REPORT:

PHASE II GEOTECHNICAL FEASIBILITY STUDIES

ENGINEERING CONSULTING SERVICES

PROPOSED DEVELOPMENT OF CRANEY ISLAND DISPOSAL AREA

PORT OF HAMPTON ROADS, VIRGINIA

FOR THE VIRGINIA PORT AUTHORITY

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July 24, 1979

Virginia Port Authority
1600 Maritime Tower
Norfolk, VA 23510

Attention: Mr. J. Robert Bray
Executive Director
Commonwealth of Virginia
Virginia Port Authority

Gentlemen:

Attached are thirty-five copies of our "Report - Phase II Geotechnical Feasibility Studies - Engineering Consulting Services, Proposed Development of Craney Island Disposal Area, Port of Hampton Roads, Virginia, for the Virginia Port Authority." Our earlier Phase I - Background Studies (submitted on December 18, 1978) uncovered a substantial amount of existing background information pertaining to the Craney Island area. However, we concluded that the proposed development of the Craney Island Dredge Disposal Area required more complete geotechnical information.

The objectives of this Feasibility Study were (1) to develop more complete information regarding the subsoil conditions underlying the proposed development area, (2) to investigate various site stabilization alternatives which could be used, and (3) to evaluate foundation types which appear feasible to support the various types of structures considered for the proposed commercial, industrial, and port development in the area.

Additional field investigations during this Feasibility Study confirmed the presence of soft dredged materials and soft natural clays in the area, underlain by dense sands below the investigated area at depths ranging from 120 to 140 feet below dike grade. At some locations these sands were in turn underlain by stiff clays. Both the dense sands and stiff clays are suitable for supporting piles for the heavier structures to be built in the area.

Lighter structures which can tolerate differential settlement can be supported on shallow surface foundations, if the soft dredged materials and the soft natural clays are first stabilized by means of preloading and drainage. A careful field instrumentation program is recommended to monitor the site stabilization process.

It is important to keep in mind that the information and recommendations presented in this Feasibility Study must be considered preliminary. Additional detailed site studies should be planned at the location of each major structure to investigate the subsoil conditions and develop foundation design information for the specific project.

We hope you will find this Feasibility Study useful. If there are any questions, please feel free to contact us.

Very truly yours,

DAMES & MOORE

G. Andrew Reti

G. Andrew Reti
Partner

GAR:emv

cc Virginia Department of Highways
and Transportation
1401 East Broad Street
Richmond, VA 23219
Attention: Mr. Waverly L. Brittle, Jr.
Director of Engineering

U.S. Army Engineer District, Norfolk
Corps of Engineers
803 Front Street
Norfolk, VA 23510
Attention: Mr. Frank T. Wootton, Jr.
Chief, Water Resources Planning Branch

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1.0 INTRODUCTION

1.1 BACKGROUND

The Craney Island Dredge Disposal Area is located in Hampton Roads, Virginia north of Craney Island (within the city limits of Portsmouth, Virginia) and west of the Norfolk Harbor Channel. The site covers an area of approximately 4 square miles and is bounded by sand dikes roughly 2 miles long on each of three sides and by the mainland on the south. The general layout and location of the area with respect to Norfolk and Portsmouth is shown in Figure 1, Site Plan.

The disposal area was constructed in the early 1950's by the U.S. Army Corps of Engineers to dispose of dredged materials removed during the maintenance dredging of shipping channels and those resulting from construction activities in the Hampton Roads area. Initial estimates of the ultimate storage capacity of the disposal areas was on the order of 96 million cubic yards, and a useful life of about 22 years was projected. The construction of the disposal area was completed in 1956 and the first dredged materials were placed in 1955 prior to levee closure.

As of October 1972, the estimated volume of dredged materials placed in the disposal area is reported to be 100 million cubic yards. Currently, the Corps of Engineers estimates that by 1979 or 1980 the ultimate capacity of the disposal area will be reached with about 125 million cubic yards of materials stored at the site. The increased storage capacity is believed to have resulted from the additional space made available by the settlement of the area under the weight of the dredged materials and due to the gradual raising of the dikes from elevation +8 (MLW*) to their present elevation (approximately +15 to +28 feet).

Currently, several alternate dredged material disposal sites are being studied to replace Craney Island. When a suitable site has been located, the State of Virginia anticipates that Craney Island will become available for development. In expectation of that event, the Virginia Port Authority

*All elevations mentioned in this report refer to Mean Low Water (MLW).

is undertaking a feasibility study to investigate alternate ways of stabilizing the area and possible methods for supporting structures related to the potential commercial, industrial, and port development at the site.

Our geotechnical feasibility study was undertaken as a part of this overall development program, to obtain a more thorough understanding of the subsoil conditions in the area proposed for development, and to study and evaluate various site stabilization and foundation support alternatives for the anticipated construction.

1.2 PREVIOUS STUDIES

Late in 1978, the Port Authority retained Dames & Moore to perform a preliminary background investigation to review all existing geotechnical information available on the subsoil conditions in the vicinity of the Craney Island Area. A report, summarizing these findings, was submitted to the Port Authority on December 18, 1978.

During our background investigations, we uncovered a substantial amount of information pertaining to the Craney Island Area in the files of the Corps of Engineers (Norfolk District) and at the Virginia Department of Highways and Transportation in Richmond. The Corps conducted extensive subsoil investigations, laboratory tests, and engineering analyses prior to the construction of the Craney Island Dredge Disposal Area. More recently the Corps also conducted additional investigations in connection with a proposal to raise the dikes to elevation +30 in order to increase the storage capacity of the containment area. The Virginia Department of Highways and Transportation has conducted extensive subsoil investigations to the west of Craney Island, for a new bridge crossing for Highway I-664 at Hampton Roads.

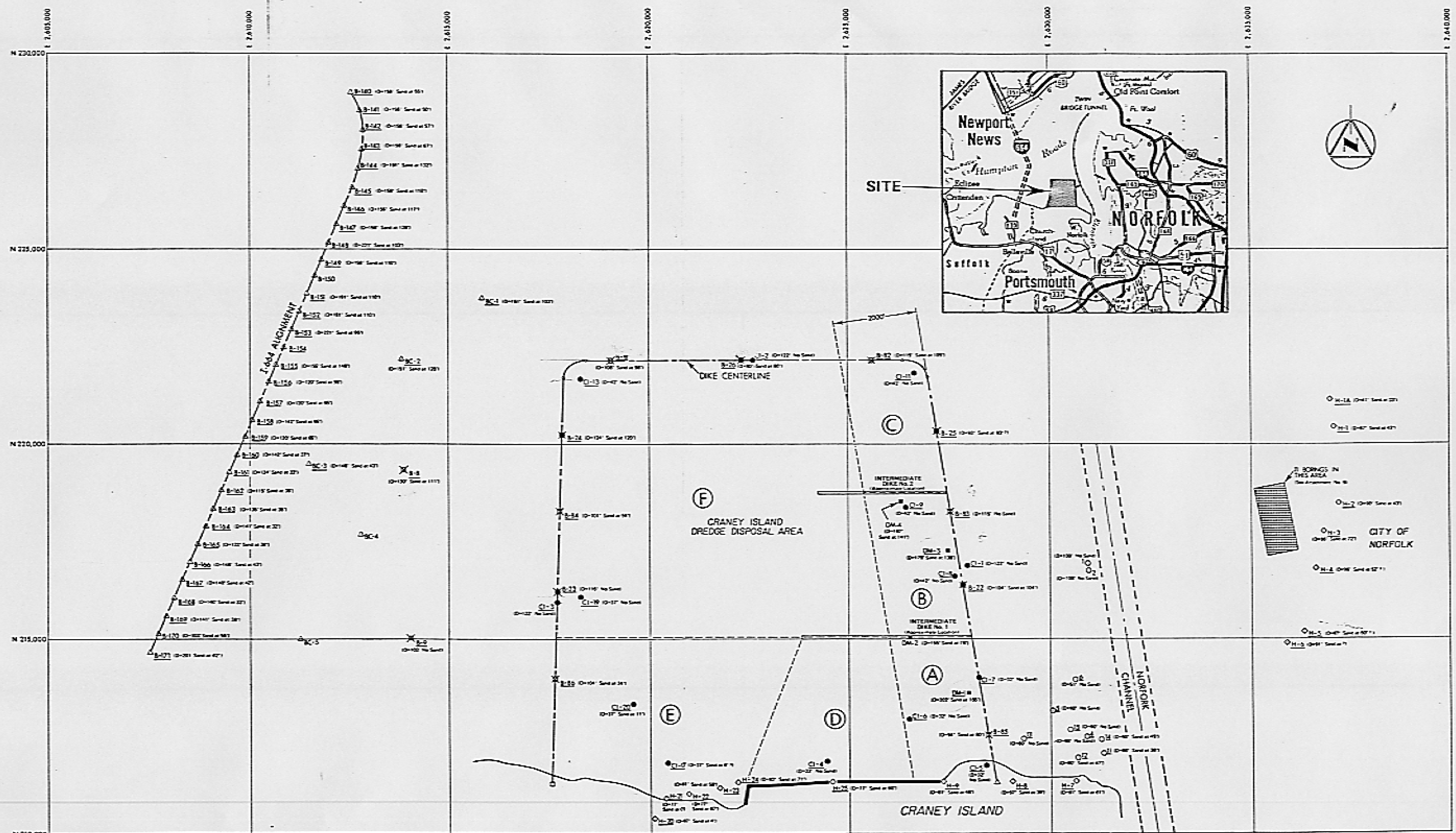
Information collected during the Phase I Background Study proved to be very useful in defining the general subsoil conditions in the area and in identifying the potential foundation problems to be anticipated in the development of commercial, industrial, and harbor facilities on Craney Island. However, the Background Study concluded that more complete geotechnical information would be required on the depth and extent of the soft marine sediments, the depth and supporting capacity of the underlying compact sandy soils, and the general extent and physical characteristics of the dredged materials placed in the area.

1.3 PROPOSED DEVELOPMENT

Based on our recent discussions with the Virginia Port Authority, we understand that initially future development will be concentrated along an approximately 2,000-foot wide on-shore strip along the eastern edge of the Dredge Disposal Area. For discussion purposes, the disposal area was divided into six zones, labeled A through F as shown in Figure 1, Site Plan. Based on our discussions with the Virginia Port Authority, areas labeled A, B, and C appear to be most attractive for development at this time. These areas are followed by areas D, E, and F in decreasing order of desirability. It is expected that areas A and B would be developed first, probably followed by the development of areas C and D.

We understand that the proposed commercial and industrial structures would include warehouses, low buildings related to light manufacturing activities, small office buildings, sheds, and other structures of this type. Structures associated with port development may include piers, wharves, various types of retaining structures, and other typical port facilities. Certain areas would be paved for vehicular access, to provide storage areas for automobile shipments, for the outdoor storage of merchandise, and for the movement of rubber-tire mounted cranes used for moving cargo. In addition, the U.S. Navy is planning to construct a pipeline along the east side of the disposal area, between its fuel storage facilities in Portsmouth and its ship loading facilities on the east side of Norfolk Channel. Deepening of the Norfolk shipping channel to a depth of 55 feet by dredging, is also being considered.

During our discussions with the Virginia Port Authority, we reviewed some of the current design practices related to port facilities in the general area. While settlement and stability considerations are of primary concern in the designs, we understand that a substantial amount of differential settlement can be tolerated by the types of structures being considered in the area proposed for development. A certain amount of maintenance and repairs to



NOTE: All Boring Locations are Approximate and are Based on Information Obtained From Previous Investigations By Others

LEGEND

1948 - Corps of Engineers Boring
1948 - Corps of Engineers Boring
1956 - Raymond Concrete Pile Co. Boring
1970 - Dames & Moore Boring
1971 - Corps of Engineers Boring
1972 - Sverdrup & Parcel Boring
1976 - URS / Madigan - Praeger, Inc. Boring
1976 - Virginia Dept. of Highways & Trans. Boring
1979 - Dames & Moore Boring
H-8 - Boring Number
D=81' - Total Depth of Boring
Sand at 48' - Approximate Depth at Which Further Sand Layers Considered Probably Suitable for Foundation Support Were Encountered

VIRGINIA PORT AUTHORITY

PROPOSED DEVELOPMENT OF CRANEY ISLAND DISPOSAL AREA
PORT OF HAMPTON ROADS, VIRGINIA

SITE PLAN

JULY 1979 COMPILED BY DAMES & MOORE

FIGURE 1

correct the effects of differential settlements in the paved areas, such as releveling and resurfacing, would be acceptable as part of a regular maintenance program. Some structural or aesthetic repairs of structures could also be tolerated as long as there was no question related to their safety and performance.

2.0 PURPOSE AND SCOPE OF STUDY

2.1 OBJECTIVES

The purposes of our feasibility study were to conduct more detailed investigations to verify the subsoil conditions found during previous studies and to extend the subsoil investigations to depths greater than those reached during the previous investigations. More specifically, our objectives were to:

- Investigate the extent, depth, and physical properties of the near-surface dredged materials which may be used to support lighter structural loads.
- Investigate the extent, depth, and physical properties of the soft natural marine sediments underlying the dredged materials.
- Investigate the depth and supporting capacity of the firmer sandy and clayey soils underlying the soft marine sediments to be used to support heavier structural loads.

Our field investigations were to concentrate on areas selected for initial development along the southeast portion of the Dredge Disposal Area, designated as A and B on Figure 1, Site Plan.

Our feasibility studies also included a preliminary investigation of the following engineering aspects of site development:

- Various stabilization alternatives which could be utilized to improve the supporting capacity of the near-surface soils.
- Various foundation types which appear feasible for the support of various structures contemplated for construction in the proposed areas.
- Preliminary design alternatives for the proposed marine structures.
- Special problems that may be related to the construction of a proposed pipeline along the east side of the area and to the proposed deepening of Norfolk channel.

2.2 LIMITATIONS

In keeping within the limitations of an initial feasibility study, this report is not intended to provide detailed design or specific construction recommendations for the structures to be built in the Dredge Disposal Area.

It is anticipated that, as more detailed plans for utilizing the area are developed, detailed site-specific foundation engineering studies will be performed for each major structure planned in the area. In keeping with the general nature of our investigations, our current studies have not addressed questions related to construction costs or to the future uses of the disposal area based on various design elevations for the dredged fill materials.

During the initial planning of our studies, it was decided to concentrate our investigations in areas labeled A and B (See Figure 1) of the Dredge Disposal Area in order to develop the maximum amount of geotechnical information related to the area proposed for initial development. Therefore, the information presented in this report is mostly related to the conditions prevalent in the southeastern corner of the Dredge Disposal Area.

During the early planning stages of this investigation, it was proposed to explore the subsoil conditions by drilling borings at locations accessible to truck-mounted drilling equipment and by performing Dutch-Cone penetration tests in the areas underlain by the very soft dredged material deposits. Subsequently, it was necessary to delete the Dutch-Cone tests because of the extremely high costs associated with transporting the equipment into the impassable soft areas. Therefore, our field investigations were limited to four soil borings along the east side of the Dredge Disposal Area at locations accessible to truck-mounted drilling equipment.

3.0 FIELD INVESTIGATIONS AND LABORATORY TESTS

3.1 SURFACE CONDITIONS

The Dredge Disposal Site is surrounded by sand dikes on three sides with crest elevations ranging between approximately +15 and +28 feet. Two intermediate dikes, extending westward from the eastern perimeter dike for about one-third the distance through the disposal area, serve as channelization guides for the deposited dredged materials. A dirt road follows the perimeter of the disposal area and is paralleled by a higher inner road on the eastern dike with road spurs extending onto the two intermediate dikes. The eastern and northern dikes are roughly 500 feet wide for most of their extent.

Based on construction drawings obtained from the Corps of Engineers, we understand that the underwater slopes of the dikes were constructed on a 30 horizontal to 1 vertical inclination. All dike slopes appear relatively stable with infrequent spots of minor localized erosion. Most portions of the main dike are well drained and compact enough to be accessible to vehicles, although off the roads the dike surface is often rough.

The dredged materials placed inside the dike are practically level. Dredged materials are usually placed along the eastern portion of the disposal area and flow westward toward three sluice gates where the water is discharged into the ocean. No ordered surface drainage pattern is evident except for the flow of the contained dredged materials toward the west. Water is observed to stand in low spots on the dike (for example between the two eastern roads) for several days after a rain. A discontinuous cover of marshy grasses occurs along some portions of the dike and in some of the filled-in areas immediately adjacent to the dike.

Certain portions within the Dredge Disposal Area, notably those north of the northern intermediate dike, have received relatively granular dredged-disposal materials. In these areas there were some earth moving operations going on at the time of our field investigations. Some of these areas are accessible to trucks, and others to four-wheel drive vehicles. In other areas, located to the north and south of both intermediate dikes and at both interior northern corners of the main dike, granular materials make it possible to access limited areas between the dikes.

With the exception of the above-described relatively firm areas blanketed by granular materials, the remainder of the Dredge Disposal Area contains primarily very soft silty and clayey dredged materials. In places a thin crust has formed due to dessication, which makes the surface appear dry and firm. However, the crust is easily broken under a man's weight and the underlying material is very wet, soft, and spongy. Other portions of the soft areas are covered by a thin layer of water and are totally untrafficable. None of the areas underlain by the very soft dredged materials can support the weight of a vehicle. Indeed, most of the areas cannot be traversed on foot.

3.2 SUBSURFACE CONDITIONS

As described in Section 1.2 of this report, there have been numerous subsurface investigations conducted in the Craney Island area over the past thirty years. The results of these investigations have been summarized in our Phase I - Background Studies Report, submitted to the Port Authority on December 18, 1978.

During our current investigation, four additional deep borings were drilled at the locations shown in Figure 1, Site Plan. Two of the borings were located on the main eastern dike and one on each of the intermediate dikes. Our borings generally confirmed the subsoil conditions encountered by the earlier, shallower borings in the area and yielded additional information on the subsoils located below the depths reached by the earlier borings.

The dikes consist of a mixture of sandy and clayey soils interspersed with varying amounts of shells and occasional pieces of wood and steel scrap. The sandy soils in the dikes are usually of medium density but the clays are relatively soft. Our boring on the first intermediate dike encountered several large timbers. We were told by the Port Authority that the intermediate dikes were constructed by placing timbers over the soft dredged materials and then pushing the dike fill over the timbers. It is very difficult to identify the interface between the dike and the underlying naturally deposited sediments, since some mixing must have occurred during the construction of the dikes and during subsequent dike settlement. The transition between the dike materials and the natural soils appears to have occurred between about 25 and 35 feet below present dike surface.

The dikes are underlain by relatively soft clays and silty sand. These Pleistocene and Recent clays are highly plastic and have a very high moisture content. They were often soft enough to require that the drill rods be held during drilling to prevent their dropping through the materials of their own weight.

Geophysical studies by others have uncovered an ancient river valley filled with soft sediments to the northwest of the Dredge Disposal Area. Extrapolations have shown that this river valley probably passes diagonally from the northwest to the southeast through the Dredge Disposal Area. This old river erosion channel, probably created during the Post-Miocene period of low ocean levels, is now filled with Recent deposits of soft and highly plastic clays. The thickness of these soft river valley deposits ranges from a few feet, near the south end of the disposal area, to over a hundred feet toward the northern end of the site. The thicknesses and depths of these soft deposits in our borings correlate reasonably well with those encountered in borings by others.

Underlying the soft clays are varying thicknesses of clean and silty sands with occasional pockets of clay or clayey sands. These materials range from loose to dense in consistency; however, a very dense layer of clean sand appears in each boring at the depth of about 120 to 140 feet. In our borings the thickness of the dense sand layer varied from about 20 to 70 feet.

The dense sand layers are underlain by stiff silty clay and clayey sand, possibly representing the older Miocene deposits. The details of the soil stratifications encountered in our borings and the classifications of the soils are shown on the boring logs presented in Appendix A.

Groundwater levels were measured during the course of our field investigations and were found to fluctuate within five feet below the surface of the dike from which the borings were drilled. The ground elevations and coordinate locations of each of our borings are shown on each boring log in Appendix A.

As mentioned previously, it was not possible to access the soft dredged materials with a drill rig in order to obtain soil samples for laboratory testing. However, based on a visual inspection of the surface, the dredged materials appear to consist of a highly variable mixture of very soft sandy, silty, and clayey soils with a very high water content and very low shear strength. As described in Section 3.1 of this report, the placement of the sandier dredged materials was controlled during deposition and was concentrated in certain portions of the disposal site. These sandier materials consist of silty or clean sands which are currently being sold to contractors for fill materials.

3.3 LABORATORY TESTS

Laboratory tests were performed on representative soil samples extracted from our borings. The purpose of these tests was to evaluate the physical properties of the principal subsoil types encountered at the site and form the basis for subsequent engineering evaluations and analyses. A detailed description of the testing methods that were used and of the test results is presented in Appendix B of this report.

Laboratory soil identification tests were conducted to supplement the classification of the soils in the field and to help correlate our soil samples with those obtained during previous investigations. These soil classification tests included particle-size analyses, specific gravity determinations, plasticity index tests, and moisture-density determinations. These laboratory tests generally confirmed the classification of the soils in the field and correlated well with the classification of soils during previous investigations. The results of the laboratory soil indentification tests are presented next to each sample symbol on the boring logs in Appendix A.

The shear strengths of the soil samples obtained in the borings were investigated by means of unconfined compression tests, unconsolidated undrained triaxial compression tests, and isotropically consolidated undrained triaxial tests. These tests are designed to reproduce in the laboratory the shear stress conditions to which the soils in the field will be subjected under the anticipated structural loads. Based on these tests, and on information from the results of standard presentation tests in the borings, the following ranges of shear-strength values are typical of the different types of soils encountered in the field:

Soil Type	Shear Strength
Soft Natural Clays	cohesion: 200 - 500 psf friction: 0 degrees
Loose to Medium Sands	cohesion: 0 psf friction: 28 - 33 degrees
Dense Sands	cohesion: 0 psf friction: 30 - 35 degrees
Stiffer Natural Silty Clay	cohesion: 700 - 1000 psf friction: 0 degrees

Laboratory consolidation tests were performed to evaluate the compressibility of the various soil layers encountered in our borings. These tests are used to estimate the amount of settlement and the time rate of settlement when the soils are subjected to the weight of fills or of various structural loads. The results of these tests are presented in Appendix B.

In reviewing these laboratory test results it must be remembered that they are based on a relatively small number of soil samples, obtained from a small number of borings drilled at widely spaced locations. The subsoils at the site are highly variable and therefore the physical soil property parameters presented in this report are only general values, and may vary substantially even within a particular soil layer. Engineering predictions based on these soil properties must be considered indicative only of the general order of magnitude of the various calculated values. These observations emphasize the necessity and importance of conducting site-specific investigations whenever the location and design of a particular structure is finalized.

3.4 CONCLUSIONS

Based on our field and laboratory investigations and on the information obtained during previous investigations by others in the Craney Island Area, the following conclusions regarding the subsoil conditions in the areas where our borings were drilled can be reached:

(1) The subsoils encountered in our borings were generally similar to those encountered during previous investigations in the general vicinity. However, the previously drilled borings in the southeastern portion of the Dredge Disposal Area ranged in depth from 32 to 122 feet. Since our current borings ranged in depth from 154 to 201 feet, we were able to obtain additional information regarding the deeper subsoils in the area.

(2) Because of the difficulty in accessing the soft dredged fill, no undisturbed samples of these materials were obtained during our investigations. Based on the visual inspection of the surface materials, the upper dredged materials consist of very soft and very wet silts and clays. Certain portions of the Dredge Disposal Area are blanketed by medium to fine sands at locations previously described in this report. These soft silts and clays are unable to support any significant structural loads and are expected to compress and flow laterally if subjected to the weight of fill or to structural loads. The sandy dredged materials would be able to support some surface loads. However, since they are probably also underlain by soft silts and clays, it is unlikely that they could support substantial structural loads without undesirable deformation or displacement.

(3) The natural silts and clays underlying the dredged materials are also very wet and very soft. These are highly plastic soils and their natural moisture content is at or above their liquid limit. Consolidation tests show that the soft clays are still undergoing compression under the weight of the dikes.

(4) Firmer sandy soils were encountered in all four of our borings at depths ranging from 76 feet (in Boring DM-2) to 140 feet (in Boring DM-4).

These sandy soils are judged to be suitable to support structural loads if their penetration resistance exceeds 25 to 30 blows per foot on the standard penetration sampler used during our field explorations. The thickness of these sandy layers ranges from 20 feet (in Boring DM-3) to 71 feet (in Boring DM-2). These sandy soils generally contain less than 15 percent of fines (passing the number 200 sieve), and an angle of internal friction on the order of 30 - 35 degrees.

(5) Stiff clayey soils were encountered in three of the four borings we drilled at the site, below the dense sandy soils described above. These stiff clayey soils were somewhat less plastic and their moisture content was substantially less than those of the soft clay soils located near the surface. Their shear strength is higher, ranging on the order of 700 to 1000 pounds per square foot, and their compressibility is lower than those of the soft clayey soils.

(6) In 1953 the Corps of Engineers estimated that the settlement of dikes constructed to elevation +8 would be on the order of 7½ feet and that approximately one half of the predicted settlement (3 to 4 feet) would take place during the first 15 years. However, during recent years additional construction has been taking place along the dikes and dike elevations currently range between about +15 and +28 feet. These additional loads have influenced the predicted settlements substantially. In 1972 the Corps of Engineers revised their settlement estimates in view of the plan to raise the dikes to elevation +30, and additional settlements on the order of 6 to 7 feet were predicted. About half of this additional settlement (on the order of 3 to 4 feet) was expected to occur in about 40 years. The variable thicknesses of soft clay along the dike alignment and the continuous placement of additional dredged materials also makes the prediction of dike settlements very difficult.

Based on our review of subsoil conditions and our estimate of the additional loads imposed by the increased dike heights, we estimate that the dikes will ultimately settle about 10 feet from their present elevations. However, these settlements will take place very slowly - about 5 feet of settlement can be expected during the next 20 years.

Due to the uncertainty of these settlement predictions we recommend that a regular settlement monitoring program be instituted in the proposed construction area in order to measure the rate and the magnitude of the settlements. More details regarding the proposed settlement monitoring program are given in Appendix C of this report.

(7) Early stability analyses performed by the Corps of Engineers indicate that the sand dike, constructed to an elevation of +8 feet, would have adequate stability against rotational sliding failure. However, the analyses predicted the possible lateral displacement of the soft clays. This type of failure was subsequently confirmed by the large mud waves which formed during the construction of the dike. Later analyses by the Corps indicate that raising the dike to elevation +30 would not endanger its stability if the raised portion was set back approximately 700 feet from the centerline of the earlier dikes.

4.0 SITE STABILIZATION ALTERNATIVES

4.1 STABILIZATION METHODS

The presence of the very soft dredged materials and of the underlying soft, silty natural clays in the proposed construction area make it necessary to consider stabilizing the site in order to facilitate its development. The following is a brief review of the principal stabilization methods used for improving the supporting capacity of the soils or accelerating their settlement and reducing their compressibility.

(1) The injection of chemicals or of a cement slurry improves the physical properties of the soil by filling the voids and by binding together the soil particles. Some injection methods rely on the permeability of the soil for carrying the chemical or cement slurry into the zone to be stabilized. This requires that the soil be pervious enough for the injected material to flow through it. The silty and clayey soils at the Dredge Disposal site are not pervious enough for this method.

Another procedure is to mix the chemicals or cement with the soil in-place by utilizing large mechanical mixing devices. The Japanese Deep Chemical Mixing (DCM) method is based on such in-place mixing of chemicals or cement with the soil. We believe that the probable high cost of this method does not favor its use at this site. However, actual cost data regarding this method were not available to us at the time of the preparation of this report.

(2) Another method is to compact the soil in place by means of vibroflotation or dynamic consolidation. In vibroflotation a large pipe is pushed into the soil while water is injected into the soil mass through openings near the bottom of the pipe. The combined action of the driving and of the water jets causes the soil particles to rearrange into a denser structure, thus resulting in compaction of the soil. Additional sand is placed into the hole left by the pipe to make up the void left by the compacting action. For vibroflotation to be effective, the soils must be

sandy enough to accept the injecting action of the water. The silty and clayey soils are too impervious for this method to be practical at this site.

Dynamic consolidation is performed by dropping a heavy weight from a great height to compact the soil by means of the dynamic energy of the dropping weight. Steel plates, weighing 20 to 50 tons, are dropped from heights ranging from 50 to 100 feet. This method has been tried at several locations with some success. However, relatively granular soils are required for the method to be workable. The silty and clayey soils at the site would, in our opinion, not compact well by the dynamic consolidation method.

(3) Stabilization by surcharging and draining consists of placing a load on the soil, equal to or in excess of the anticipated final loads on the area, and facilitating the escape of the water from the compressed soil. The combined action of the surcharge and of the escaping water causes the soil to consolidate, thereby increasing its shear strength and reducing its compressibility. Surcharging usually consists of placing fill over the area to a height which produces a load equal to or greater than the anticipated permanent load on the site. Drainage is accelerated by installing sand drains, gravel drains, or other means to assist the escape of water from the compressed clay layers under the surcharge. We believe that given sufficient time the proposed construction area can be best stabilized by means of surcharging and drainage. Details on this method are presented in the next section of this report.

4.2 RECOMMENDED SITE STABILIZATION

We recommend that the area of the proposed development be stabilized by surcharging and draining. Such a program would achieve the following objectives:

- It would provide a working platform over the existing soft ground for supporting construction equipment and for supporting shallow foundations for light structures that can tolerate differential settlements. In addition, the working platform would also serve as a base for paved areas.

- It would help accelerate the consolidation of the underlying soft soils thereby improving their shear strength and reducing their compressibility.

The following construction sequence would be used:

(1) Prepare a working platform by carefully placing a good quality granular fill up to 5 feet in thickness to obtain an even working surface.

(2) Install sand drains or wick drains.

(3) Preload the area by placing additional granular soil fill over the working platform.

(4) Monitor the settlement and drainage of the soft subsoils by means of adequate field instrumentation.

(5) Remove the preloading after the soils have consolidated to a sufficient degree.

(6) Install the foundations for the structures to be constructed in the area.

The specific construction methods, design details, and the effectiveness of surcharging and draining are greatly influenced by the specific soil conditions at the site. Because of the uncertainties regarding the clay thickness and compressibility at a particular building site, it would be desirable to field test the above described site stabilization procedure at a specific location in Area A. The effectiveness of the procedure would be monitored by means of settlement plates and pore water pressure measurements. Based on these measurements, the construction procedures could be modified by changing the spacing of the drains and/or by increasing or decreasing the amount of the surcharge. The test area could also provide a means for trying out various construction methods for placing the working platform and for installing the drains.

Since Area B is underlain by a deeper layer of soft clay, we believe that its development should be postponed until after the recommended stabilization procedure has been tried out in Area A. We anticipate that the greater thickness of soft marine clays in Area B will pose greater construction difficulties for stabilizing that area.

The first step will be to construct a working platform over the area to be developed. A 4- to 6-foot thick layer of sandy soils should be placed, to level the area and to provide a base for subsequent construction operations. We believe that some of the on-site sandy soils could be utilized to construct this platform. To ensure adequate drainage, the sand should contain no more than 5 percent fines (finer than a #200 sieve).

It is very likely that some mud waves will form as the platform is placed over the soft, clayey dredged material. Various construction techniques should be tried to avoid producing such mud waves. The materials could be end dumped from trucks at the dikes and then spread in thin layers over the soft dredged materials. If mud waves cannot be avoided in this manner, it may be necessary to place filter cloth, logs, old timber piles, or other debris over the soft dredged material prior to placing the sand platform. However, these may make the subsequent installation of drains and piles more difficult. The sand fill may also sink into the soft dredged materials as it is placed. Again various construction techniques need to be tried in order to develop a suitable method for placing the platform.

After a suitable working platform has been constructed, vertical drains should be installed through the platform and into the dredged materials and the natural soft clay layer in order to accelerate the drainage of water. Stone columns, sand drains, and wick drains (Kjellman-Franki paper drains) are three possible alternatives which may be suitable. While stone-columns can serve both to reinforce the soft clay and to accelerate its consolidation, the fine-grained clay soils will probably plug the pores in the stone columns and thus reduce their effectiveness as drains. We believe that sand drains would be more suitable than stone columns.

Wick drains are essentially long plastic strips, installed vertically and containing holes to drain the water. They are relatively easily installed and may be economically attractive. Our preliminary discussions with the Franki Pile Foundation Co. (licensees for these drains), indicate that this method may be an effective substitute for sand drains and may result in economies of time and materials. Further details regarding the best drainage methods for stabilizing the site can be developed after more specific plans for the area have been prepared.

After the installation of the drains, more fill would be placed over the site to accelerate drainage and compression. This preload would subject the clay to at least the loading level anticipated to be applied by the foundations. Thus, compression in the clay would take place prior to the construction of the structures. Some preload would be removed before erecting the structures. Assuming light industrial and commercial structures for the proposed development area, at least 4 or 5 feet of fill would have to be placed over the working platform to induce the required precompression. However, a higher level of surcharge would accelerate the compression and drainage of the soft soils. The most effective surcharge height should be determined based on the location of the site to be surcharged, and the thickness of the soft soils at the site. The preload would cause the water in the clay to flow up through the drains and into the working platform. Adequate means for draining this water would be installed either by extending the drains through the platform and the surcharge or by placing horizontal drainage channels within the fill.

The progress of the surcharging and drainage program must be monitored by means of an adequate number of settlement plates and pore water pressure measuring devices. The rate of settlement of the fill and the rate of drainage of the pore water are important indicators of the success and progress of the stabilization procedures. Surface settlement plates would be installed no further than 200 to 400 feet apart, and six to ten pore water pressure measuring devices would be installed at key locations. A systematic monitoring program would be instituted to perform the measurements at regular time intervals. The data obtained would be analyzed to evaluate the success of the stabilization program and determine when it is nearing completion. More details regarding the proposed field instrumentation program are presented in Appendix C.

Lateral movement markers near the edge of the dredge area dikes should also be monitored for horizontal movement, in order to detect if the surcharge is producing any lateral flow in the underlying clay layers. Excessive surcharging may cause the dikes to be subjected to undesirable lateral loads, thereby endangering their stability. The surcharging program may need to be modified if these readings indicate a potentially dangerous condition.

5.0 FOUNDATION SUPPORT ALTERNATIVES

5.1 SURFACE FOUNDATIONS

We understand that most of the buildings to be constructed in the Dredge Disposal Area will consist of relatively light and flexible one-story commercial, industrial, and warehouse structures. Such buildings would have relatively low column or wall loads and can be designed to accept fairly large differential settlements without structural distress. Foundations for such structures could be supported near the surface of a 5-foot thick working platform consisting of compacted granular soils and placed in connection with the soil stabilization program described in Section 4.3 of this report.

The following preliminary design recommendations can be used in the planning of such foundations. More specific design values must be developed for each structure based on a more detailed foundation investigation conducted at each specific location and on the results of the soil stabilization program, as described in Section 2.2 of this report.

(1) Surface foundations can be used if column loads on individual footings are less than 15 kips or if wall loads on continuous footings are less than 3 kips per linear foot.

(2) Surface footings can be designed for a maximum soil bearing pressure of 1,500 pounds per square foot for dead loads and can be increased by 1/3 for transient live loads, such as those due to wind.

(3) The minimum depth of footings below adjacent ground must be at least 12 inches for frost protection. This depth can include the thickness of any pavement placed immediately adjacent to the footings.

(4) There must be a minimum clear thickness of 3 feet of well-compacted sandy fill below each footing. In no case should the width of the footing exceed the depth of sandy fill below it.

(5) Lateral loads applied to the footings can be resisted by a combination of bottom friction and passive pressure against the sides of the footings. Bottom friction can be estimated as that resulting from a friction angle of 20 degrees between the footing and the soil below it. Passive pressure against the sides of the footings can be estimated as that resulting from hydrostatic pressure of an equivalent fluid with a unit weight of 300 pounds per cubic foot. The lateral resistance of surface foundations due to bottom friction and passive pressure should not be used simultaneously to resist lateral loads. One or the other must be reduced by a factor of one half if both are used together.

(6) We understand that buildings currently being considered in the area will have no basements. However, if underground structures are to be considered in the future, design recommendations for their foundations will need to be developed specifically for each case.

The settlements of foundations designed in accordance with the above recommendations will be controlled by the thickness and compressibility of the soft subsoils underlying the specific structure and by the extent and success of the soil stabilization applied to the soft soils underlying the sandy fill work pad. These settlements will therefore depend on the intensity of the stabilizing surcharge, the effectiveness of the drainage measures, and the amount of time allowed for soil stabilization to take place.

Settlements of surface foundations can be estimated based on the results of the soil stabilization program, as monitored by means of the field measurements recommended in this report. We recommend that the settlement criteria for surface foundations be evaluated after the types of structures to be constructed in area have been further defined. The soil stabilization program should be designed to be consistent with these tolerable settlements.

In addition to settlements, due to their own structural loads, buildings will also settle as a result of the overall settlements occurring in the Dredge Disposal Area as a result of the loads imposed by the dikes and the dredged fill.

5.2 FLOOR SLABS

It is anticipated that some areas within or outside of buildings will require concrete floor slabs. Such slabs should not be designed to support heavy concentrated loads, and the average distributed pressure over the slab should not exceed 500 to 600 pounds per square foot. The slabs should be designed to undergo significant settlements without structural distress by including flexible joints along which displacement without cracking can take place. Slab grades should be designed for adequate drainage even if they settle due to the compression of the soft subsoils.

5.3 DEEP FOUNDATIONS

Heavier structures or structures which cannot tolerate differential settlements will have to be supported on long piles deriving their support in the deeper dense sandy and stiff clayey soils. In general, structures which will require pile foundations include buildings over one story in height, and those imposing more than 15 kip permanent column loads or more than 3 kips per lineal foot wall loads. Structurally sensitive buildings such as those including gantry rails, sensitive instruments and equipment, or relatively rigid structures could not tolerate the differential settlements which would occur if they were supported on surface footings and will therefore have to be supported on long piles.

The bearing strata suitable for supporting long piles include the dense sands and stiff clays generally located about 120 to 140 feet below present dike elevation (or at approximately -100 feet elevation). We have assumed that piles will derive adequate support in soils for which the standard penetration-test blow resistance exceeds 25 to 30 blows per foot.

Because of the relatively long pile lengths, the high structural loads on each pile, and the downdrag due to the compression of the soft upper clays, it is expected that heavy pile cross-sections would be required to support the anticipated loads. Generally, the following pile types should be considered for supporting heavy structural loads: steel H piles, steel pipe piles, step-taper cast-in-place concrete piles, solid prestressed concrete piles, or prestressed concrete cylinder piles. Because of the variable depth of the supporting dense sand and stiff clays, pile types which can be easily adjusted in length to varying field conditions by field splicing should be given preference. The final selection of the pile types should be based on the subsoil conditions at each specific building site and on the experience and cost records of pile contractors active in the area.

As the soft subsoils compress under their own weight or due to structural or fill loads, they are expected to apply a downward (negative) skin friction to the piles. This negative skin friction decreases the net structural load that the pile can carry. The magnitude of the negative skin friction applied to the pile depends on the lateral surface area of the pile and the pile length along which it is acting. Negative skin friction may range from about 20 tons for 18-inch piles embedded in 20 feet of soft clay to as much as 160 tons for a 36-inch pile embedded in 100 feet of soft clay.

Based on a pile penetration of at least 20 to 30 feet into the stiff clayey subsoils, preliminary ultimate net (not including downdrag loads) supporting pile capacities ranging from 25 tons for 18-inch piles to 100 tons for 36-inch piles can be considered. In the dense sandy soils, ultimate net pile capacities on the order of 350 tons for 18-inch piles can be used for initial estimates. If the pile is underlain by a layer of dense sand or stiff clay at least 20 feet thick, these pile supporting capacities can be increased to at least twice the above given values. A factor of safety ranging from 1.5 to 2.0 should be applied to the ultimate pile capacities for computing working loads. The supporting capacities of piles driven in close groups must be reduced to account for their group action.

The specific design of pile foundations for individual structures should be developed after the type of structure and its location have been determined. Then borings should be drilled at the specific location of the structure to determine the thickness of soft soil and the depth and supporting capacity of the dense sand and stiff clay layers at the specific site. At this stage in a feasibility study there are too many variables to permit the development of specific pile recommendations.

Certain problems must be kept in mind during the design of pile foundations for supporting heavier structures. The following is a brief discussion of these potential problems and possible means for overcoming them:

(1) Vertical piles will be subjected to downdrag loads (negative friction) as the soft clays consolidate. Although there have been experiments in coating piles with slippery materials to reduce downdrag loads, there is some question regarding the long term efficiency of such coatings. Since the clay layers are expected to be consolidating for many years, it is questionable if coatings can remain effective for such long periods of time. It would be more appropriate, in our opinion, to design the piles to resist downdrag loads.

(2) The lateral load resistance of long piles driven through soft soils is expected to be relatively low. Piles driven through the deep soft clay layers will have long cantilever lengths and will thus be subjected to substantial bending moments under lateral loads. Significant lateral loads will have to be resisted by means of batter piles designed to act in tension, or in compression.

(3) Batter piles will be subjected to transverse loads (bending loads) caused by the settlement of the soft clays. Therefore, batter piles should be installed at a steep angle to reduce the transverse loads. Batter piles must be designed to resist the bending moments produced by these loads.

(4) Structures supported on long end-bearing piles, deriving support in the stiff clays, are expected to settle 1-3 inches, depending on the type of pile and on the thickness of the supporting layers. Such piles supported in deep dense sand are expected to settle less than one inch. More detailed settlement estimates must wait until more specific information regarding the structure and its exact location is available.

(5) Structures supported on long piles are expected to settle much less than those supported on surface foundations. In addition, as the soft soils

consolidate, the surface of the ground will also undergo some settlement even in unloaded areas. Therefore, structures supported on long piles will appear to be rising out of the ground. Such differential settlements must be accomodated by means of special structural details such as joints, flexible connections, and hinged plates. The differential settlement between pile supported structures and other structures can be reduced if the piles are driven after site stabilization has been substantially completed. Therefore, it would be desirable to delay the driving of piles for the heavier structures until much of the settlement due to stabilization has been completed.

(6) The variability of the subsoils across the area and the variations in the depth to the dense sands and stiff clays makes it difficult to predict the supporting capacity of long piles. We recommend that field pile load tests be planned to develop more accurate and more reliable information on pile supporting capacities. These load tests should be conducted at the specific sites where the heavier structures will be located.

6.0 FOUNDATIONS FOR MARINE STRUCTURES

6.1 TYPES OF MARINE STRUCTURES

It is expected that the proposed site development will include various types of port structures to accommodate ships of various sizes. These structures are expected to include piers, wharves, and possibly mooring and breasting dolphins. It is also expected that some dredging will be required to develop channels deep enough for ships to access the marine facilities.

6.2 FOUNDATIONS FOR MARINE STRUCTURES

The very soft soils at the site will require that heavy marine structures be supported on long piles deriving support in the dense sandy and stiff clayey soils located approximately 120 to 140 feet below present dike grade. The soft dredged materials and the soft natural clays will impose substantial lateral forces on marine structures which are required to retain embankments or soil masses. These lateral forces will be particularly significant if additional dredging is performed to deepen the ship channel adjacent to the structures.

The great depth of the supporting soils and the low shear strength of the soils near the surface will require the use of heavy pile sections to resist the lateral forces applied to the marine structures. Heavy prestressed solid concrete piles, prestressed concrete cylinder piles or large diameter pipe piles will be required to provide adequate supporting capacity for the marine structures. Batter piles in tension or in compression will be required to resist lateral forces. General design principles for deep pile foundations are presented in Section 5.3 of this report.

6.3 EARTH-RETAINING STRUCTURES

It is expected that earth-retaining structures such as sheet pile walls or concrete retaining walls will be required along the nearshore edge of the piers to hold back the fill, the soft dredged materials, and the soft natural clays. It may be necessary to replace a portion of the soft materials immediately adjacent to the marine structures with select granular fills in order to reduce the lateral forces applied on earth-retaining structures. However, stabilization of the onshore area by preloading and drainage, before constructing the walls would also improve the strength of the soils and thereby reduce the lateral forces.

Lateral forces would have to be resisted by means of batter piles embedded into the deeper dense sandy or stiff clayey soils, since the soft soils near the ground surface would not provide sufficient lateral resistance for tie-back anchors or deadmen. In addition, soft soils adjacent to the pile-supported marine structures are expected to undergo settlements which would apply significant downdrag loads on the piles supporting the marine structures. These piles must therefore be designed to support the additional load applied by these downdrag forces.

7.0 SPECIAL PROBLEMS

7.1 PIPELINE

We understand that the U.S. Navy is proposing to construct an oil pipeline along the eastern side of the Dredge Disposal Area to connect its fuel storage facilities on Craney Island with its ship loading facilities in Norfolk. The pipeline will be placed in a 5-foot deep trench on the Dredge Disposal Area and will cross the Norfolk Channel in an underwater trench.

On land the trench is expected to have 2½ horizontal on 1 vertical slopes. Such slopes are expected to be stable in the sandy and silty surface soils in the Dredge Disposal Area. However, in areas of soft clayey soils, the walls of the trench would have to be shored and braced for stability. In areas where the groundwater is high, the trench would have to be dewatered during construction.

As the surface of the Dredge Disposal Area settles due to the compression of the soft dredged materials and the soft natural clays, the pipeline will also settle. Therefore, flexible joints and pipe loops should be included in the pipeline to accommodate such settlement. If the pipe trench is backfilled with compacted granular materials, these will settle less than the surrounding soft soils. Therefore, after some settlement has taken place, the pipeline alignment may shown a "hump" which may interfere with the utilization of the area. Periodic maintenance may be required along the pipeline alignment to relevel the surface of the ground.

The pipeline crossing in the Norfolk channel is far enough from the proposed development areas and is therefore not expected to interfere with the proposed construction on the Dredge Disposal Area.

7.2 CHANNEL DREDGING

We understand that the Norfolk channel may be dredged to elevation -55 feet to accommodate larger ships. In addition, dredging will be required to provide a channel for accessing the marine facilities to be constructed on the east side of the Dredge Disposal Area. Based on information from the Corps of

Engineers, we understand that the existing underwater slopes of the dikes constructed for the Dredge Disposal Area were constructed on a slope of 30 horizontal to 1 vertical. These dikes are underlain by the soft natural clay deposits. Dredging the soft clays would undermine the stability of the east dike of the Dredge Disposal Area--therefore, retaining structures would have to be installed prior to dredging to retain the soft dredged deposits and the soft natural clays.

7.3 SLOPE PROTECTION

After the proposed marine facilities have been completed, there will be exposed slopes consisting of soft clays in some areas. Even if stabilized by preloading and drainage, these clays will be unable to resist erosion by currents and waves. To prevent scouring and undermining, the clay slopes must be protected by means of rip-rap revetments. To prevent the fines from washing out, the rip-rap must be supported on a filter cloth or on a filter bed of coarse granular soils. Detailed designs for filter beds and for rip-rap revetments will be developed after the design of the waterfront structure has been finalized.

7.4 SEISMIC FORCES

Craney Island is located in Seismic Zone I (Minor Damage Category) - as classified in the Southern Standard Building Code in use in the area. The U.S. Coast and Geodetic Survey map for the area shows the maximum seismic design ground acceleration for Zone I to be 0.08 g. The California Applied Technology Council map for the area shows an affective peak acceleration on the order 0.05 g. Because of these relatively low accelerations, static design methods with adequate factors of safety are expected to be adequate for structures in the area.

7.5 PAVEMENTS

We expect that the proposed marine facilities and storage areas will be surrounded by large paved areas used for outdoor storage, access to the storage areas and the facilities, and support of rubber-tired cranes.

Although concrete pavements will be used near some of the structures, it is expected that most of the area will have flexible pavements.

We expect that flexible pavements, supported on areas stabilized by preloading and drainage and underlain by no less than 4 feet of compacted granular fill, will perform satisfactorily. Differential settlements may cause uneven pavement surfaces with resulting ponding after rains or uneven riding surfaces. Therefore, some pavement maintenance must be anticipated.

7.6 SOIL CORROSIVENESS

The dredged materials and the groundwater in the area may contain chemicals corrosive to steel or concrete. Therefore, chemical tests should be performed on select samples of dredged materials and the groundwater in the vicinity of proposed structures. Based on these tests, protective measures, if required, would be applied to steel or concrete to be placed in the area.

APPENDIX A - FIELD EXPLORATIONS

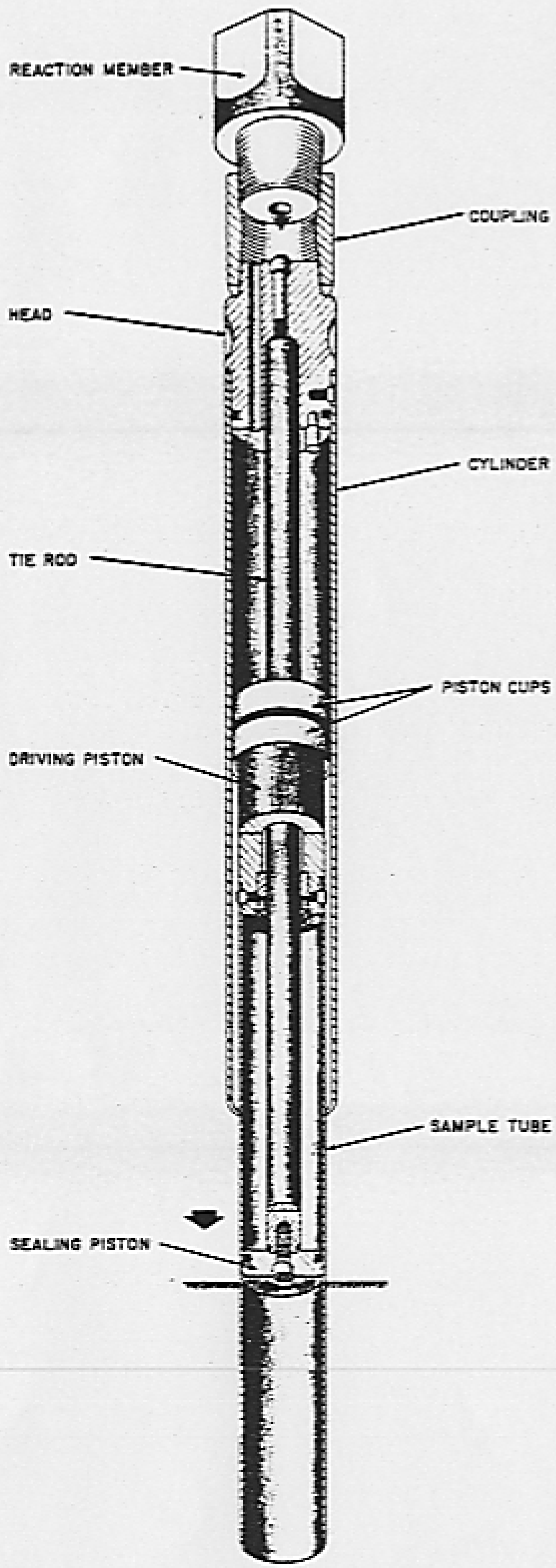
The subsurface conditions in the area of the proposed development were investigated by drilling four borings, ranging in depths from 154.5 feet to 201.5 feet below existing ground surface. The locations of these borings are shown in Figure 1, Site Plan.

The borings were drilled by the Girdler Foundation & Exploration Company of Clearwater, Florida, utilizing a truck mounted, Failing 1500 drill rig. The borings were advanced through the subsurface soils and fill materials by the rotary-wash method. Samples of the various soil deposits encountered were obtained at approximately 5-foot intervals. Standard Penetration Tests were performed at frequent intervals as shown on the boring logs. Relatively undisturbed samples of the soils in the borings were obtained by means of Shelby tubes, and Dames & Moore piston and Type U samplers, the latter two illustrated on Plates A-1 and A-2. The tube samplers were pushed hydraulically into the soil, while the Type U sampler was driven by a 300-pound hammer falling a distance of 24 inches. The standard split spoon used for the Standard Penetration Test (disturbed samples) was driven by a 140-pound hammer falling 30 inches. The blow counts on the driven samplers represents the number of blows required to advance the sampler a distance of 1 foot.

The drilling operations conducted during this investigation were supervised by one of our engineers, who classified the soils, obtained the soil samples, and maintained a continuous log of the borings. Ground water levels were also observed during the drilling and at the completion of each test boring. Final ground water level readings were obtained after the completion of the boring and are presented below the log of each boring.

Ground surface elevations and the coordinates for each boring are presented above the log of each boring. The completed borings were located by Corps of Engineers' surveyors in relation to the Virginia State Grid System.

Graphical representation of the soils encountered in each boring is presented on the Log of Borings, Plates A-4 and A-8. The nomenclature used in describing the soil types is presented on Plate A-3, Unified Soil Classification System.



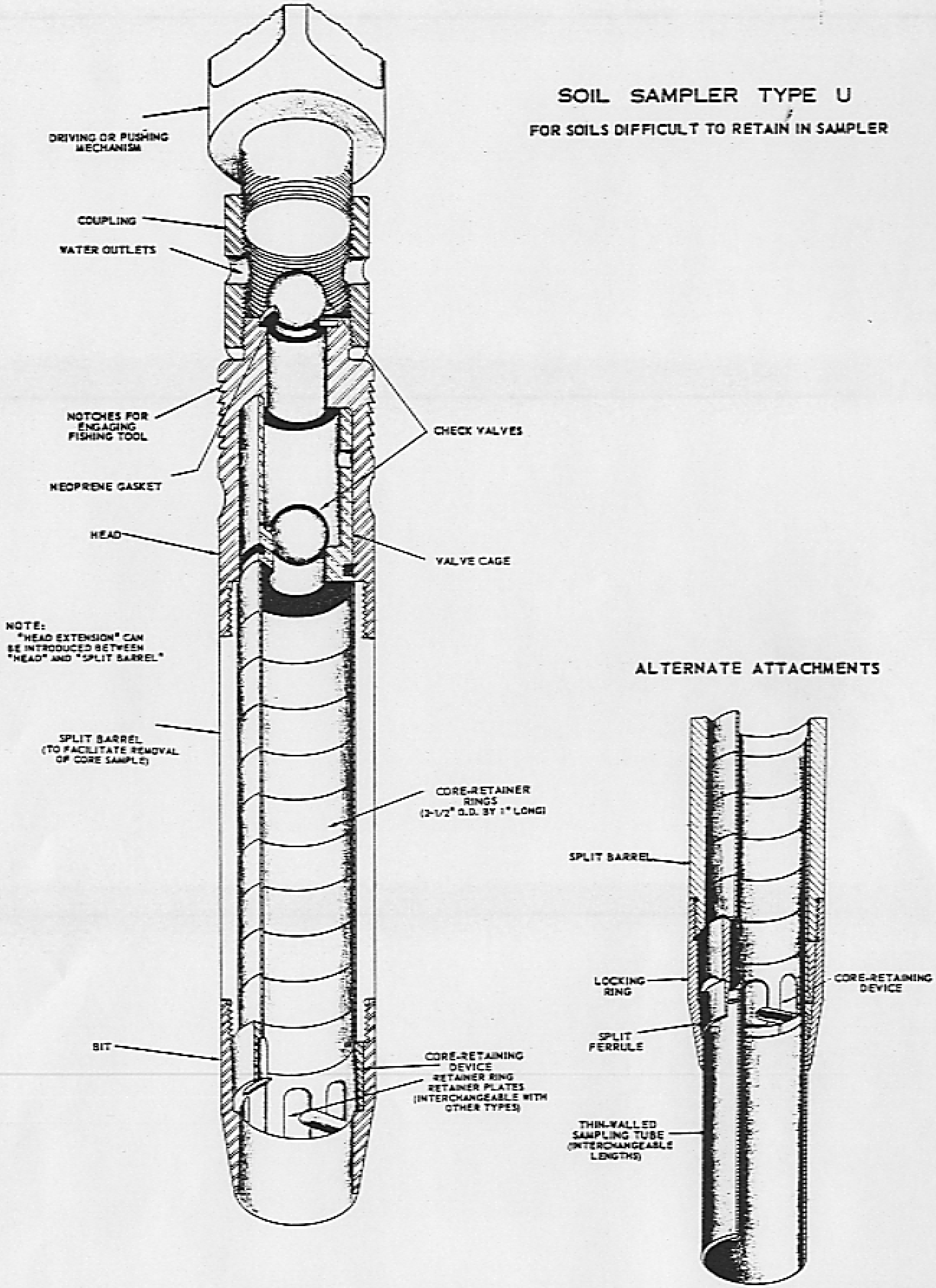
PISTON SAMPLER

THE DAMES & MOORE PISTON SAMPLER HAS BEEN DEVELOPED TO OBTAIN SAMPLES OF SOFT SOILS WITH A MINIMUM OF DISTURBANCE. THE MOST SIGNIFICANT FEATURES ARE THE SEALING PISTON WHICH CONFINES THE SOIL DURING SAMPLING AND THE SAMPLE TUBE WHICH HAS A WALL THICKNESS OF ONLY 0.042 INCHES.

AT THE START OF THE SAMPLING, THE LOWER END OF THE SAMPLE TUBE IS ADJACENT TO THE SEALING PISTON AT THE BOTTOM OF AN EXPLORATION TEST BORING. THE SEALING PISTON, CYLINDER, HEAD, AND REACTION MEMBER REMAIN STATIONARY DURING SAMPLING. COMPRESSED AIR, COMPRESSED NITROGEN, OR WASH WATER ARE FORCED INTO THE CYLINDER THROUGH THE SAMPLING RODS FROM THE DRILLING EQUIPMENT. THE DRIVING PISTON MOVES THE SAMPLE TUBE DOWNWARD INTO THE SOIL.

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



DAMES & MOORE

PLATE A-2

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
MORE THAN 50 % OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50 % OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTES:

1. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE CLASSIFICATIONS.
2. WHEN SHOWN ON THE BORING LOGS, THE FOLLOWING TERMS ARE USED TO DESCRIBE THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE COMPACTNESS OF COHESIONLESS SOILS.

COHESIVE SOILS

	(APPROXIMATE SHEARING STRENGTH IN KSF)
VERY SOFT	LESS THAN .25
SOFT	0.25 TO 0.5
MEDIUM STIFF	0.5 TO 1.0
STIFF	1.0 TO 2.0
VERY STIFF	2.0 TO 4.0
HARD	GREATER THAN 4.0

COHESIONLESS SOILS

VERY LOOSE
LOOSE
MEDIUM DENSE
DENSE
VERY DENSE

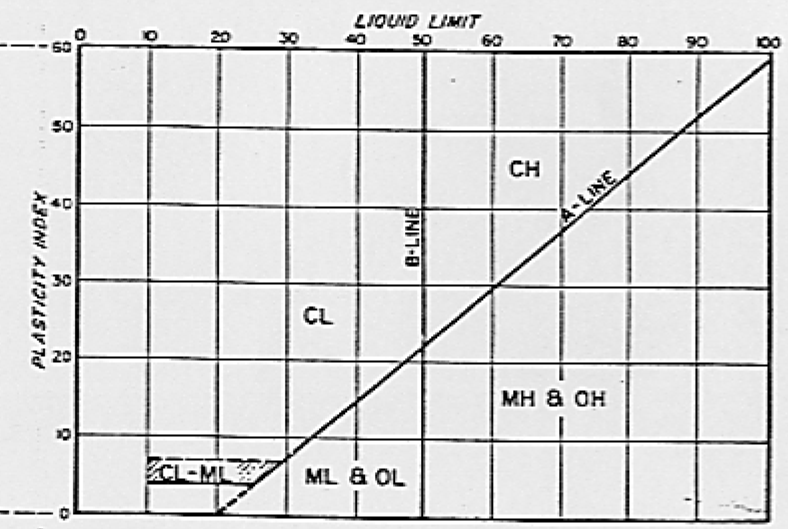
THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE, AND SOIL DENSITY DATA.

GRADATION CHART

MATERIAL SIZE		PARTICLE SIZE			
		LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE•	MILLIMETERS	SIEVE SIZE•
SAND	FINE	.074	#200•	0.42	#40•
	MEDIUM	0.42	#40•	2.00	#10•
	COARSE	2.00	#10•	4.76	#4•
GRAVEL	FINE	4.76	#4•	19.1	3/4"*
	COARSE	19.1	3/4"*	76.2	3"*
COBBLES		76.2	3"*	304.8	12"*
BOULDERS		304.8	12"*	914.4	36"

•U.S. STANDARD *CLEAR SQUARE OPENINGS

PLASTICITY CHART



FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

SAMPLES

■ INDICATES UNDISTURBED SAMPLE
⊠ INDICATES DISTURBED SAMPLE
□ INDICATES SAMPLING ATTEMPT WITH NO RECOVERY

I INDICATES LENGTH OF CORING RUN

NOTE:
DEFINITIONS OF ANY ADDITIONAL DATA REGARDING SAMPLES ARE ENTERED ON THE FIRST LOG ON WHICH THE DATA APPEAR.

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

1 NO. 14-68

KEY TO SAMPLES:

15 ■ INDICATES APPROXIMATE DEPTH OF RELATIVELY UNDISTURBED SAMPLE OBTAINED WITH A DAMES & MOORE TYPE U SAMPLER.

— NUMBER OF BLOWS REQUIRED TO ADVANCE A DAMES & MOORE TYPE U SAMPLER, ONE FOOT WITH A 300-POUND WEIGHT FALLING APPROXIMATELY 30 INCHES.

W ☒ INDICATES APPROXIMATE DEPTH OF DISTURBED SAMPLE OBTAINED WITH A DAMES & MOORE TYPE U SAMPLER.

— INDICATES THAT SAMPLER WAS ADVANCED UNDER THE STATIC WEIGHT OF THE DRIVING HAMMER AND DRILL RODS.

R ☐ INDICATES DEPTH OF A DAMES & MOORE TYPE U SAMPLING ATTEMPT WITH NO RECOVERY.

— INDICATES THAT SAMPLER WAS ADVANCED UNDER THE STATIC WEIGHT OF THE DRILL RODS.

8 ◪ INDICATES DEPTH OF DISTURBED SAMPLE OBTAINED WITH A STANDARD SPLIT SPOON SAMPLER.

— NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD, SPLIT SPOON SAMPLER, FALLING 30 INCHES (N-VALUES FROM SPT).

⧅ INDICATES DEPTH OF STANDARD SPLIT SPOON SAMPLING ATTEMPT WITH NO RECOVERY.

P ▮ INDICATES APPROXIMATE DEPTH AND SAMPLING INTERVAL OF RELATIVELY UNDISTURBED SAMPLE OBTAINED WITH A SHELBY TUBE SAMPLER.

— INDICATES THAT THE SAMPLER WAS HYDRAULICALLY PUSHED TO OBTAIN THE SAMPLE.

INDICATES APPROXIMATE DEPTH AND SAMPLING INTERVAL OF DISTURBED SAMPLE OBTAINED WITH A SHELBY TUBE SAMPLER.

INDICATES APPROXIMATE DEPTH AND SAMPLING INTERVAL OF SHELBY OR DAMES & MOORE PISTON TUBE SAMPLING ATTEMPT WITH NO RECOVERY.

INDICATES APPROXIMATE DEPTH AND SAMPLING INTERVAL OF RELATIVELY UNDISTURBED SAMPLE OBTAINED WITH A DAMES & MOORE PISTON TUBE SAMPLER.

INDICATES APPROXIMATE DEPTH AND SAMPLING INTERVAL OF DISTURBED SAMPLE OBTAINED WITH A DAMES & MOORE PISTON TUBE SAMPLER.

KEY TO SOIL TEST DATA ABBREVIATIONS:

%F	PERCENT FINES (FRACTION PASSING No. 200 SIEVE) ANALYSIS
UNC	UNCONFINED COMPRESSION TEST
TX/CIP/PP	TRIAXIAL COMPRESSION TEST - CONSOLIDATED ISOTROPICALLY W/PORE PRESSURE MEASUREMENT
TX/UU	TRIAXIAL COMPRESSION TEST - UNCONSOLIDATED UNDRAINED
P	PERMEABILITY TEST
PSA	PARTICLE SIZE ANALYSIS
CONSOL	CONSOLIDATION TEST
GS	SPECIFIC GRAVITY

NOTES:

1. ELEVATIONS REFER TO MEAN LOW WATER LEVEL DATUM.
2. COORDINATES REFER TO THE VIRGINA STATE GRID SYSTEM.
3. SOILS ARE CLASSIFIED ACCORDING TO THE UNIFIED SOIL CLASSIFICATION SYSTEM.
4. THE DISCUSSION IN THE TEXT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.

KEY TO LOG OF BORINGS

DAMES & MOORE

BORING DM-1

SURFACE ELEVATION: 21.4 FT.
LOCATION: N213,043 E2,628,050

LABORATORY TEST DATA

Depth (ft, approx.)	Other Tests	Triaxial* Compression $\frac{\sigma_1-\sigma_3}{2}$	Triaxial* Compression σ_3	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index %	Moisture Content %	Dry* Density
0–5						28.6	
0–5						26.8	91.1
0–5						18.2	97.7
15–20						24.2	90.1
30–35	UNC	195.5				62.6	61.7
35–40				46	23	56.4	
40–45	1DSF					26.4	
55–60						16.3	
65–70						20.2	100.7
75–80	PSA						

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

Boring log

Depth in Feet (approx.)	Blow Count	Symbols	Description
0–5	20, 7	SC	GRAY-BROWN FINE TO MEDIUM SAND WITH SOME SILTY CLAY WITH A TRACE OF FINE SHELL FRAGMENTS (MEDIUM DENSE) GRADING WITH LESS FINES GRADING WITH STRINGERS OF SILTY CLAY
5–12	4	SM	DARK GRAY-GREEN FINE TO MEDIUM SAND WITH A TRACE OF COARSE SAND A LITTLE SILT AND A TRACE OF FINE TO COARSE SHELL FRAGMENTS (LOOSE)
12–18	8	SP	LIGHT GRAY WITH ORANGE SPOTS LOOSE FINE SAND WITH A TRACE OF SILT (LOOSE) GRADING WITH MEDIUM SAND AND A TRACE OF FINE TO COARSE SHELL FRAGMENTS
18–28	8, 8	GM	GRAY-GREEN FINE TO MEDIUM SAND WITH A LITTLE SILT AND OCCASIONAL STRINGERS AND ORANGE-BROWN SILTY CLAY AND FINE TO COARSE GRAVEL AND SHELL FRAGMENTS (LOOSE) STRINGERS OF SILTY CLAY GRADING OUT GRADING WITH A 3" COBBLE
28–40	P	CL	DARK GRAY-GREEN SILTY CLAY WITH A TRACE OF FINE TO MEDIUM SAND AND A TRACE OF FINE SHELL FRAGMENTS GRADING WITH A STRINGER OF FINE TO MEDIUM SAND SHELLS GRADING OUT
40–50	3, P	SP/SM	DARK GRAY-GREEN FINE SAND WITH A TRACE OF SILT AND FINE SHELLS (VERY LOOSE) SHELLS GRADING OUT
50–60	P, 17	SM	GRAY-GREEN FINE SAND WITH A LITTLE SILT AND A TRACE OF FINE TO COARSE SHELL FRAGMENTS (MEDIUM DENSE) GRADING WITH A TRACE OF MEDIUM TO COARSE SAND AND A TRACE OF CLAY
60–72	P, 6, 4	SP/SM	GRAY-GREEN FINE TO MEDIUM SAND WITH A TRACE OF SILT (LOOSE) GRADING WITH A TRACE OF CLAY AND SHELLS GRADING WITH MORE FINES
72–79	P	SM/ML	DARK GRAY-GREEN FINE SAND AND A TRACE OF FINE SHELLS WITH STRINGERS OF CLAYEY SILT GRADING WITH COARSE SHELL FRAGMENTS
79–85	P	CH	DARK GRAY-GREEN SILTY CLAY WITH TRACE OF FINE SAND AND SHELLS

FILL

LOG OF BORINGS

A-6

DAMES & MOORE

BORING DM-1 (CONT'D)

SURFACE ELEVATION: 21.4 FT.
LOCATION: N213,013 E2,628,050

LABORATORY TEST DATA

Depth in Feet	Other Tests	Triaxial* Compression $\frac{\sigma_1-\sigma_3}{2}$	Triaxial* Compression σ_3	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index %	Moisture Content %	Dry* Density
85	PSA			81	43	81.7	50.9
90	TX/UU	231	4200				
95							
100				69	32	66.5	
105	UNC	562.0				64.5	59.1
110							
115							
120						23.1	97.4
125							
130	34SF					29.4	
135						26.0	96.5
140	PSA					20.7	
145							
150							
155	UNC	321				26.1	99.7
160	-						
165	CONSOL			63	38	37.3	84.8

Depth in Meters	Depth in Feet	Blow Count	Symbols	Description
26	85	2	CH	
28	90	P	CH	
29	95	P	CH	GRADING WITH A TRACE OF ORGANICS
30–31	100	5	CH	GRADING MORE SILT
32	105	P	CH	ORGANICS GRADING OUT
33–34	110	P	CH	
35	115	9	SM	GRAY-GREEN FINE SAND WITH STRINGERS OF CLAYEY SILT AND A TRACE OF ORGANICS (LOOSE)
37	120	11	SM	GRADING WITH MEDIUM SAND
38	125	P	SM	GRADING WITH A TRACE OF SHELL FRAGMENTS
40	130	30	SM	GRADING WITH MORE CLAYEY SILT SHELL FRAGMENTS GRADING OUT
41	135	24	SP/SM	GRAY-GREEN FINE SAND WITH A TRACE OF SILT (MEDIUM DENSE)
43	140	72	SP/SM	GRADING TO VERY DENSE
44	145	18	SM/ML	GRADING WITH MEDIUM SAND GRAY-GREEN FINE SAND AND SILT WITH A TRACE OF SHELLS AND WITH OCCASIONAL LARGE SHELL FRAGMENTS (MEDIUM DENSE)
45–46	150	25	SM	GRAY-GREEN FINE SAND WITH SOME SILT AND A TRACE OF CLAY AND SHELLS (MEDIUM DENSE)
47	155	20	SM	
49	160	31	SM	SHELLS GRADING OUT GRADING TO DENSE
50	165	12	CH	DARK GRAY-GREEN SILTY CLAY (MEDIUM STIFF)
51	170			

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

LOG OF BORINGS

A-7

DAMES & MOORE

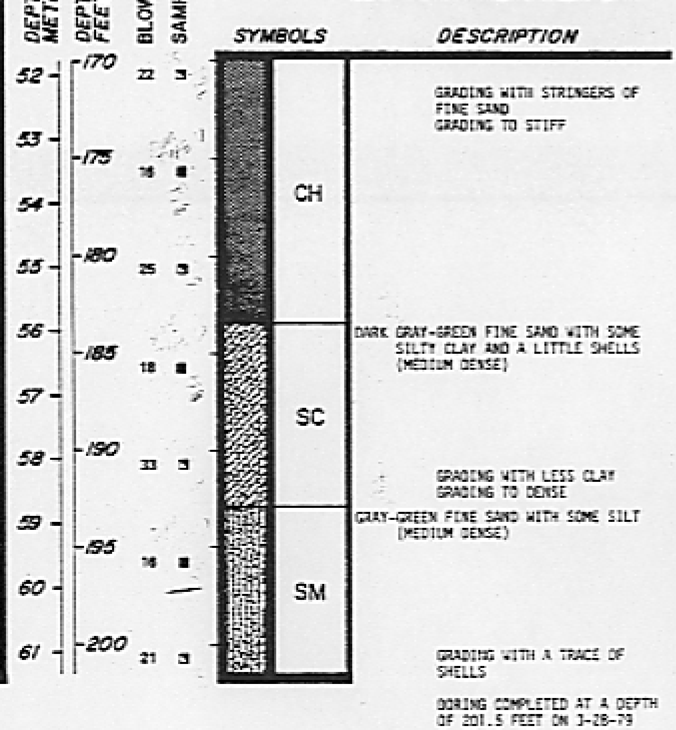
PLATE A-5 (CONT'D)

BORING DM-I (CONT'D)

SURFACE ELEVATION: 21.4 FT.
LOCATION: N213,013 E2,628,050

LABORATORY TEST DATA						
OTHER TESTS	TRIAXIAL* COMPRESSION $\frac{\sigma_1-\sigma_3}{2}$	TRIAXIAL* COMPRESSION σ_3	ATTERBERG LIMITS LIQUID LIMIT %	ATTERBERG LIMITS PLASTICITY INDEX %	MOISTURE CONTENT %	DRY* DENSITY
UNC	730				42.3	78.5
			56	36		
					29.0	92.9
27:F					27.1	

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF



LOG OF BORINGS

BORING DM-2

SURFACE ELEVATION: 21.4 FT.
LOCATION: N214,474 E2,626,571

LABORATORY TEST DATA

Depth interval (feet)	Other Tests	Triaxial* Compression $\frac{\sigma_1-\sigma_3}{2}$	σ_3	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index %	Moisture Content %	Dry* Density
0–5						47.6	
5–10						28.8	
10–15							
15–20				54	24	65.7	59.6
20–25						25.0	
25–30							
30–35							
35–40							
40–45	TX/CIU/PP560	350				59.3	64.4
45–50				53	29	45.4	
50–55							
55–60							
60–65	PSA					28.6	
65–70							
70–75						30.0	
75–80							
80–85						31.9	87.6

BLOW COUNT

Approx. Depth (feet)	Blow Count
2.5	2
7	2
11	3
14	2
18.5	W
23	8
28	7
33	1
38	W
44	P
49	3
54	P
59	P
64	12
69	P
74	1
79	26
84	14

SYMBOLS / DESCRIPTION

Symbol	Description
CL	DARK GRAY-GREEN SILTY CLAY AND FINE TO MEDIUM SAND WITH A TRACE OF COARSE SAND AND FINE TO COARSE SHELL FRAGMENTS AND ORGANICS (VERY SOFT)
SM	BROWN MOTTLED WITH GRAY-GREEN FINE SAND WITH SOME SILT AND STRINGERS OF SILTY CLAY WITH TRACES OF ORGANICS AND SHELLS (VERY LOOSE)
SP	GRAY-GREEN FINE TO MEDIUM SAND WITH SHELL FRAGMENTS AND A TRACE OF SILT (VERY LOOSE)
MH/CH	BLACK CLAYEY SILT WITH A TRACE OF FINE SAND AND A TRACE OF ORGANICS (VERY SOFT)
SP	GRAY-GREEN FINE TO MEDIUM SAND WITH A TRACE OF SILT AND A TRACE OF FINE SHELL FRAGMENTS (LOOSE) GRADING WITH MORE SHELLS AND A TRACE OF COARSE SAND GRADING TO VERY LOOSE WITH MORE SHELLS
CH	DARK GRAY-GREEN SILTY CLAY WITH A LITTLE SHELL FRAGMENTS AND A TRACE OF FINE SAND (VERY SOFT) GRADING TO A TRACE OF SHELL FRAGMENTS GRADING TO SOFT GRADING WITH MORE SHELLS
SC/CL	LIGHT BROWN-GREEN FINE SAND AND SILTY CLAY WITH A LITTLE SHELL FRAGMENTS (MEDIUM DENSE) GRADING TO GRAY-GREEN GRADING TO VERY LOOSE
SM	GRAY-GREEN FINE SAND WITH SOME CLAYEY SILT AND A TRACE OF SHELL FRAGMENTS (MEDIUM DENSE)

FILL (0 to approx. 40 ft)

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

LOG OF BORINGS

A-9

DAMES & MOORE

PLATE A-6

BORING DM-2 (CONT'D)

SURFACE ELEVATION: 21.4 FT.
LOCATION: N214,474 E2,626,571

LABORATORY TEST DATA

OTHER TESTS	TRIAXIAL* COMPRESSION $\frac{\sigma_1-\sigma_3}{2}$	TRIAXIAL* COMPRESSION σ_3	ATTERBERG LIMITS LIQUID LIMIT %	ATTERBERG LIMITS PLASTICITY INDEX %	MOISTURE CONTENT %	DRY* DENSITY
PSA					33.8	87.0
					25.6	91.6
UNC	239.5				30.3	93.6
					27.6	
46SF					28.6	
			60	33	39.7	

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

DEPTH IN METERS: 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47

DEPTH IN FEET: 85, 90, 95, 100, 105, 110, 115, 120, 125, 130, 135, 140, 145, 150

BLOW COUNT (top to bottom): 35, 8, 19, P, 20, 40, 23, 37, 12, 21, 10, 28

SYMBOLS	DESCRIPTION
SM	GRADING TO DENSE
	GRADING TO MEDIUM DENSE
	GRADING WITH OCCASIONAL 1/2 INCH LAYERS OF FINE TO COARSE SHELL FRAGMENTS
	SHELLS GRADING OUT
	GRADING TO DENSE
	? CLAY
SC/CL	GRAY-GREEN FINE SAND AND SILTY CLAY WITH A TRACE OF FINE SHELL FRAGMENTS (MEDIUM DENSE)
CH	GRAY-GREEN SILTY CLAY (VERY STIFF)
	BORING COMPLETED AT A DEPTH OF 154.5 FEET ON 3-30-79

LOG OF BORINGS

A-10

DAMES & MOORE

BORING DM-3

SURFACE ELEVATION: 26.4 FT.
LOCATION: N216,541 E2,627,442

LABORATORY TEST DATA

OTHER TESTS	TRIAXIAL* COMPRESSION $\frac{\sigma_1-\sigma_3}{2}$	TRIAXIAL* COMPRESSION σ_3	ATTERBERG LIMITS LIQUID LIMIT %	ATTERBERG LIMITS PLASTICITY INDEX %	MOISTURE CONTENT %	DRY* DENSITY
					31.2	
					10.9	110.3
			65	30	67.1	56.5
CONSOL			68	35	79.5	63.7
					44.2	69.5
					70.9	
PSA						
					71.7	
TX/UU	372	1800			84.9 84.0	51.1 50.6

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

DEPTH IN METERS: 0 – 25
DEPTH IN FEET: 0 – 85

BLOW COUNT (top to bottom): 9, 17, 18, 2, 3, 4, 14, W, P, 5, P, P, 2, P, P, 2, P, P

SAMPLES

SYMBOLS	DESCRIPTION
SM	BROWN SAND SITH SOME CLAYEY SILT AND A LITTLE FINE SHELL FRAGMENTS (LOOSE)
SP/SM	BROWN FINE TO MEDIUM SAND WITH A LITTLE SILT AND A LITTLE FINE TO COARSE SHELL FRAGMENTS (MEDIUM DENSE)
GM	BROWN FINE TO COARSE GRAVEL AND FINE TO COARSE SAND WITH A TRACE OF SILT AND FINE TO COARSE SHELL FRAGMENTS (MEDIUM DENSE) GRADING WITH A TRACE OF CLAYEY SILT GRADING TO VERY LOOSE
MH/CH	BLACK CLAYEY SILT WITH A LITTLE FINE SAND AND SHELL FRAGMENTS (SOFT)
SP	GRAY-GREEN FINE SAND WITH A TRACE OF SILT (LOOSE) GRADING WITH A TRACE OF FINE SHELL FRAGMENTS (MEDIUM DENSE) 4 INCH LAYER OF DARK GRAY-GREEN SILTY CLAY
CH	DARK GRAY-GREEN SILTY CLAY (VERY SOFT) 6 INCH LAYER OF CLAYEY FINE SAND GRADING WITH SHELLS GRADING WITH A LITTLE FINE SAND GRADING TO SOFT GRADING WITH MORE SHELLS GRADING TO A TRACE OF SHELLS SHELLS GRADING OUT GRADING WITH A TRACE OF FINE SHELLS

FILL

LOG OF BORINGS

DAMES & MOORE

PLATE A-7

BORING DM-3 (CONT'D)

SURFACE ELEVATION: 26.4 FT.
LOCATION: N216,641 E2,527,442

LABORATORY TEST DATA

OTHER TESTS	TRIAXIAL* COMPRESSION $\frac{\sigma_1-\sigma_3}{2}$	TRIAXIAL* COMPRESSION σ_3	ATTERBERG LIMITS LIQUID LIMIT %	ATTERBERG LIMITS PLASTICITY INDEX %	MOISTURE CONTENT %	DRY* DENSITY
GS,P CONSOL TX/CIU/PP	1030	1262			63.3 72.3	60.4 54.5
UNC	509				72.7 56.9	56.2 63.8
					76.9	
PSA					52.7	
16SF					27.0	
					29.1	91.4
			61	36	40.5	84.5

DEPTH IN METERS: 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51

DEPTH IN FEET: 85, 90, 95, 100, 105, 110, 115, 120, 125, 130, 135, 140, 145, 150, 155, 160, 165, 170

BLOW COUNT (SAMPLES, top to bottom): 3, P, P, W, P, P, 5, 6, 9, 9, P, 42, 31, 52, 19, 27, 10

SYMBOLS	DESCRIPTION
CH	SHELLS GRADING OUT GRADING TO STIFF GRADING TO VERY SOFT GRADING TO MEDIUM STIFF GRADING WITH STRINGERS OF FINE SAND
SP	GRAY-GREEN FINE SAND WITH A TRACE OF SILT (DENSE) GRADING TO VERY DENSE
CH	GRAY-GREEN SILTY CLAY (VERY STIFF) GRADING WITH A TRACE OF SHELL FRAGMENTS

BORING COMPLETED AT A DEPTH OF 169.5 FEET ON 4-5-79

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

LOG OF BORINGS

A-12

DAMES & MOORE

PLATE A-7 (CONT'D)

BORING DM-4

SURFACE ELEVATION: 25.4 FT.
LOCATION: N217,850 E2,626,663

LABORATORY TEST DATA

Depth in Feet	Other Tests	Triaxial* Compression $\frac{\sigma_1-\sigma_3}{2}$	σ_3	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index %	Moisture Content %	Dry* Density	Blow Count
0–5						16.0		13
5–10								9
10–15						21.3	99.8	1
15–20								3
20–25	15SF					23.7		10
25–30						20.6	104.1	19
30–35								6
35–40						65.3		7
40–45						43.2	78.4	2
45–50								5
50–55								2
55–60	UNC	418				58.2	63.8	1
60–65						62.9		W
65–70				80	49			1
70–75	CONSOL TX/UU	289	1200	67	38	80.6 80.0	52.6 52.4	P
75–80	PSA							R
80–85						79.2		R

SYMBOLS / DESCRIPTION

- **SP** (0 to ~10.5 ft): BROWN FINE TO MEDIUM SAND WITH A TRACE OF SILT (MEDIUM DENSE)
  - GRADING TO GRAY-GREEN WITH A TRACE OF SHELL FRAGMENTS
  - GRADING TO LOOSE
- **SM** (~10.5 to ~25 ft): GRAY-GREEN FINE SAND WITH SOME CLAYEY SILT (VERY LOOSE)
  - GRADING WITH MEDIUM SAND
  - GRADING WITH A LITTLE SILT AND SHELL FRAGMENTS
- **SP** (~25 to ~28 ft): GRAY-GREEN FINE TO MEDIUM SAND WITH A TRACE OF SILT AND A TRACE OF SHELL FRAGMENTS (MEDIUM DENSE)
- FILL (0 to ~28 ft)
- **CH** (~28 to 85 ft): BLACK SILTY CLAY WITH A LITTLE SHELL FRAGMENTS (MEDIUM STIFF)
  - GRADING TO DARK GRAY-GREEN WITH STRINGERS OF FINE SAND
  - GRADING TO SOFT
  - SAND GRADING OUT
  - GRADING TO A TRACE OF SHELL FRAGMENTS
  - GRADING TO VERY SOFT
  - SHELLS GRADING OUT

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

LOG OF BORINGS

A-13

DAMES & MOORE

PLATE A-8

BORING DM-4(CONT'D)

SURFACE ELEVATION: 25.4 FT.
LOCATION: N 217,850 E2,626,663

LABORATORY TEST DATA

OTHER TESTS	TRIAXIAL* COMPRESSION $\frac{\sigma_1-\sigma_3}{2}$	TRIAXIAL* COMPRESSION σ_3	ATTERBERG LIMITS LIQUID LIMIT %	ATTERBERG LIMITS PLASTICITY INDEX %	MOISTURE CONTENT %	DRY* DENSITY	DEPTH IN FEET (approx.)	BLOW COUNT	SAMPLES
							91	2	▫
					55.9		101	W	▫
					50.2		116	R	▫
4TSF					33.1		126	18	▫
			63	30	57.5		136	6	▫
							146	40	▫
PSA					22.7	100.1	156	140	■
							166	30	▫

DEPTH IN METERS: 26–50; DEPTH IN FEET: 85–165

SYMBOLS	DESCRIPTION
CH	GRADING WITH A TRACE OF SHELL FRAGMENTS SHELLS GRADING OUT
SC/CL	DARK GRAY-GREEN FINE SAND AND SILTY CLAY (MEDIUM DENSE)
CH/MH	DARK GRAY-GREEN SILTY CLAY (MEDIUM STIFF)
SM	GRAY-GREEN FINE SAND WITH A LITTLE CLAYEY SILT AND A TRACE OF SHELL FRAGMENTS
SP/SM	GRAY-GREEN FINE TO MEDIUM SAND WITH A TRACE OF SILT (VERY DENSE)
SC	DARK GRAY-GREEN FINE TO MEDIUM SAND WITH SOME CLAY WITH A TRACE OF COARSE SAND
CH	DARK GRAY-GREEN STIFF SILTY CLAY

BORING COMPLETED AT A DEPTH OF OF 166.5 FEET ON 4-7-79

*NOTE
SHEAR STRESS, PSF
DRY DENSITY, PCF

LOG OF BORINGS

A-14

DAMES & MOORE

PLATE A-8(CONT'D)

APPENDIX B - LABORATORY TESTING

Representative soil samples were tested at Dames & Moore laboratories in Washington, D.C., and in Cranford, New Jersey. The laboratory tests performed include moisture-density determinations, Atterberg limit determinations, particle-size analyses, unconfined compression tests, unconsolidated-undrained and isotropically-consolidated-undrained triaxial compression tests, consolidation tests, specific gravity determinations, and permeability tests.

Moisture-Density Determinations - The moisture content and dry density of soil samples were determined in conjunction with each shear strength and consolidation test. Additional moisture-density tests were performed for correlation purposes. Moisture determinations were performed in accordance with ASTM Test Designation: D2216-71. The results of all moisture-density tests are presented at the left side on each boring log.

Atterberg Limit Determinations - Atterberg limit tests were performed on selected soil samples obtained from the test borings. The Atterberg limit tests include a determination of the liquid limit and plasticity index of the soil. These tests facilitate the classification of the soils and are also used for correlation purposes. The liquid limit and plasticity index were determined in accordance with ASTM Test Designations D423-66 and D424-59, respectively. The results of the Atterberg limit tests are presented at the left side on each boring log.

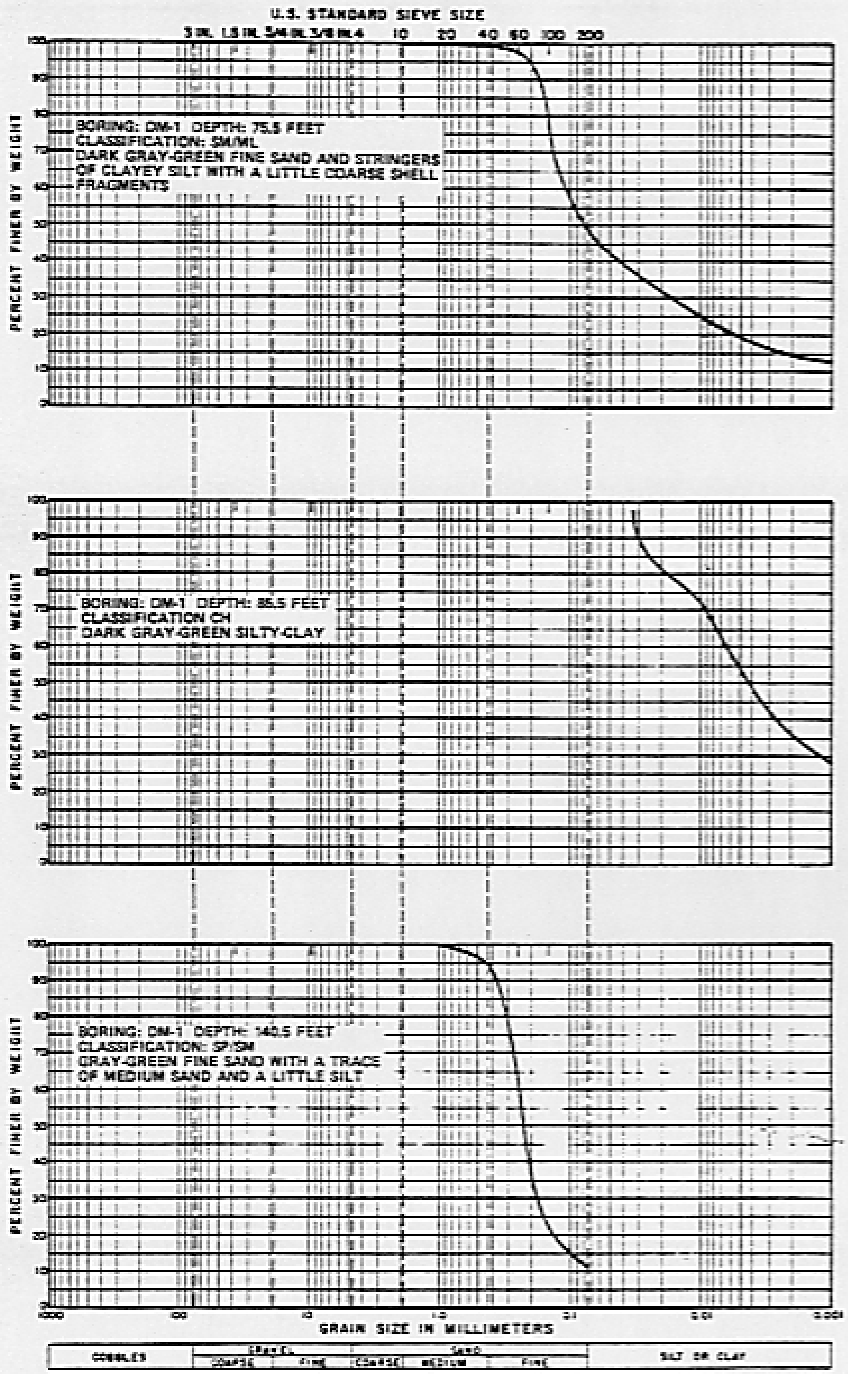
Particle-Size Analyses - Particle-size analyses were performed on selected soil samples obtained from the borings. These tests were used for classification and correlation purposes and were performed in accordance with ASTM Test Designation: D422-72, Particle-Size Analysis for Soils. The results of the particle-size analyses are presented on Plates B-1 through B-4.

Unconfined and Triaxial Compression Tests - Unconfined and triaxial compression tests were performed on representative samples to determine strength characteristics of the soil strata. Triaxial tests included unconsolidated-undrained tests and tests with isotropic consolidation to approximate in-situ stress conditions. Stress-strain curves were plotted for each compression test, and shear strength results shown on the left of the logs were determined utilizing the maximum axial stress developed. The strength tests were performed in the manner described on Plate B-5, Methods of Performing Unconfined Compression and Triaxial Compression Tests.

Consolidation Tests - Consolidation tests were performed on four samples of the clay materials in order to provide data for estimating the settlements of structures. The consolidation tests were performed in accordance with the method described on Plate B-6, Method of Performing Consolidation Tests. The results of the consolidation tests are presented on Plates B-7 through B-10, Consolidation Test Data.

Specific Gravity Tests - The specific gravity of representative clay samples were determined for correlation purposes in accordance with ASTM Test Designation: D854-58.

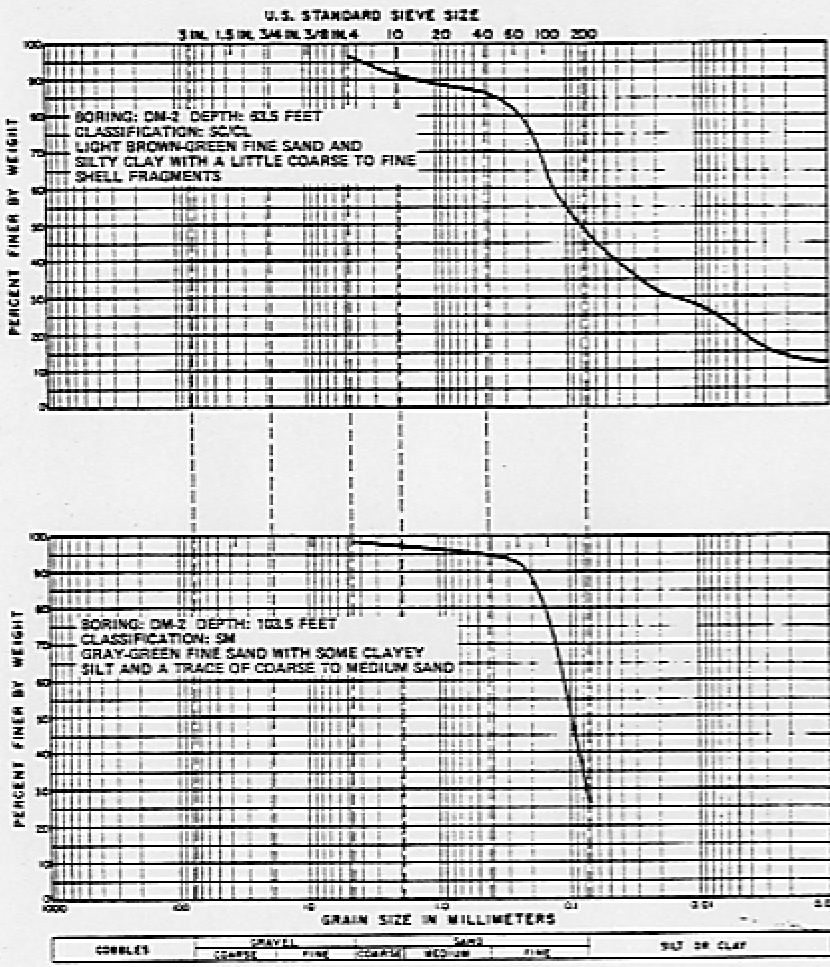
Permeability Test - A permeability test was performed on a representative clay sample as described on Plate B-11, Method of Performing Percolation Tests.



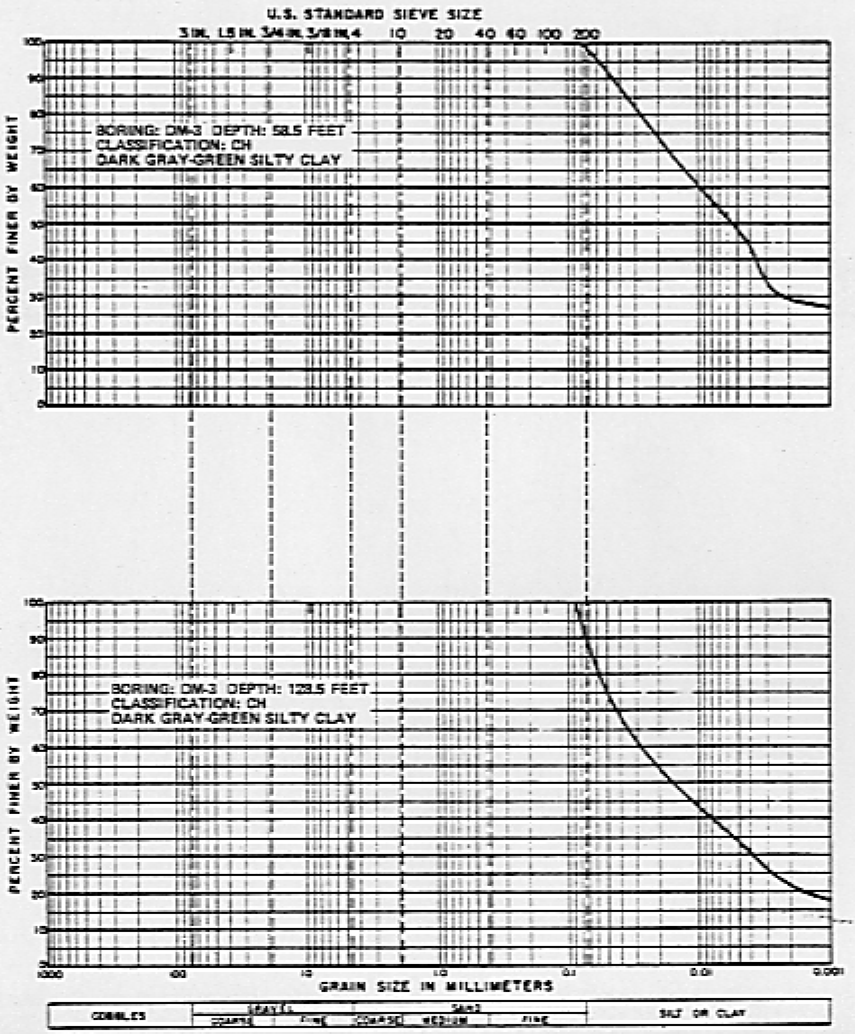
PARTICLE SIZE ANALYSES

DAMES & MOORE

PLATE B-1

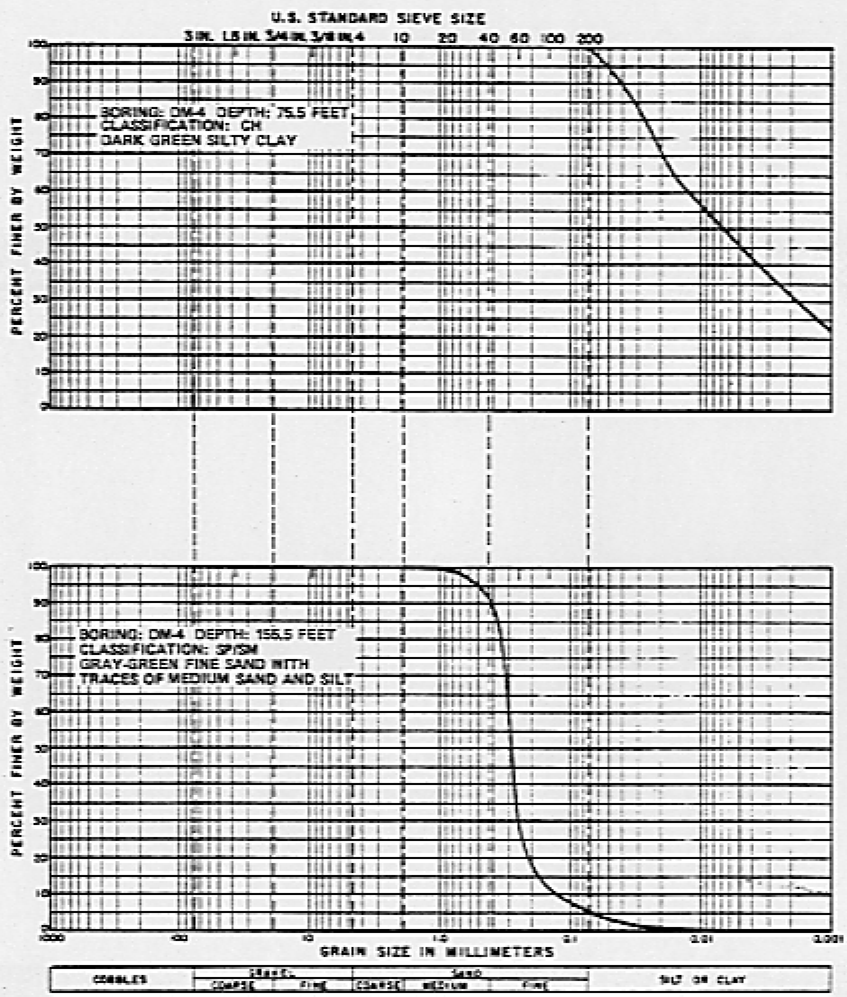


PARTICLE SIZE ANALYSES



PARTICLE SIZE ANALYSES

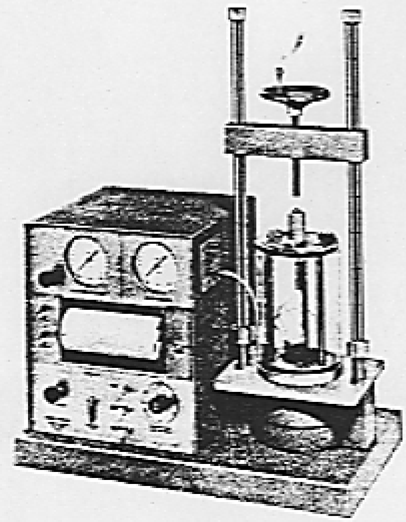
DAMES & MOORE



PARTICLE SIZE ANALYSES

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.



TRIAXIAL COMPRESSION TEST UNIT

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

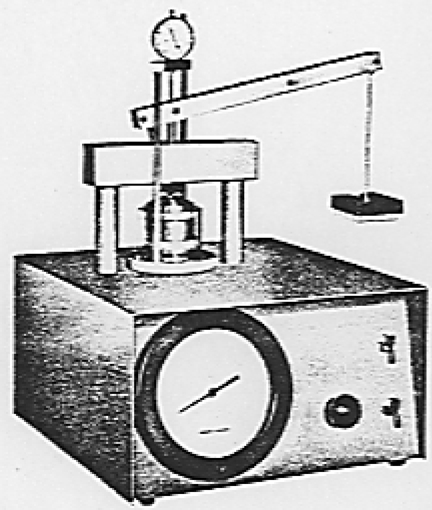
DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

Method Of Performing Consolidation Tests

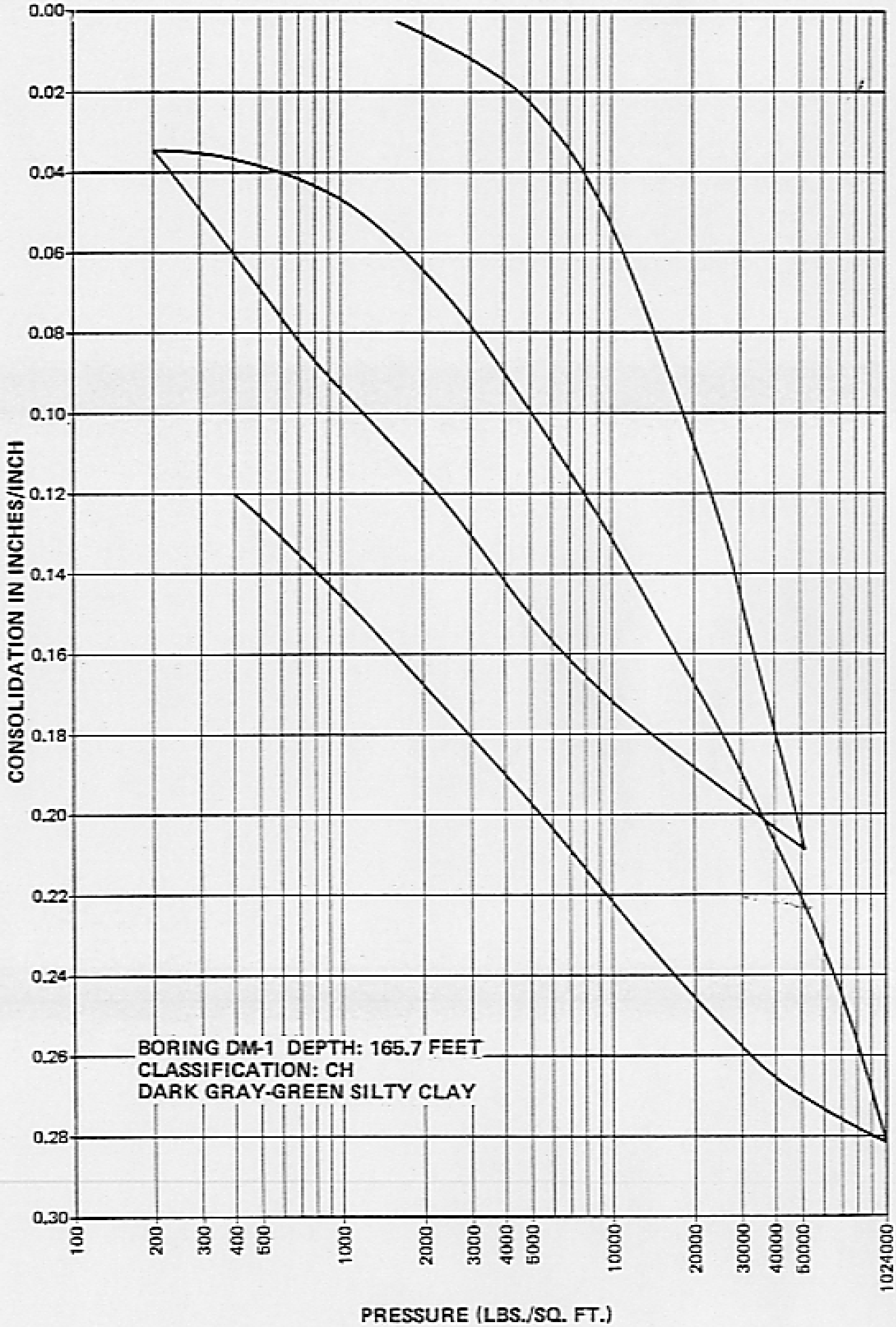
Consolidation tests are performed to evaluate the volume changes of soils subjected to increased loads. Time-consolidation and pressure-consolidation curves may be plotted from the data obtained in the tests. Engineering analyses based on these curves permit estimates to be made of the probable magnitude and rate of settlement of the tested soils under applied loads.

Each sample is tested within brass rings two and one-half inches in diameter and one inch in length. Undisturbed samples of in-place soils are tested in rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.

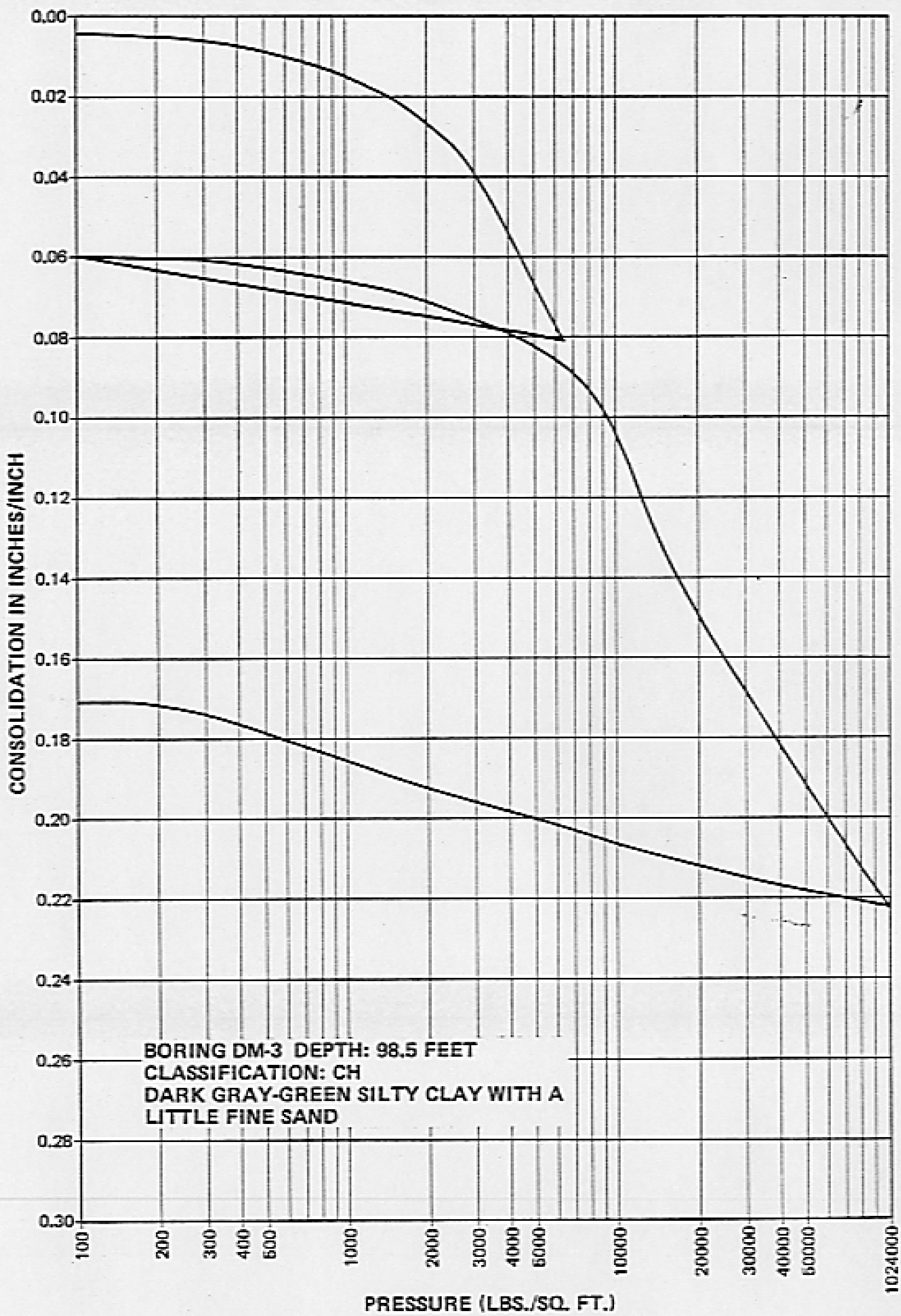


DEAD LOAD-PNEUMATIC CONSOLIDOMETER

In testing, the sample is rigidly confined laterally by the brass ring. Axial loads are transmitted to the ends of the sample by porous disks. The disks allow drainage of the loaded sample. The axial compression or expansion of the sample is measured by a micrometer dial indicator at appropriate time intervals after each load increment is applied. Each load is ordinarily twice the preceding load. The increments are selected to obtain consolidation data representing the field loading conditions for which the test is being performed. Each load increment is allowed to act over an interval of time dependent on the type and extent of the soil in the field.

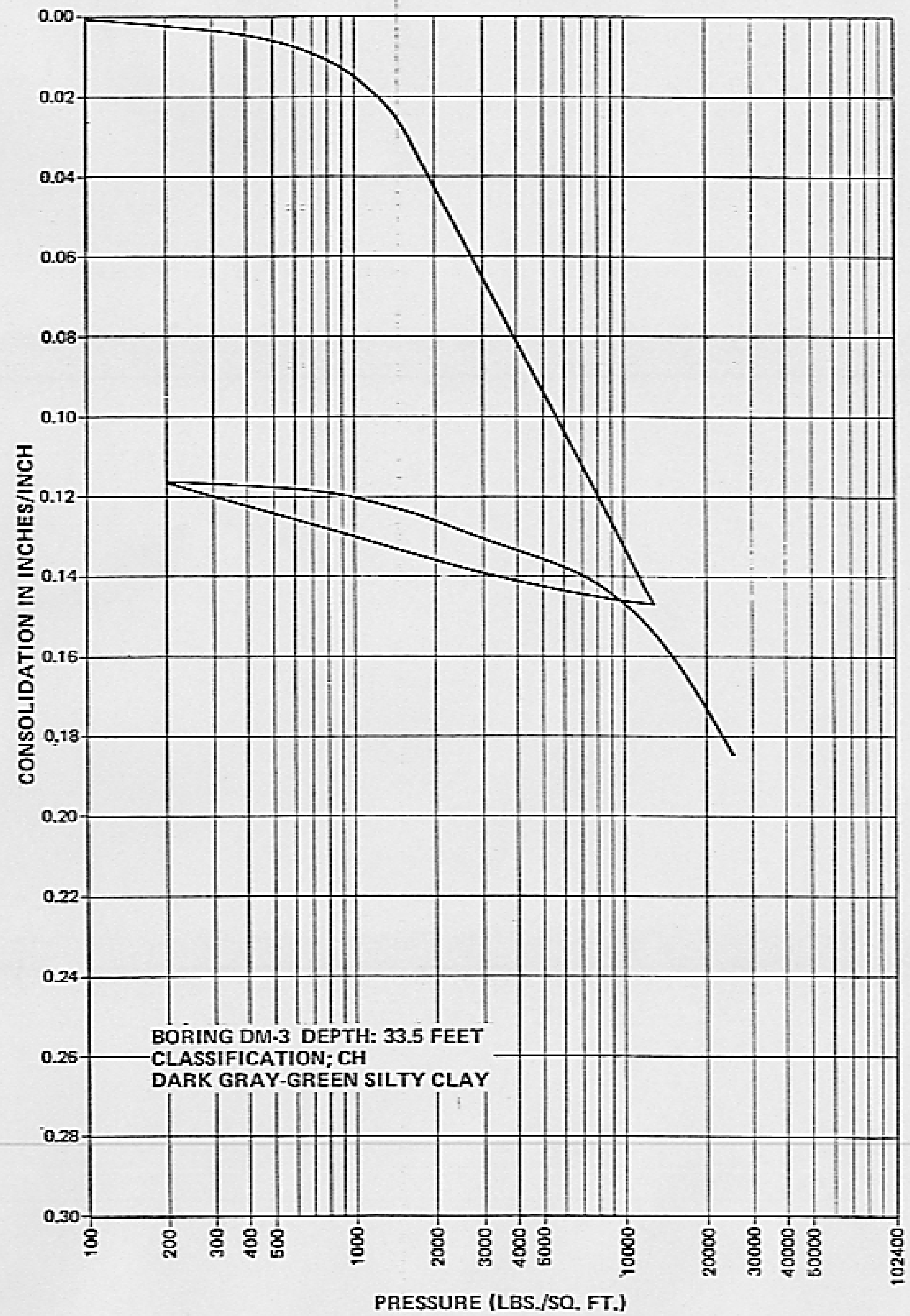


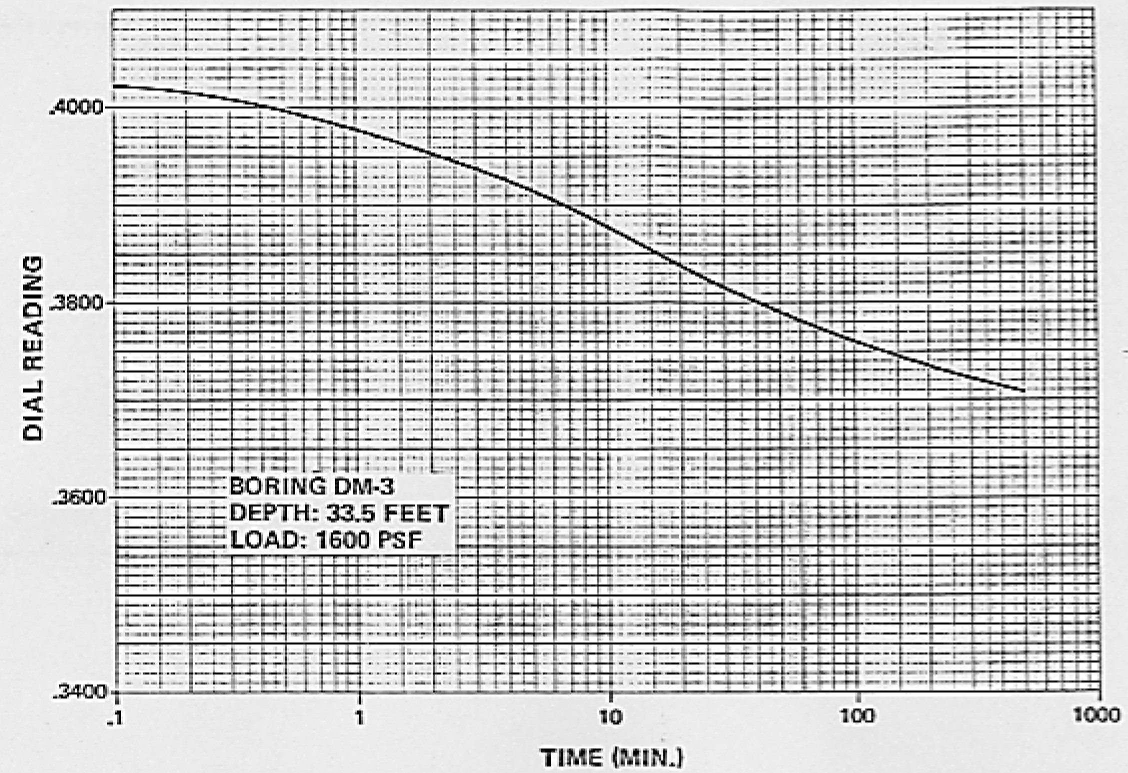
CONSOLIDATION TEST DATA

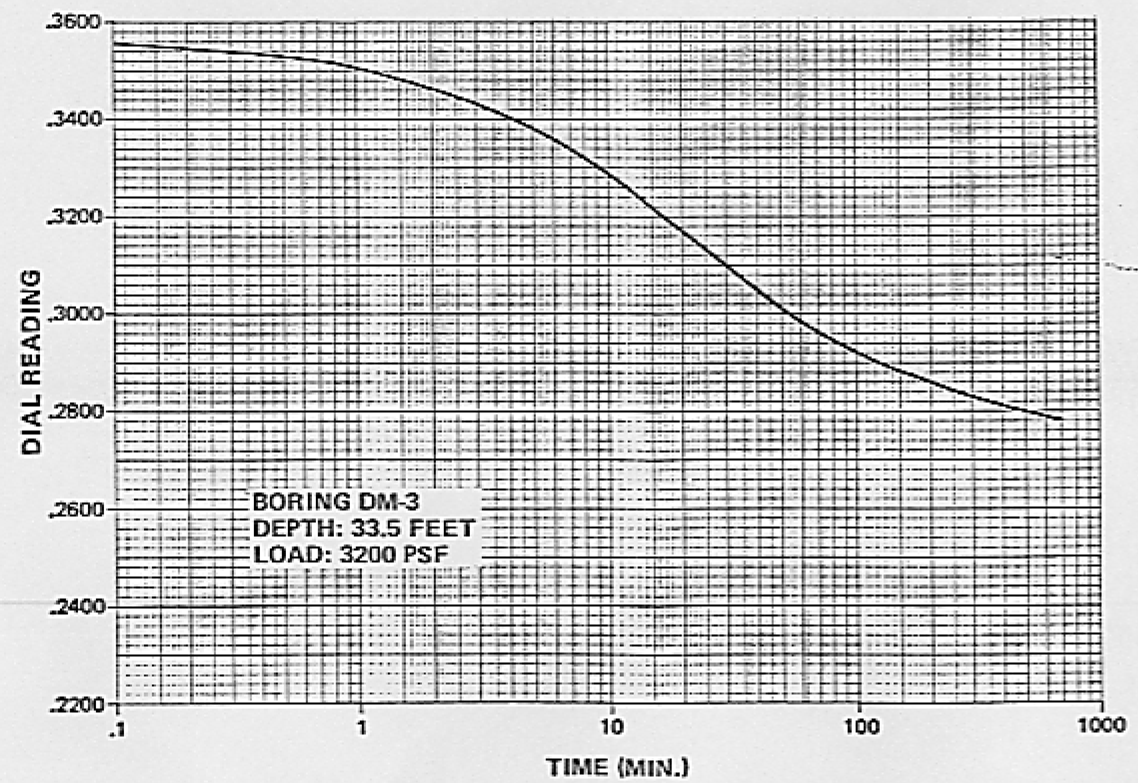


CONSOLIDATION TEST DATA

DAMES & MOORE



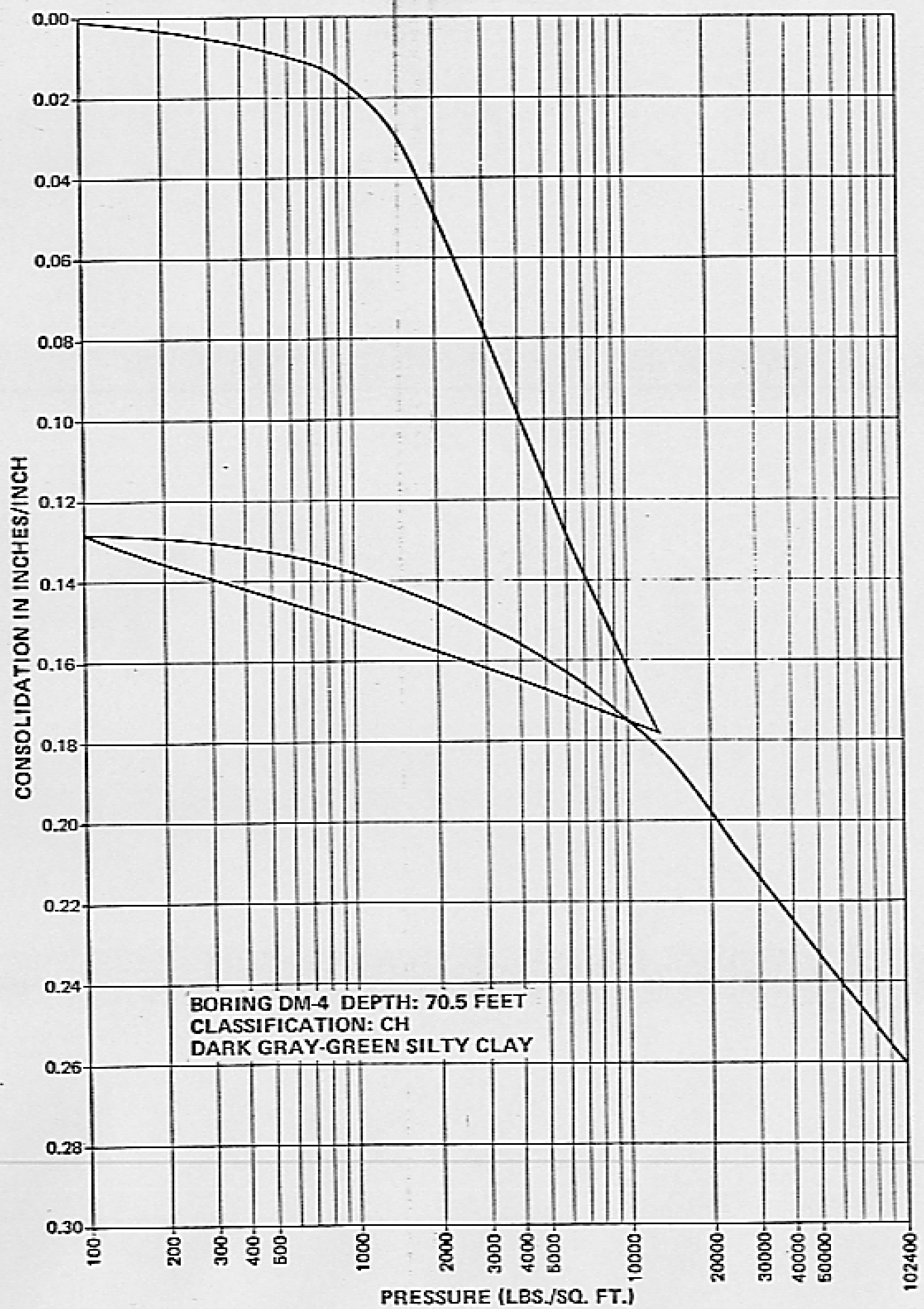


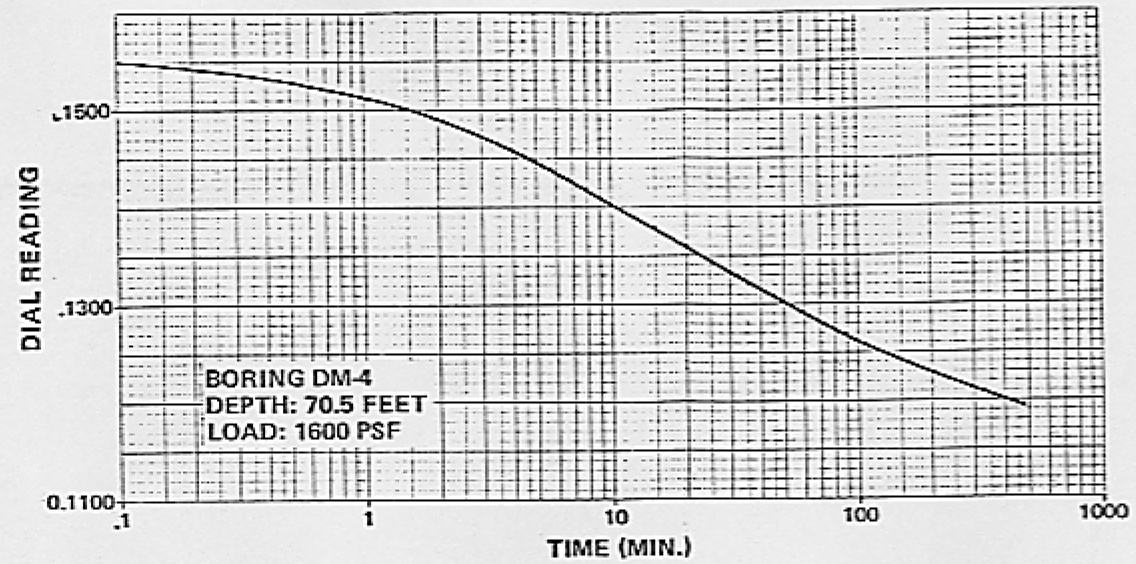


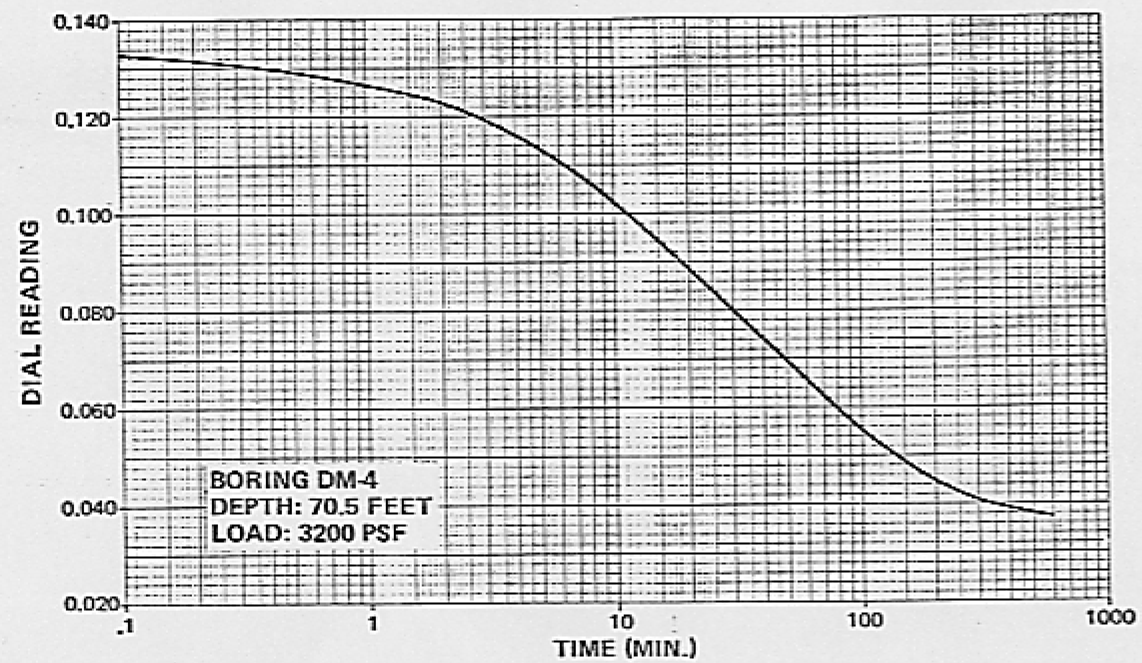
CONSOLIDATION TEST DATA

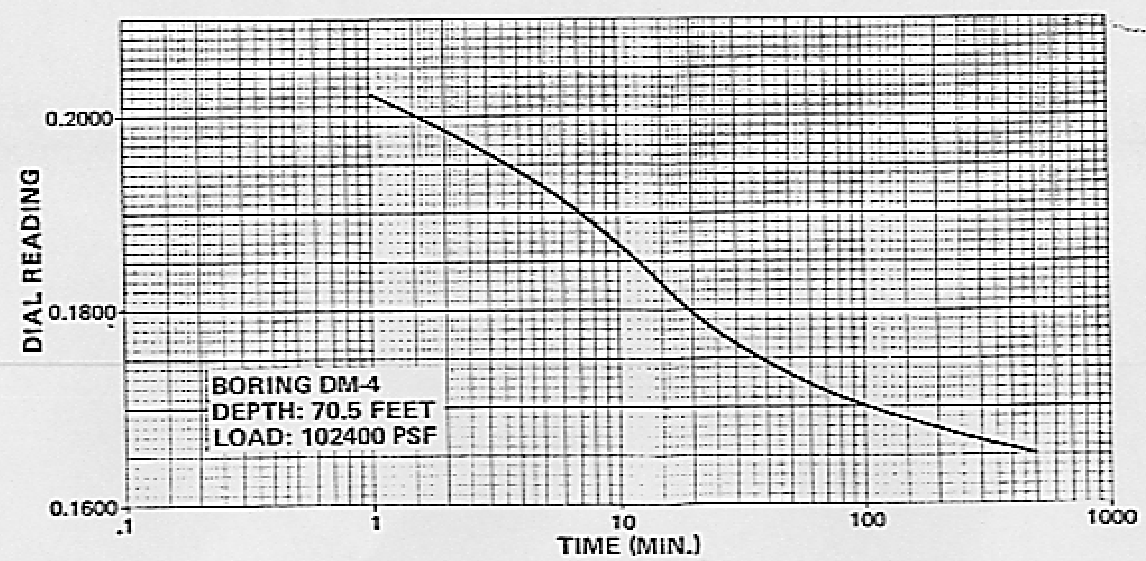
DAMES & MOORE

PLATE B-3







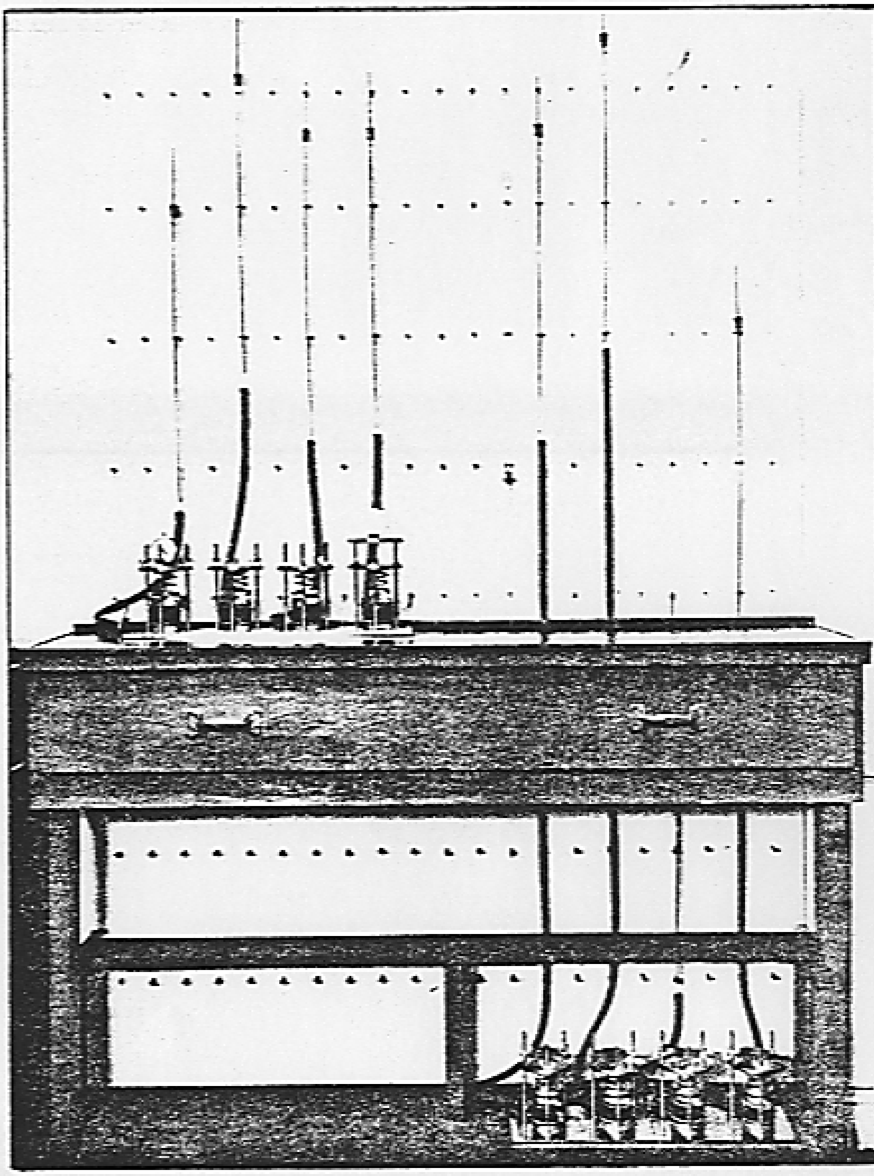


CONSOLIDATION TEST DATA

DAMES & MOORE

PLATE B-10

METHOD OF PERFORMING PERCOLATION TESTS



APPARATUS FOR PERFORMING PERCOLATIONS TESTS
Shows tests in progress on eight samples simultaneously.

The quantity and the velocity of flow of water which will escape through an earth structure or percolate through soil are dependent upon the permeability of the earth structure or soil. The permeability of soil has often been calculated by empirical formulas but is best determined by laboratory tests, especially in the case of compacted soils.

A one-inch length of the core sample is sealed in the percolation apparatus, placed under a confining load, or surcharge pressure, and subjected to the pressure of a known head of water. The percolation rate is computed from the measurements of the volume of water which flows through the sample in a series of time intervals. These rates are usually expressed as the velocity of flow in feet per year under a hydraulic gradient of one and at a temperature of 20 degrees Centigrade. The rate so expressed may be adjusted for any set of conditions involving the same soil by employing established physical laws. Generally, the percolation rate varies over a wide range at the beginning of the test and gradually approaches equilibrium as the test progresses.

During the performance of the test, continuous readings of the deflection of the sample are taken by means of micrometer dial gauges. 'he amount of compression or expansion, expressed as a percentage of the original length of the sample, is a valuable indication of the compression of the soil which will occur under the action of load or the expansion of the soil as saturation takes place.

APPENDIX C - PROPOSED FIELD INSTRUMENTATION

GENERAL

The anticipated performance of the subsoils in the proposed construction area is based mainly on engineering analyses and a review of measured settlements occuring under the existing dikes. These predictions are based on the results of a rather limited field exploration program and a small number of laboratory tests on undisturbed samples. We recommend that the actual performance of the subsoils during and after the development of the area be monitored by means of a field instrumentation program. The purpose of this Appendix is to briefly describe the proposed instrumentation program. When the decision to install the field instrumentation is made, additional details and specifications will be developed for the field instrumentation.

PROPOSED INSTRUMENTS

Three types of instruments are proposed in the area:

(1) Surface settlement plates
(2) Pore water pressure piezometers
(3) Lateral movement inclinometers

Surface Settlement Plates

The surface settlement plates are installed at the surface of the ground and are used to accurately measure the settlement occuring at various locations of the site. The settlement plates consist of a vertical steel plate Embedded in the ground, welded to a surface plate and including a vertical extension above the ground. Sometimes the vertical pipe Embedded in the ground is encased in concrete for additional protection. The vertical pipe above the ground can be extended through any fiLL placed above the plate. A typical settlement plate is shown schematically on Plate C-1. Periodic, accurate surveying is conducted to measure the elevation of the settlement plate. The surveys must be tied in to a stable benchmark which is not influenced by construction operations. Therefore the benchmark must be far enough away from the Dredge Disposal Area in order to be outside the settlement area imposed by the weight of the dredged material. The frequency of the settlement measurements depends on the rate of field placement and the rate of movement observed during the surveys.

Pore Water Pressure Piezometers

As the fill for preloading is placed, excess pore water pressure is expected to develop in the soft clay. As this pore water pressure dissipates, the clays will compress and will gain in shear strength. Measuring the rate of pore water dissipation can provide direct information as to the rate at which the clays are consolidating and additional shear strength is developing. Pore water pressure measurements are therefore very useful in predicting the behavior of the soft clay layers and the time rate of settlements. A typical pore water piezometer installation is shown schematically in Plate C-2. However, many types of instruments are available and the selection of the specific type of pore water pressure measuring gauge must be made after the decision to install these has been finalized.

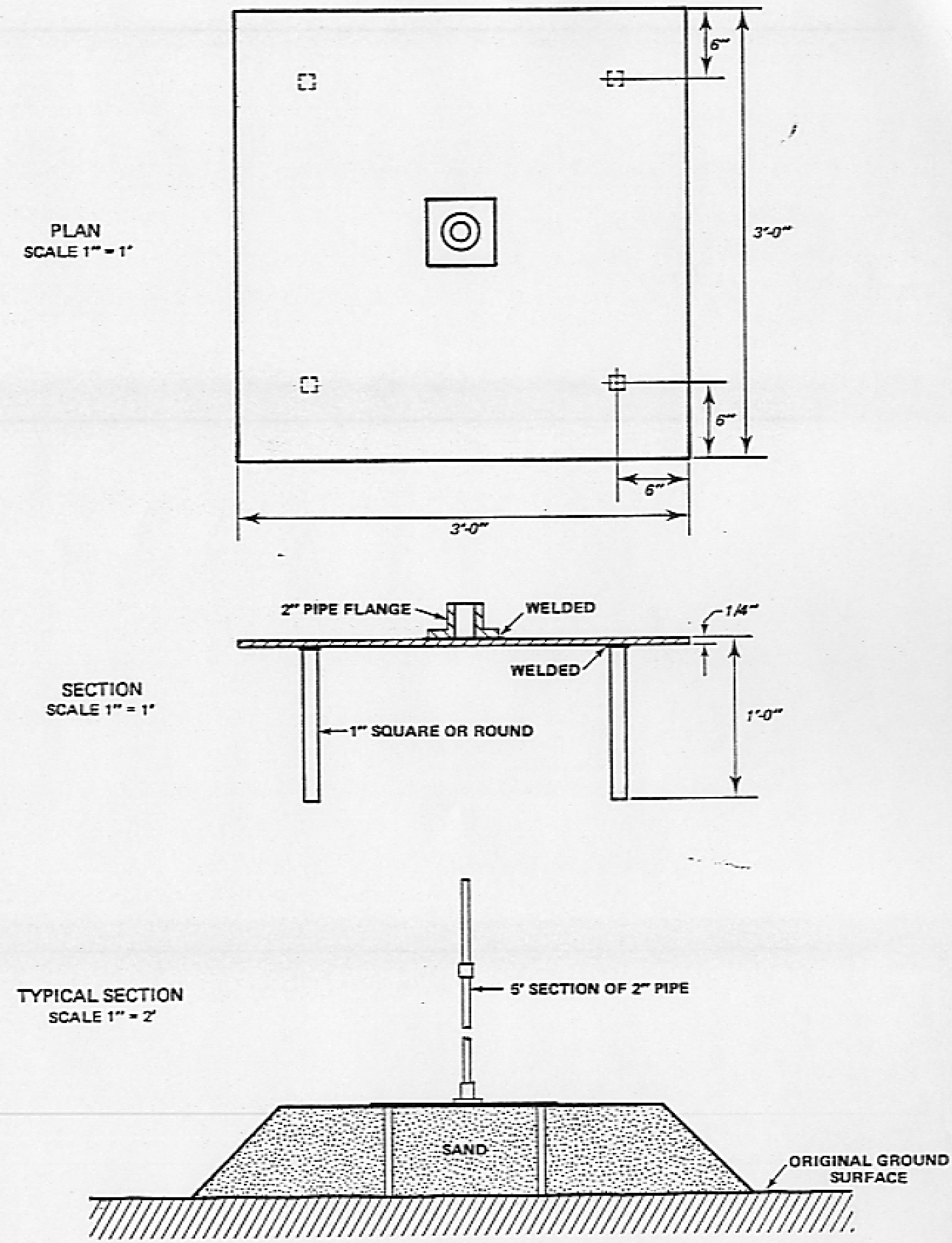
Slope Inclinometers

The soft clay below the dikes may undergo some lateral movements if the surcharge load is excessive. This would endanger the stability of the dikes and may cause slope failures. The measurements of lateral movements below the dikes would assist in detecting any incipient lateral movement before it becomes excessive. A typical inclinometer installation with the necessary measuring instrumentation is shown in page C-3. The selection of the exact type of inclinometer and its installation procedure can be developed after the decision to install these instruments has been made.

PLAN
SCALE 1" = 1'

SECTION
SCALE 1" = 1'

TYPICAL SECTION
SCALE 1" = 2'



SETTLEMENT PLATE

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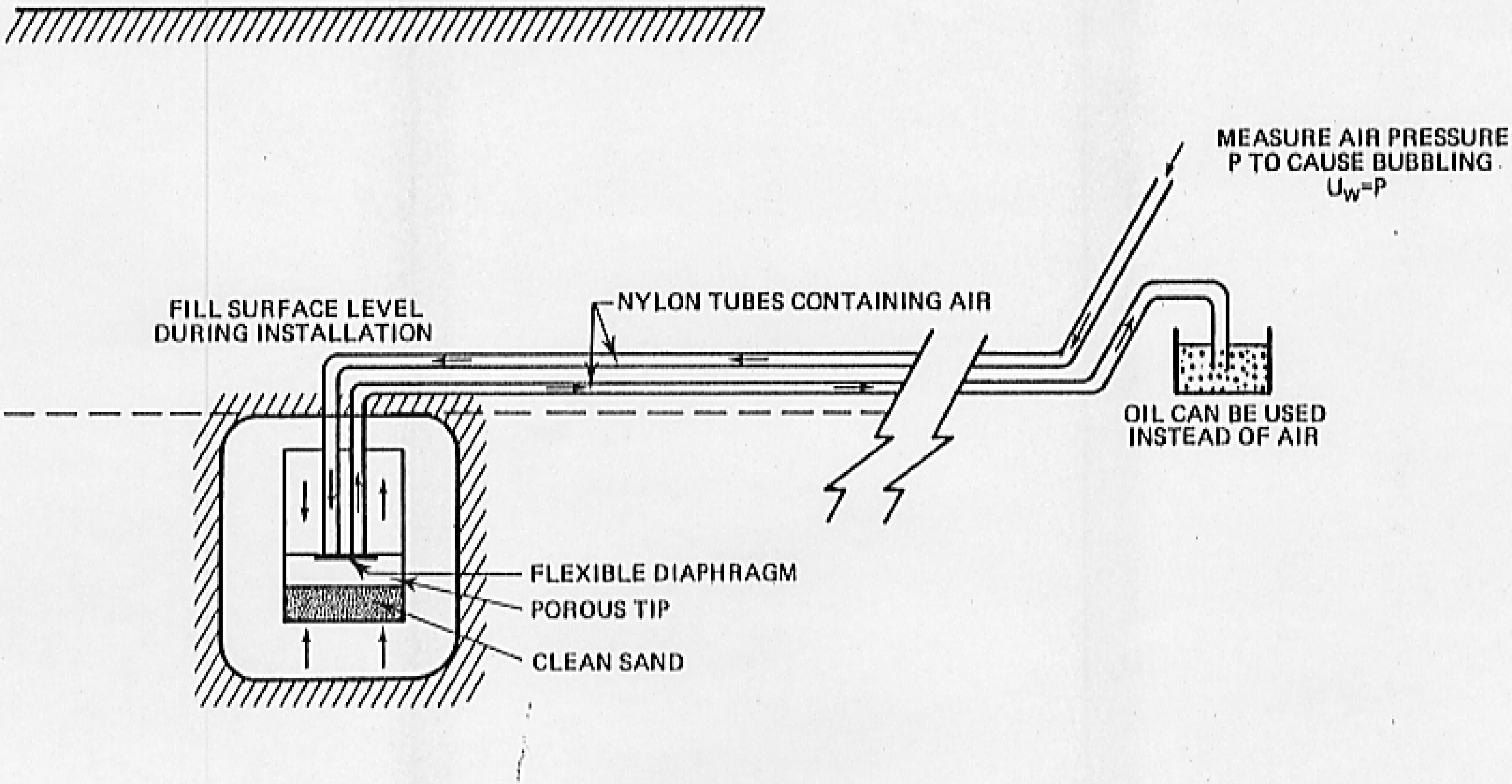
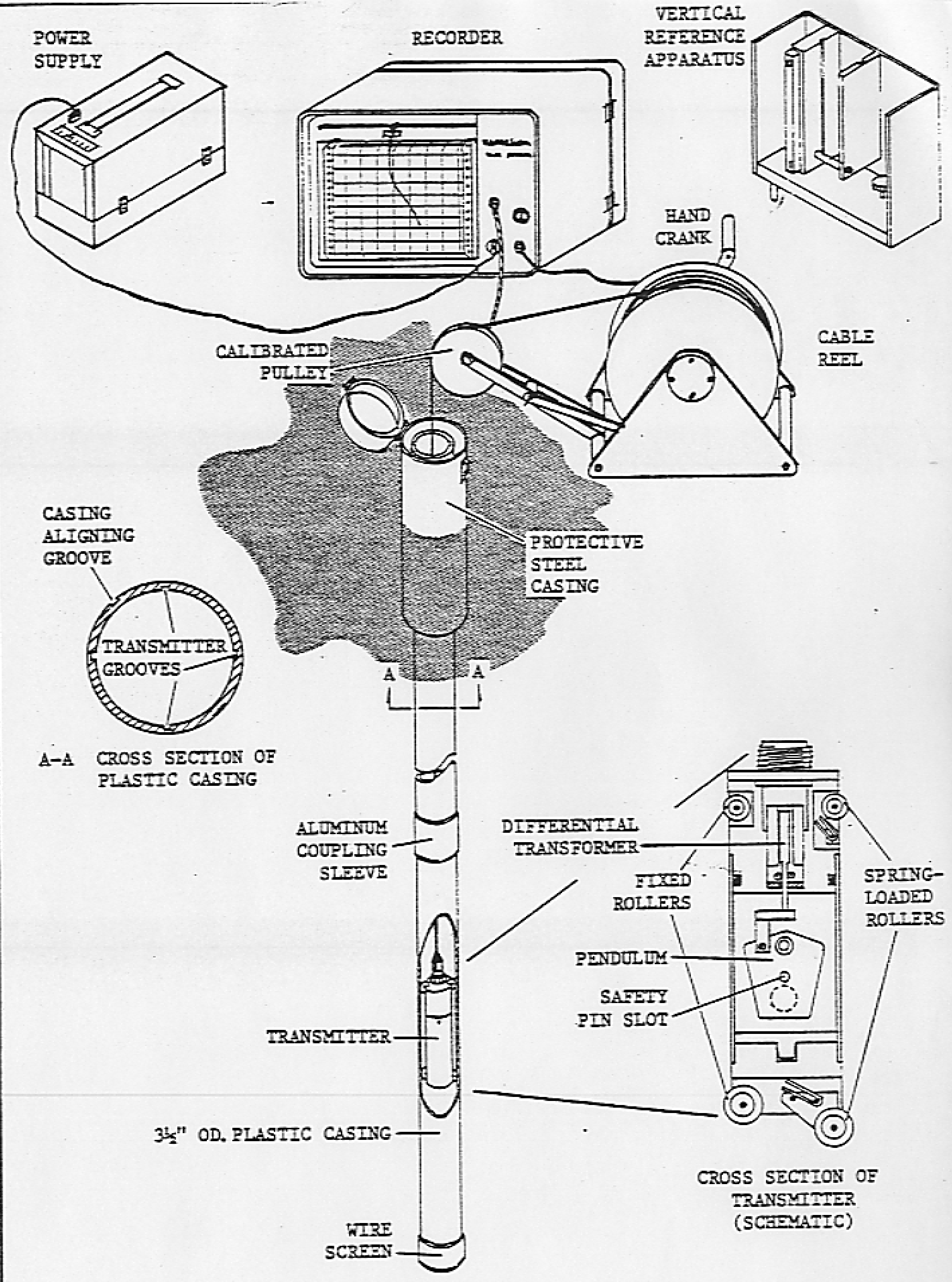
PLATE C-1



PNEUMATIC PIEZOMETER



SLOPE INCLINOMETER

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