has been the subject of many geological studies and field investigations, and no doubt some of you have been instrumental in the collection, preparation and presentation of some of the data I refer to. With particular reference to the Virginia Coastal Plain, the publications of Messrs. Stephenson, Cooke, Mansfield, Wentworth, Cederstrom, Clark, Miller, Ewing, Cory, Rutherford, Sinnott, Tibbitts, Sanford, Richards, Ryan, Hack and others were reviewed; and the generous assistance of the Virginia Department of Highways, the Virginia Department of Conservation and Development, Division of Geology, and the U. S. Geological Survey aided greatly in determining a basic conception of the geology of the adjacent shorelines. The available information showed that basement rock in the area is located at depths greater than 2000 feet and is overlain by geologically unconsolidated marine sediments belonging to the Lower and Upper Cretaceous, Tertiary, Pleistocene and Recent Periods. Within the depth penetrated by our preliminary borings, to a maximum depth of 300 feet below mean low water, it was determined that the Tertiary soils are of the Miocene Age and probably belong to the Yorktown formation. The Quaternary deposits, which are of primary significance in our study, were formed during the various glacial cycles of the Pleistocene Period. During these glacial cycles alternate depositing of materials and scouring by the master streams of the Atlantic slope, such as the Delaware, Susquehanna, Potomac and James Rivers, formed what is presently referred to as drowned system of stream channels. Evidence of these buried channels is apparent in the logs of borings taken during the construction of bridges over the James, York, Rappahannock, Patuxent and Potomac Rivers and at the upper Chesapeake Bay Bridge near Annapolis. Since these boring logs showed the drowned or buried channels to be filled with mud, silt and organic materials which represented unsatisfactory foundation material, we were anxious to determine if this condition existed at our proposed bridge location.

Prior to our preliminary borings we were able to examine well-drilling logs, foundation-boring logs and pile-driving records for many installations located on the adjacent shorelines; but, except for superficial information obtained from dredging operations, we were never able to uncover any sub-bottom information near the centerline of the 18-mile open water portion in which our major structures were to be built. Therefore, it was apparent that we must secure borings as quickly as possible. Like most projects in this initial stage, there was neither enough time nor enough money available to obtain complete and final sub-bottom information before the proposed project financing could be arranged. The problem was solved by making a careful study of the plan layout to determine the minimum number of soil borings which would give us maximum information intermittently across the 18-mile water portion, and by supplementing this information with a sub-bottom sonar reflection survey. Twenty-four water locations were selected for borings. The major portion of these were concentrated at the two tunnel locations and the North Channel Bridge.

In order to carry out this exploratory program our first operation was to establish the location of the boring holes on the proposed centerline of the structure across the 18 miles of open water. This is sometimes done by use of optical observations using standard surveying instruments or by the use of sextant from the deck of a vessel or drilling barge. The use of either of these methods was impractical from a time standpoint because our boring operations were to be performed during the winter months when visibility is intermittent and very poor, and conditions of the sea eliminated the use of floating drill barges and required that all drilling be done from prefabricated steel towers resting on the bay bottom. Our schedule required that all operations be on a 24-hour basis, and since navigation regulations would not permit the placing of marker buoys in the sea lanes more than two hours in advance of setting the drilling towers, we knew that we would be required to establish some of the boring locations at night. This problem was solved by using equipment manufactured by the Hastings-Raydist Co. of Hampton, Va., which is commercially known as "Raydist". The DM Raydist system is a phase comparison system based on the fundamental principle of determining distances, or differences in distances, in terms of the relative phase of an audio heterodyne. Basically, continuous-wave transmitters emit continuous radio signals, which differ from one another by audio frequencies. These heterodyne audio frequencies, when transmitted from a common location, such as a survey vessel, are received and relayed back to the survey vessel from two or more shore stations, the locations of which are accurately known. By comparing the phase of the signals as received and relayed back to the survey vessel, accurate distance determinations can be made. The Raydist equipment used in our survey consisted of two shore based stations as shown, at Oyster and Grandview, and duplicate Raydist position indicators mounted on the survey vessel. These indicators consisted of two dials and electro-sensitive paper recorders which continuously indicated and recorded automatically in terms of direct range, the distance from the survey vessel to the two shore based Raydist stations. Each dial reading represents a distance equal to a half wave length of the frequency on which the Raydist is operating. For the frequency which we were using each dial reading represented a sensitivity of 18 inches. The concentric circles shown indicate multiples of the dial readings given on the Raydist equipment and represent a distance from the shore station. The intersection of these circles, representing a given set of dial reading, provide an accurate fix of the position of the survey vessel (Fig. 6).



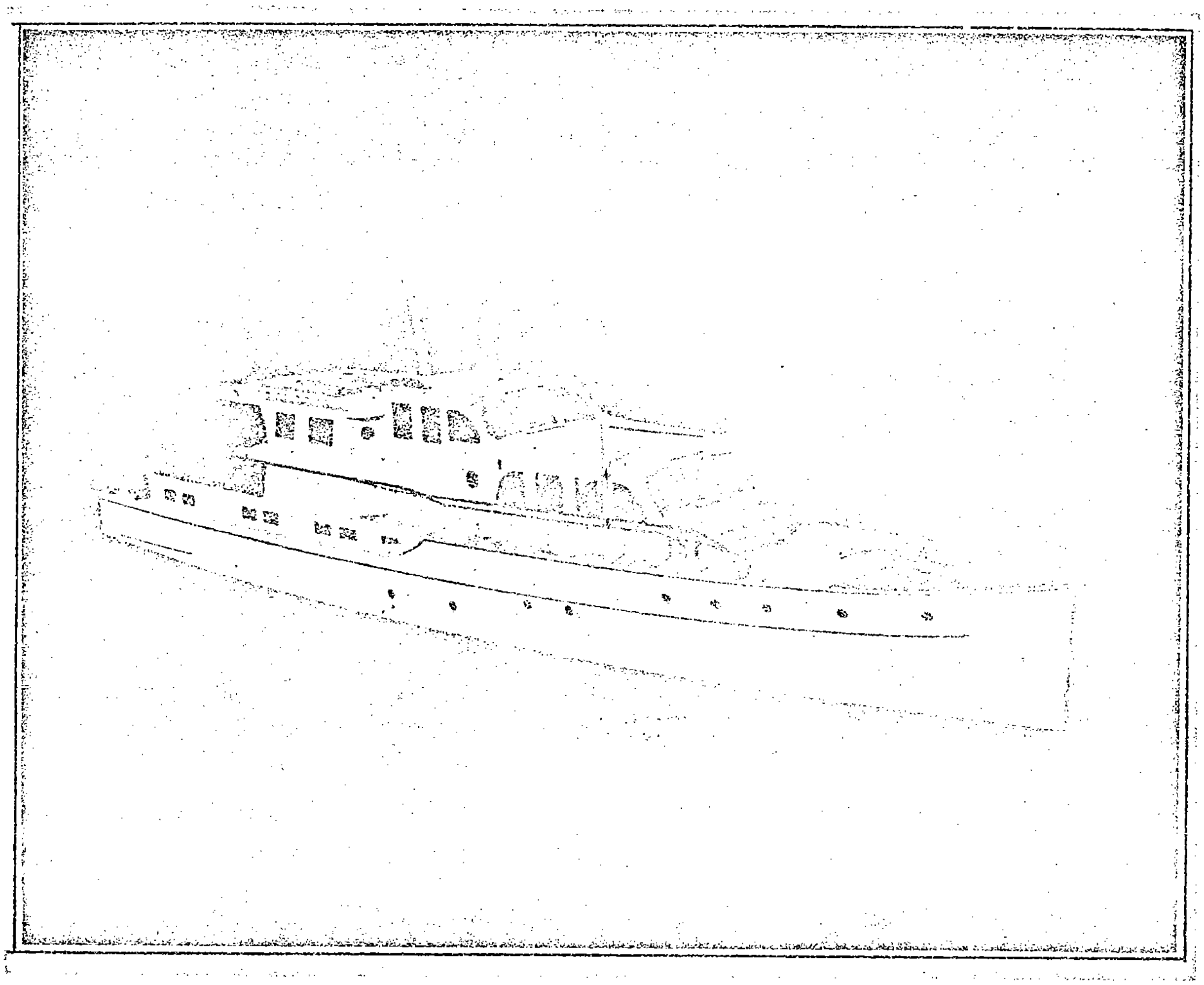


Figure 7

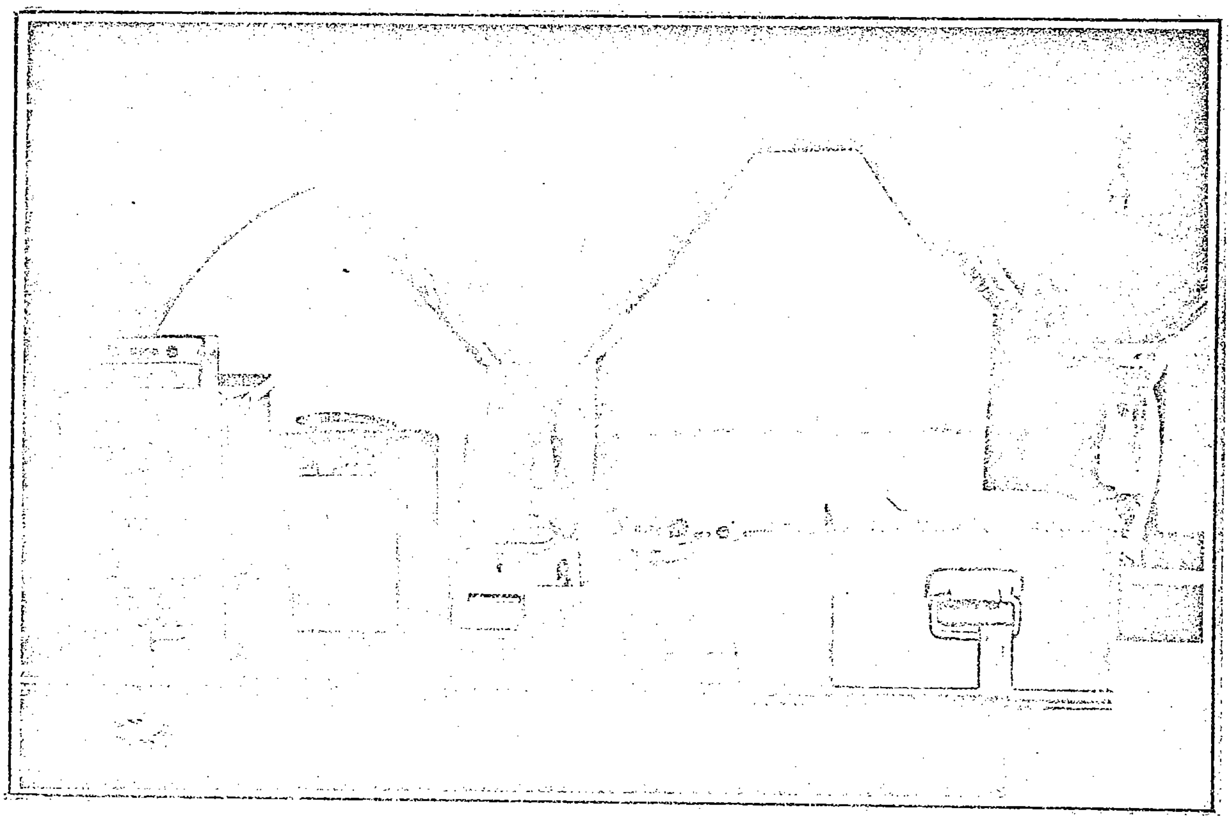


Figure 8

During locating operations the vessel positions were continuously plotted on a USC&GS map on which concentric circles had been superimposed and the vessel could be steered on any predetermined course without visual reference to landmarks. When the vessel was steered to the exact predetermined location of a boring hole, a buoy was dropped as a reference and the boring tower was set up on this location. Several of our boring positions were located at night. An accuracy of about 1 part in 5000 can be expected under normal field conditions, but final accuracy depends to a great extent on wind, tide, interference from passing ships and to a larger extent on the ability of the survey vessel pilot. When visibility permitted, the locations of the boring towers were checked with sextant and by use of surveying instruments. All locations were found to be within area limits specified.

Figure 7 shows the motor ship "Robin" on which the "Raydist" equipment was installed. Note the two Raydist antenna.

Figure 8 shows the "Raydist" equipment installed in the main cabin. To eliminate errors in readings, the numbers of the Raydist dials were made "red" and "green". It takes about 4 hours to install the equipment, and two men are needed to read and plot the course during navigation.

Figure 9 shows one of the boring towers in position for drilling. The actual boring work was done by Raymond Concrete Pile Co., New York, and the Tidewater Construction Corp., of Norfolk, Va., as joint venturers on a negotiated contract. The prefabricated steel platforms were constructed with 12-inch pipe sleeves at the four corners thru which 10-inch pipe piles 120 feet long were driven into the Bay bottom for support.

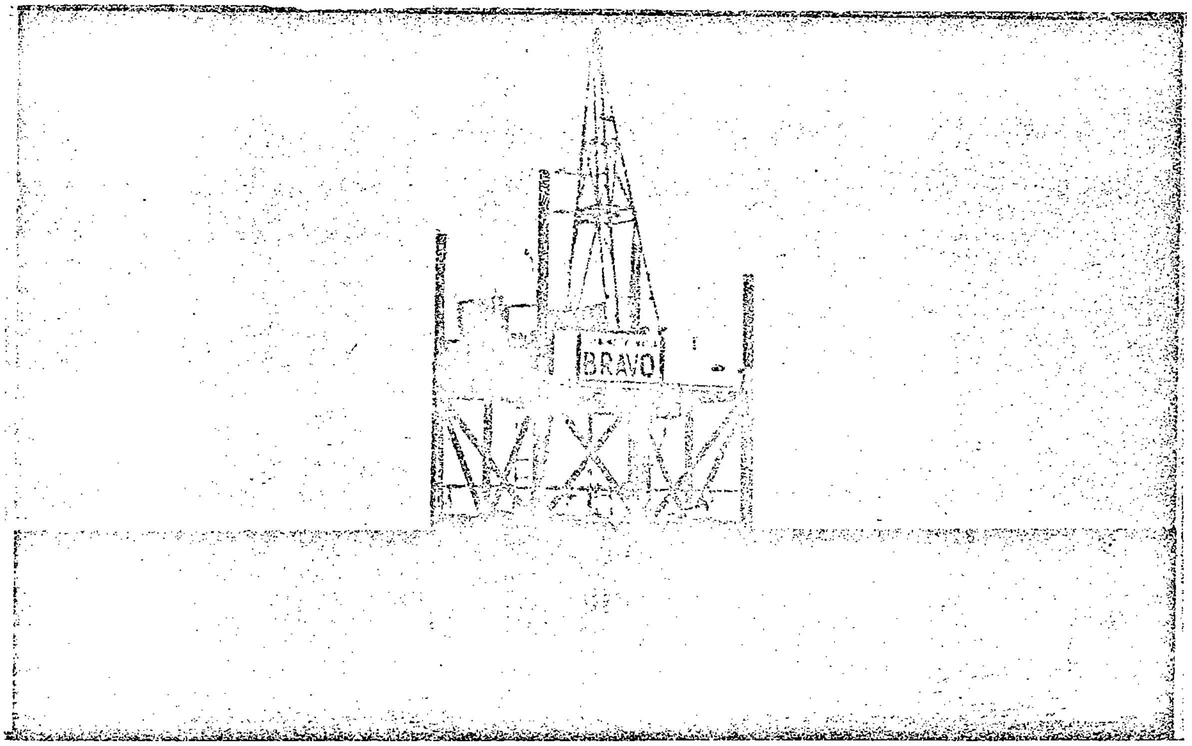


Figure 9

During the boring operations the temperature often dropped to below freezing and the sea was high, often slashing the lower bracing platform and considerable difficulty was encountered in getting the boring crews off and on the drilling platform. Each drilling tower was equipped with two-way radio, life raft, fire extinguishers, navigation lights, fog horn, electric generator and housing for the protection of records and samples, and for the comfort of the men from inclement weather. All the drilling work was accomplished with only minor accidents; one small fire, and one close call from being run down by a merchant vessel which came too close, out of curiosity. The drilling towers were stable and adequate for our operation.

To move the tower, the platform was disconnected, the piles pulled, and the whole installation lowered onto outriggers mounted on the front of the floating crane (Fig. 10).

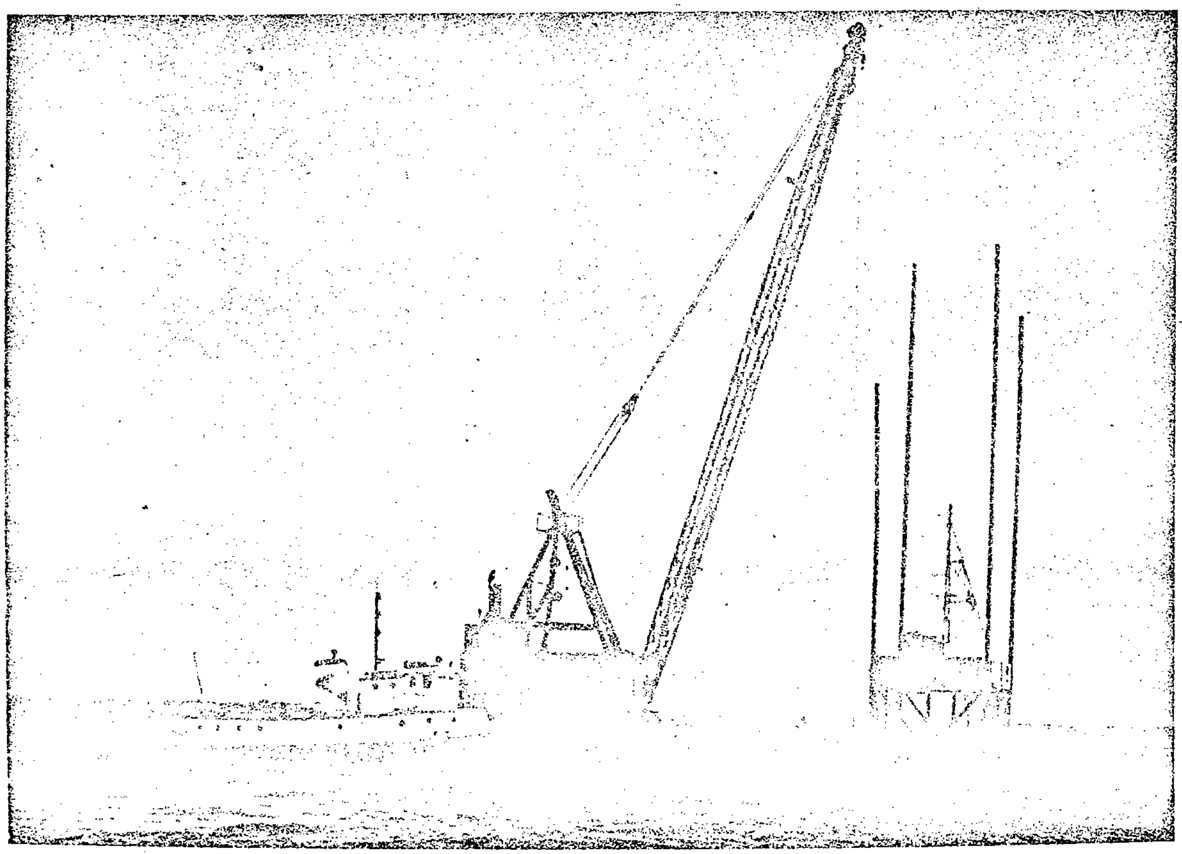


Figure 10

Since some of our borings were spaced approximately one mile apart, a sonar reflection survey was made to determine the continuity of the various soil strata identified by the actual borings. The sonar survey was made by Alpine Geophysical Associates, Inc., of Norwood, New Jersey under a subcontract. The equipment used in this work was designed and built by the company. It utilizes a spark discharge in the water as a broad band sound source and a non-directional hydrophone to pick up the reflected sound. The spark source and hydrophone are towed in a "fish" about 100 feet astern of the survey vessel. The incoming signals are amplified and recorded on a dry electro-sensitive paper. The speed at which the sound travels thru the water and subsurface material indicates the density and approximate depth of the underlying material stratas. A soft organic silty material will absorb the sound and give no reflection. A hard layer near the surface of the sediments will prevent deeper penetration. If either condition is encountered it would indicate the need for additional borings to determine the identity and classification of the material and its suitability for foundations.

When the results of the sonar survey are correlated with the actual borings a fair indication can be established regarding the continuance of a stratum between the known boring holes.

The sonar equipment was installed in the main cabin of the survey vessel alongside the "Raydist" equipment and was interconnected so that all records were correlated on a time basis, giving the sonar reflection for each recorded "Raydist" location. The "Raydist" equipment was used to guide the survey vessel over a predetermined course which covered an area approximately one mile on each side of the project centerline. The sonar survey operations required only seven days including installation, calibration, operation and removal of equipment and a total distance of approximately 200 miles was covered by the survey vessel during the operation.

Figure 11 shows a very poor reproduction of a small portion of the recording produced by the sonar equipment. The large letters and

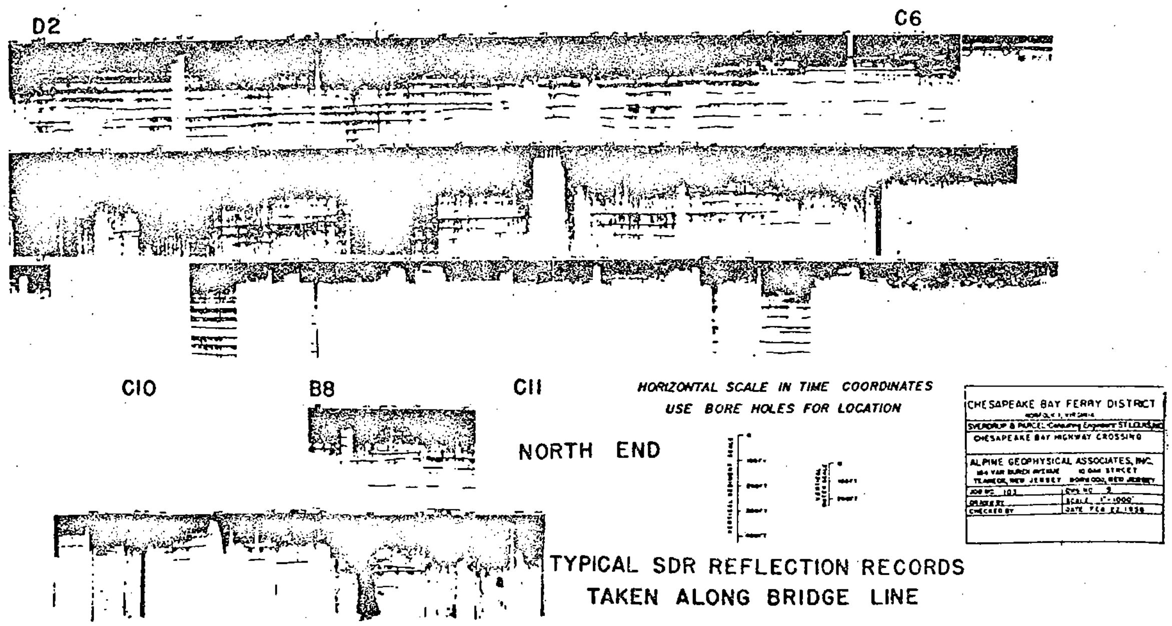


Figure 11

figures D2, C6, etc., indicate the location of actual borings. The small figures above the black portion indicate the day and the hour the recording was made. The horizontal pencil lines shown were added by the interpreter. Between D2 and C6 the 170 foot horizon is probably continuous although it is masked by multiple reflection over part of the area. A deeper horizon rises from 295 feet to 240 feet. To the right of C6 there is no penetration through an erratic horizon at an average depth of 70 feet. This area is not defined by the borings but appears to owe its character to the presence of soft material. The actual recording of the data is, of course, mechanical but the interpretation of the results is another matter requiring broad experience and knowledge of the physical properties of the various types of soils and their reaction to the sonic phenomenon. We selected Alpine Geophysical Associates to do our work because the experience and background of these men qualified them as specialist in the interpretation of the records.

Throughout the preliminary sub-surface exploration, Sverdrup & Parcel retained the services of Moran, Proctor, Mueser & Rutledge as foundation and soils consultants. Their representative in the field observed the drilling and sonar survey operations, advised regarding the shifting of boring locations and securing of additional samples, made on the spot examinations of the soil samples and supervised packing and shipping of all samples to their New York laboratory for analysis.

Upon completion of the laboratory tests and analysis, representatives of the foundation consultants, the sonar survey group and Sverdrup & Parcel assembled for a general discussion of the findings and correlation of the sonar and boring records.

Figure 12 shows the preliminary geologic profile indicated by the borings and sonar survey. We have indicated what we believe to be the top of the Tertiary and what appear to be buried channels. The general description of the upper materials shown as "A" is "loose to medium compact gray fine sand, some silt and traces of shells". The materials at the top of the Tertiary, shown as "F", is described in general as "stiff to very stiff greenish gray silty clay, some fine sand and traces of shell". It is proposed to carry all piling into the Tertiary formation unless otherwise indicated by additional test borings or test loads. At two locations, pockets of organic silt and peat were found. The extent of these pockets will be explored further at the time final borings are made. It is proposed to accelerate the settlement of materials under the artificial islands by the use of sand

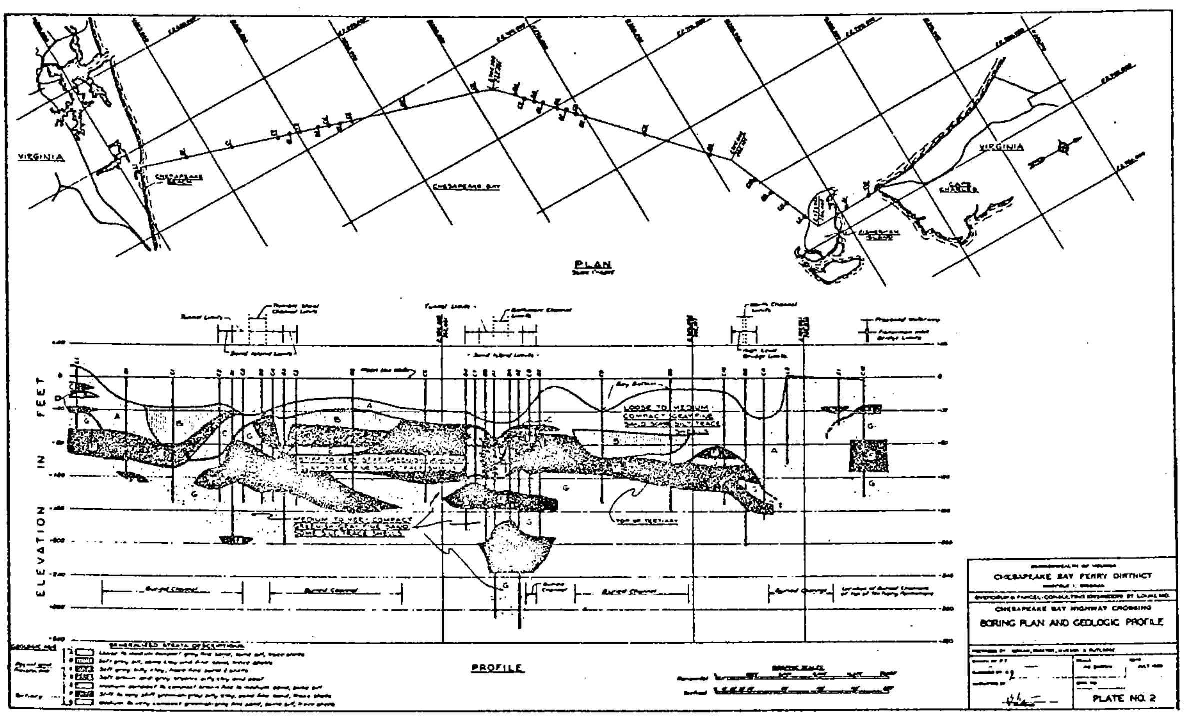


Figure 12

All of the field work required on the sub-surface exploratory program was accomplished in slightly less than 2 months. Two borings penetrated to a depth of 300 feet below mean low water. Six holes were carried to minus 200 feet, sixteen holes to minus 150 feet and two land holes were carried to about minus 100 feet, making a total of approximately 4,700 feet.

The samples recovered included 350 split spoon samples, 14 Shelby tube samples and 36 three inch diameter undisturbed samples.

It is presently estimated that approximately one hundred additional borings will be required to provide final foundation information necessary for the completion of the final plans and specifications for the project.

THE MD ENGINEERING SEISMOGRAPH AND ITS APPLICATION TO HIGHWAY ENGINEERING

By

Axel M. Fritz, Jr.[1]

As you may know, there has been considerable interest during the last year or two in the application of the MD Engineering Seismograph to highway engineering problems. Many dozens of these instruments are now in use by contractors and highway departments, and numerous papers have appeared in trade journals such as Engineering News Record and Contractors and Engineers. In view of its widespread application, it was felt that a discussion of the MD Seismograph in more detail might be of general interest. I would like to (a) describe the types of subsurface problems which can be solved, (b) explain the principle of operation, and (c) give some examples of highway engineering applications.

The subsurface problems which can be solved by the MD Seismograph can be classified as follows:

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[1]Geophysical Specialties Co., 15409 Robinwood Drive, Hopkins, Minn.

1) Determination of thickness of a surface layer, or depth to a layer boundary.

2) Determination of depths to two layer boundaries, for example the thickness of soil overlying gravel together with depth to an underlying bedrock.

3) Dip of the rock surface, if it is not horizontal.

4) Identification of materials in each of the layers, as for example gravel, clay, solid rock, weathered rock.

5) Engineering classification of the material properties, such as bearing capacity, rippability.

The operating principle of the MD Seismograph can be illustrated by Figure 1. Sound waves from a sledgehammer blow (or explosive) will travel through the subsurface along various paths. Some

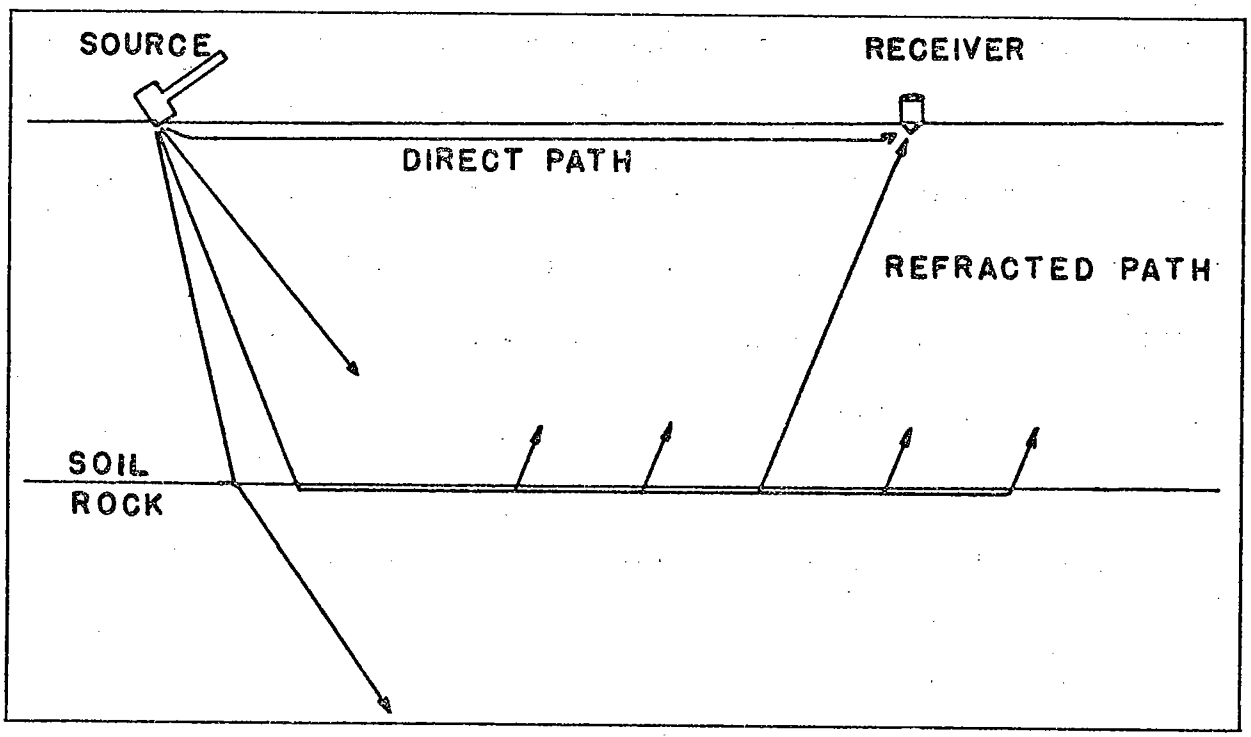


Figure 1

of the waves travel a "Direct" path to the receiver, which is located on the surface a few tens of feet away. Other waves travel down to the high-speed rock layer (which we assume here to be present) and back to the receiver. It is the function of the MD Seismograph to measure the time of travel for the fastest wave, whatever may be its path. As can be seen in Figure 2, the "Direct" wave will be the fastest at short distances, but for longer distances, the "Refracted" wave may arrive sooner.

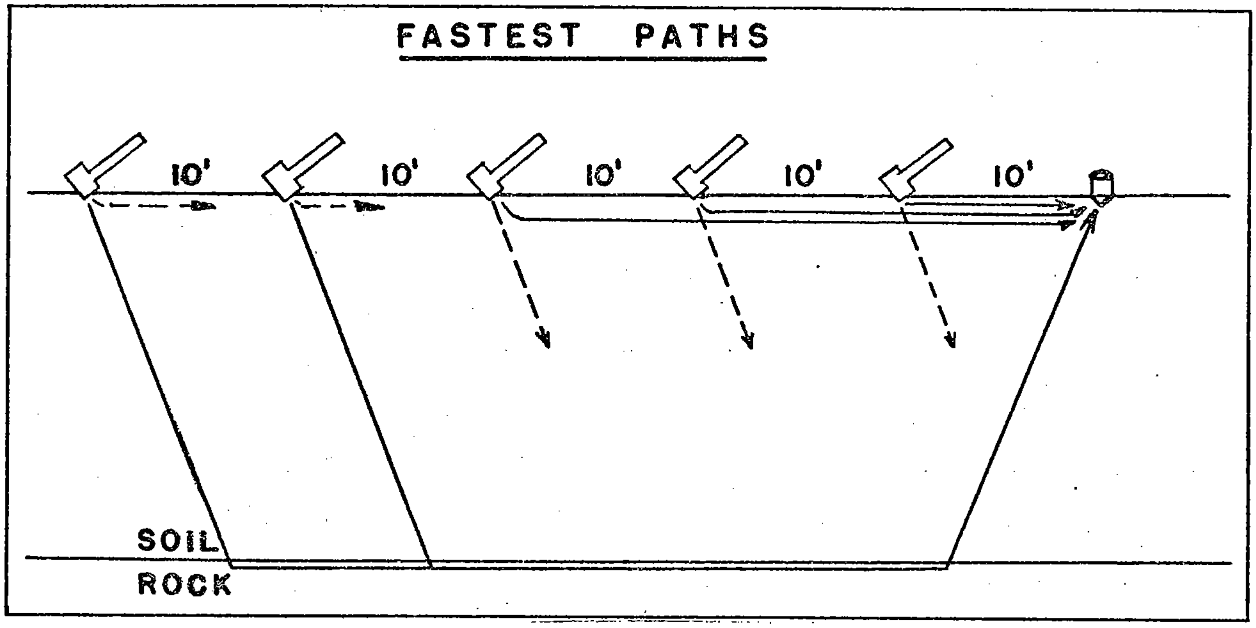


Figure 2

An analogy which we find helpful is that of an auto race between two cities which are separated by a mountain. One auto travels the winding road over the mountain, while the other auto travels along a high-speed highway around the mountain. If the total distance is short enough, the auto on the "Direct" path over the mountain will win, but for larger distances, the higher speed along the "Refracted" path around the mountain may overcome the distance handicap.

Figure 3 shows a typical set of readings, taken along a line of 5 hammer stations. When graphed, the data appear as in Figure 4.

FIELD DATA

DISTANCE	TIME
10 FEET	10 MILLISECONDS (= .010 SECONDS)
20	20
30	30
40	32
50	34

Figure 3

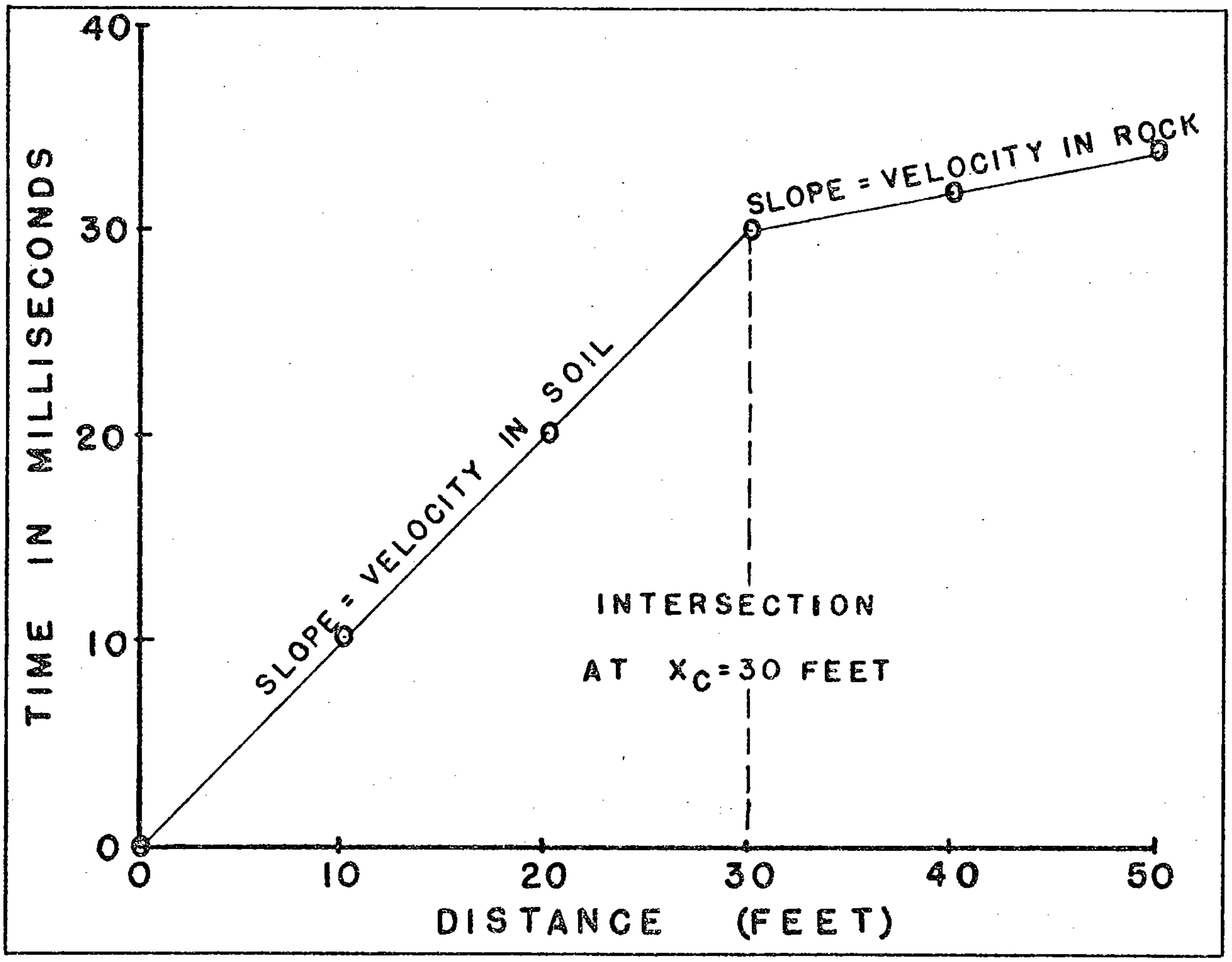
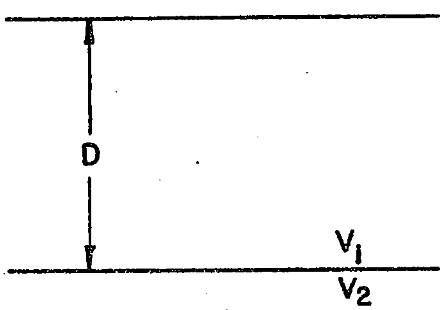


Figure 4

Three quantities can be read directly from this graph: the distance at which the intersection occurs, the slope of the first line segment (which equals the speed of sound in soil), and the slope of the second line segment (which equals the speed of sound in rock). The interpretation is shown in Figure 5: the materials are identified by their characteristic velocities, and the depth is found to be 12 feet to rock.

INTERPRETATION



1. V_1 = 1000 FEET/SECOND
   (CHARACTERISTIC OF SOIL)

2. V_2 = 5000 FEET/SECOND
   (CHARACTERISTIC OF ROCK)

3. DEPTH $D = \frac{X_C}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$

   $D = \frac{30}{2}\sqrt{\frac{5000 - 1000}{5000 + 1000}}$

   D = 12 FEET

Figures 6 and 7 show how the MD Seismograph performs its function of measuring travel time. As shown at the bottom of Figure 7, the time reading is obtained instantaneously by means of lights on the instrument. Figure 8 pictures the unit with accessory equipment.

One of the important applications to highway engineering has been in the prediction of rock rippability. Rock velocity, as measured by the MD Seismograph, can be correlated with degree of consolidation and with rippability. Figure 9 shows how this correlation is made.

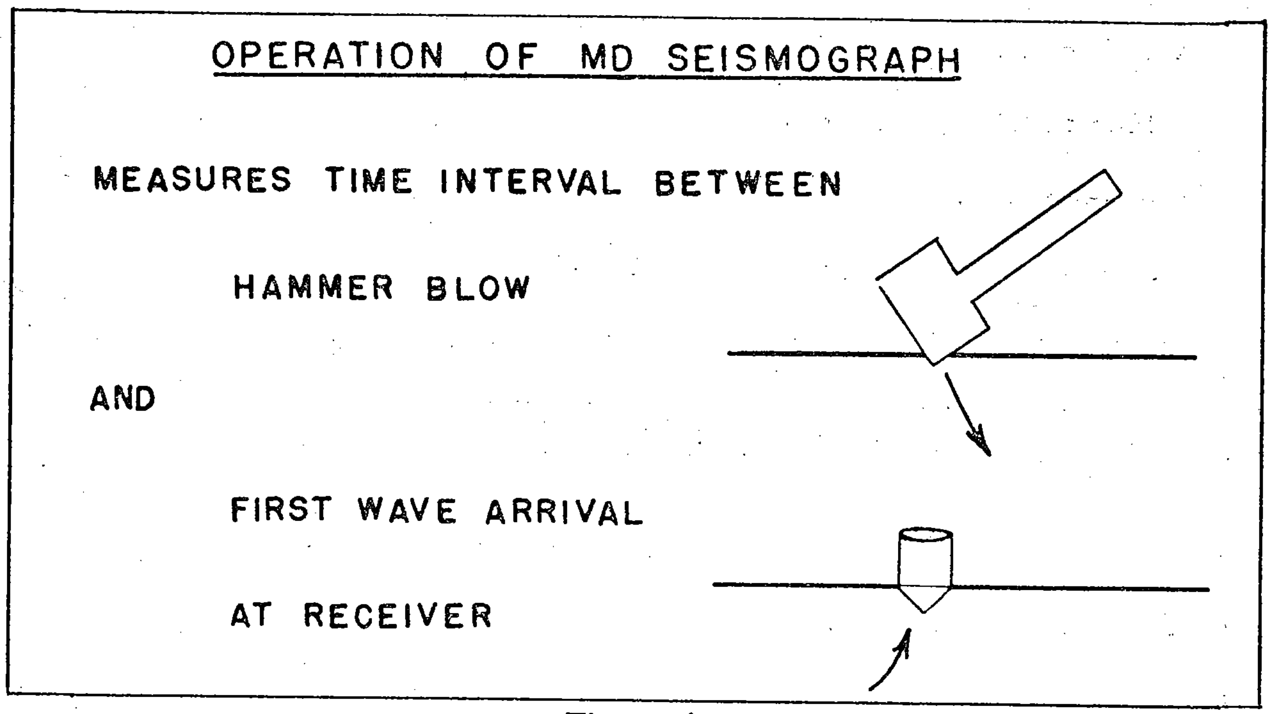


Figure 6

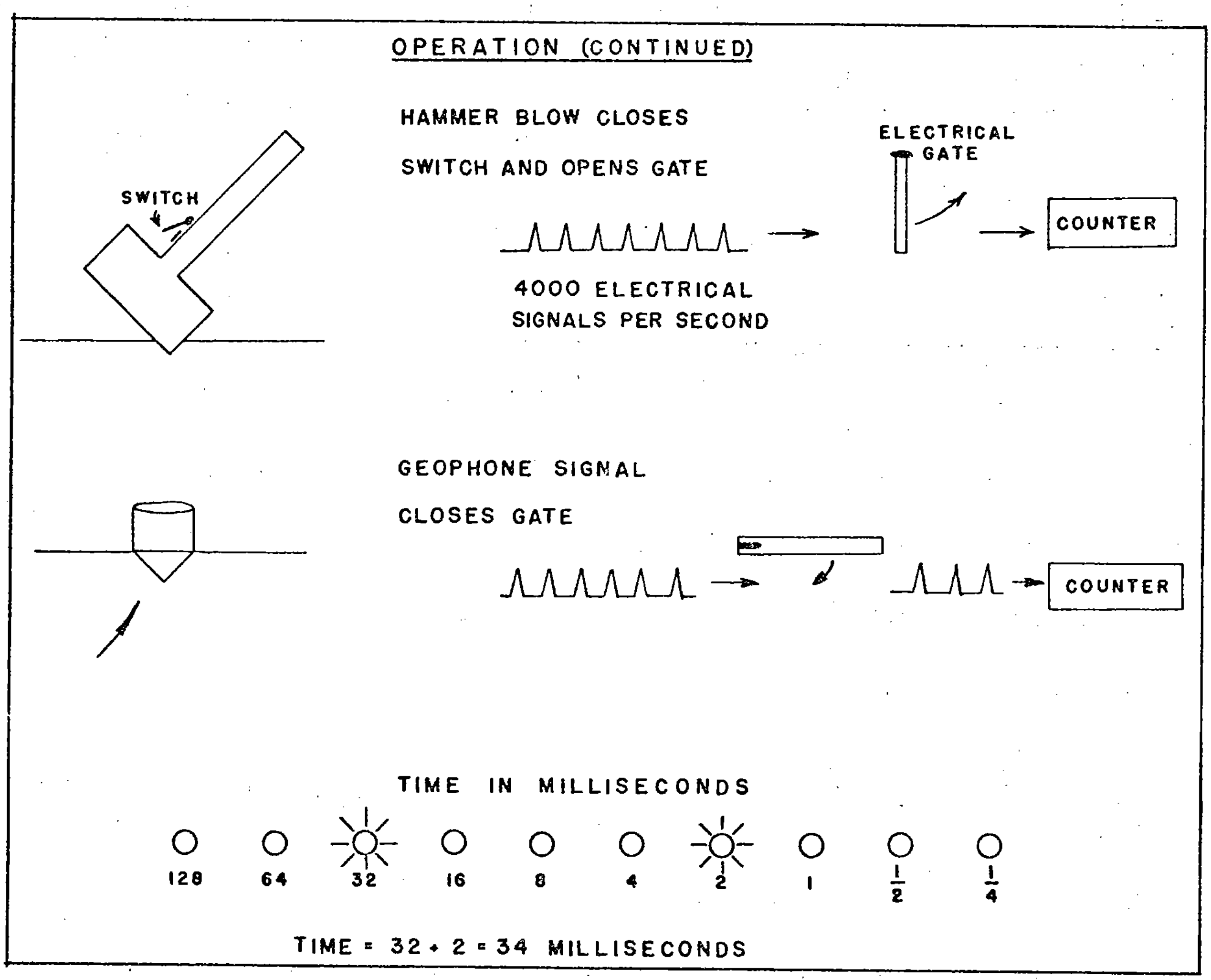


Figure 7

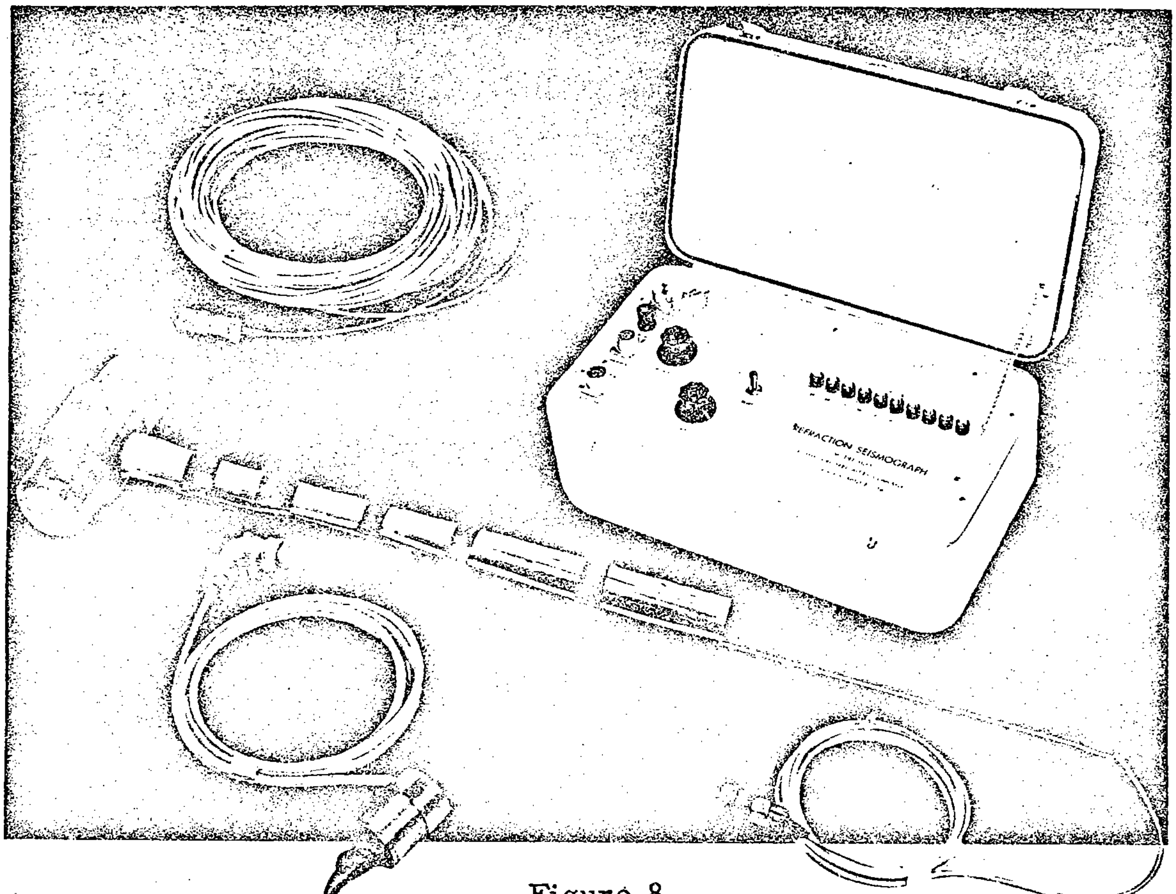


Figure 8

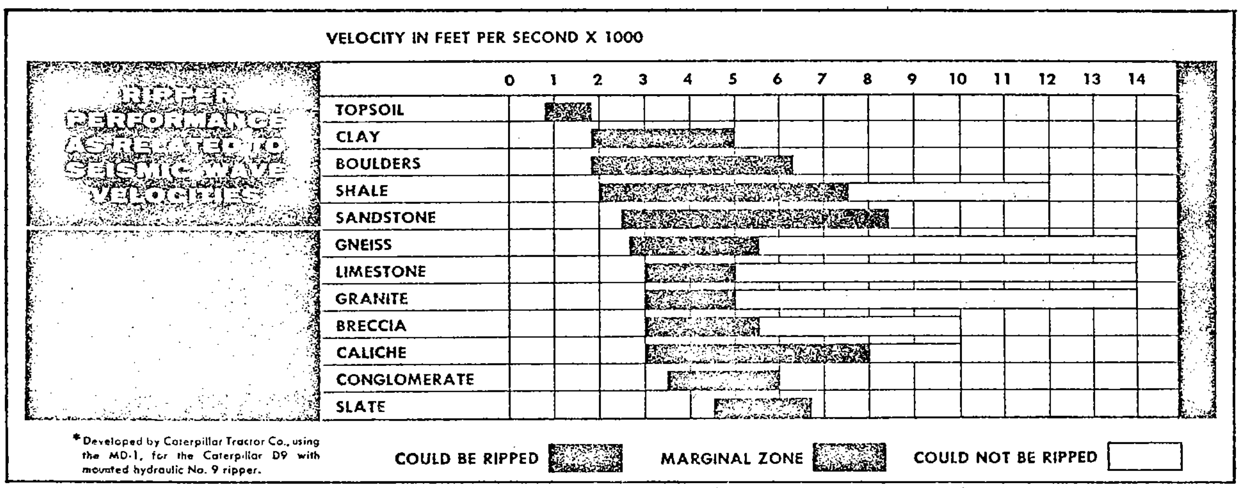


Figure 9

By predicting rippability in advance of bidding, cost estimates can be greatly improved. This offers a tremendous competitive advantage.

The first example discussed here, a relocation of U. S. Highway 127, is typical of the most frequent type of request for subsurface information made to the Michigan State Highway Test Laboratory in Ann Arbor. Bedrock was suspected to be within the depth of the proposed cut. Seismic soundings taken by the Soils Section showed the bedrock to exist at one end only, and there it was well below the grade line. The seismic investigation located a body of compacted clay and loam, which was shown to be slightly above grade at the center of the cut. From this information, a profile was constructed (as shown in Figure 10) and given to the design engineers for their use in designing the sub-base and for finalizing the grade line. The information was also made available to contractors bidding on the job. The overall result was lower bids, with substantial savings for the State of Michigan.

As can be seen from this slide, borings were made to correlate the seismic findings. They proved to be useless in establishing bedrock depth because they met refusal at points well above the bedrock, due to scattered boulders in the area.

Figure 11 shows another highway re-location job. The information made available from the seismic survey permitted the Soils Section to classify materials to be excavated; this resulted in lower bids. In this case they determined which materials could be moved by

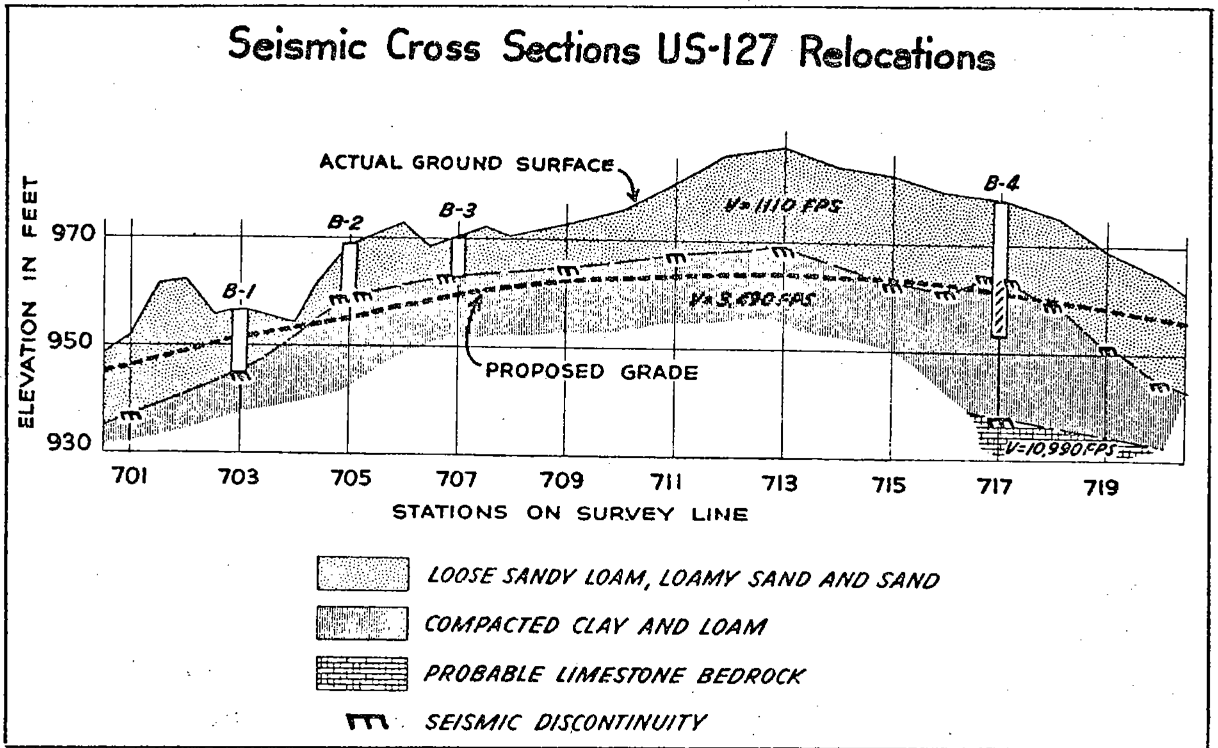


Figure 10

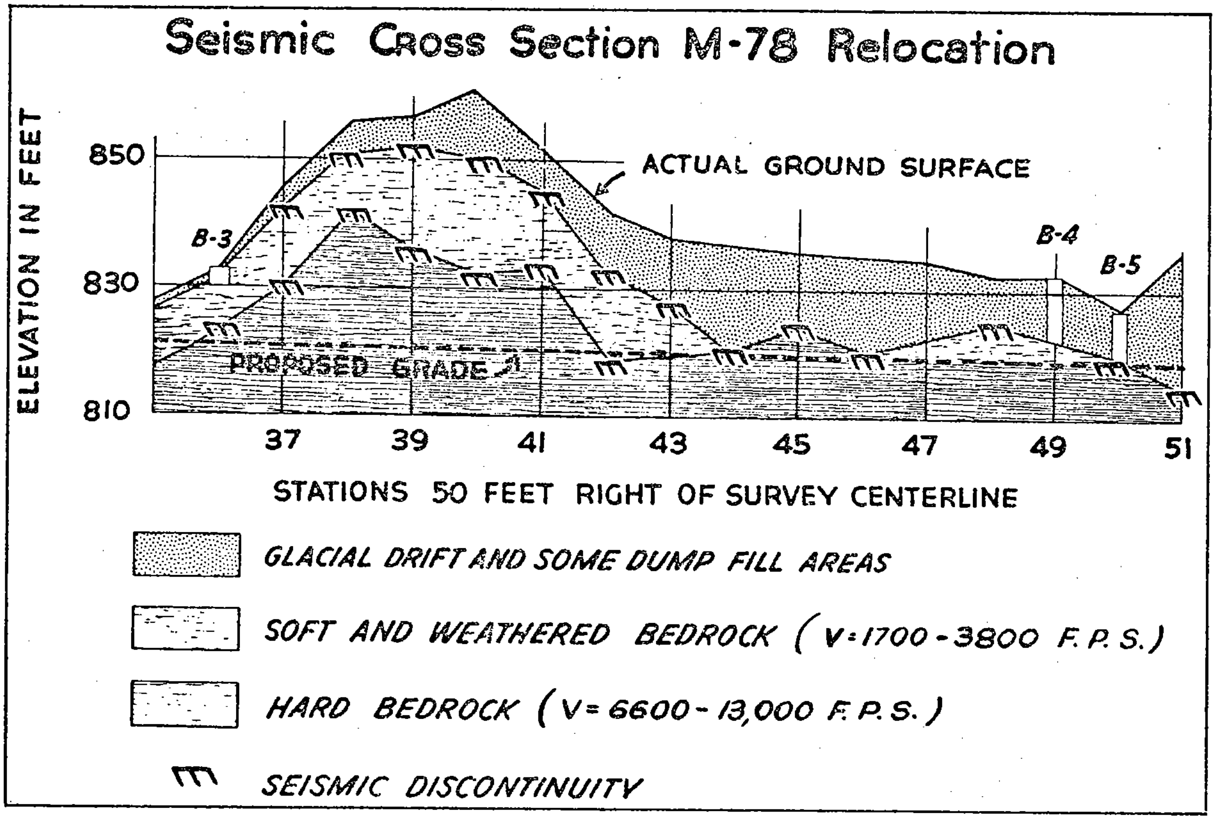


Figure 11

scraper alone, which would have to be ripped out by a heavy tractor-ripper combination and then moved with scrapers, and which would have to be drilled and shot.

Figure 12 shows an application in locating a borrow pit. Using the MD, Michigan State Highway personnel proved its effectiveness in establishing depth of overburden and quantity of deposit of suitable aggregate materials. Figure 12 shows the seismic profile for this Michigan State Highway Department borrow pit. It was lying on a slope and covered with thick underbrush. It was difficult for any other

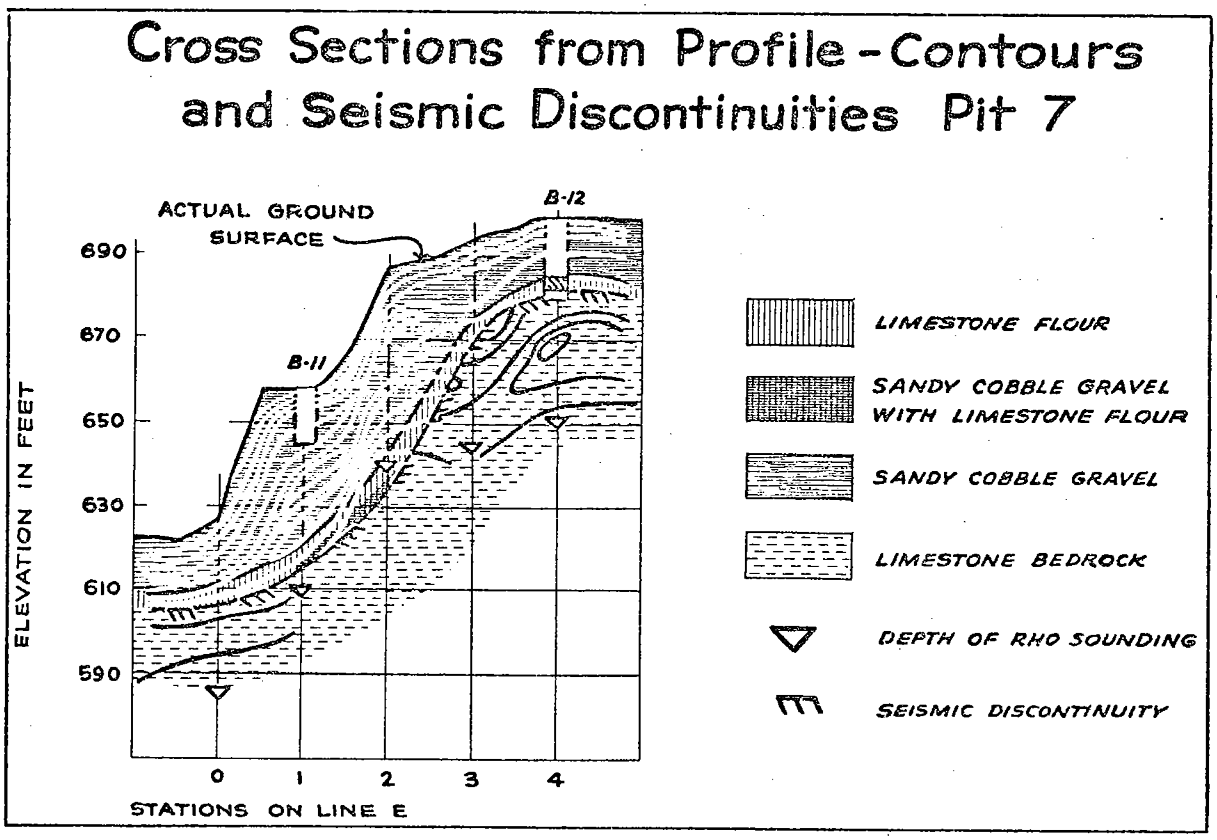


Figure 12

exploration method to be used because of access. Through the use of the MD Seismograph, the existence of suitable borrow was established and delineated.

Electrical resistivity equipment proved helpful in providing more exact information about the nature of the materials in this glacial drift deposit, but proved useless for providing depth information because as is shown here, the lines of electrical resistance did not follow the layers of soil materials. Bedrock and overlying drift had similar electrical properties, but vastly different seismic properties as was demonstrated by seismic analysis.

These are just a few of the wide variety of problems which can be handled by the MD Engineering Seismograph. The unit has been field proved for over three years in climates as varied as Greenland, Alaska, Guam, Pakistan. It is now in use in 23 countries of the world.

BEACH EROSION AND PROTECTION IN FLORIDA

By

Per Bruun[1]

Florida has about 1300 miles of general shoreline, not including islands, of which no less than 800 miles is sandy beach.

The Florida peninsula was built up of sand on top of a limestone footing. Sand material for this process came from the Appalachian Highland, carried to the sea by rivers and streams; it then driften southward along the shoreline, moved by the action of waves and currents and was deposited in barriers, spits and recurved spits, cuspate forelands, etc.

Florida is now at war with coastal problems. Waves and tides on two seas threaten to wipe out its priceless heritage of sandy tropcal beaches which bring the state about 1/3 of its income.

Figure 1 gives an impression of the general situation in regard to shore erosion in Florida classified according to the rate of shoreline recession. Erosion which is caused by natural forces only is generally low, e.g. about 1 ft. per year, if not interfered with by coastal

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[1] Director, technical sciences, Coastal Engineering Laboratory, Gainesville, Fla.

inlets. Meanwhile, there is considerable evidence that natural erosion is increasing not only in Florida, but elsewhere along the shores of the United States. In Florida we find such evidence on the upper East coast at Ormond Beach which now experiences an increasing amount of coquina shell eroded from outcroppings, emerged as well as submerged. On the Gulf coast evidence is found in (still) undeveloped areas on the lower coast in Sarasota and Lee counties.

The slightly rising sea level - when considered over longer periods - may be partly responsible for this development. Sea level rose 3 - 4 inches in the 1930-1950 period but now seems to be stable again. Another reason may be that our limestone, coquina or coral reefs gradually erode and leave the shore behind them more exposed. Natural erosion is something we have to live with. The only way we can counteract it in the long run is by barring the littoral longshore as well as transversal drift completely (and it is most unlikely that it will ever be possible to do that), or by replacing the material eroded.

Beach erosion is caused by wave and current action which moves the beach material along the shore. The material so moved, usually sand, is called "littoral drift". Wave action is always the predominant factor on the open seashore, while currents play an important role at channels and inlets. Littoral drift on the Florida East coast is predominantly southward and the same is true for the lower West coast, while it is westward on the upper West coast.

Erosion begins when more material is removed from a coastal area than is deposited in the same area over a given period of time. At some places it is the beach itself which is eroded and the shoreline recedes rather slowly. At other places both the beach and the offshore bottom are eroded and the shoreline usually recedes more rapidly. The most extreme case of erosion occurs when the erosion of the offshore bottom exceeds the erosion of the beach itself. This condition develops instability in the beach and offshore profile which often demonstrates itself by a pronounced recession of the shoreline during heavy storms when the profile, being too steep, suddenly flattens out again.

It is unfortunately not a matter of coincidence that in Florida the heaviest erosion has been experienced in the areas of most concentrated development along our shores, e.g. in the area between Palm Beach and Miami on the east coast and between Clearwater and Sarasota on the lower Gulf coast. Fig. 1 gives a vivid impression of the erosion rate which demonstrates shoreline recessions of up to 30 ft. per year. The responsibility for the increase of erosion in these highly populated areas lies in man's interference with the natural shore regime.

The author is convinced that groins at some places in Florida have done considerably more harm than good. The same statement cannot be made when it comes to seawalls. In numerous cases they have protected valuable property or dune faces, but when built as vertical bulkheads, they have in just as many cases caused a lowering and increased erosion of the beach itself. Sloping walls of various designs have now been introduced in Florida and will usually do a much better job of beach preservation than vertical walls. But they should be built with skill and care.

Our most formidable erosion enemy is, as demonstrated by Fig. 1, our improved inlets which work as barriers to the natural littoral drift. Nobody will deny the great importance of these inlets, past and present, in Florida. When they were established or improved, erosion was no concern; but it is now. Under the newly adopted state permit procedure for coastal structures - in which the state is cooperating with the Corps of Engineers to mutual benefit - it is unlikely that any inlet will be improved in Florida without eliminating the adverse effects of erosion by introducing by-passing plants or other by-passing arrangements. The applicant will be made fully responsible for any damage to downdrift beaches and will not be granted a permit unless he agrees on corrective measures to be taken simultaneously with the improvement, or, in any case, as soon as adverse effects have appeared. Bond issues are required to assure that funds are available for proper action. This practice is followed in all cases of groin construction (public property excepted) and it has been encouraging to experience only little objection against such practice introducing a higher ethical standard on coastal protection.

COASTAL PROTECTION

Florida is now losing approximately 15,000,000 cu. yd. of beach sand per year from its most important 200 miles of tourist beaches on the lower east and west coasts. Shoreline recessions up to 20-30 ft. per year occur. The extent of the problem is fully recognized from the fact-finding engineering side. But, what can we do to protect our shores?

Coastal protection is demonstrated by nature in some instances. A natural headland acts as a jetty or a groin. Islands and reefs serve as breakwaters and often cause material to be accumulated behind them. Accumulations of stones from land areas which have been eroded, or outcroppings of rock or shell serve as seawalls or breakwaters. Rivers supply material to the beach in a manner similar to methods of artificial nourishment.

Along the Florida shoreline there is little natural coastal protections of importance. There are no real headlands but in some cases reefs serve as seawalls or submerged breakwaters. As a general rule it can be said that the maintenance of Florida beaches depends on the natural equilibrium between the supply and erosion of material. Unfortunately, unimproved and, in particular, improved inlets have seriously disturbed this balance.

Coastal protection in Florida, heretofore, has been almost entirely in the hands of individuals who, in many cases, have built their homes and other installations too close to the shoreline. As a result of this, very large sums of money have been spent in some localities for construction of protective works. On other beaches the owners are facing the alternative of complete loss of valuable property or very heavy expenditures to protect what is left of their land.

There has been a tendency to grasp at almost any suggested method which seems to promise some measure of success at the least cost. Types of construction which have been abandoned elsewhere, such a permeable groins, have recently been built in Florida. The results, almost without exception have been disappointing and there are unfortunately too many examples of such work, conceived in desperation.

The following is a short review of coastal protection measures:

Seawalls have the advantage of being able to protect the area behind them under storm and high tide conditions. But they cannot accumulate material, and when built as vertical or steep impermeable walls (sheet-pilings), located close to the water's edge, they have the disadvantage of increasing erosion of the beach in front of them. Vertical walls should therefore be avoided on seashores if they cannot be built at such distance from the shoreline so that they are touched by wave action only under storm conditions and not at every high tide.

Sloping walls of different types, impermeable as well as permeable (rubble mounds), are more considerate to the beach and can therefore be built closer to the water's edge. Fig. 2 shows a sand asphalt revetment with a boardwalk suggested for an open sea coast on the southeast coast of Florida. It is supposed to be built of natural beach sand but it is often necessary to add fine (filter) material as well as coarse sand to the natural sand. The asphalt mixture should vary according to sand composition, location and steepness of revetment and may be 10-20% asphalt in the base layer which is "painted" with a layer of asphalt with higher penetration "surfaced" with a (usually) light colored material, coarse sand or shell. The asphalt cover can be replaced by interlocking concrete blocks (22" x 22" x 5") resting on a filter layer of fiberglass material as shown in Fig. 3. Great care should be shown in the construction of such revetment and all joints should be covered adequately with filter material.

In bays and lagoons wave action is less and vertical sheet-pile walls may be the most practical solution. Fig. 4 shows prestressed concrete sheet-pile walls of Florida State Road Department standard type. Protection against oversplash - erosion behind the wall can be obtained by different designs as suggested including gravel layers separated from the sand fill by fiberglass filter material or by an asphalt or interlocking concrete block slab. For exposed sections a rubble mound in front of the wall is practical.

Groins are built perpendicular to, or at a slight angle toward the shoreline. They have the advantage of accumulating material and building up a beach and, as already mentioned, the disadvantage of interposing a total or partial obstruction to the littoral drift which causes leeside erosion.

Groins can be classified as:

a. permeable or impermeable, and

b. adjustable or non-adjustable

Most groins are of the impermeable, non-adjustable type. Meanwhile this type, particularly when built rather high, causes lee-side erosion. Permeable groins of various designs - often patented - are in use but have generally been unsatisfactory because they fail to work under storm conditions when they are most needed.

The best groin is either the very low impermeable non-adjustable groin or the impermeable adjustable groin. The latter usually consists of prestressed concrete king piles (e.g. 12 x 14 in.) with tongue and groove connected with horizontal boards (e.g. 4 x 8 in.) of wood, preferably creosoted pine, greenheart or other imported hardwood. At exposed places a rock fill resting on nylon or plastic cloth sheets at the extreme end is advisable, in particular when stability difficulties arise from pile-driving. Fig. 5 shows such a groin which should be kept well adjusted at all times according to needs.

Sand asphalt groins of low type have been built in Maryland and New Jersey, and in Florida at Fernandina Beach. While the experience up north (using gentle slopes) have been quite satisfactory, the Fernandina installation started deteriorating quickly. The slopes were too steep.

In order to minimize the leeside erosion, a group of groins should be built with the extreme ends on a smooth curve as indicated on Fig. 6 with "Headland" and "Z-groins". The most advantageous distance between the single groins in a group depends upon local conditions of beach and offshore profiles, beach material, and wave and current conditions. Distance between groins varies between about 1.5 and about 3 times the length of the groin in the sea under normal storm conditions.

Dikes can be classified as a special type of seawall. Dikes are used when valuable, low-lying areas are to be protected. On sea coasts dikes are often replacements for natural dunes which have been washed away by the sea. They will then have to be located at a certain safe distance from the shoreline and be protected from wind erosion by proper vegetation as e.g. species of ammophila which is used extensively in Europe, and in the United States on Long Island and in North Carolina.

If a dike has to be located fairly close to the shoreline it will have to be "surfaced" with a protective layer. Sand asphalt is very useful for this purpose and has been used extensively in the Netherlands.

It is customary to build coastal roads behind such dikes usually with a drainage ditch between the dike and the road.

Artificial Nourishment is the newest device in our fight against erosion.

For thousands of years man has tried to protect himself by the construction of forts, whether such structures were earth walls, or walls of wood or brick. This method of defense can be compared with our groin, jetty and seawall coastal protection technology.

Later man tried to defend himself by throwing all sorts of material such as rock, lumber, hot water, tar and bullets against the enemy and this is in great measure the present war strategy. It took some time for man to realize that he could adopt a similar strategy in the defense of our shores against erosion. The artificial nourishment of beaches is now generally recognized as being the atomic weapon against beach erosion. But it is a new field and we are still on the testing level. This is true for the kind of artificial nourishment which is based on material dredged in bays and waterways and deposited on the beach - as we in Florida are practicing on a continuous basis at Jupiter Island and by the Florida State Road Department at the new public beach on the southern tip of Anna Maria Key - as well as for by-passing sand arrangements, whether this is done by dredging material on bay shoals just inside the inlet such as at Hillsboro Inlet on the lower East coast or accomplished by the employment of by-passing sand plants which may be permanently installed (Palm Beach Inlet) - possibly including a feeder arrangement as planned at Ft. Pierce - or with the pump running on a trestle picking up the material where it is to be found in ample quantities. In regard to pumping from land to ocean, it is quite obvious that we cannot continue hauling sand out in the sea thereby only eroding the shore from the rear also. It will soon be necessary to extend our supply lines for material to the ocean bottom. The lower Gulf coast of Florida is a good example of the absolute necessity of having such "sand-wells" established on the ocean bottom where we may be able to find suitable material. Thanks to the numerous bay developments it is already difficult to find sand material for beach nourishment in the Tampa and Clearwater Bay areas on the Gulf coast, and the situation may be similar in other counties. Fig. 7 shows certain ideas of how such ocean-based dredging operations may be accomplished. The simplest and also most economical solution is the "shallow water dredge" which can pump material from very shallow water such as bay (and offshore) shoals. The "Texas tower solutions" may be useful where extensive sources exist. Mentioning the dredging atomic powered submarine may cause the reader to smile right now, but it may be in operation within the next five to ten years. Needless to say, the problem is not only a matter of just dredging but also a question of pumping the material to shore. A new type discharge practice will have to be developed but it is, without doubt, all possible.

Financing - Meanwhile, even with "atomic" equipment, we can accomplish little or nothing if our projects are not backed up dollar-wise and here is the weak point, in the U. S. A. in general, and in Florida in particular. We have a federal law according to which a contribution of 1/3 toward improvement of fully publicly owned beaches can be given by Congress, but we have only a little public beach. In 1957 Florida passed a law mentioning a 1/2 state-contribution to public beaches, but this law was - because of shortage of funds - never put into effect.

A practical financing plan may be secured by proper coastal zoning as indicated in Fig. 8. There has been good experience with that kind of zoning elsewhere in the world because consideration is taken of all concerned in the matter, directly as well as indirectly.

Acknowledgment - The author wishes to take this opportunity to express the appreciation of the Coastal Engineering Laboratory of the University of Florida for the good cooperation of the Geology Department of Florida State University and the Florida State Road Department in solving problems of mutual interest on the Florida shores.

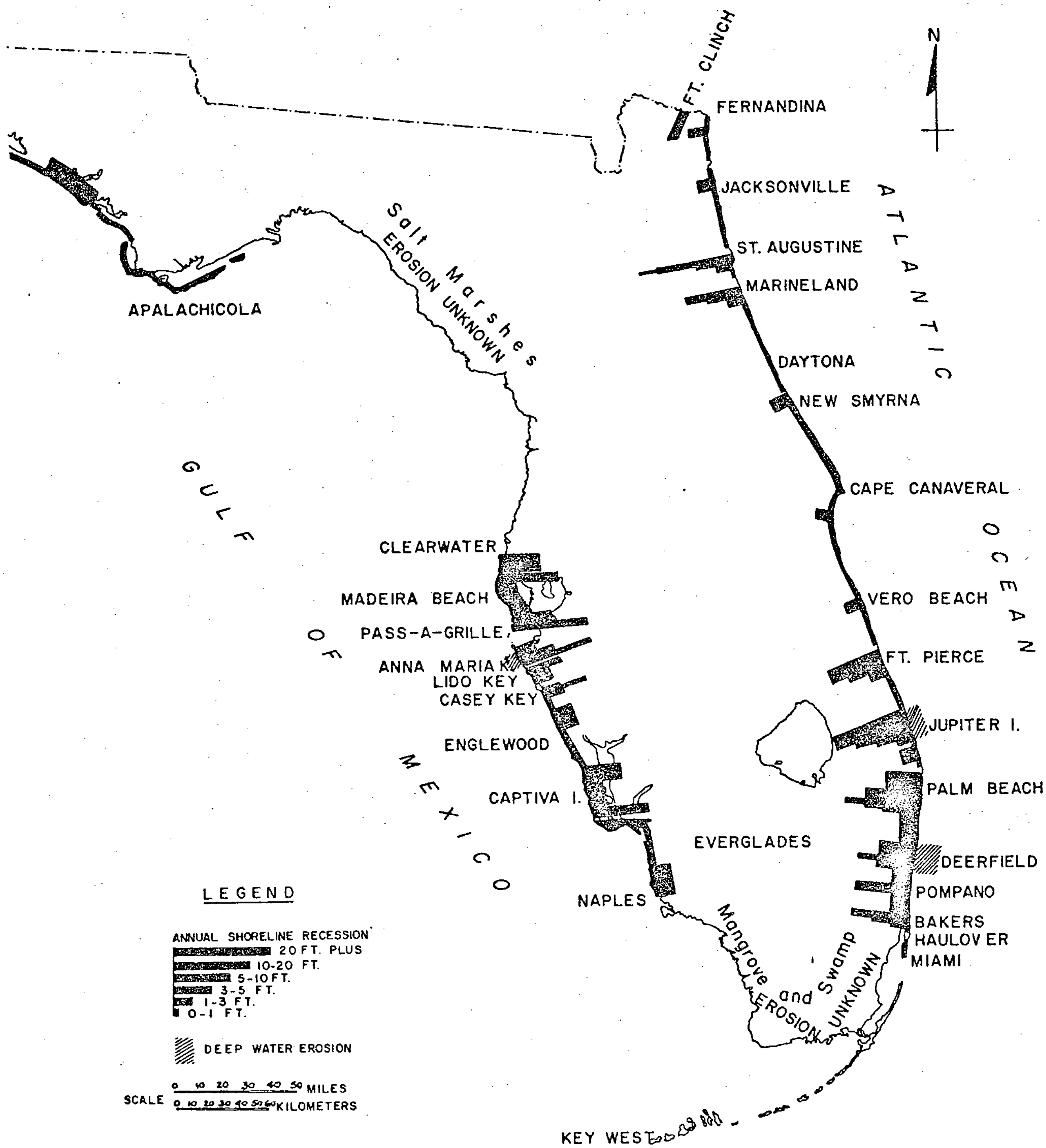
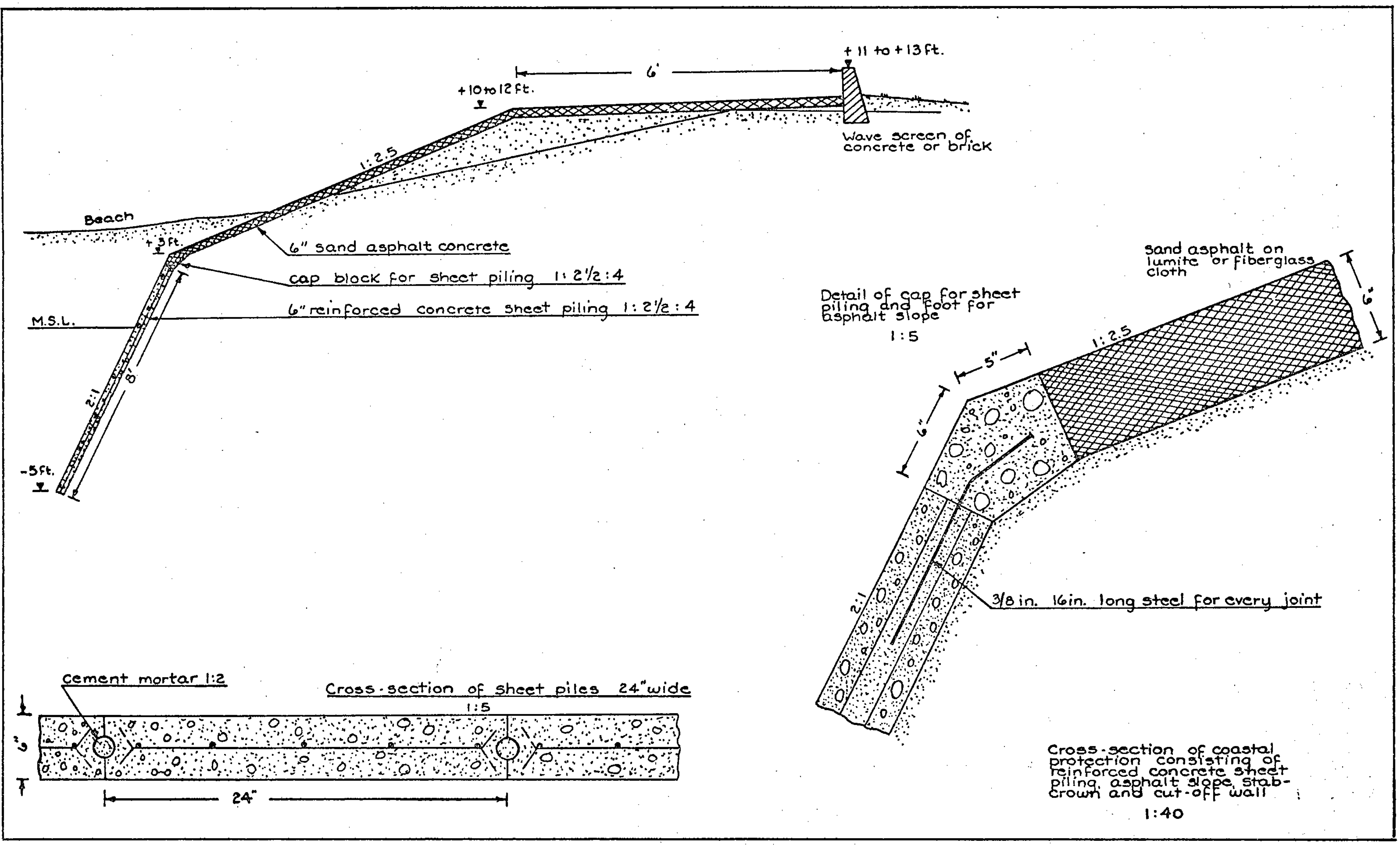

N
FT. CLINCH
FERNANDINA
JACKSONVILLE
ST. AUGUSTINE
MARINELAND
DAYTONA
NEW SMYRNA
CAPE CANAVERAL
VERO BEACH
FT. PIERCE
JUPITER I.
PALM BEACH
DEERFIELD
POMPANO
BAKERS
HAULOV ER
MIAMI
ATLANTIC
OCEAN
APALACHICOLA
Salt Marshes
EROSION UNKNOWN
GULF
OF
MEXICO
CLEARWATER
MADEIRA BEACH
PASS-A-GRILLE
ANNA MARIA K
LIDO KEY
CASEY KEY
ENGLEWOOD
CAPTIVA I.
EVERGLADES
NAPLES
Mangrove and Swamp
EROSION UNKNOWN
KEY WEST
LEGEND
ANNUAL SHORELINE RECESSION
20 FT. PLUS
10-20 FT.
5-10 FT.
3-5 FT.
1-3 FT.
0-1 FT.
DEEP WATER EROSION
SCALE
0 10 20 30 40 50 MILES
0 10 20 30 40 50 60 KILOMETERS

Figure 1



+ 11 to + 13 ft.
6'
+10 to 12 ft.
Wave screen of concrete or brick
1:2.5
Beach
+3 ft.
6" sand asphalt concrete
cap block for sheet piling 1:2½:4
6" reinforced concrete sheet piling 1:2½:4
M.S.L.
8'
2:1
-5 ft.
Detail of cap for sheet piling and foot for asphalt slope
1:5
Sand asphalt on lumite or fiberglass cloth
6"
1:2.5
5"
6"
3/8 in. 16in. long steel for every joint
2:1
Cement mortar 1:2
Cross-section of sheet piles 24" wide
1:5
6"
24"
Cross-section of coastal protection consisting of reinforced concrete sheet piling, asphalt slope, stab-crown and cut-off wall
1:40

Figure 2

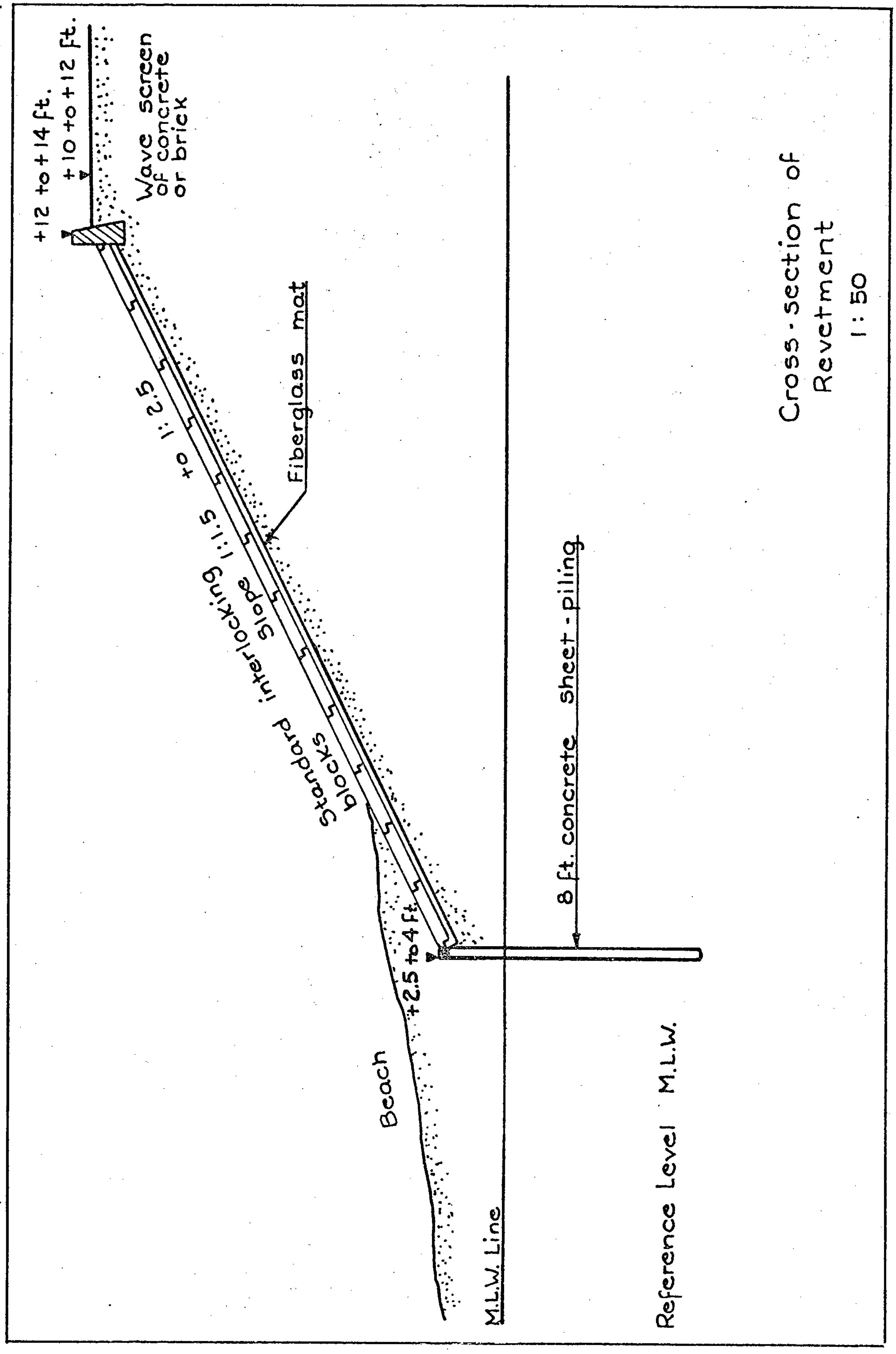

+12 to +14 ft.
+10 to +12 ft.
Wave screen of concrete or brick
Fiberglass mat
Standard interlocking blocks
Slope 1:1.5 to 1:2.5
8 ft. concrete sheet-piling
+2.5 to 4 ft.
Beach
M.L.W. Line
Reference Level M.L.W.
Cross-section of Revetment
1:50

Figure 3

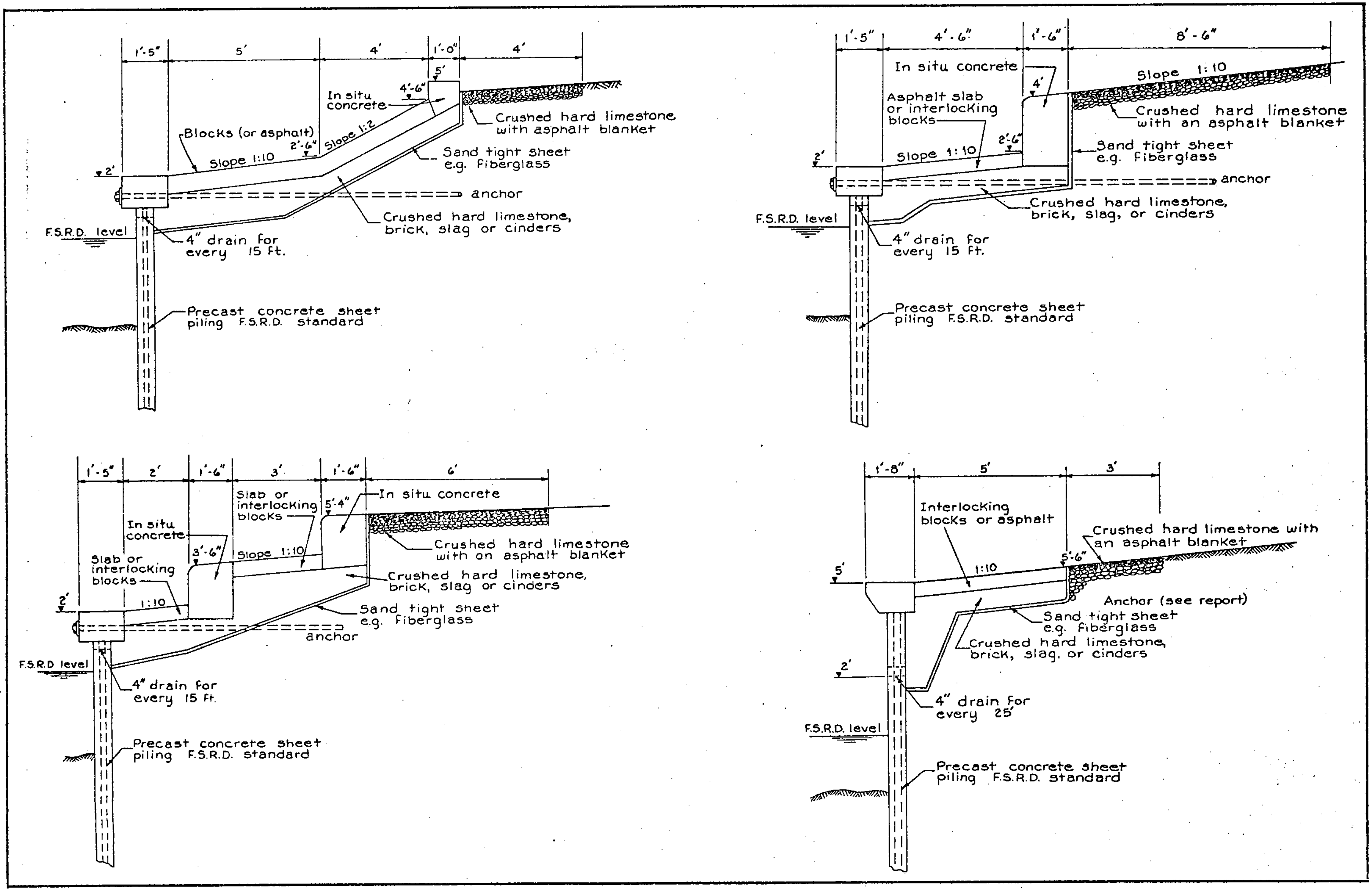

1'-5"
5'
4'
1'-0"
4'
5'
4'-6"
In situ concrete
Blocks (or asphalt)
2'-6"
Slope 1:2
Slope 1:10
2'
Crushed hard limestone with asphalt blanket
Sand tight sheet e.g. fiberglass
anchor
Crushed hard limestone, brick, slag or cinders
F.S.R.D. level
4" drain for every 15 Ft.
Precast concrete sheet piling F.S.R.D. standard
1'-5"
4'-6"
1'-6"
8'-6"
In situ concrete
Slope 1:10
4'
Asphalt slab or interlocking blocks
Crushed hard limestone with an asphalt blanket
2'-6"
2'
Slope 1:10
Sand tight sheet e.g. fiberglass
anchor
Crushed hard limestone, brick, slag, or cinders
F.S.R.D. level
4" drain for every 15 Ft.
Precast concrete sheet piling F.S.R.D. standard
1'-5"
2'
1'-6"
3'
1'-6"
6'
Slab or interlocking blocks
5'-4"
In situ concrete
In situ concrete
3'-6"
Slope 1:10
Crushed hard limestone with an asphalt blanket
Slab or interlocking blocks
Crushed hard limestone, brick, slag or cinders
1:10
2'
Sand tight sheet e.g. fiberglass
anchor
F.S.R.D level
4" drain for every 15 Ft.
Precast concrete sheet piling F.S.R.D. standard
1'-8"
5'
3'
Interlocking blocks or asphalt
Crushed hard limestone with an asphalt blanket
5'
1:10
5'-6"
Anchor (see report)
Sand tight sheet e.g. fiberglass
Crushed hard limestone, brick, slag, or cinders
2'
4" drain for every 25'
F.S.R.D. level
Precast concrete sheet piling F.S.R.D. standard

Figure 4

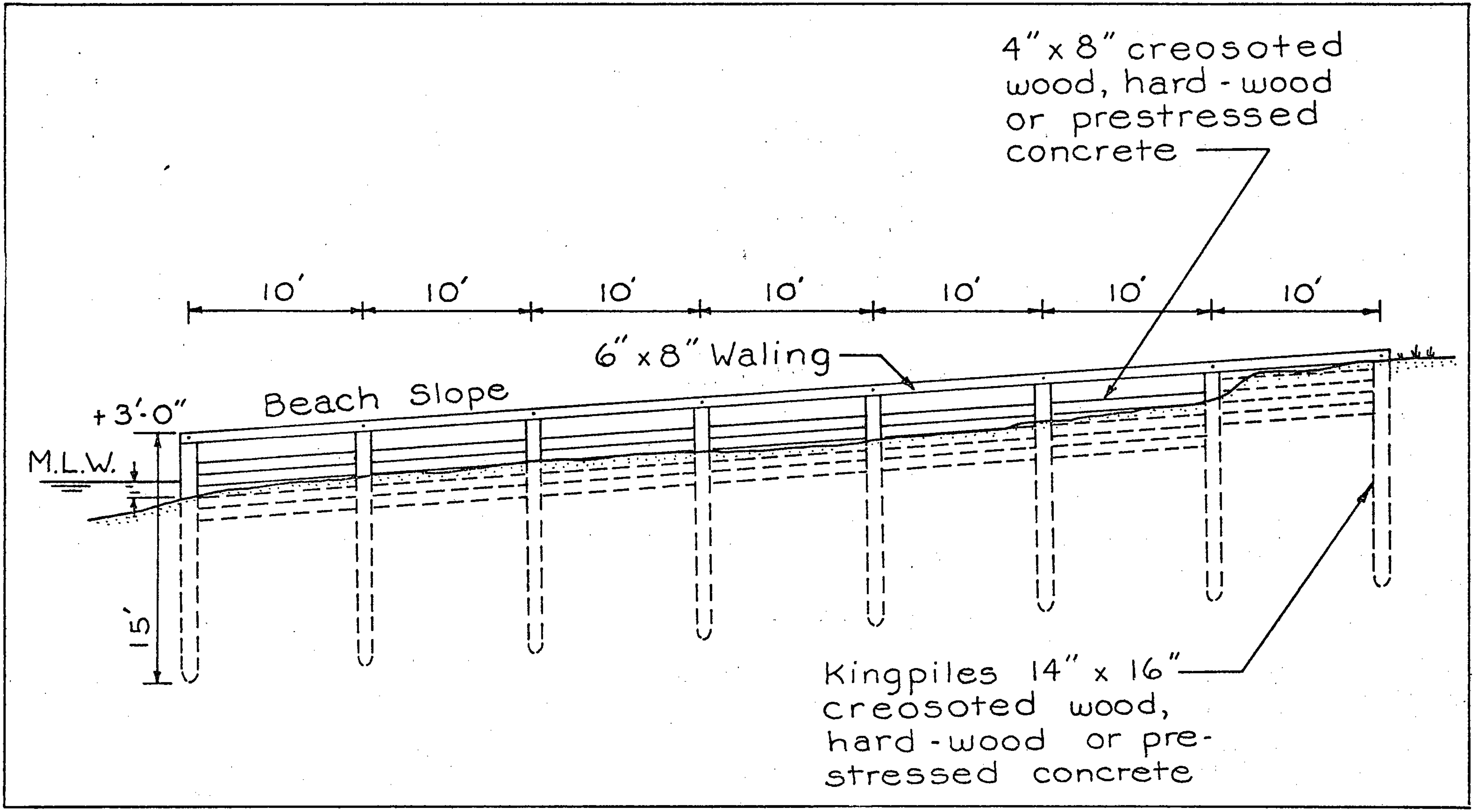

4" x 8" creosoted wood, hard-wood or prestressed concrete
10'
10'
10'
10'
10'
10'
10'
6" x 8" Waling
Beach Slope
+3'-0"
M.L.W.
15'
Kingpiles 14" x 16" creosoted wood, hard-wood or pre-stressed concrete

Figure 5

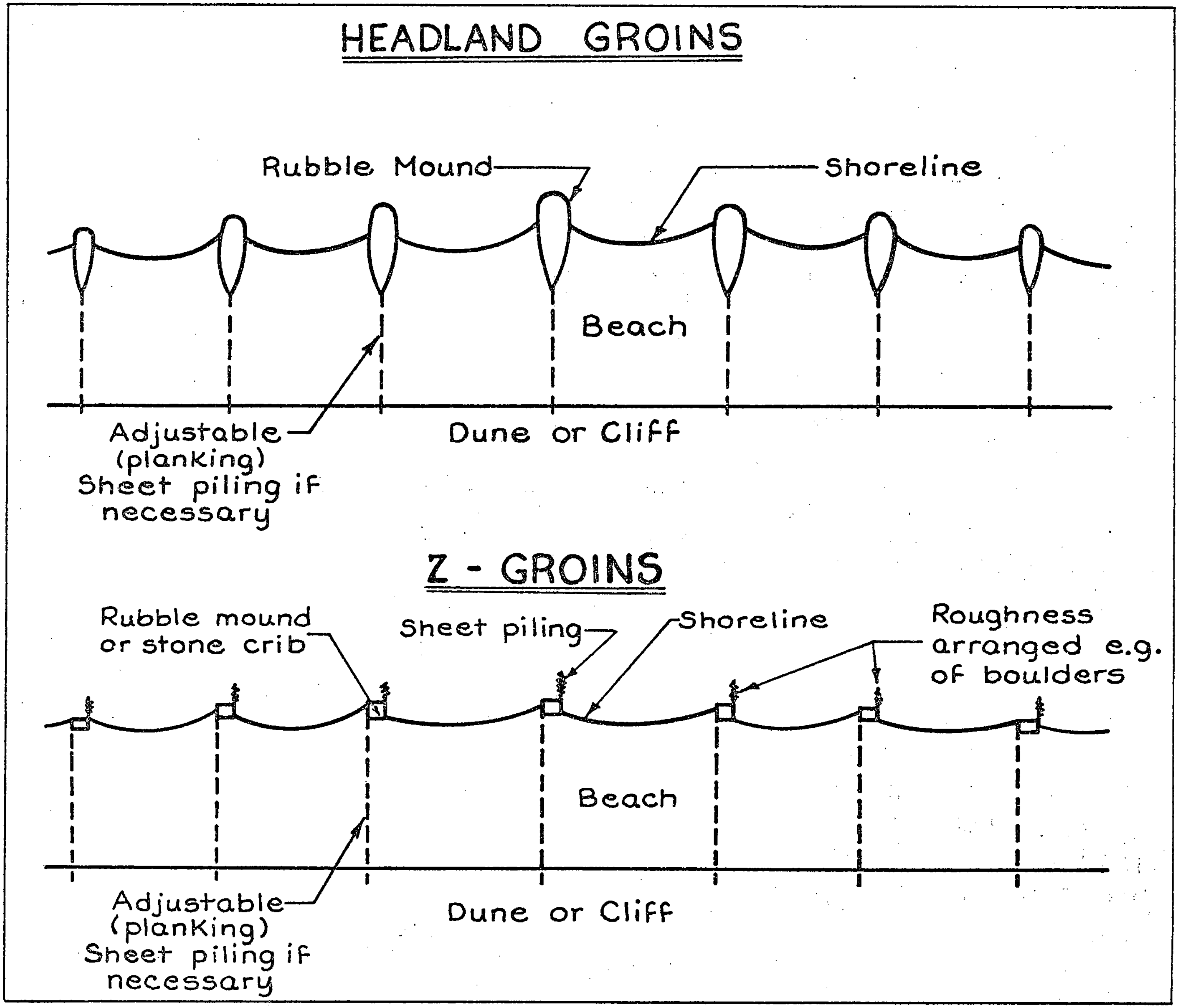

HEADLAND GROINS
Rubble Mound
Shoreline
Beach
Adjustable (planking) Sheet piling if necessary
Dune or Cliff
Z - GROINS
Rubble mound or stone crib
Sheet piling
Shoreline
Roughness arranged e.g. of boulders
Beach
Adjustable (planking) Sheet piling if necessary
Dune or Cliff

Figure 6

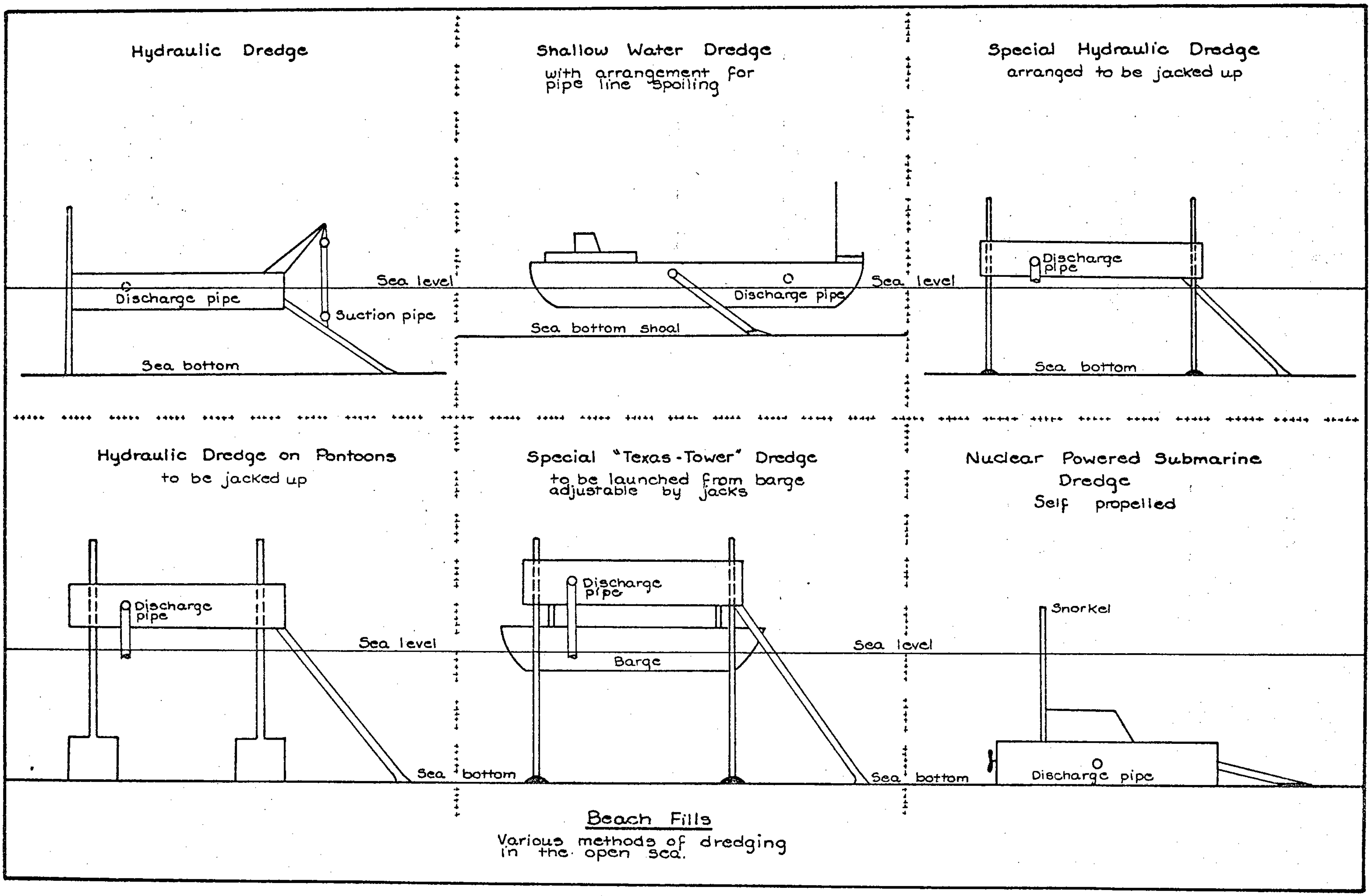
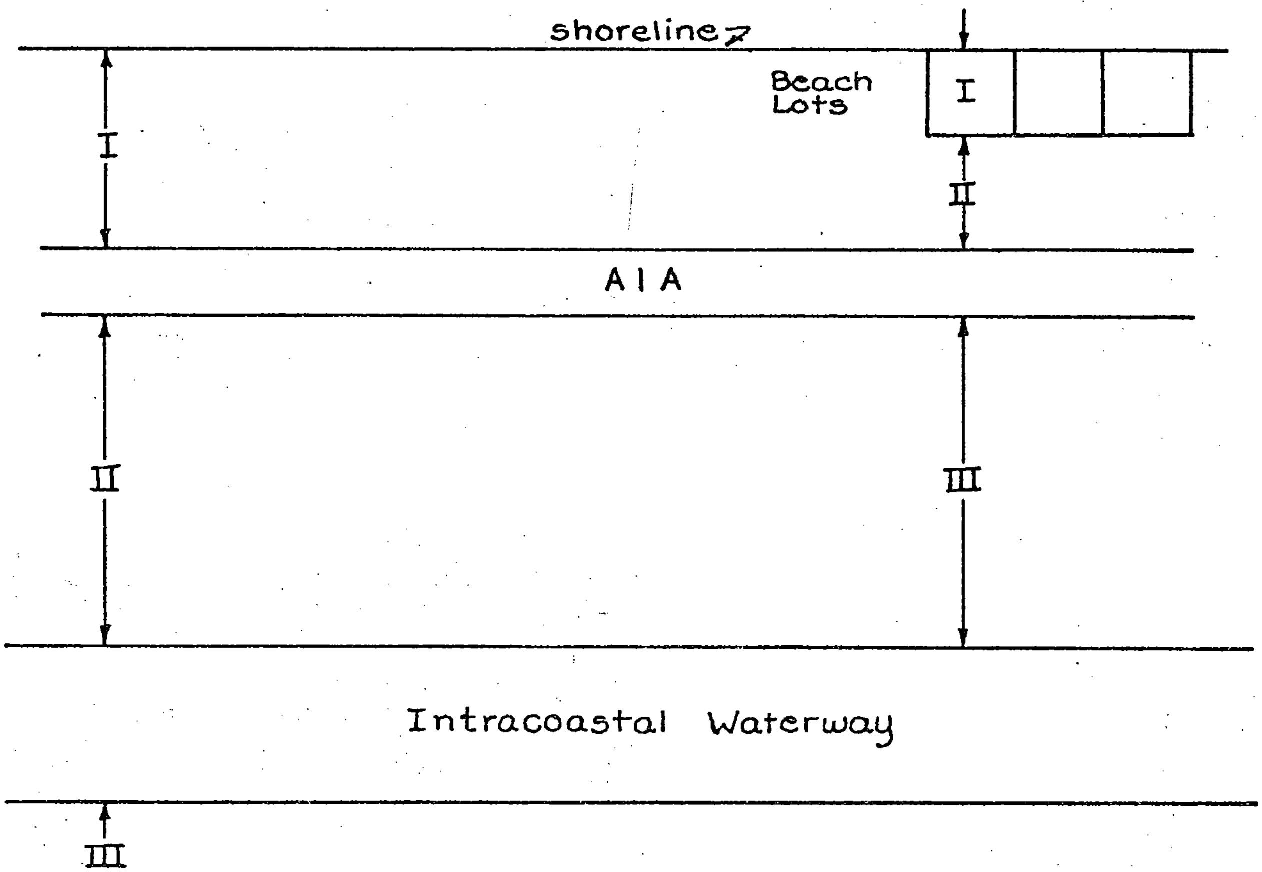

Hydraulic Dredge
Discharge pipe
Sea level
Suction pipe
Sea bottom
Shallow Water Dredge
with arrangement for pipe line spoiling
Discharge pipe
Sea level
Sea bottom shoal
Special Hydraulic Dredge
arranged to be jacked up
Discharge pipe
Sea bottom
Hydraulic Dredge on Pontoons
to be jacked up
Discharge pipe
Sea level
Sea bottom
Special "Texas-Tower" Dredge
to be launched from barge adjustable by jacks
Discharge pipe
Barge
Sea level
Sea bottom
Nuclear Powered Submarine Dredge
Self propelled
Snorkel
Discharge pipe
Beach Fills
Various methods of dredging in the open sea.

Figure 7

Coastal Zoning



Financing

Overall Plan

Individuals according to zoning	50%
County and/or City	40%
State and/or Federal	10%

Local Zoning Plan

Zone I	60%	=	30%	of total
Zone II	30%	=	15%	of total
Zone III	10%	=	5%	of total

Figure 8

PAVEMENT DISRUPTION BY RECENT EARTH MOVEMENTS

By

S. A. Lynch[1] and Paul Weaver[2]

Introduction.

Movements of the land surface in historic times have been observed in many areas and are due to many causes but this discussion concerns earth movements in the Texas coastal plain and especially those earth movements causing disruption of man-made surface structures. Most of these earth movements involve subsidence of the earth.

Subsidences.

Major.

The phenomenon of local surface subsidence is probably best known from the examples at Mexico City, Mexico and Long Beach, California. Mexico City was six feet above the level of a nearby lake in 1900 but it was over 15 below lake level in 1958. In 1954 the rate of subsidence was reported at 11.8 inches per year. Long Beach,

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California has had a total of 26 feet of settlement in the Terminal Island area between 1937 and 1958.

Minor.

Only a few cases of subsidence involve many feet of settlement and therefore command wide-spread public interest, but the engineer and geologist must pay attention to locating and following the progressive development of movements in which settlement may be only a few inches. In areas of even small subsidence there may be a need for remedial action to pavements, pipelines and buildings.

Obviously the consolidation or rupture resulting from a force depends upon the properties of the material. These properties may only be understood from the geological sub-base.

The Texas coastal plain includes that belt from the shore inland in which the near-surface rocks are unconsolidated sands, fine sands, and clays. These same rocks extend for many hundreds of feet downward. There are local exceptions such as the cemented rocks at the surface called caliche, intermittently present in the area which is more arid, that is, to the southwest; and the salt domes, which are vertical cylindrical masses of salt, topped by limestone and anhydrite and located in the near-shore portion of the coastal plain.

Kinds.

Some of the subsidences are connected with salt domes; others affect only unconsolidated layers, which have properties more analogous to soils than to elastic rocks.

Similar geological setting is also found in the coastal areas of Louisiana and in parts of Mississippi.

This paper will give examples of those classes of subsidence found in the Texas coastal region, but is not a complete review of all forms of subsidence.

Regional.

The first class of subsidence is the result of tilting seaward of large areas of the region. Not enough coverage has been affected by precise leveling repeated at long time intervals to establish whether this is generally present throughout the region. The evidence for it in the Galveston region (1) seems to show its existence, but at such a time-rate that it need not be a hazard to roads although it may affect freeboard of docks. This class may be called regional tilting.

Tectonic.

The second class is shown by the difference in elevation between two large blocks along a boundary which is nearly a straight line. Commonly the lower block is toward the coast. Instead of a topographic bluff, the surface expression is a steepening slope. These seem to have connection with larger displacements in the subsurface as faults*, along which movement occurred at intervals over a long geologic period.

The Hitchcock fault in Galveston County, which is considered of this class, has had movement of about 14 inches in the past 15 years; both breakage of pavement and shifting alignment on railway tracks were results.

The subsidence, or rather faulting, of this class may be called tectonic. No earthquake noises have been reported from these localities in the Texas coastal plain, which is one of generally low seismicity (2).

Landslide.

A third class of surface displacement which is mostly horizontal, but gives rise to subsidence, has been observed along some streams where a bank is high and steep. Even without undercutting by the stream itself, lack of lateral support of the material of the bank makes it unstable. Sinking under a house just a slight distance back from the bank of Bray's Bayou inside Houston has occurred. The largest observed movement of this kind was reported by Russell in Wilkinson County, Mississippi on the east side of the Mississippi

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*On the surface, the present movement seemingly resulted from tension and frequently is evidenced by a number of closely spaced openings, each small. The largest opening of record is a break 4 inches wide in a 24-inch water line near Clarkwood, Neuces County, in 1958. This was a supply line to the city of Corpus Christi, Texas.

River (3). Subsidence associated with these movements may be called landslide.

Loading.

The fourth class of surface subsidence is that due to consolidation of the uppermost layers of unconsolidated material by loading, which may be by a building, by flooding with water as in some irrigation procedures or by off-channel storage; finally, by an ore-bin, of which the so-called "sulphur vats" of the Texas coastal plain are examples.

These subsidences have their outer edge very close to the loaded area and roads nearby are usually not affected. They are loading subsidences.

Mining.

The fifth class of subsidence is that which results from the removal of minerals from underground, using the term mineral in the broad sense to include fluids as well as solids. We call this mining subsidence.

Solids in Fluid Form.

Sulphur.

Extraction of minerals in the solid state through shafts from underground workings is done only in one case (Hockley, Harris County) so it will not be discussed.

Extraction of solid minerals after they have been made fluid is done in this area for sulphur and salt. The sulphur is melted by use of superheated steam (Frasch process) and the melted sulphur and the cooled water is removed through wells. The rock containing the sulphur is incompetent, once the sulphur which bonded its fragments is removed, so it collapses, and the roof progressively stopes to the surface. The shape and the amount of area of subsidence beyond the area of the sulphur deposit is roughly proportional to the depth and to the tonnage extracted (4).

As the water moves underground during the Frasch operation, it sometimes gets outside the sulphur ore, and into the salt dome on which each sulphur deposit is situated. The result is salt solution which may result in a stope reaching the surface.

Salt.

Mining of salt in the coastal plain of Texas is also effected by pumping water into a salt dome through a well, where it dissolves the salt to form a brine which is pumped out through the same well or through another well connected with the first one underground in the salt. If mining of this salt is not deep enough, the roof of the resulting water-filled cavity underground may also sink and stoping also take place to the surface. Such a subsidence occurred at Blue Ridge, Fort Bend County, Texas, damaging many structures, including the packing plant. A similar subsidence occurred at Orchard (Moore's Field), also in Fort Bend County, during sulphur mining and some of the wells were engulfed.

Since highways generally avoid such mining sites, hazards might not be expected; however, there are salt domes covered by roads which are not nowbeing mined, but which might be in the future, at which time relocation of the roads crossing them might well be considered.

Fluids.

The removal of minerals which are fluids underground is by far the most important cause of subsidence, because this category includes pumpage of underground fresh water, produced in the Texas coastal region in by far the greatest tonnage of all minerals, and which as petroleum, natural gas, and some salt water comes here from the unconsolidated (and therefore subject to consolidation) coarse sands, fine-grained sands and clays. Fresh water is also extracted over large areas also by a natural process from very shallow depths and even under our highways. Since consolidation results at shallow depth similar to the result of extraction at great depth through wells, this shallow movement of water will be discussed first.

Natural Extraction.

Evaporation.

The most common natural method of extracting water from the subsurface is by evaporation to the atmosphere as part of the ever operative and ever present cycle of moisture movement into and out of the soil.

As highway pavement structure depends upon support from a consolidated base, shrinkage of the latter and its subsidence because of evaporation, which is more severe at the edges of the pavement, may cause failure along the edges of flexible pavement or produce longitudinal edge cracking in rigid pavements (5).

Transpiration.

The occurrence of distorted pavements, often producing a series of waves of rather definite amplitude and period, may be the result of shrinking of foundation soil due to the presence of rows of trees or large shrubs.

Croney (6) found that the suction exerted by the roots of vegetation can have considerable effect on the moisture distribution under pavements. Normally, the water removed by vegetation is replaced by rainfall. If, however, the supply of rainfall is removed by a prolonged drought or due to paving of the soil around the vegetation, the roots extract moisture as they continue to develop rapidly due to the presence of moisture. "When heavy clays are dried by trees to a moisture content lower than that previously reached in the soil, the soil will not reassume its initial moisture condition after removal of the trees, and some permanent settlement results" (5, p. 30).

This situation is well illustrated by numerous tree-lined streets on the campus of Texas A. & M. College at College Station. Medium-sized live oak trees were transplanted and set at intervals of approximately 35 feet and 4 feet back from the rigid concrete pavement. Presumably the soil had a somewhat uniform moisture content at the time of pavement construction and tree planting.

As the trees grew and their roots extended under the pavement the moisture was withdrawn from the clayey soil. While exact figures are not available for live oaks, plant physiologists believe transpiration by live oaks approaches that of the mesquite which uses 1,725 pounds of water for every pound of wood fiber it produces.

The pavements were deformed, curbs were cracked and shattered near the joints adjacent to the trees, with the curbs settling a maximum of six inches. It was necessary to partially rebuild the curbs and to resurface some streets.

Similar disruptions of pavement by dewatering and subsequent transpiration were noted on State Highway 21 west of Bryan adjacent to College Station. In this case a number of hackberry trees are located at least 20 feet from the pavement, yet obviously they are the cause of the damage as the pavement is not damaged in nearby areas free of trees.

Locally, unusual drainage conditions may be as serious a factor of dewatering as transpiration by trees. In local areas where pavements are built on outside hill-cuts and deep ditches are cut on the inside adjacent to the hill, dewatering of the soil beneath the pavement frequently causes pavement disruption.

Artificial Withdrawal.

Considering finally subsidence through artificial withdrawal of fluids, we have the extraction of petroleum, natural gas, and salt water from oil fields in the unconsolidated sands in the Texas coastal region, and of fresh water for domestic, municipal and industrial purposes, usually through a group of wells fairly close together. We therefore have what is called either an oil field or a water-well field.

Petroleum.

Surface subsidence as a result of petroleum production with accompanying natural gas and salt water at Goose Creek, Harris County, Texas between 1918 and 1925 has been discussed by Minor (7), Pratt and Johnson (8), Snider (9), and Sellards (10). The rate at which the surface was subsiding there was followed by repeated leveling in that period by George H. Lacy, and recorded in testimony in a lawsuit. The data showed submergence of the land into the bay in the oil field proper, and faults and subsidence even beyond the wells' area (8). Maximum subsidence was 3.3 feet. Oil produced was about 50,000,000 barrels, with a large proportion of salt water towards the end (11, Figure 15). Production is still continuing.

At the Wilmington oil field in California and also to a lesser amount in Playa del Rey, Torrance, Santa Fe Springs, and Inglewood oil fields, surface subsidence is being measured. At Wilmington, total subsidence from 1937 to 1958 was 26 feet at the center and the surface area involved was about four times that of the oil field. During this period about 1,000,000,000 barrels of oil were produced (12).

Some of the hazards of mining petroleum involve city streets and state highways. In drilling oil and gas wells often it is impossible to secure the surface rights directly above the desired subsurface location, and it is common practice to use a favorable surface location and directionally drill the well to the subsurface objective. This is particularly true when the subsurface objective directly underlies buildings, streets and railroads. It is also common practice to pump both solids and fluids into wells to fracture and/or acidize the producing formation to boost its production. There is considerable difference of opinion whether or not it is possible to control the direction of fractures in subsurface formation.

It is readily seen that fluids pumped into relatively shallow formations under sufficient pressure to fracture the formations may produce vertical fractures that may extend to or near the surface, thereby causing damage to engineering structures by the fractures themselves or by fluids passing through the fractures. Even when surface fracturing does not take place, the overburden may be lifted.

Except that the production of petroleum and natural gas is from much greater depth and is in lesser volume when subsidence occurs, it is in effect a part of the same basic process as the shallow dewatering which we discussed as operative near the surface.

Subsidence in oilfields is more likely to occur if there be in the oil producing unconsolidated sands a lack of effective water-drive.

<u>Fresh Water</u>.

Fresh water from wells into consolidated sands, called <u>aquifers under confined conditions</u>, is extracted in the coastal region of Texas in enormous volumes, and at high rates locally. Records show that wells capable of producing in excess of 1,000,000 gallons per day are possible in large areas (1, Plate 13 and p. 21).

Here again, subsidence as a result of this water extraction involves the process like the shallow dewatering near the surface, namely, the consolidation of clays. Instead of roots to pull water, we have wells with pumps, and also the energy of the underground water under compression initially, and the sands and clays above and below each of the sands also under compression.

This pressure head was such in the Texas Gulf Coast region that when the first wells were drilled, they flowed above the land surface in the area near the coast line and inland for some miles (13). Now static levels in the Houston area are about 150 feet below sea level, whereas fifty years ago they were more than 50 feet above sea level, a drop equivalent to 12 tons per square foot loss in the support the former head afforded the beds above the aquifers. As to the aquifer itself there are two effects, expansion of the water and compression of the aquifer. These are not large proportionally to dewatering of adjacent clays, which is a third effect caused by the hydraulic gradient created into the aquifer from the clays next to it. The clays lose water, and overburden net pressure on them increases as the counter-pressure from the aquifer below has been reduced. Consolidation of the clay is of course under greater forces than in the analogous case of dewatering at the surface, where overburden is very thin.

If the clays had been already consolidated into shales, and their porosity were low and also the permeability practically zero, the resulting effects would be confined to those within the aquifer itself. But in the Texas coastal region, clays are unconsolidated, porosity is of the order of 35 percent; permeability, although probably immeasurable by present techniques, is existent, so that the clays give up much water with enough time.

How much subsidence results from any particular water production? In the first place, this would require a knowledge of the properties of the sands and clays, the original water pressure, and the rate of withdrawal. Carothers and Newnam have shown the manipulation of these data to obtain the answer for an idealized case (14).

For the civil engineer, the salient results are (1) the area of the surface subsidence will be larger than the well field; (2) the subsidence will have a saucer shape, flat at the bottom of the depression, then a rapid slope upward at the edge of the bottom; and finally a gentle long outer part of the saucer, but concave upward; (3) due to variation in each different layer, inequalities of consolidation will develop and reach the surface, (4) any well-field which has operated at large rate will have more and more production proportionally from the clay compared to the sand for the same time.

An excellent example, studied by Gray (11), shows how these factors have operated, and particularly some of the surface manifestations of damages to roads and other structures.

The locality is the Baytown, Harris County, Texas area. Although there are many water wells in this area, close-spacing of them to constitute a large water-well field occupies only a small part of the region.

The interpretation of the subsidence around this well field shown in Figure 1 is complicated by several other factors which affect the surface:

1) The most important of these is the effect of large water withdrawals west of Baytown towards Houston from the same aquifer and from deeper ones, giving rise to a very large lowering of water tables and surface subsidence of its own during the same period as that tabulated by Gray for Baytown, namely 1943 - 1953.

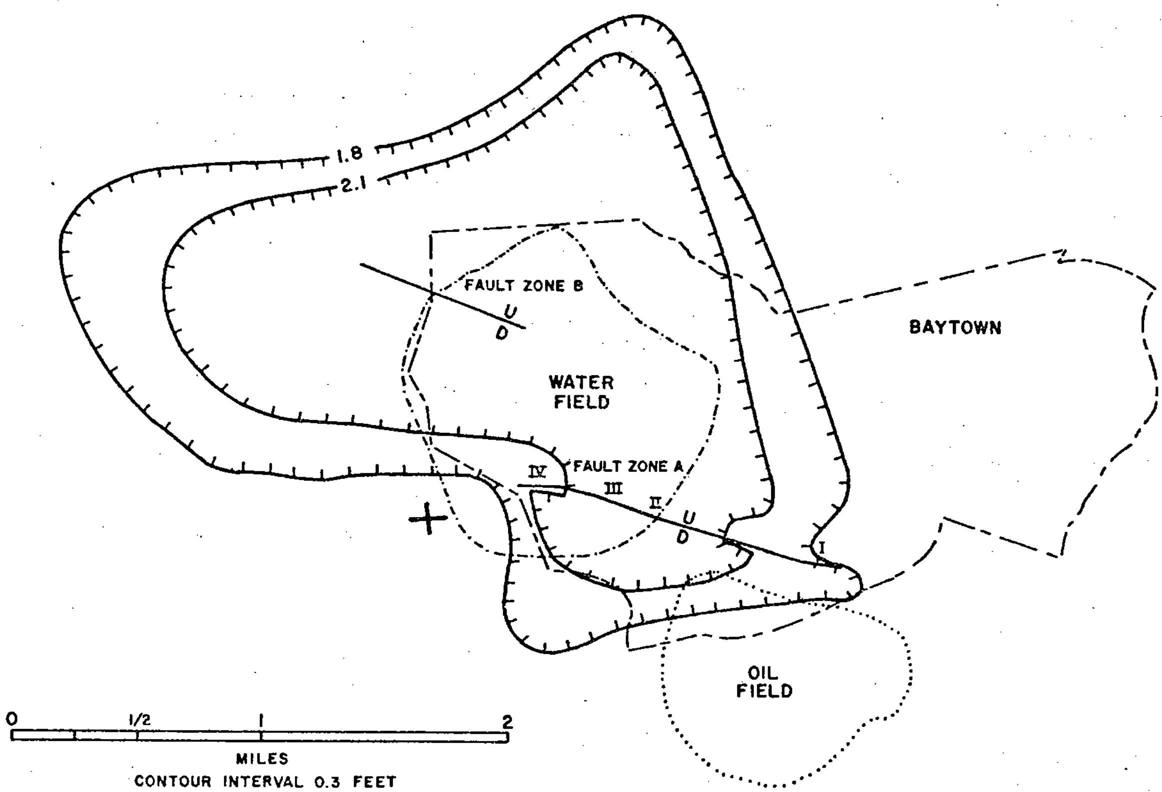

FIGURE 1. SUBSIDENCE AND FAULTING IN BAYTOWN, TEXAS AREA CONTOURED ON DIFFERENCES IN ELEVATION FROM 1943 THROUGH 1953.

Figure 1

2) The second factor has been the regional tilt postulated for the whole area, Houston and Baytown alike. Its vector is unknown, but the local subsidence mapped possibly should be reduced by several tenths of a foot.

3) Third are the effects from oil, gas, and water withdrawals which are still continuing, but not at the same rate as when the subsidence was originally studied in 1926.

4) Fourth, the placing of spoil from the ship channel and other surface activities. This is believed responsible for the re-entrant in the subsidence contours near the point "X".

Broadly, the two inside contours of this subsidence at Baytown of 2.1 and 1.8 feet from 1943 to 1953, are parallel, and show to be an oval outside of the water-well field, but parallel to its outer edge. These two contours correspond to the rim of the bottom of the saucer. (Outer contours as well as inner contours are omitted because of limited control data. The 1.2 foot subsidence contour, north and east would be beyond the 1.8 contour one mile to the north and 2 miles to the east.)

How is the subsidence related to the water withdrawal from this large well field? The observed value of total subsidence was calculated by Gray as 105,800 acre-feet (11, page 29) from 1943 to 1953. If we consider the effect of the other factors listed immediately preceding, this should be corrected by removing them. A rough calculation suggests reduction to 60,000 acre-feet.

In this same period Gray tabulates water withdrawals from the water-well field (11, Plate VI) as amounting to 235,000 acre-feet for 1943-1953.

Winslow and Wood have estimated that in the Houston area generally, the percentage of the water produced which comes from the clays is 22 percent (15). Using this conversion, the clay water loss at Baytown, which is also the clay volume loss, would be about 52,000 acre-feet, of the same approximate magnitude as the surface subsidence, corrected for other factors.

But a subsidence, even of this magnitude evident over an area of about 100 square miles would not be very serious if the depression were smooth. At Baytown, in addition to the bending downward, there are fault-lines, so-called, where the slope is accentuated. Gray mapped two of these, designated A and B, near the well-field. Figure 2 shows four profiles across A, designated I, II, III, and IV on the map. Other disturbances will be discussed, not on the fault-lines.

At point IV on zone A, at Market Street and Avenue F, Baytown, the displacement is 1.16 feet. A fault, with 0.7 foot displacement, passes diagonally under a house near zone B, causing the brick footing to separate from the house. The pavement and curbs of several streets in Baytown show offsets and frequent repairs are required.

Burnet School, near zone B, has had one corner of the building displaced by the fault movement and tie-rods were installed to hold the building together. A water well for Burnet School had the electric pump set about three feet above ground level on a concrete slab set six inches into the ground. The concrete slab, formerly six inches

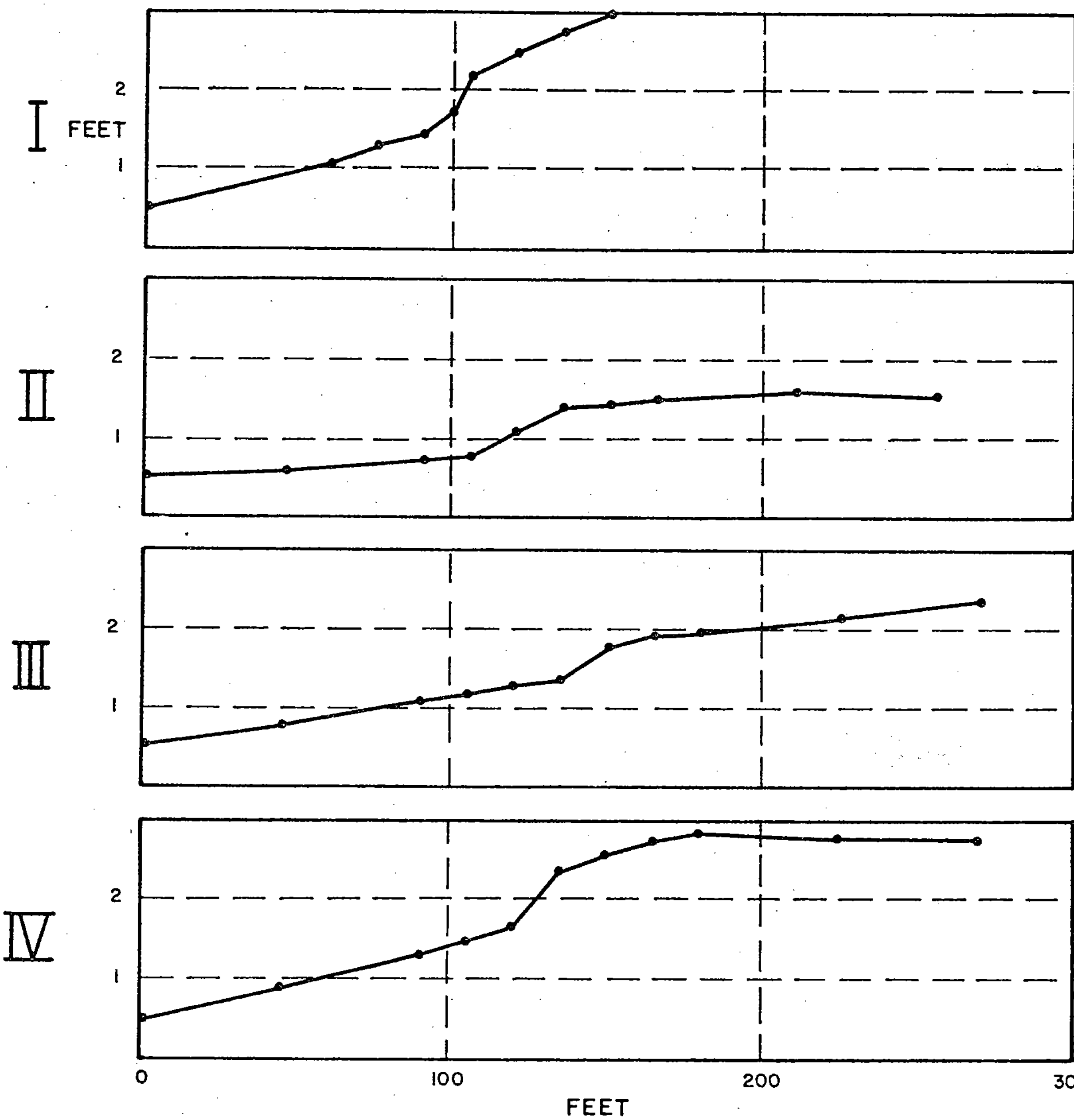

FIGURE 2. PROFILES ACROSS ZONE "A" FAULT, BAYTOWN, TEXAS.

Figure 2

into the soil is now eight inches above it and is now supported only by its bond to casing cemented at 200-foot depth. Allowing for compression of this casing, the total subsidence is evidently greater than 14 inches.

Baytown is not the only area where subsidence is presenting problems to engineers and geologists. Very shallow water wells produced at high rates from confined aquifers as well as in unconfined areas frequently crater and not always around the casing. Similar cratering may take place during the completion of an oil or gas well, or afterwards. Also there may be surface effects from an underground leak from these wells.

In the Brazos River flood plain, extensive irrigation for cotton is common. In a few instances the withdrawal of water from the coarse, shallow gravels has been so great that the fine surface soils ran downward near the wells, forming cone-shaped depressions. Many of these wells are located along the edge of the highway right-of-way. Likewise, many irrigation water wells in the rice area of Texas and Louisiana are as near as possible to the property line and highway. There are numerous instances where the withdrawal of water has allowed enough local subsidence to damage the pavement in the vicinity of the wells.

Numerous faults are being observed on roads in the Houston district, which includes the Baytown local area just described. Winslow and Wood report a total water withdrawal through wells in the district for the 1943-1954 period of 114 by 10^{10} (1140 billion) gallons and a corresponding total subsidence of 25.4 by 10^{10} gallons (obtained by converting the volume of subsidence to gallons of water). The maximum subsidence in the district in this period was a little over three feet.

Large industrial plants in the Houston area are turning to ponds supplied from surface water for cooling water due to the subsidence problems involved in the use of shallow subsurface waters. One company stated their water requirements were 2.8 pounds of water per kilowatt hour produced under the best forced evaporation system. They cannot take the risk of extracting this quantity of water for many years from the subsurface beneath their plant area.

If the use of heat pumps for both heating and cooling becomes widespread, great quantities of shallow subsurface water will be used. It is obvious that this practice may cause damage to local streets and pavements, near the extraction wells.

With the ever increasing use of subsurface water, the engineer and geologist should no longer ignore declining artesian pressures as they are a source of potential danger to engineering structure.

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ROCK AND EARTH IN THE HIGHWAY PROGRAM

By

H. A. Radzikowski[1]

Your organization is to be congratulated. Last September, in keeping with the growing importance of geology to modern highways, you changed your stature from a regional to a national symposium. With your broad national horizon, you will appreciate more fully the significant role the geologist can play in this greatly expanded highway program.

For in your workshop - this land of ours, are the rocks and earth which have a two-fold impact on the highway program. In the first place, until this long-range highway program is completed some 6000 highway contractors will be moving nearly 20 million cubic yards of rock and earth each working day. How efficiently and economically they accomplish this phase of the highway program will depend to a certain extent upon you - the geologists.

This is not a new role for the geologist. About half the State highway departments have from one to ten geologists on their staffs.

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All but one State has the equivalent of a geological survey, although the names of the organization vary. All State highway departments utilize the geological maps and data produced by these State organizations, as well as those of the U. S. Geological Survey. The geologists employed by the highway departments augment, rather than duplicate, this geological information.

No, this is not a new role for the members of this symposium. It is a job that many of you have been doing for some time. And it has been a job well done, too. Let me show you why.

Back in 1925, the average bid price for common excavation on highway work was 39 cents per cubic yard. The average bid price for the same item last year was 38 cents. Ten years ago it was also 38 cents. This stability in the cost of moving rock and earth, in spite of the higher costs of labor, supplies and overhead, is due primarily to the greater degree of mechanization and the continuous improvement of earthmoving equipment.

But there is another factor, too. When a contractor moves to a highway construction project today, he usually has considerably more information concerning the geology of the area than was the case 35 years ago. With this information he is in a better position to select the best equipment to perform the grading and compaction operations most efficiently and economically. Because you have helped reduce the unknowns in moving rock and earth in highway construction projects, you have helped us to hold the price line on common excavation.

I wonder how many people appreciate the full impact of the fact that this price line has been held. The savings in cost alone are substantial since grading costs amount to about 30 percent of the total expenditures for the highway program. These savings can be converted into more miles of badly needed highway improvements.

Modern earthmoving equipment coupled with up-to-date applied geology have had even farther reaching benefits for the highway program. Because they have enabled us to greatly increase our productive capacity to move rock and earth, there has been an evolution in highway design standards. These new standards feature such improvements as flatter grades, easier curves, longer sight distances and greater widths for rights-of-way, shoulders and surfaces. It would not have been possible to attain these improved design standards with the methods and equipment for earthmoving and for geological subsurface explorations that were in use 35 years ago. Because we have advanced in these areas in the quantity and quality of our productive capacity, the economy, the welfare and the safety of our citizens will benefit from these improved design standards.

There have been many other new developments in highway engineering which will benefit the road program. They are significant to you because they denote an atmosphere of open-mindedness - a willingness to try out new ideas in the highway industry.

Not long ago, a set of plans was unrolled in my office. This set of plans was an excellent illustration of the renaissance that has taken place in highway engineering in the past five years. Faced with a greatly expanded program, a serious shortage of engineers and a pressing need to maintain or improve the quality of construction, highway officials undertook an all-out effort to increase the productive capacity of their engineers, streamline their procedures and improve their control over quality. The set of plans unrolled in my office that day demonstrated many of their accomplishments.

This set of plans detailed the construction of 14 miles of highway on new location through fairly difficult terrain. The only field work performed until the plans were completed was installing ground control for aerial surveys. Even this phase employed a new development - the tellurometer - which permits the accurate measurement of lines from 500 feet to 30 miles or more in length in a matter of 30 or 40 minutes.

Because we are here primarily to discuss highway geology, the various new developments involved in arriving at the preliminary location and design for this road will be treated briefly. They are pertinent to our discussion, though, because some are already under consideration for extension into the field of highway geology.

Aerial surveys coupled with the latest photogrammetric techniques, the electronic computer, and other new electronic equipment, produced and analyzed the data required for preliminary location and design. Many more alternate locations, alignments and grades were investigated than would have been possible with the accepted methods of a few years ago because of the time required, particularly for field work and hand computations. Where a major structure was necessary, four different types of bridges were investigated thoroughly. Stress analyses were made and the sizes of members in the superstructures were determined in each case. Preliminary designs of the approaches also were made. The selection of the bridge type was based upon a more comprehensive evaluation of many more variables than possible a few years ago since the computations now can be performed even hundreds of times faster than before.

Right-of-way estimates and plans were made from large scale enlargements of the aerial mosaics. This procedure minimized the danger of skyrocketing real estate prices which sometimes occur when ground surveys are made.

Drainage areas were established, drainage structures located and pipe sizes determined. Slope intercepts also were established. Cross-section data was obtained and preliminary estimates of earthwork quantities were calculated on the electronic computer.

But now, let us look at some of the new developments in applied geology that are being used to an increasing extent on highway projects such as this and others across the Nation. There are others, too, in various stages of development.

Just before I left Washington, I was informed that the development of a new electronic computer program for soil analysis had been completed and was now in use. With this program, laboratory test data from soil samples can be analyzed and the samples classified on the electronic computer. This can be done at the rate of six or seven samples per minute in contrast to about thirty minutes per sample where former hand calculations were used.

My office in the Bureau of Public Roads worked closely with the Michigan State Highway Department in the testing of a new device that should be of interest to you. This is a nuclear instrument, developed by the Michigan State Highway Department, for determining moisture and density of highway embankments.

Radium-D beryllium is used in this instrument as the source of nuclear energy. This source radiates both gamma rays and high speed neutrons. The gamma rays are reflected to a degree dependent upon the density of the material through which they travel. The high velocity neutrons are slowed by any hydrogen atoms they contact. Hence, more neutrons will be slowed down if a given material has a higher moisture content. Both the reflected gamma rays and the slowed neutrons are picked up by Geiger-Muller counter tubes separately and are read on the gage. The gage readings are interpolated into density and moisture content of the material from curves.

Determination of density and moisture content of a compacted embankment with this instrument takes one man two minutes. Contrast that with the requirements in time, and equipment for conventional methods. And these are not the only benefits resulting from this new development. The density and moisture content of undisturbed material are determined. This reduces the chance of human error in taking and testing samples. Quality goes up too since we can now check density and moisture content more frequently. Not only is there less interference to the contractor but we actually can expedite his operations because he knows sooner when the specified results have been achieved.

Hundreds of tests have been made to compare densities and moisture contents determined with this instrument and by conventional methods. A high degree of correlation resulted. As a consequence, the Michigan State Highway Department has now taken the gage out of the experimental stage and turned it over to the operating divisions. The gage should be available commercially soon.

Another instrument has been taken out of the laboratory and put to work on highway construction projects. This instrument was developed by the Bureau of Public Roads and is known as the soils resistivity gage. This instrument is calibrated in the field either by actual soils sampling or by observation in cuts of the character of the soil types. Once this calibration is accomplished, the nature of the subsurface soils types in the area can be readily ascertained. Not long ago this gage was used by the contractor who had been awarded the construction of the south half of the Washington-Baltimore Parkway. With the aid of the gage the contractor was able to locate suitable borrow material within and adjacent to the right-of-way. He reported that hundreds of thousands of dollars were saved because the cost of buying and hauling material was reduced.

New developments have taken place in seismic equipment. One of the latest of these utilizes a metal plate which is struck with a sledge hammer. The time required for the vibration to travel through the subsurface soils can be interpolated into the various soils types.

Both of these instruments have many applications in highway construction. For example, contractors formerly went to considerable expense to determine whether a given material requires blasting and shovel loading or whether it could be broken up with the new powerful rippers that are now available. Of course, if the ripper could be used and the material loaded by fast economical scraper methods, the cost of excavation could be substantially reduced. Either of these instruments properly calibrated could be used to solve this question.

Earlier we pointed to the growing use of aerial surveys and photogrammetry in highway engineering. Soil classification by photo-interpretation also has grown. While this is not a brand new technique the procedures and equipment have been considerably refined in recent years. The classification of soils by this method does not by any means replace the normal borings and sampling. It does, however, indicate the most likely places for borings or sampling to obtain desirable materials. In this way the number of borings is reduced to a minimum. Not long ago, the Bureau had to close down a section of the Natchez Trace Parkway because no suitable material could be located within a reasonable haul distance. Shortly thereafter an aerial survey of the proposed route was made. Through photo-interpretation an adequate quantity of good material was located right on the section that was to be constructed. In fact, the center line of the Parkway passed through the bed of material. More States make a practice of furnishing this information to contractors and they report that bid prices are favorably influenced.

The location of suitable rock, gravel and other materials for highway construction is another area where geologists can make important contributions to the highway program. The importance of this contribution can be understood from tables prepared by the Bureau of Public Roads periodically which show the highway construction usage factors for aggregates. These aggregates include sand, gravel, clay gravel, crushed gravel, slag, crushed stone, etc. The tables give the weighted average over periods of three years on Federal-aid construction projects. The latest one shows that for each million dollars of contract construction cost, contractors purchased 49,000 tons of aggregates. For each million dollars of contract construction cost they also produced 65,000 tons of aggregates.

This ia a big job for the geologist. For not only must we have large quantities of rock, gravel and other aggregates but we must also have quality. The degree of service which these roads and bridges we are building will provide in the future will depend in a large measure upon the aggregates that are used in their construction.

The geologist and the highway bridge engineer should continue to team up in foundation explorations for the many structures that will be required in the highway program. Some 375,000 structures are estimated will be constructed or improved in the Nation's highway programs.

The location of sound bedrock or other suitable foundation material is an important factor in site selection, in economy of construction and service life. The location of water tables, surface and subsurface drainage, and the characteritistics of the bedrock or other foundation material also must be determined. Frequently, geological data as a result of borings and other methods of exploration serve as a basis for solving landslide problems and for designing slopes.

When rock excavation is necessary the seismic method of determining subsurface characteristics has application. Most States now regulate the amount of the explosive a contractor can use in a blasting charge by the American Standard Table of Distances from homes, buildings or other vulnerable installations. Frequently, the allowable charge is not sufficient to give good breakage to the rock. At least one State now provides that where seismic data has been obtained, such data will govern the size of the charge permitted.

These, then, are some of the new developments that are being integrated into the highway program. Most of them are related to the rocks and earth that will play an important part in carrying out the program. I have tried to show to you that highway officials are looking constantly for ways and means of improving the quality and economy of highway construction.

They are looking to you - the geologists - for your ideas on how improvements can be made. There are new developments taking place in the field of applied engineering geology. Airborne radioactivity measurements are being used to define the boundaries of rock and earth foundation. The magnetometer has gone airborne too and has been linked with an electronic recorder to obtain geological data on bedrock foundations. Radioisotopes have been used to trace the flow of subsurface water, locate water tables and explore the internal structure of mineral aggregates.

Are these and other developments in geology now operational or are they still in the experimental stage? Will they help us further improve the quality or economy of highway construction? Highway officials would welcome your ideas, your advice and your cooperation. And I am sure that their cooperation and support of your activities would be forthcoming in return.