GEOTECHNICAL INVESTIGATION
GOLD CREEK RECLAMATION PROJECT
JUNEAU, ALASKA
FOR THE
CITY AND BOROUGH OF JUNEAU

APRIL 1, 1982
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Dames & Moore

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April 1, 1982

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P.O. Box 2317
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Attention: Mr. George Davidson, P.E.

Gentlemen:

We are pleased to transmit herewith our final report entitled "Geotechnical Investigation, Gold Creek Reclamation Project, Juneau, Alaska," for the City and Borough of Juneau.

We provided you our preliminary conclusions and recommendations for review and comment in our February 23, 1982 draft report. Your comments and questions provided during our February 26, 1982 meeting have been incorporated in this report.

We appreciate the opportunity to provide these services and look forward to assisting you in the future. Should you have any questions, please call.

Yours very truly,

DAMES & MOORE

By J. Michael Blackwell

J. Michael Blackwell
Partner

JMB mb
10 copies submitted
cc: Sverdrup & Parcel (2)

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*Our preliminary recommendations for foundation support are provided in a supplementary letter report.

GEOTECHNICAL INVESTIGATION
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INTRODUCTION

This report presents the results of our geotechnical investigation at the site of the proposed Gold Creek Reclamation Project in Juneau, Alaska. The project area is illustrated relative to the City of Juneau on the Vicinity Map, Plate 1.

The site, which encompasses about 24 acres of tidelands, is bounded by Egan Drive on the north, the extension to West 8th Street on the east, and the subport pier on the west. Current site grades vary from about Elevation 10* to about Elevation -2. The project will involve filling portions of the site to about Elevation 25 using soils dredged from the Gastineau Channel. At the time of our investigation specific land use plans had not been identified and location/type of structures to be included in the development were not planned.

The potential sources of fill that have been considered during this investigation are located on the Project Area and Potential Borrow Source Location Map, Plate 2. The project site is illustrated on Plate 3, Gold Creek Reclamation Project Area.

SCOPE

The purpose of this geotechnical investigation is to provide recommendations for design and construction planning of containment dikes and the site fill. Our study has not addressed issues related to land use or specific foundation support requirements for future structures. Our investigation has included the following elements:

*Elevations in the text and appendix of this report refer to Mean Lower Low Water (MLLW).

- Gathering and review of existing geotechnical and environmental information pertaining to the site and adjacent regions. The results of this review were presented in our progress report of November 9, 1981.

- Field investigation
  -- Subsurface conditions at the site were explored by means of 11 borings.
  -- Several potential borrow sources were evaluated by data review and exploration; 10 borings, each about 20 feet deep were completed during this investigation.
  -- A bathymetric survey of the site and the area immediately offshore of the site was completed.

- Laboratory testing
  -- Conventional testing to evaluate the character of the soil at the site.
  -- Elutriate and turbidity tests on selected samples from potential borrow sources.

- Engineering analysis, with the objective of supporting conclusions and recommendations on the following topics:
  -- Slope stability
  -- Soil liquefaction potential
  -- Site dynamic response (expressed as site period)
  -- Relative suitability of candidate borrow sources
  -- Site settlement upon filling
  -- Site filling procedures
  -- Containment dike design and construction requirements
  -- Wave defense requirements
  -- Pile capacity for support of waterfront structures

Dames & Moore is also providing support to this project by undertaking an environmental evaluation. Fieldwork for this element of our services was begun during March 1982.

DESIGN AND CONSTRUCTION CONSIDERATIONS

The primary geotechnical and environmental issues that have been identified and that must be considered during design and construction are as follows:

1. Slope Stability. Inclination of submerged slopes, which extend from the project area into Gastineau Channel average about 2-1/2:1 (horizontal to vertical) but are steeper than 1:1 in some areas. The results of our study indicate that these slopes will be unstable during the design seismic event. Setback of the site fill from the top of existing slopes will be required in order to reduce the potential for loss of portions of the reclaimed area during an earthquake.

2. Liquefaction. Temporary loss of soil strength during an earthquake resulting in general surface settlement and differential building settlement may be caused by liquefaction. The soils underlying the project area are characterized by moderate density and strength and may be susceptible to this phenomenon.

3. Fill Placement. Construction of containment dikes will be required prior to filling in order to reduce turbidity and to retain dredge soils at the project area. Dredging and filling must be scheduled to avoid work during in- and outmigration of salmonids, especially if soils are dredged near active spawning streams. A short-term variance of Alaska water quality requirements may be solicited with the permit application to the Corps of Engineers. The long-term impact of dredging/fill placement on water quality is expected to be negligible.

Balancing the costs and risks related to the principal geotechnical issues (1 and 2 above) will be complex, requiring a reflection of the City's land use planning policies and economic factors. As plans for this project progress, we should be kept informed of geotechnically related decisions, and be given the opportunity to review and advise on these topics. The risks associated with site performance during earthquakes may be better defined by accomplishing a seismic evaluation and companion geologic study. We recommend these studies be implemented prior to final design in order to assess the issues of cost and risk.

PROJECT AREA AND POTENTIAL BORROW SOURCE CONDITIONS

PROJECT AREA

General

Conditions at the site of the Gold Creek Reclamation Project were evaluated by review of existing information, a site bathymetric survey, and subsurface exploration. Our findings follow. Results of the bathymetric survey and location of site borings are shown on Plate 3. A description of the field equipment and procedures used for the bathymetric survey are presented in Appendix A. Equipment used for site exploration and laboratory procedures, including logs of the site borings designated as "SB," are presented in Appendix B.

Surface Conditions

The site of the planned reclamation project is located on about 24 acres of tidelands along the outer margin of the Gold Creek delta. Beyond the landward margins of the site on the northwest, north, and east the delta has been filled previously, and the developed land lies about 10 to 15 feet above the presently-exposed delta. The seaward slopes of the existing fill are protected by riprap.

A fuel transfer dock extends about 650 feet offshore from Egan Drive near the east margin of the site. We understand that sand and gravel was occasionally extracted from near the mouth of Gold Creek as late as the early 1950s. Other than the existing dock and the reported gravel extraction, the site is essentially undeveloped and in its natural condition.

About half of the site is covered by a brown algal mat with mussels, barnacles, and other invertebrates. Surficial soils are fine to coarse sand with a variable gravel and cobble content. The coarser soils appear to be more prevalent near the north margin of the site.

Subsurface Conditions

Our knowledge of soil conditions at the site is based on the results of the 11 borings drilled during this investigation, supplemented by logs of borings drilled by the Alaska Department of Transportation and Public Facilities (DOTPF) along Egan Drive and at the new Gastineau Channel Bridge. The soils encountered in the borings consist generally of very loose to medium dense very fine to coarse sand with a variable silt, gravel and shell content. A stratum of soft to medium stiff silt was encountered in Boring SB-2-81 at the surface and at the location of SB-3-81 and SB-6-81 at depths on the order of 110 to 128 feet.

The soils identified during this investigation are typical of a delta for a fast-moving stream. The dominant soil type identified by the explorations at or near the project area consists of slightly silty to silty fine to medium sand. A greater percentage of coarse soil deposits including gravel, cobbles, and boulders appear to underlie the area near the site's north margin. This particle size variation is primarily the result of the decreasing energy gradient of Gold Creek where the stream empties into Gastineau Channel.

Based on subsurface conditions inferred from State of Alaska DOTPF explorations along the alignment of the new Gastineau Channel Bridge, we expect that the sand soils underlying the project area will extend to depths on the order of 150 to 170 feet below current site grade. A dense to very dense strata of glacial soil, typically referred to as glacial till, is anticipated to underlie the sand soils. For the purposes of this study, we have assumed that bedrock is present beneath the site at depths ranging from about 170 to 210 feet. These estimates are based on extrapolation of data developed by the DOTPF at the new Gastineau Channel Bridge site.

POTENTIAL BORROW SOURCES

General

Soil conditions at 10 potential borrow sources including submerged slopes adjacent to the site were evaluated by review of existing information and by 10 borings drilled at four of the sites. The approximate locations of explorations completed during this and previous investigations by others are shown on Plate 2. A description of each of the areas evaluated is presented in a following section. Logs of the borings completed during our site investigation are illustrated in the appendix and are designated "BB."

Soils which underlie the potential borrow sites can be categorized into three distinctly different units as follows:

Unit 1: Slightly silty to silty fine to coarse sand with some gravel and cobble layers. Unit 1 soils typically occur as deltaic deposits where streams discharge into Gastineau Channel.

Unit 2: Sandy silt, silty gravelly very fine to medium sand, and sandy gravel. Unit 2 deposits are represented by soils present within the limits of the intertidal zone and shallow reaches of Gastineau Channel. These soils are the result of stream deposition in the channel and subsequent transport and redeposition by tidal action.

Unit 3: Angular rock fragments and fine to coarse sand with angular gravel. Unit 3 soils are tailings from the Alaska-Juneau (AJ) and Alaska Gastineau mine sites.

CONCLUSIONS AND RECOMMENDATIONS

SLOPE STABILITY ANALYSES

General

The south periphery of the project area is bordered by submerged slopes with average inclinations of about 2-1/2:1; some areas are, however, steeper than 1:1. We have evaluated the stability of these slopes under dynamic (earthquake) conditions. The conventional pseudo-static method of computer analysis based on Bishop's simplified method was used to represent earthquake loading of the slopes by simulating the horizontal acceleration with horizontal static forces. Seismic criteria developed for design of the Gastineau Channel Bridge by the DOTPF were adopted for this study. A discussion of the seismic parameters and soil properties used during our analyses, are presented in Appendix C.

Safety Factors

The adequacy of a factor of safety obtained from slope stability analyses must be examined considering the following:

1. Potential effect of failure on loss of life, liability, facility damage, and operations.

2. Replacement/repair of damaged facilities and loss of use.

3. Sensitivity of the factor of safety with respect to the representativeness of parameters used in the analysis, including:
   (a) soil parameters
   (b) subsurface profiles
   (c) ground acceleration

4 The uncertainty of definition of seismic criteria based on seismic history, geologic structure, site response characteristics, attentuation of earthquake events, etc.

5. The method of analysis used, especially for the dynamic loading conditions where the ground motion and soil behavior are simulated by pseudostatic analyses.

Factors of safety against slope failure for the dynamic case were developed during our study. The safety factor for slope stability was computed using the modified Bishop method of stability analysis, which computes the safety factor by comparing the driving forces with the available soil shear resistance along a postulated failure plane. The computerized analysis program arrives at minimum safety factors for given input parameters which include soil unit weight, soil shear strength, soil profile, and slope geometry.

For most slope stability projects, safety factors on the order of 1.0 to 1.2 for pseudostatic analysis of temporary dynamic loading conditions are usually appropriate but are dependent to a large degree on the probability of occurrence of the design seismic event. It is our opinion that a minimum safety factor of about 1.05 for the dynamic case is appropriate to evaluate the effect of slope failure on the planned development.

Results of Slope Stability Analyses

Safety factors for several slope inclinations were evaluated using a ground acceleration based on the "operating earthquake" (Magnitude 8.5 at a distance of 90 miles which results in a horizontal ground acceleration "a" of 0.15 g) which has been described in a study by the DOTPF (Appendix C) as the highest magnitude earthquake which could occur during the design life of the new Gastineau Channel Bridge. The results of the DOTPF study indicate this event has a high probability of occurrence. A range of soil friction angles was input for each slope inclination for comparative analysis and to evaluate the change in safety factor as a function of soil strength. One slope inclination was analyzed using a ground acceleration based on the contingency earthquake (Magnitude 6.5 at a distance of 25 miles which results in a = 0.18 g). The DOTPF study concluded this event has a low probability of occurrence. The results of the analyses are presented as a family of curves, which are a function of friction angle and safety factor, on Plate 4, Results of Pseudo-Static Slope Stability Analyses.

It should be noted that conventional seismic risk evaluations define the operating earthquake as less severe but more probable than the contingency earthquake. Some confusion can occur when the operating earthquake is of higher magnitude but at further distance from a site than the contingency earthquake. The confusion is compounded when there is a very low probability of exceeding the magnitude of the operating earthquake. Because of its high magnitude and low probability of exceedance, the DOTPF operating earthquake chosen for design of the new bridge may be considered by some to represent a contingency level event. The implication is that the horizontal ground acceleration corresponding to the DOTPF operating earthquake and used for our slope stability analyses may be somewhat high. We expect, however, that other factors not introduced during the stability analyses, such as pore pressure increases and liquefaction potential of the soils, balance the apparent conservative values of ground acceleration.

It is our conclusion that the safety factor of slopes with inclinations greater than about 4:1 will be less than unity during the operating earthquake. This is based also on a range of soil strengths which we have evaluated from the results of laboratory tests and blow count data. A comparative analysis using the ground acceleration for the contingency earthquake indicates that safety factors for slopes inclined at about 5:1 should be close to unity for the same range of soil strength. Use of the higher ground acceleration is somewhat conservative, however, since the recurrence interval for the contingency earthquake is longer than that of the operating earthquake as defined by the DOTPF.

The stability analyses have been used to establish a fill boundary in order to limit the risk of damage should slope failure occur. We have selected a fill set back limit as illustrated on Plate 5, Recommended Site Fill and Construction Boundaries, based on failure of existing slopes to a stable 4:1 configuration (approximate) during the operating earthquake. We recommend that this fill limit be included in the design to reduce loss of and damage to facilities located near the south side of the site. We further recommend that a construction set back, as illustrated on Plate 5, be located at least 75 feet north of the preliminary fill limit. The intent of the construction set back is to establish a zone where no construction will be planned, with the possible exception of marine structures.

Slope failure will affect to some degree marine structures located near the top of the failure plane. Failure of slopes can be expected to exert lateral and vertical forces on piles and pier abutments which may exceed the structural capacity of the supporting elements. The magnitude of these loads and risk of structural failure will be dependent on the locations of the structures and the degree of earth movement which may accompany a seismic event.

Accordingly, we recommend that the slope inclination in the vicinity of future marine structures be reduced to less than 4:1 by means of dredging. We expect that dredging will be limited by equiment capabilities to water depths on the order of 40 feet. Soils dredged from the existing slopes may be utilized as general site fill to establish planned grades behind containment dikes which would be an initial part of the development. An illustration depicting envisioned dredge limits relative to existing slopes and required containment dikes is presented on Plate 6, Typical Section.

It is our opinion that the fill and construction set backs outlined above are appropriate considering the seismicity of the Juneau area and the character of the soils which underlie the site. It is important to note, and we emphasize that the methods of analyses, expected variation of soil parameters, and assumed behavior of the soil mass, introduce uncertainties regarding the performance of the site during a seismic event. Furthermore, our results and conclusions are based on the assumption that liquefaction of the site soils does not influence the mechanics of slope failure. If this assumption is not valid, then propagation of slope failure towards the center of the site could result in slopes much flatter than 5:1. A further discussion of the uncertainty that this phenomenon introduces is presented in a subsequent section.

The set back limits may be altered by accepting lower safety factors. However, a higher level of risk is inherent if the recommended boundaries are moved further south.

LIQUEFACTION POTENTIAL OF SITE SOILS

Liquefaction of sand deposits is the transformation of the soil mass from a solid state to a liquified condition as a result of increased pore water pressure. This transformation may be accompanied by loss of soil strength and horizontal and vertical movement of the soil mass, which could range from negligible amounts to many feet. The impact of liquefaction on structures may include minor settlement, lateral deflection, tilting or failure of structural elements as a result of reduced bearing capacity of the soils underlying the structure.

The phenomenon of liquefaction is generally associated with loose and saturated sand deposits. The factors which may affect the liquefaction potential of the soil deposit are as follows:

1. Grain size distribution of the soil deposit.
2. Initial relative density (D_r)
3. Magnitude of ground vibration
4. Location of drainage and dimensions of deposit
5. Magnitude and nature of superimposed loads
6. Soil structure which is dependent upon depositional environment
7. Duration of ground vibration
8. Previous stress history
9. Entrapped air

Results of Liquefaction Study

The empirical and analytical procedures used during our liquefaction study in addition to the graphic results of our analyses are presented in Appendix C.

It is our opinion that the potential for liquefaction of the site soils to depths on the order of 40 to 60 feet is moderate to high. The most susceptible deposits will be those soils with relative densities less than about 60 percent and with a mean effective grain size less than about 0.7 mm. Soils which underlie the project area near Egan Drive appear to be less susceptible to liquefaction due to the percentage of coarse sand and gravel. The results of our liquefaction study are based on state-of-the-art analytical procedures. However, these procedures do not include consideration of previous site stress history or a conclusion regarding the consequences and magnitude of settlements resulting from liquefaction.

Our conclusions regarding the depth of liquefaction have been evaluated using case history studies and the apparent increase in density (based on blow count data) between depths of about 40 to 60 feet. This increase in density could be influenced by many factors, including densification during previous earthquakes which may have preceded deposition of the overlying soils.

IMPLICATIONS OF STABILITY ANALYSES AND LIQUEFACTION STUDY

The issues which must be considered during an evaluation of these analyses include definition of the level of risk associated with a seismic event, limitation of the impact of that event on the development, and reduction of the potential for loss/damage as a result of earthquake loading. The level of risk can be associated with the design seismic event. Studies conducted by the DOTPF indicate that the operating earthquake has a high probability of occurrence for a 50-year project life.

A method for limiting the impact of a seismic event would include zoning of the project area for different levels of risk associated with liquefaction potential prior to siting facilities and structures. Although we have concluded that the potential for liquefaction is lower along the north side of the project area, field and laboratory data developed during this investigation are insufficient to delineate zones with respect to a specific degree of risk.

The combined effects of liquefaction and slope failure must be considered also. Current methods of analyses including finite element analysis cannot provide any degree of certainty of site performance for a combination of slope failure and liquefaction of adjacent level or near level ground. It is our opinion that liquefaction could contribute significantly to reduced safety factors of submerged slopes which border the project area. This phenomenon could result in general site settlements accompanied by failure of the slopes and progressive movement of the south portion of the site toward the center of Gastineau Channel.

We have recommended fill and construction limits as described previously. These limits will reduce the potential for damage which may be the result of slope failure, but do not include consideration of soil liquefaction potential. We have indicated that site zoning using available data is not practical. Therefore, we recommend that three alternative development schemes and the corresponding relative risk for each be considered. The alternatives for site development as we envision them are as follows:

Alternative Description	Relative Development Cost	Relative Risk
1. Construct containment dikes, fill the site, and accept the risk of liquefaction and slope failure associated with the design seismic event.	Low	High
2. Stabilize the soils underlying the containment dike, construct dikes, fill the site, and accept the risk of liquefaction where stabilization is not accomplished.	Low to Moderate	Moderate to High
3. Construct dikes, fill the site, and stabilize all areas of potential liquefaction to 50-foot depths.	High	Low to Moderate

It is our opinion that the second alternative represents the best balance between cost and seismic hazard. This alternative, if implemented, should reduce the potential for catastrophic site failure resulting from the combined effects of slope failure and liquefaction. However, the risk that liquefaction of the remaining unstabilized soils will occur must be accepted. We expect that settlement of unstabilized potentially liquefiable soils will occur but that the risk of translation of the site toward Gastineau Channel will be reduced.

SOIL IMPROVEMENT

Assuming partial stabilization is the selected alternative, we conclude that stabilization of the site soils by one or more of several proprietary methods currently in use should be implemented to reduce the potential for liquefaction beneath the recommended containment dikes along the south side of the site and to improve the performance of soils near the top of submerged slopes. Stabilization of soils underlying other portions of the project area will depend upon the purpose of the development and type of structure. In general, we expect that stabilization will be appropriate beneath major structures, such as multi-story buildings. Low-level wood frame structures may not require stabilization.

The soil stabilization procedure which may be most effective for a specific site is dependent upon characteristics of the soils, effect of the procedure on existing facilities, topographic features, and cost. Soils underlying the project area consist of relatively clean, fine to coarse sand and very fine to medium sand deposits. Several soil improvement procedures could be implemented for these conditions, including blasting, vibratory probe, and dynamic consolidation. The principal for each procedure is to initiate localized liquefaction by imparting energy into the soil mass. Blasting and dynamic consolidation represent the lowest cost procedures. However, quality control may be difficult. Blasting has the added disadvantage that near-surface soils cannot be densified.

do limited dyn consolidation trials.

The proprietary methods developed by Terra-Probe and Vibroflotation are approaches which also may be appropriate for use at the Gold Creek reclamation site. Each procedure involves insertion of a pipe pile section or probe by means of vibration and water jetting at various intervals across the site to increase the relative density of the soils. Backfilling during this procedure is required as a result of settlements which occur at each probe location. Spacing of probes is typically on the order of 6 to 8 feet, depending on the required improvement and specified final soil relative density. The following tabulation adapted from Mitchell (1981) summarizes the methods and relative cost of each procedure.

Method	Effective Treatment Depth (feet)	Advantages, Limitations	Relative Cost
Blasting	More than 100	Rapid, inexpensive; final density varies, no near-surface improvement, dangerous	Low
Dynamic Consolidation	100+	Good improvement, reasonable uniformity	Low
Terra-Probe	10 to 60	Relative densities up to 80 percent, ineffective in some sands	Moderate
Vibroflotation	100*	High relative densities, good uniformity	Moderate

*Equipment for improvement to 60-foot depths available in the United States only.

The improvement procedure selected will require additional fill to raise site grades during and after stabilization due to surface subsidence related to soil densification. The amount of additional soil required will depend upon the method implemented. We estimate that about 0.2 to 0.4 cubic yards of additional fill will be required for each square foot of stabilized area. This estimate will vary depending on the method used, effectiveness of the procedure, and initial soil densities.

(0.4)(27) = 10.8' settlement?

SITE FILLING AND DEVELOPMENT

General

We have identified four potential borrow source areas which appear to be viable from the standpoint of proximity to the site and cost in place. Prior to filling, construction of containment dikes using select borrow from onshore sources will be necessary in order to retain the dredge soils and to reduce turbidity of the effluent before return into Gastineau Channel.

A phased site filling program may be required depending on available funding. This will require division of the project area into two or more sections, each of which will comprise a major cell to be filled during successive phases. The number of major cells will depend, again, on budget allocations for the project.

Compaction of the dredged soils may be accomplished after filling and drainage of the fill mass. However, segregation of fine-grained particles from the coarse fraction of the dredge fill may create zones of weak, compressible soil deposits depending on cell geometry and method of fill placement. Those areas are expected to drain and gain strength more slowly than other portions of the dredge fill, and may require preparation procedures, such as preloading, different than for the coarse fraction dredge spoil.

Phased Containment Dike Construction

We recommend that the project area be divided into at least two cells (that may be divided into smaller cells to accommodate various phases of site filling). The two main cells would be separated by a channel which will be required for Gold Creek.

The main cells should be formed by construction of a containment dike constructed within the limits of the fill setback, as noted on Plate 5, and parallel to the new channel formed for Gold Creek. These peripheral containment dikes should be constructed with select granular fill imported from on-shore sources. The fill should consist of a sand and gravel or fine angular rock mixture with less than 5 percent passing the No. 100 sieve to facilitate placement during periods of high tides. The select fill will provide a dike section with sufficient strength to retain the lower strength dredged soils. Soil or rock fill placed during containment dike construction may be placed without compactive effort, if the materials conform to the recommended gradation.

If angular rock fill is used for dike construction, increased turbidity resulting from leaching of dredge effluent through the dike may result. Should the coarse rock fill be more economical than a sand/gravel mixture, the gradation of the rock should be carefully examined to evaluate the possible increase of turbidity and to assess alternative procedures to limit leaching of fine particles through the dike section.

The outer dike slopes should be inclined no steeper than 1.5:1 and 2:1 for angular rock fill and sand/gravel mixtures, respectively. Soils placed to achieve steeper inclinations will require compaction and may be susceptible to instability. The inner dike slopes may be constructed as steep as is practical. The dike crest width should be planned at about 10 feet to provide access for equipment.

Slope Protection Requirements

The outboard containment dike slopes will be subjected to wind- and vessel-generated wave forces which will erode the dike. The lower portion of the dike slopes may be exposed to the action of breaking waves depending upon embankment toe elevation, inclination of near-shore slopes, and tide levels. If dredging is accomplished in the vicinity of marine structures as recommended previously then the less critical condition of a nonbreaking wave would control design of slope protection in those areas. Where dredging is not accomplished, the breaking wave condition should be considered during design.

We have evaluated slope protection requirements for the outboard dike slopes using wind and wave data developed by the DOTPF for the new Gastineau Channel Bridge. Four alternatives have been selected based on our analyses. The approach for slope protection which is chosen will depend upon economic considerations related to initial construction costs, long-term maintenance costs, and, possibly, land value. The alternatives are tabulated below:

	Slope Inclination	Riprap Slope Protection*	Estimated Level of Risk, Percent Damage
Alternative 1:	2:1	Class II	Low to moderate; less than 10%
Alternative 2:	1.5:1	Class II	Moderate to very high greater than 20%
Alternative 3:	1.5:1	Class III,† bottom 4 feet Class II upper slope	Low; less than 5%
Alternative 4:	2:1 bottom 4 feet 1.5:1 upper slope	Class III† Class II	Low; less than 5%

*Based on State of Alaska DOTPF specification 611-2.01.

†Class III riprap not required if dredging adjacent to containment dikes is accomplished.

We do not advocate implementing Alternative 2. It is our opinion that the cost associated with maintenance/repair will be greater than the savings realized by using the lighter weight Class II rock.

We recommend that Class II and Class III riprap slope protection consist of a rock cover about 2-1/2 and 3-1/2 feet thick, respectively. Provision for protection of the rock cover at the end of the containment dikes should be included in the design to reduce the potential for outflanking of the slope protection.

The riprap should be constructed on a synthetic fabric (geotextile) in order to reduce the potential for infiltration of embankment soils through the riprap as a result of wave action and differential water pressures. A graded rock filter is not recommended since Class I riprap in addition to fabric would be required.

Toe protection should be included in the design of the selected alternative. We recommend that the riprap be keyed into existing soil at the toe of the containment dike in order to reduce the potential for undercutting. The key should extend at least 3 feet below the dike toe and have a minimum bottom dimension of 3 feet with side slopes inclined at 1:1 as illustrated on Plate 7, Suggested Toe Protection Detail.

We estimate that runup of nonbreaking waves for slopes inclined at 1.5:1 or 2:1 will be in the range of 2 to 2-1/2 feet assuming that slope protection consists of graded rock as recommended. The higher value of runup applies to the steeper slope inclination.

BORROW SOURCE EVALUATION

Dredge Fill

We have identified the anticipated soil conditions at 10 potential borrow sources for general site fill. Review of existing data obtained during our study, in addition to the results of our borings, have been used as the bases for our conclusions regarding the suitability of these soils. Approximate cost information pertaining to distance of source from the project area, type of soil available, and equipment required to dredge/transport soil has been provided by Manson Construction and Engineering Company. Elutriate testing of mine tailing deposits indicate that total volatile solids, chemical oxygen demand (COD), and heavy metals do not exceed limits established by the EPA. These results and other test data are presented in Appendix B.

The results of our evaluation indicate the soils generally available for use as fill consist of fine to coarse sand with a silt content in the range of 5 to 20 percent and a variable gravel content. The soils encountered have been classified into three units as described in a previous section titled "Site Conditions, Potential Borrow Sources."

We have considered soil gradation, distance from the project area, types of equipment required, and available quantity, although the latter issue has been essentially a judgmental determination. The Unit 1 and Unit 3 soils are the most suitable because of their granular characteristics and low silt content. These soils will drain more rapidly after placement than the Unit 2 soils.

The methods and equipment required to dredge and transport soils from the borrow sites to the project area are generally dependent on the types of soils and distance from the fill site. A suction dredge with a pipeline to transport the dredge slurry would be one means of dredging and transport. The suction dredge is limited to about 1-1/4 miles of discharge line with the dredge pump, but is capable of pumping for distances up to about 2 miles with a booster pump. A submerged discharge line would be required with the suction dredge to permit passage of boat traffic. The use of a hopper dredge would be appropriate for areas further than about 2 miles from the site. The hopper dredge is loaded at the borrow source, then moved by tugboat to the fill area for offloading. During dredging at the borrow source, soils placed in the hopper tend to segregate, resulting in discharge of water with suspended fine soils over the side while the coarser sediments settle to the bottom of the hopper. The result is increased turbidity at the dredge site but a lower fines content in the dredge fill.

Our evaluation has resulted in a relative engineering ranking for each of the potential borrow sources in order to delineate the more suitable areas. The qualitative rankings are as follows:

Rank*	Description
1	Most suitable
2	Some constraints
3	Least suitable
4	Impractical

*These rankings do not consider the implications of seasonal environmental constraints.

The results of our study including the ranking for each site are presented in Table 1.

Potential Armor Stone Sources

We made brief visits to several potential sources of armor stone, making an initial assessment of stone availability and suitability. From this reconnaissance, we have identified two existing quarries (inactive at the time of our visit) that merit consideration as sources. A final decision and evaluation should be made after project stone requirements are established -- in terms of stone size and quantities. An assessment of ownership and possible royalty costs has not been done.

The estimates of produced stone size given below are to be considered preliminary. These estimates are based on limited exposures, and do not account for variations in blasting techniques.

Fish Creek: On a spur, about 1/2 miles west of Eagle Crest Road. Haul distance about 5.9 miles. Rock type is greenstone with irregular lenses of slate and argillite, 1 to 4 inches thick. The effective joint spacing is about 3 to 4 feet (maximum) and 1/2 to 1 foot average. Occasional shattered zones were noted. Ten to 40 percent of this rock could probably be produced in the 1- to 2-ton size range.

The face is located on the side of a 30-foot high hill, oval in plan and about 100 feet wide and 300 feet long. No other development is in the area.

Bonnie Brae: About 1,000 feet uphill from North Douglas Highway, 3.3 miles north of the Douglas Bridge. A large face is not open, but based on existing exposures, the rock type is dominately schistose-greenstone. Effective joint spacing 2 to 3 feet (maximum), 1/2 foot (average). Five to 15 percent of this rock would probably be produced in the 1- to 2-ton size range.

There are no other developments in the area, but a residential subdivision is planned nearby.

Dredge Slurry Turbidity Control

Proper design and operation of overflow structures will be essential to avoid degrading water quality excessively. The State of Alaska water quality standards, which are administered by the Department of Environmental Conservation (DEC), are dependent on timing restrictions related to fish migration, the type of dredging equipment, and fill

TABLE 1

(A) Potential Source Name and Location	(B) Haul Distance, Center of Source-Center of Site (miles)	(C) Soil Characteristics and Unit	(D) Estimated Silt Content (percent)	(E) Esimated Quantity (cubic yards)	(F) Environmental and/or Seasonal Constraints	(G) Dredging, Transportation and Placement Methods	(H) Estimated Cost[(a)] (per cubic yard)	(I) Comments and Rank
1. Douglas Boat Harbor	1.8	Unit II silty fine to medium sand and sandy silt	less than 10 for sand soils	80,000 to 120,000	turbidity and siltation of adjacent benthic populations due to poor flushing (not certain of flushing characteristics)	Suction Dredge Hopper Dredge	$3.50 + $3.30	Congested area, submerged line with booster pump Rank = 4
2. Lawson Creek Delta	0.7 to 0.8	Unit I fine to coarse sand with variable silt and gravel content	less than 10	200,000 to 240,000	juvenile pink salmon outmigration in spring/early summer juvenile salmon feeding/refuge habitat waterfowl feeding/resting	Suction Dredge Hopper Dredge	$3.50 $3.30	Submerged line; quantity based on 10-foot dredge depth Rank = 1
3. Eagle Creek Delta	1.6	Unit I silty fine to coarse sand and gravel	less than 10	200,000 to 240,000	juvenile salmon outmigration in spring/early summer juvenile salmon feeding/refuge habitat waterfowl feeding/resting	Suction Dredge Hopper Dredge	$3.50 + $3.30	Submerged line with booster pump works with high tides only; quantity based on 10-foot dredge depth Rank = 2
4. Gastineau Channel between Eagle and Falls Creeks on west side of channel	2.1	Unit II sandy silt and silty fine sand	10 to 60+	1.2 to 1.3 million	spring/summer feeding and refuge for outmigrating juvenile salmonids	Suction Dredge Hopper Dredge	$3.50 + $3.50	Hopper dredge, works with high tides only; high turbidity; quantity based on 10-foot dredge depth; includes intertidal zone Rank = 3
5. Falls Creek Delta	2.7	Unit I sandy fine to coarse gravel with variable silt content	less than 10	160,000 to 200,000	juvenile salmon outmigration in spring/early summer feeding/refuge for outmigrating salmonids waterfowl feeding/resting	Hopper Dredge	$3.30	Hopper dredge works with high tides only; quantity based on 10-foot dredge depth; Rank = 2
6. Gastineau Channel Salmon Creek to Swietzer Creek	2.9 to 4.8	Unit II silty fine sand with some medium sand and zones of sandy silt	10 to 15 percent for sand soils	360,000 to 400,000	outmigration pathway during spring/early summer for juvenile salmon	Hopper Dredge	$3.30	Hopper dredge, work with high tides only; high turbidity; quantity based on Plan 2 as described in COE feasibility report dated 1977 Rank = 3

(a) Cost could increase as a result of royalties for privately owned borrow.

TABLE 1

(A) Potential Source Name and Location	(B) Haul Distance, Center of Source—Center of Site (miles)	(C) Soil Characteristics and Unit	(D) Estimated Silt Content (percent)	(E) Esimated Quantity (cubic yards)	(F) Environmental and/or Seasonal Constraints	(G) Dredging, Transportation and Placement Methods	(H) Estimated Cost[a] (per cubic yard)	(I) Comments and Rank
7. Gastineau Channel Swietzer Creek to Fritz Cove	4.8 to 8.0	Unit II silty fine sand with some medium sand and zones of sandy silt	10 to 15 percent for sand soils	480,000 to 500,000	outmigration pathway during spring/early summer for juvenile salmon	No		Quantity based on 5-foot dredge depth; includes intertidal zone Rank = 4
8. AJ Mine Tailings	1.5	Unit III slightly silty fine to coarse sand with some gravel	less than 10 percent	Unknown	turbidity effects during juvenile salmon outmigration	Suction Dredge Hopper Dredge	$3.25 $3.25	Suction dredge with submerged line and booster pump; quantity depends on owners willingness to supply borrow Rank = 1
9. Alaska-Gastineau Mine Tailings at Thane (Sheep Creek)	4.2	Unit III slightly silty fine to medium sand	less than 10 percent	Unknown	turbidity effects during juvenile salmon outmigration	Hopper Dredge	$3.30	Hopper dredge; quantity depends on owners willingness to supply borrow Rank = 1
10. Submerged slopes at Project Site	--	Unit I fine to coarse sand with some gravel	10 to 15 percent	200,000 to 250,000	same as for filling at site	Suction Dredge	less than $3.00/c.y.	Quantity should be based on dredging to Elevation 40 with 4:1 slope inclination Rank = 1

placement procedures. The criteria which will most likely apply to the Gold Creek Reclamation Project dredge fill program is turbidity. The state regulations indicate that water quality cannot exceed 25 nephlometric turbidity units (NTU) above the natural background water turbidity level. This criterion is difficult, if not impossible, to meet for most projects. The state usually grants a short-term variance for projects which are accomplished in accordance with other stipulations. However, reduction of dredge slurry turbidity will be an essential element of the design and site filling phases.

Based on the results of laboratory analyses on soils obtained at potential borrow sources, we expect that water quality in the cells will be in the range of 80 to 120 NTU after about 60 minutes. Some variations should be expected due to mixing.

Location and design of the overflow structure will be important to reduce turbidity of the effluent prior to discharge from the cell. Various forms of adjustable weirs have been used successfully for control of discharge on other projects. Synthetic fabric curtains may be implemented also to limit the discharge of suspended fine particles. If weirs are used, we recommend a flow height of 2 inches.

The overflow structure should be designed assuming a dredge flow velocity in the range of 12 to 18 feet per second and 22 hours of dredge operation per day. The volume of dredge effluent, then, will be dependent on the diameter of the discharge line. Specifics of the overflow structure design may be determined by the contractor at the time of construction.

Some flow of water through the containment dikes will occur as filling begins. However, the granular containment dike should act as a filter and should effectively reduce the turbidity of the slurry water. Flow rate through the dikes will decrease as the voids within the dike structure become clogged with fines.

We recommend that the dredge line discharge be located so that the coarse fraction of the soils dredged are deposited near the peripheral containment dikes. As these coarse soils are deposited, they should be graded into place with a small cat behind the dikes in order to improve dike stability.

FILL AND SUBGRADE SETTLEMENT

Settlement of the dredge fill surface will result both from consolidation of the underlying natural soil and subsidence within the fill. We estimate that settlement due to consolidation of the natural soil below the dredge fill will be in the range of approximately 3 to 15 inches. Settlement beneath the containment dikes should be less than about 6 inches. Most of this settlement should occur fairly rapidly, probably within 1 to 3 weeks after completion of filling.

We estimate that soils of Unit 1 and 3 will have a bulking factor of about 5 to 10 percent. This implies that the average density of the soils as they exist will be reduced by a factor of approximately 1.05 to 1.1. This will result in a volume of in place dredge fill greater than the extracted volume of soil. The filled volume will decrease with time as the fill consolidates under its own weight. This volume change will result in an average areal settlement of the dredge fill in the range of 6 to 12 inches for 15 to 25 feet of dredge fill. Greater settlements, on the order of 1 to 2 feet, may occur where significant deposits of fine grain soils are present subsequent to filling.

The loads imposed by the fill mass may induce settlement of the pile foundations which support the Standard Oil dock. Filling around or near the existing structure should not be accomplished during off-loading of petroleum products. The structure and piping should be surveyed and inspected prior to, during, and after fill placement in order to assess the impact of filling and to identify any releveling of piping which may be required as a result of settlement.

The following plates and appendices are attached and complete this report.

Plate 1 - Vicinity Map

Plate 2 - Project Area and Potential Borrow Source Location Map

Plate 3 - Gold Creek Reclamation Project Area and Results of Bathymetric Survey

Plate 4 - Results of Pseudo-Static Slope Stability Analyses

Plate 5 - Recommended Site Fill and Construction Boundaries

Plate 6 - Suggested Toe Protection Detail

Plate 7 - Typical Section

Our preliminary recommendations for foundation support are provided in a supplementary letter report.

STATE OF ALASKA
49TH
JAMES B. HARAKAS
NO. CE4784
REGISTERED PROFESSIONAL ENGINEER

Respectfully submitted,

DAMES & MOORE

By

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And

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And

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Project Engineer

6842-003-20
April 1, 1982

REFERENCES

Alaska Department of Highways, Engineering Geology Branch, 1977. Foundation report, Gastineau Channel Bridge Number 740.

_____, Materials Section, 1964. Centerline soils and materials sites investigation, Juneau Outer Drive, phase two.

_____, Materials Section, 1965. Foundation investigation Gold Creek, Juneau District.

_____, Materials Section, Southeastern District, 1972. Supplementary report, materials sites investigations, Southeastern District, Glacier Expressway, Norway Point - Airport. Alaska Department of Highways, [no place].

Brown, R.E. and A.J. Glenn, 1976. Vibroflotation and Terra-Probe Comparison. Journal of the Geotechnical Engineering Division. Vol. 102, No. GT10. October.

Carlson, L.A., 1976. Design seismic ground motion for the Gastineau Channel Bridge at Juneau, Alaska. Alaska Department of Highways, Juneau, AK.

_____, 1979. The Gastineau Channel Bridge: design aspects of a long span - free cantilever prestressed segmental concrete structure. Alaska Department of Transportation, Juneau, AK.

Committee on Soil Dynamics of the Geotechnical Engineering Division, 1978. Definition of terms related to liquefaction. Journal of the Geotechnical Engineering Division. Vol. 104, No. GT9. September.

Herrmann, F.A., 1972. Navigation channel improvement, Gastineau Channel, Alaska: hydraulic model investigation. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Technical report H-72-9.

Mitchell, J.K., 1981. "Soil improvement: state-of-the-art." Paper presented at the International Conference on Soil Mechanics and Foundation Engineering (10th: 1981: Stockholm, Sweden). University of California, Berkeley, CA.

Newmark, N.M., 1975. Seismic design criteria for structures and facilities, Trans-Alaska Pipeline System. In Proceedings of U.S. National Conference on Earthquake Engineering (1975: Ann Arbor, MI). Earthquake Engineering Research Institute, Berkeley, CA, p. 94-103.

Prakash, S., 1981. Soil dynamics. McGraw Hill, New York, NY.

R&M Engineering and Geological Consultants, 1972. Soil and foundation report, proposed sewage treatment plant, City and Borough of Juneau, Alaska. R&M Engineering and Geological Consultants, Fairbanks, AK.

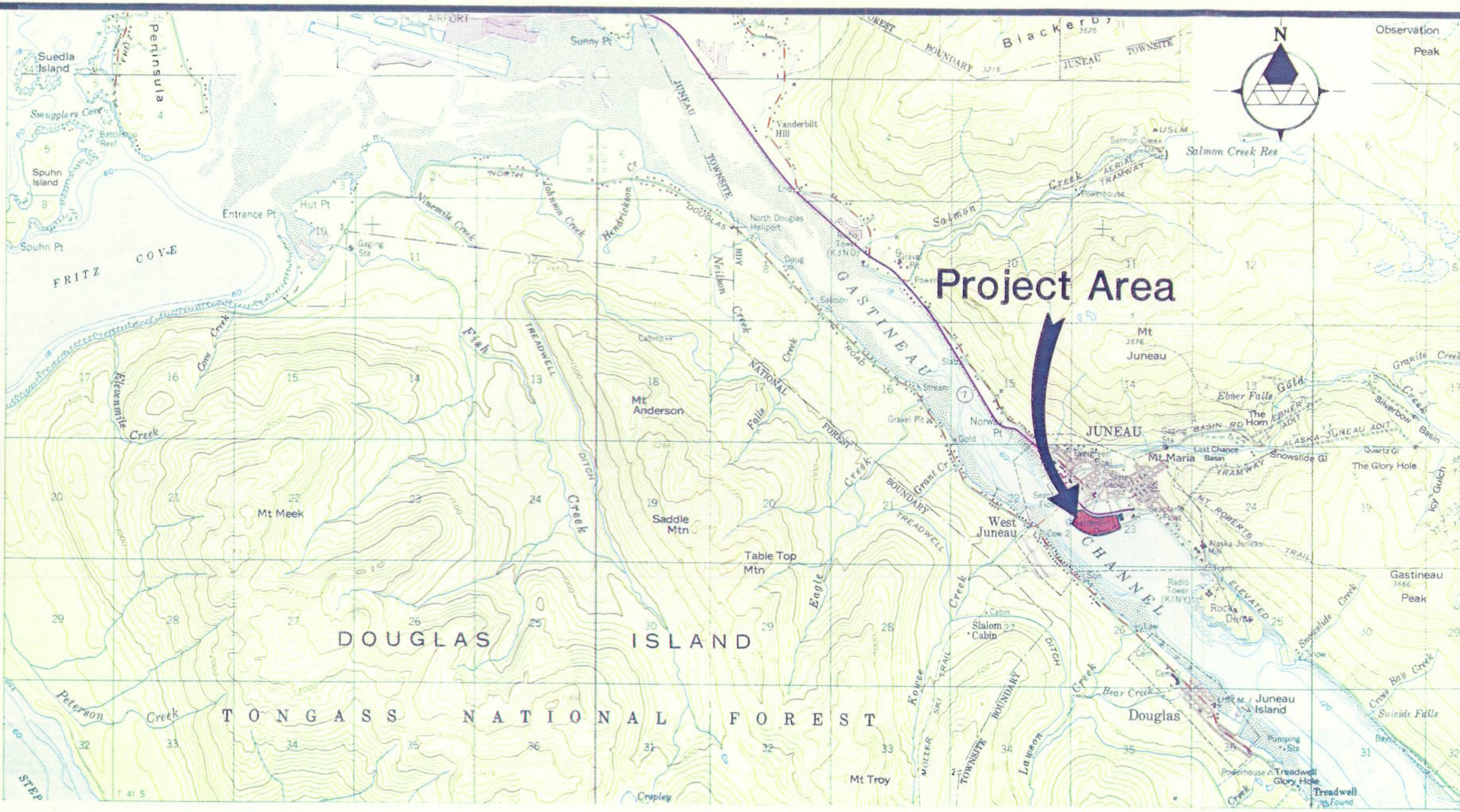
Seed, H.B., 1979. Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. ASCE Journal of the Geotechnical Engineering Division, Vol. 105, No. 2, p. 201-255.

Seed, H.B. and I.M. Idriss, 1971. Simplified procedure for evaluating soil liquefaction potential. ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 97, No. 9, p. 1249-1273.

U.S. Army Corps of Engineers, Alaska District, 1977. Gastineau Channel, Juneau, Alaska, feasibility report, channel deepening for navigation. U.S. Army Corps of Engineers, Alaska District, [Anchorage, AK].

_____, Alaska District, [1981?]. Draft Juneau small boat harbor detailed project report: Appendix E - Engineering and Cost Analysis (includes report drawings). U.S. Army Corps of Engineers, Alaska District, Anchorage, AK.

Woodward-Clyde Consultants, 1977. Evaluation of liquefaction potential, Gastineau Channel Bridge No. 740. Woodward-Clyde Consultants, San Francisco, CA.



REFERENCE:
JUNEAU (B-2), ALASKA QUADRANGLE, 1962,
MINOR REVISIONS 1966, PHOTO REVISED 1974 BY
USGS AND USC & GS.

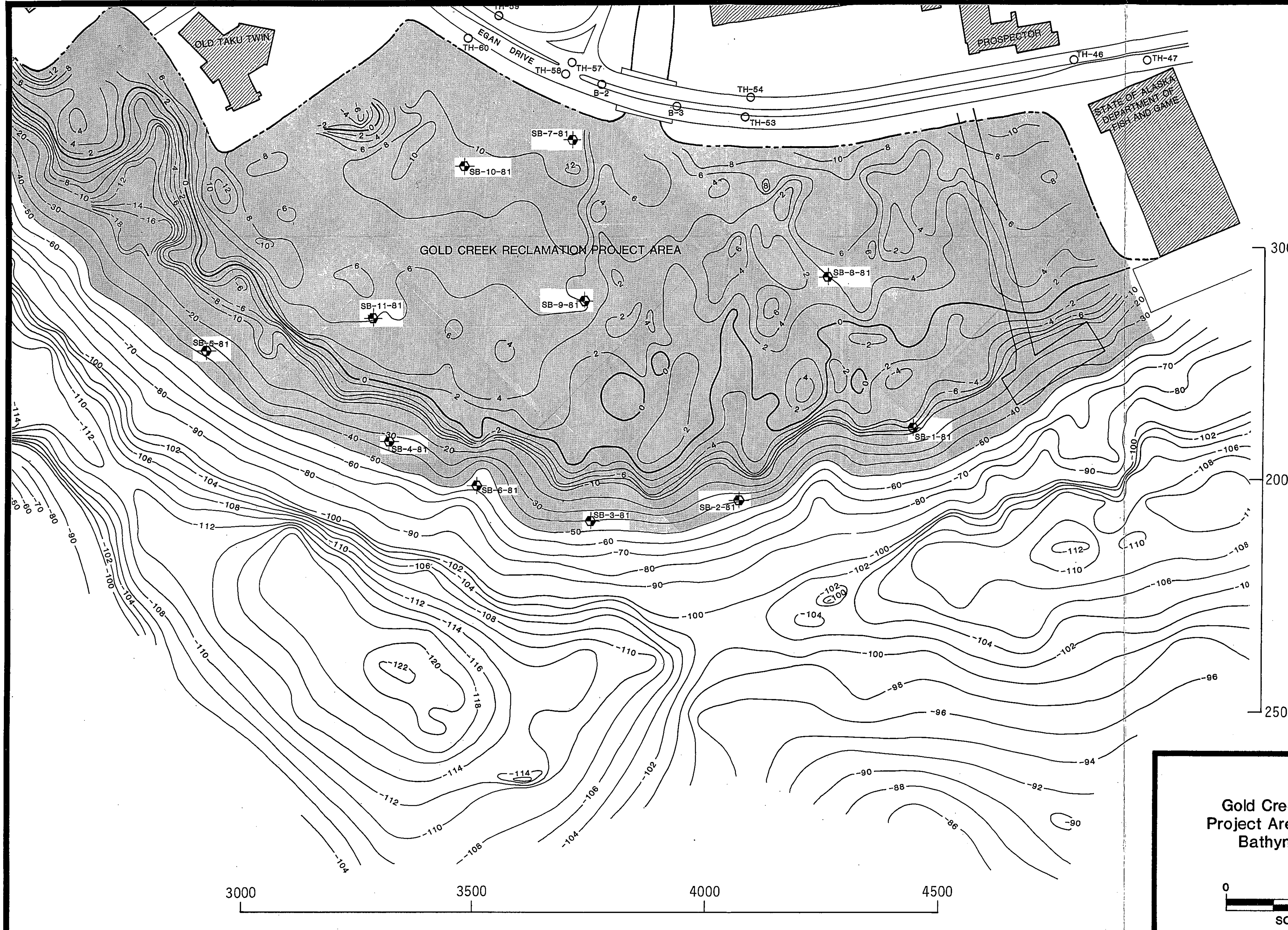
Vicinity Map

3000 0 3000 6000 9000 1200

Scale in Feet

Job No. 6842-003

Plate 1



KEY:

BORING LOCATION & NUMBER THIS INVESTIGATION BY DAMES & MOORE (SB-1-81)

BORING LOCATION & NUMBER FOR PREVIOUS INVESTIGATION BY OTHERS (TH-46, B-2)

LEGEND:

MEAN HIGHER HIGH WATER LINE(MHHW)

ELEVATION CONTOURS IN FEET

CONTOUR INTERVALS ARE AS FOLLOWS:
2 FEET FROM ELEVATION +12 TO -10
10 FEET FROM ELEVATION -10 TO -100
2 FEET FROM ELEVATION -100 TO -122
OR AS OTHERWISE LABELED ON DRAWING.

VERTICAL DATUM: MEAN LOWER LOW WATER (MLLW)

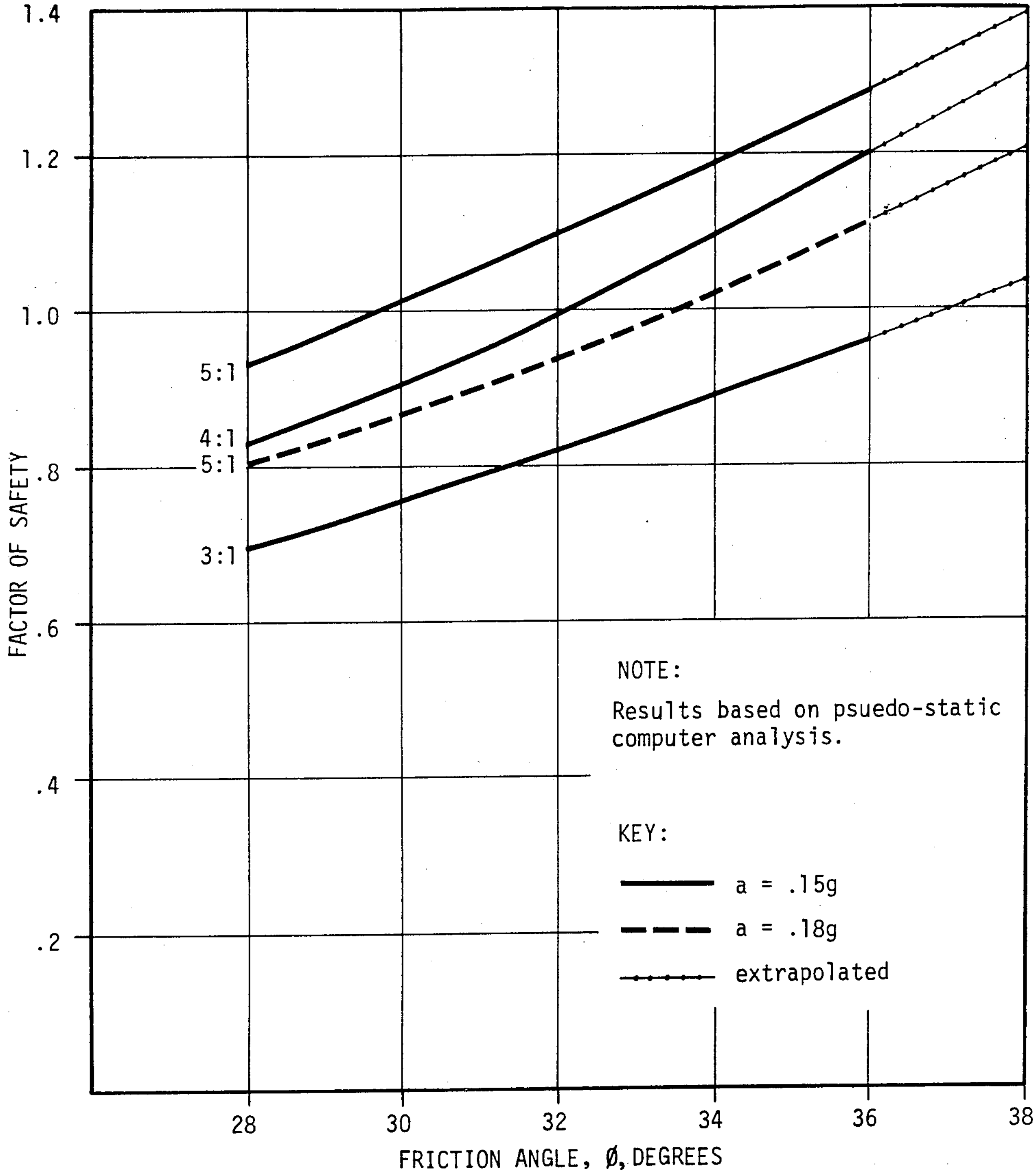
HORIZONTAL DATUM: CITY OF JUNEAU PLANE COORDINATE SYSTEM BASED ON CROW MAPS.

NOTES:

1. SURVEY DATE NOVEMBER 12, 1981.
2. ALL ELEVATIONS ARE IN FEET AND REFER TO MEAN LOWER LOW WATER (MLLW). TIDAL CORRECTIONS ARE BASED ON TIDAL ELEVATIONS RECORDED MANUALLY AT HARRIS HARBOR, JUNEAU.
3. POSITIONING FOR THIS BATHYMETRIC SURVEY WAS ACCOMPLISHED UTILIZING A MOTOROLA MINI-RANGER RATIO POSITIONING SYSTEM.
4. THE SHORELINE (MHHW) AND FACILITIES SHOWN ON THIS DRAWING WERE TAKEN FROM 1979 JUNEAU-DOUGLAS TOPOGRAPHICAL BASE MAPS (SCALE 1"=50') PREPARED BY NORTH PACIFIC AERIAL SURVEYS, INC., ANCHORAGE, ALASKA FOR THE CITY AND BOROUGH OF JUNEAU, ALASKA, DEPARTMENT OF PUBLIC WORKS.
5. WARNING - THIS MAP IS NOT INTENDED FOR NAVIGATIONAL PURPOSES.

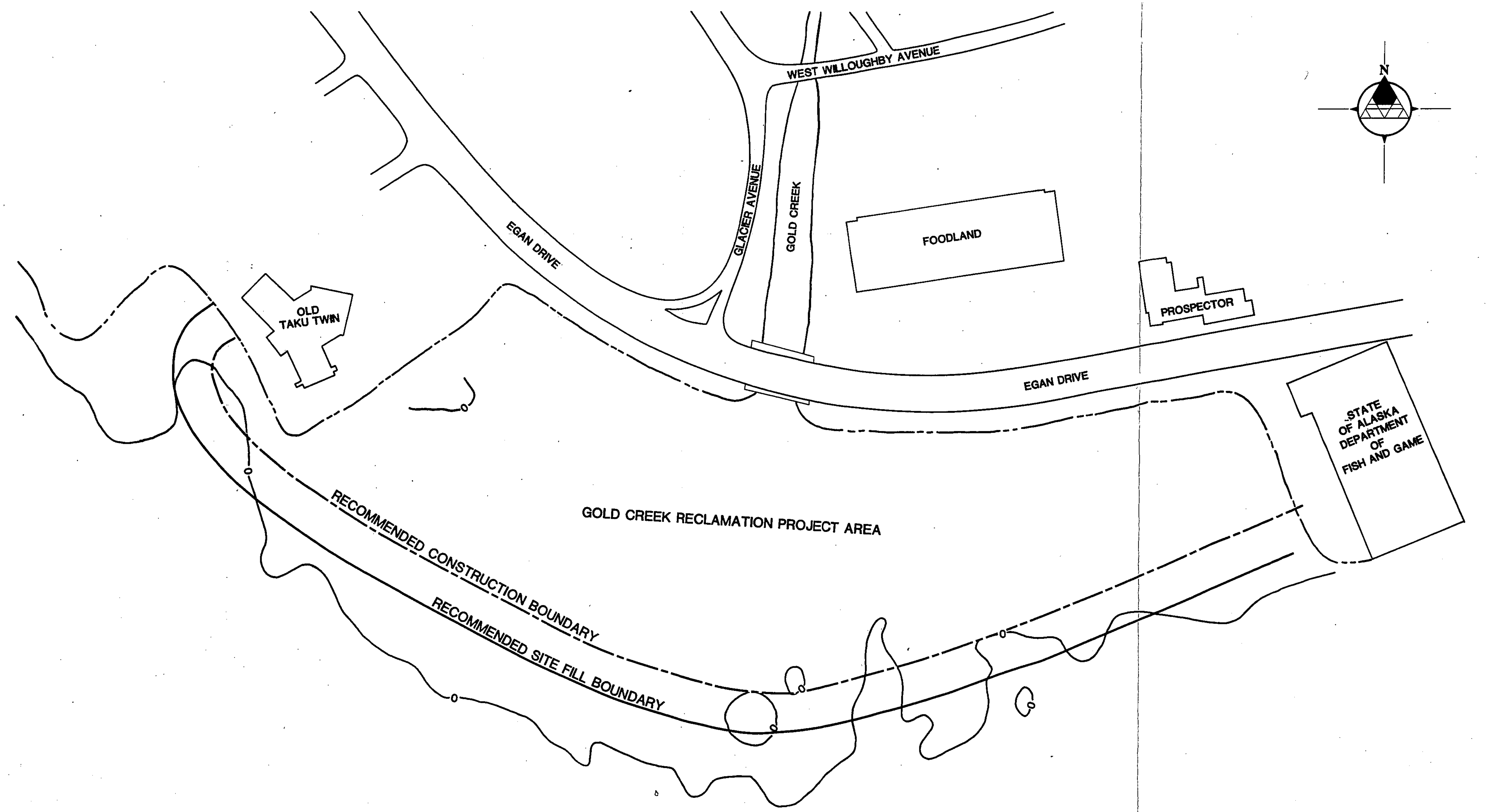
Gold Creek Reclamation Project Area and Results of Bathymetric Survey

0 200 400
SCALE IN FEET



Results of Pseudo-Static Slope Stability Analyses

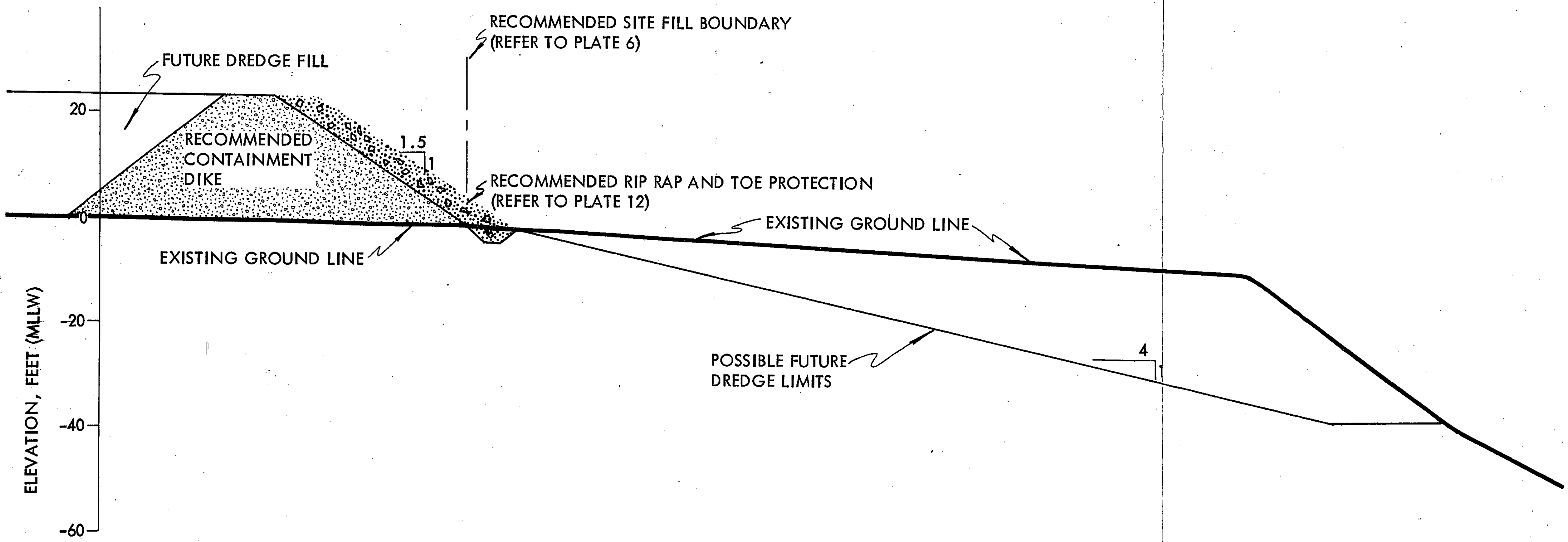
Dames & Moore



Reference:
Sketch of Gold Creek Reclamation Project,
Juneau, Alaska by EMPS dated January 1982

Recommended Site Fill
and Construction Boundaries

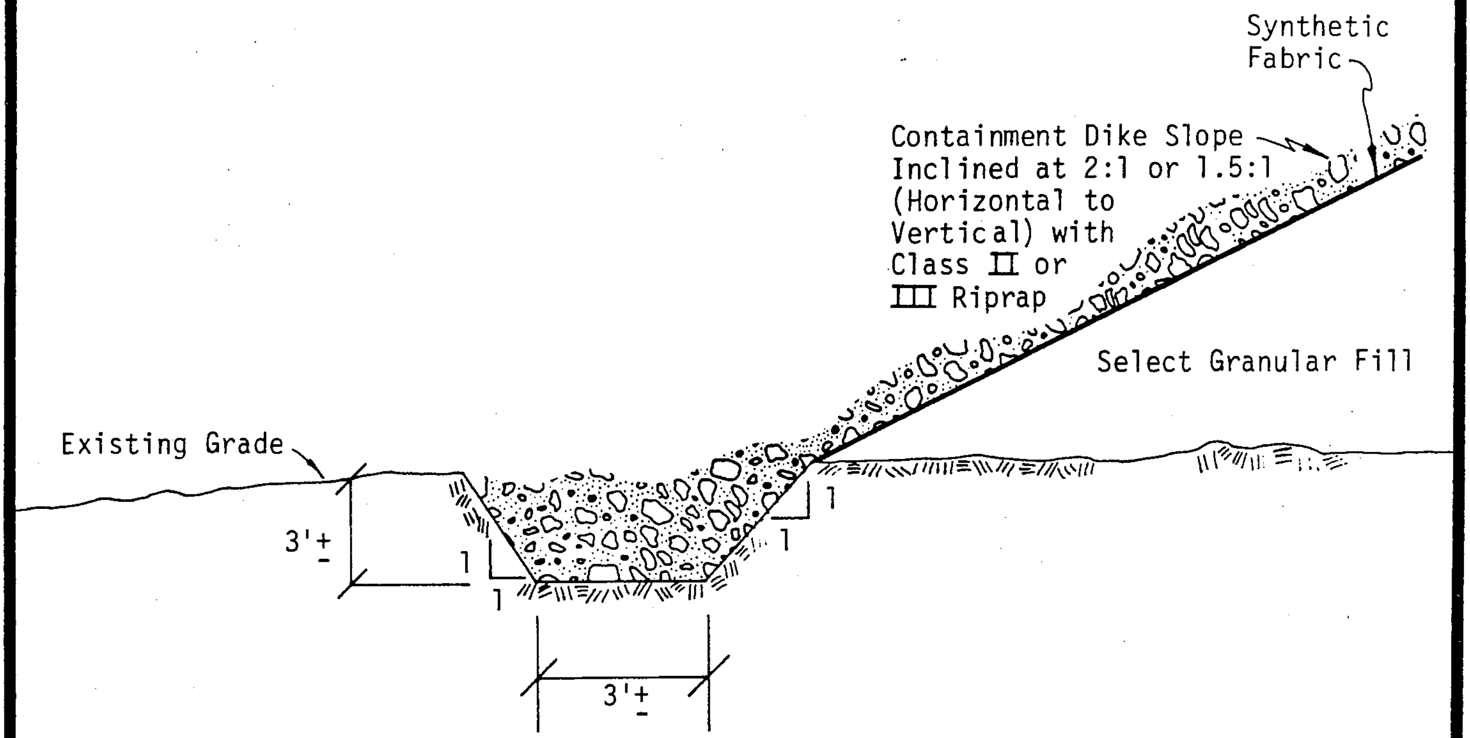
Dames & Moore



TYPICAL SECTION

Existing Conditions, Recommended Dike and Dredge Limits

SCALE 1" = 20', HORIZONTAL AND VERTICAL



TYPICAL SECTION
No Scale

Suggested Toe Protection Detail

Dames & Moore

APPENDIX A

BATHYMETRIC SURVEY METHODOLOGY

DATA ACQUISITION PROCEDURES

The bathymetric survey at the Gold Creek Reclamation Project was conducted on November 12, 1981. Utilizing a 21-foot vessel, 27 lines were surveyed on arcs approximately normal to the shoreline. Additionally, 8 shore parallel lines and 2 diagonal lines were surveyed to cross-tie the data in the survey area.

Positioning for the surveys was provided using a two-range Motorola Mini-Ranger III Radio Positioning System with a repeatable accuracy of ±3 meters on each range. This system was calibrated on a measured baseline prior to initiating the survey. Water depths were measured and recorded continuously with a Raytheon precision fathometer (Model DE-719) which is capable of accuracies of 0.5 percent ±1 inch of indicated depth. The fathometer was calibrated at the beginning and end of the survey utilizing a conventional bar check (steel plate suspended beneath the transducer to 10- and 20-foot depths on a premeasured steel cable). Positioning data, including time and boat location (two ranges), were recorded on a strip chart recorder.

Radio positioning shore stations were located at the Union 76 Terminal southeast of the survey area and at the waters edge on Douglas Island south of the survey area. This baseline orientation was selected for optimum positioning geometry. Coordinates on the shore stations were provided by EMPS. Periodic positioning interference was encountered due to reflection of radio signals off of the numerous buildings along the north shore of Gastineau Channel.

Tidal levels in Gastineau Channel were measured manually during the survey from a reference point selected near the Harbormaster's office at Harris Harbor. After the completion of the survey, EMPS determined an elevation on the reference point. The reference elevation (26.66 MLLW) was used to compute tidal elevations which were used to apply time-variable tidal corrections to survey soundings. Tidal elevations ranged from +20.0 to 11.3 feet MLLW during the survey.

DATA PROCESSING AND ANALYSIS

At the conclusion of the field data collection effort, survey positioning and depth data were transferred to a computer data file. Utilizing a data preparation program, all necessary tidal corrections, unit conversions, and triangulations were computed to prepare the survey data for plotting. Positioning post-plot, depth-plot, and contoured maps were produced utilizing a surface approximation and contour mapping program. These maps were then edited and combined as required and a final contour map was prepared.

The survey area is characterized by two topography distinct areas. Along the northern half of the survey area, a relatively shallow, gently sloping delta extends into the Gastineau Channel from Gold Creek. Elevations over this area range from +12 to -6 feet MLLW. Although the Gold Creek Channel dissects the delta and is distinct at the north of the site, it quickly branches off and is distinguishable only in the irregular character of the contours east of the creek. At an average elevation ranging between +4 and -6 feet MLLW along the delta front, the bottom slope drops off quickly to the Gastineau Channel. The apparent slope of the delta front averages about 2-1/2:1 (horizontal to vertical) but steepens locally to less than 1:1. The toe of the slope occurs at elevations ranging from -100 to -112 feet MLLW. The axis of Gastineau Channel extends to an elevation of between -104 and -127 feet MLLW in the survey area and its character varies from a fairly wide flat area to a "V"-shape. The results of the subject bathymetric survey are presented on Plate 3.

APPENDIX B

SITE EXPLORATIONS AND LABORATORY TESTS

SITE EXPLORATIONS

Subsurface conditions underlying the Gold Creek Reclamation Project area and potential borrow sources were explored by drilling 21 borings with rotary-wash drilling equipment mounted on a tracked carrier. The borings at the project area were drilled to depths ranging from approximately 46 to 130 feet below current site elevations. Borings drilled during this investigation at the location of the potential borrow sources were drilled to depths on the order of about 20 feet. The approximate locations of the borings drilled during this investigation, in addition to borings drilled by others during previous subsurface exploration, are shown with respect to existing features and the project area on Plates 2 and 3.

The site exploration program was coordinated by members of our staff who located the explorations and maintained detailed logs of the conditions encountered. The location of offshore borings completed at the project area was provided by EMPS; other borings were located approximately relative to topographic features. Logs of the explorations are presented on Plate B-1 through B-12. The borings completed at the project area are designated as SB; borings completed at potential borrow source areas are designated BB; bulk samples obtained at potential borrow sources are designated BS. The soils have been classified in accordance with the Unified Soil Classification System, which is presented on Plate B-13.

Relatively undisturbed samples of the soils were obtained at frequent intervals in the borings using a Dames & Moore Type U Sampler, which is illustrated on page B-6. A Sprague and Henwood sampler with dimensions similar to that of the Dames & Moore sampler was used also to obtain disturbed and undisturbed samples. These samplers were driven with a 300-pound hammer falling a distance of approximately 30 inches.

Standard Penetration Tests (SPT) were also accomplished in the site borings at various depths. The SPT is completed by driving a split-spoon sampler with a 140-pound hammer falling a distance of about 30 inches. The number of blows required to drive each sampler a distance of 1 foot into undisturbed soil is shown immediately adjacent to each sample notation. The correlation between the SPT blow counts and those of the Dames & Moore and Sprague and Henwood samplers is about 1:1. Samples were obtained also by pushing the sampler under the weight of the rods and the hammer. Samples obtained in this manner are noted by the letter (P) adjacent to each sample notation. In addition to the undisturbed and disturbed samples obtained using the three samplers, bulk samples were obtained at several of the borrow source locations as noted on Plate 3.

Boring elevations are approximate and have been estimated from the results of the bathymetric survey accomplished by Dames & Moore and a bathymetric survey of portions of the Gastineau Channel accomplished by the Alaska Department of Transportation and Public Facilities.

LABORATORY TESTS

Soils obtained during the site exploration program were examined in our laboratory in order to select representative samples for testing and to verify the classification recorded during the site exploration program. Laboratory tests included direct shear, triaxial, grain size, compaction, consolidation, and moisture-density tests. Turbidity and elutriate tests were performed by AM TEST, Inc. on samples obtained at the potential borrow source sites. The results of these tests are presented below in Tables B-1 and B-2.

Direct shear and triaxial tests were performed at a continuous rate of shearing deflection in the manner presented on page B-7 and B-8, respectively, in order to evaluate the shear strength of the site soils. The results of the direct shear tests are presented on Plates B-14 and B-15; the triaxial test results are shown on Plate B-16.

TABLE B-1

Test	Sample Designation Bulk 1	Bulk 6	Bulk 7	Seawater
Bulk sediment analysis:				
Total volatile solids (%)	--	0.92	0.74	--
Chemical Oxygen Demand (%)	--	1.33	2.46	--
Oil and Grease (mg/kg)	130.	--	--	--
Elutriate analysis: (4:1) seawater to sediment ratio)				
Lead (mg/l)	--	<0.02	<0.02	<0.02
Copper (mg/l)	--	<0.02	<0.02	<0.02
Zinc (mg/l)	--	0.080	0.112	0.107
Mercury (mg/l)	--	<0.0002	<0.0002	<0.0002
Arsenic (mg/l)	--	<0.001	<0.001	0.002
Cadmium (mg/l)	--	0.072	0.093	<0.010
Oil sheen	Not Detected	--	--	--

TABLE B-2

Sample Designation/ Depth	Weight (g)[a] in 600 mls	Turbidity (NTU)[b]						
		Minutes						
		0	5	10	15	20	25	30
Bulk 1/ surface	23.5	200	150	140	110	88	83	72
Bulk 6/ surface	4.7	130	110	90	85	79	69	57
Bulk 7/ surface	6.1	110	68	44	34	28	30	19
Bulk 3/ surface	3.6	320	260	230	160	160	130	120
Bulk 4/ surface	4.0	240	170	150	110	85	76	66
BB2 #3/ 9-1/2 feet	6.0	190	180	150	130	89	90	92
BB3 #1/ 1-1/2 feet	4.9	580	470	370	260	240	190	180
BB4 #2/ 4 feet	13.6	430	370	320	280	210	150	160
BB5 #2/ 4 feet	4.4	480	380	300	260	190	150	150
BB6 #1/ 1-1/2 feet	4.0	270	240	200	160	130	95	97
BB8 #4/ 14-1/2 feet	6.6	240	210	200	150	140	98	100
BB9 #1/ 1 foot	6.4	160	130	110	100	86	77	62
BB8 #2/ 4 feet	6.0	240	220	200	180	130	96	83
BB10 #4/ 16 feet	5.0	180	140	120	110	85	76	78
Bulk 2/	16.0	420	300	190	140	100	74	80
BB3 #2/ 6 feet	5.5	180	160	130	120	90	82	72

(a) Seawater.
(b) Background turbidity: seawater = 0.44 NTU.

Grain size determinations were completed on samples of the soils encountered at the project area and also at the borrow sites. The results of the tests are presented graphically on Plates B-17 through B-36.

Compaction tests of borrow source soils were completed on several borrow source samples. The results of those tests are presented on Plates B-37 through B-40.

Consolidation tests were completed in order to evaluate the consolidation characteristics of the silt soils which underlie the project area. The results of those tests are presented on Plate B-41.

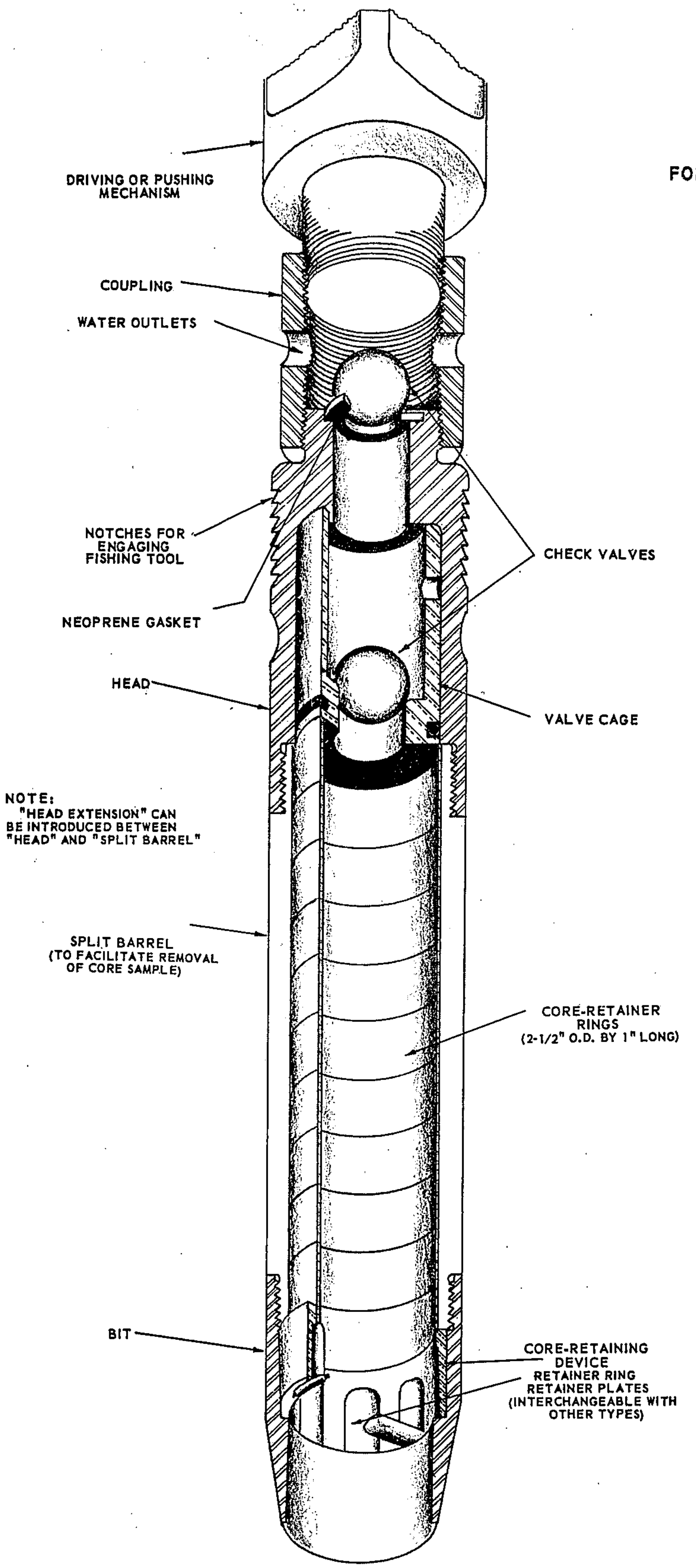
Moisture-density tests were completed in conjunction with each of the above tests and on other selected soil samples if the sample was undisturbed. The results of these tests are presented on the test summary sheets and on the logs of the borings adjacent to the appropriate sample notation.

The following plates are attached and complete this appendix:

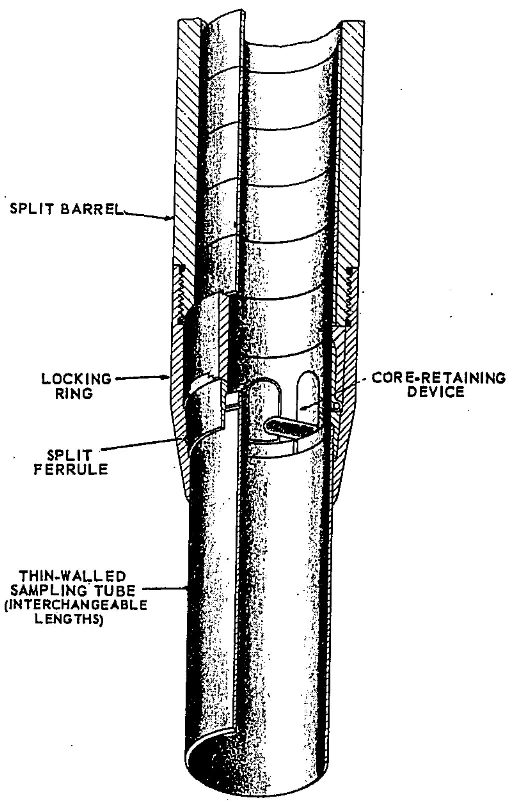
Plates B-1 through B-12, Logs of Borings
Plate B-13, Unified Soil Classification System
Plates B-14 and 15, Summary of Direct Shear Test Data
Plate B-16, Summary of Triaxial Test Data
Plates B-17 through B-36, Gradation Curves
Plates B-37 through B-40, Compaction Test Data
Plate B-41, Consolidation Test Data

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER

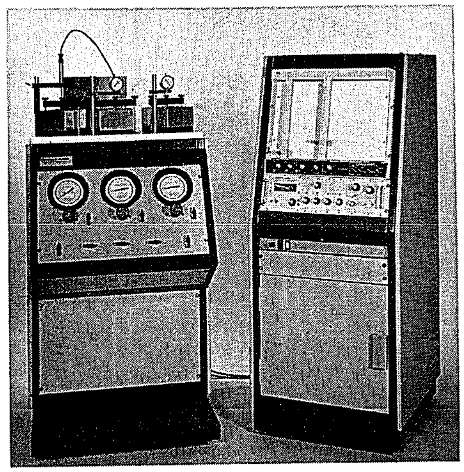


ALTERNATE ATTACHMENTS



417.9 (5-68)

Method Of Performing Direct Shear and Friction Tests



DIRECT SHEAR APPARATUS WITH ELECTRONIC RECORDER

Direct shear tests are performed to determine the shearing strengths of soils. Friction tests are performed to determine the frictional resistances between soils and various other materials such as wood, steel, or concrete. The tests are performed in the laboratory to simulate anticipated field conditions.

Each sample is tested in a split sample holder, two and one-half inches in diameter and one inch high. Undisturbed samples of in-place soils are extruded from 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.

Direct Shear Tests

A one-inch length of the sample is tested in direct single shear. A constant pressure, appropriate to the conditions of the problem for which the test is being performed, is applied normal to the ends of the sample through porous stones. A shearing failure of the sample is caused by moving the upper sample holder in a direction perpendicular to the axis of the sample. Transverse movement of the lower sample holder is prevented.

The shearing failure is accomplished by applying to the upper sample holder a constant rate of deflection. The shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soils is determined from the resulting load-deflection curves.

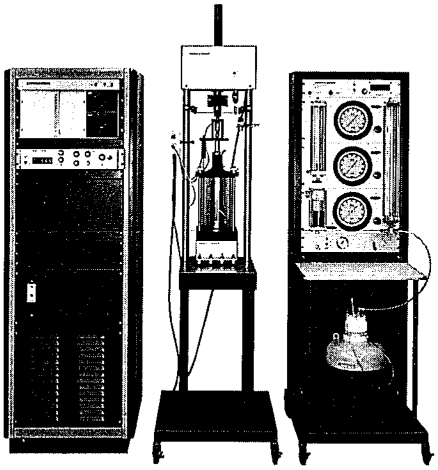
Friction Tests

In order to determine the frictional resistance between soil and the surfaces of various materials, the lower sample holder in the direct shear test is replaced by a disk of the material to be tested. The test is then performed in the same manner as the direct shear test by forcing the soil over the friction material surface.

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.

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.



TRIAXIAL COMPRESSION TEST UNIT

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:

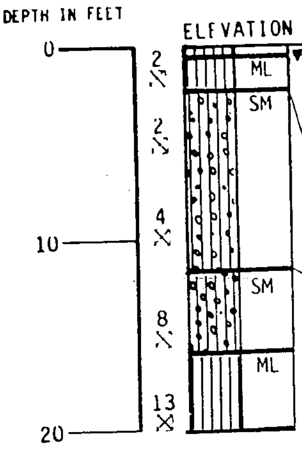
UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

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.

Boring BB-6-81



ML Dark brown silt with occasional gravel and shell fragments (very soft)

Dark gray to black sandy silt with shell fragments and trace of gravel (very soft)

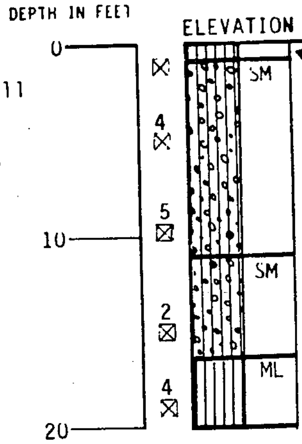
Dark gray silty fine to medium sand with trace of gravel (loose)

Dark gray silty fine sand with some gravel (loose)

Dark gray sandy silt with trace of gravel (medium stiff)

Boring completed 12-6-81

Boring BB-7-81



ML Brown silt (very soft)

Dark gray silty fine sand with occasional shell fragments (very loose to loose)

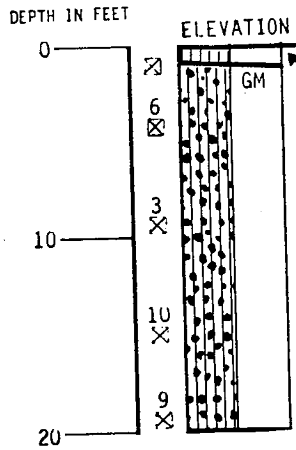
Grades with trace of gravel

Dark gray to black silty very fine sand with occasional shell fragments (very loose)

Dark gray to black sandy silt (soft)

Boring completed 12-6-81

Boring BB-8-81



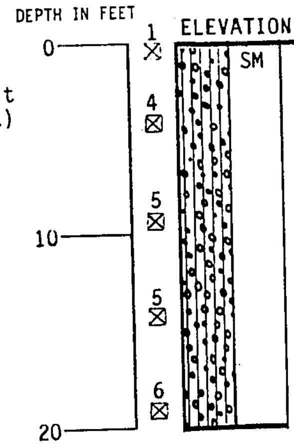
ML/OL Brown organic silt with organic matter and abundant shell fragments (very soft)

Brown and gray silty sandy fine to medium gravel with organic matter and abundant shell fragments (loose)

Grades with decreasing shell fragments

Boring completed 12-5-81

Boring BB-9-81



Dark gray to black silty fine sand with some shell fragments (very loose)

Grades to loose

Boring completed 11-21-81

LOG OF BORINGS

Dames & Moore

Job No. 6842-003-20

Boring SB-2-81

ELEVATION -38±

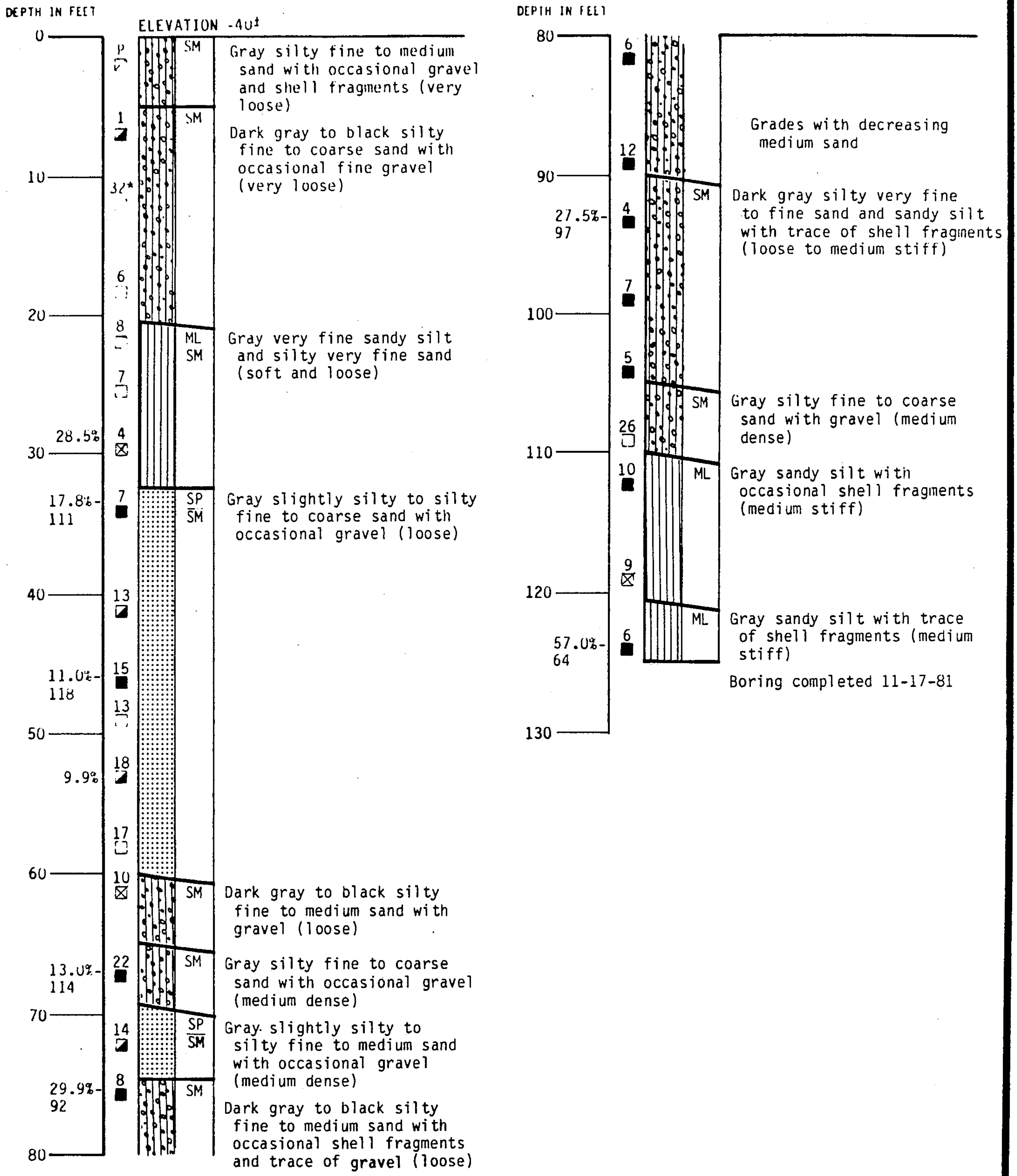
Depth (ft)	Sample (blows)	Moisture/Density	Soil	Description
0			ML	Gray silt with shell fragments (very soft)
	2	25.4%-91	SM	Gray silty fine to medium sand with some gravel and shell fragments (very loose)
	4			
10	2			
	12			Grades to loose to medium dense with decreasing shell fragments and increasing gravel
20	13	15.5%		
	9	28.1%-95	SM ML	Dark gray to black silty very fine sand and sandy silt (loose and medium stiff)
30	12		SP SM	Gray slightly silty to silty fine to coarse sand with occasional gravel (loose to medium dense)
	12			
40	7			
50	20			
	24			
60	15			
				Grades with increasing gravel
70	27			
	9	33.8%-88	SM	Dark gray to black silty very fine to fine sand with abundant shell fragments and a trace of gravel (loose)
80	8			
80	11		SM	Dark gray to black silty fine to medium sand with shell fragments (loose)
90	6	42.2%-80	SM ML	Dark gray to black silty very fine sand and sandy silt with occasional shell fragments (loose and medium stiff)
	19		SP SM	Gray slightly silty to silty fine sand with shell fragments and a trace of gravel (medium dense)
	15			
100				Grades to loose with decreasing shell fragments and gravel
	9	37.1%-83	SM	Gray silty very fine sand with occasional shell fragments (loose)
110				

Boring completed 11-17-81

LOG OF BORINGS

Dames & Moore

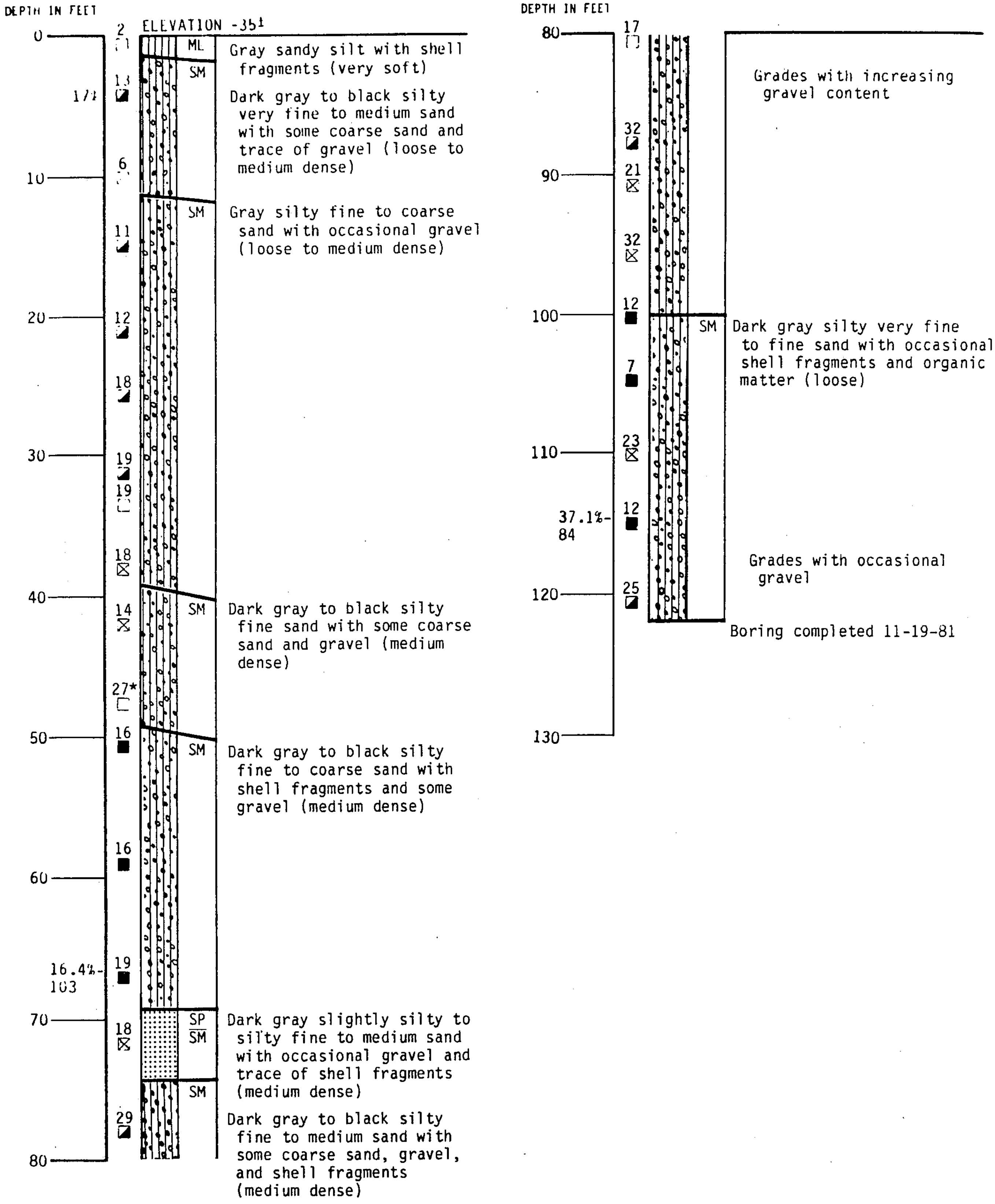
Boring SB-3-81



LOG OF BORINGS

Dames & Moore

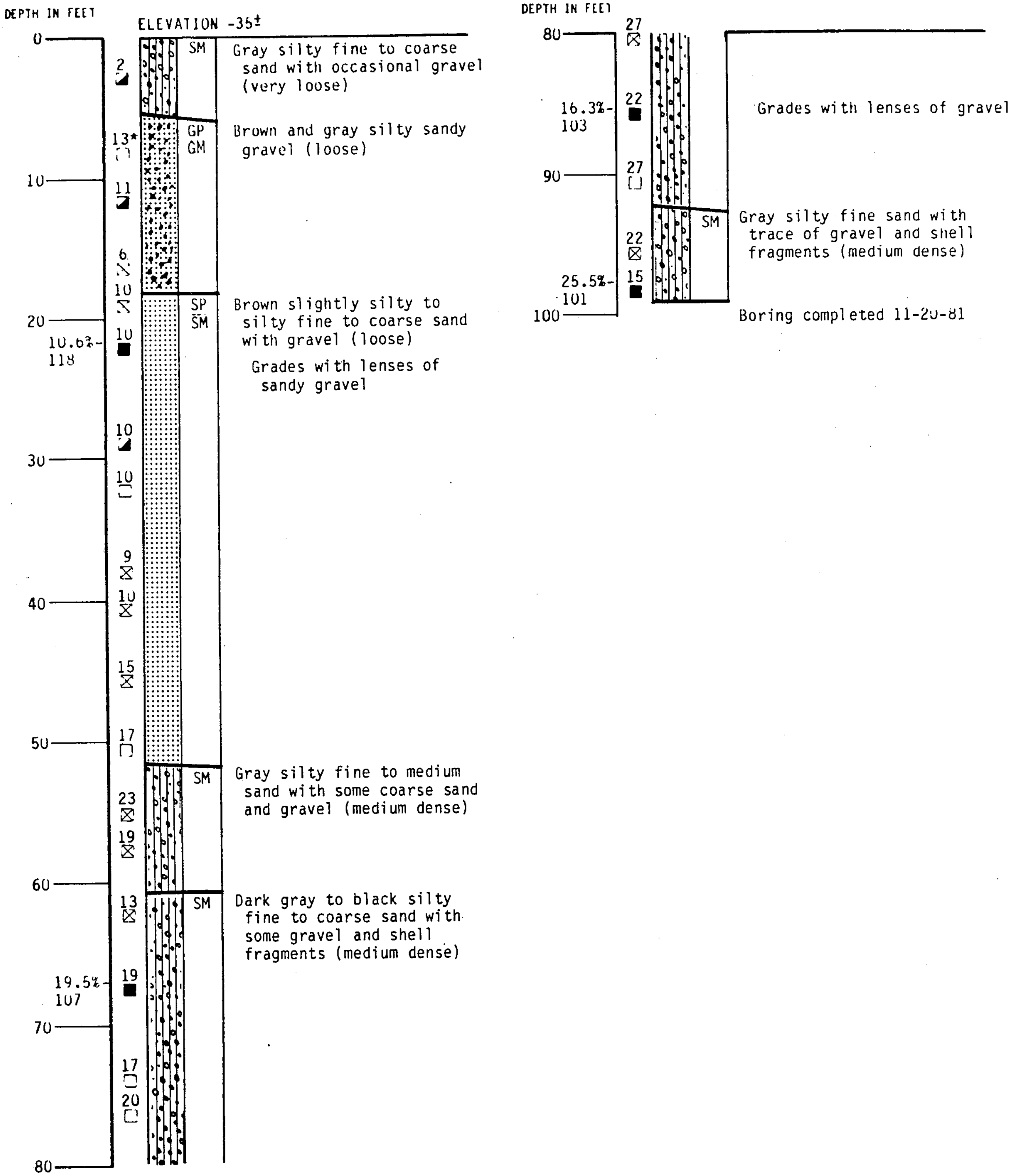
Boring SB-4-81



LOG OF BORINGS

Dames & Moore

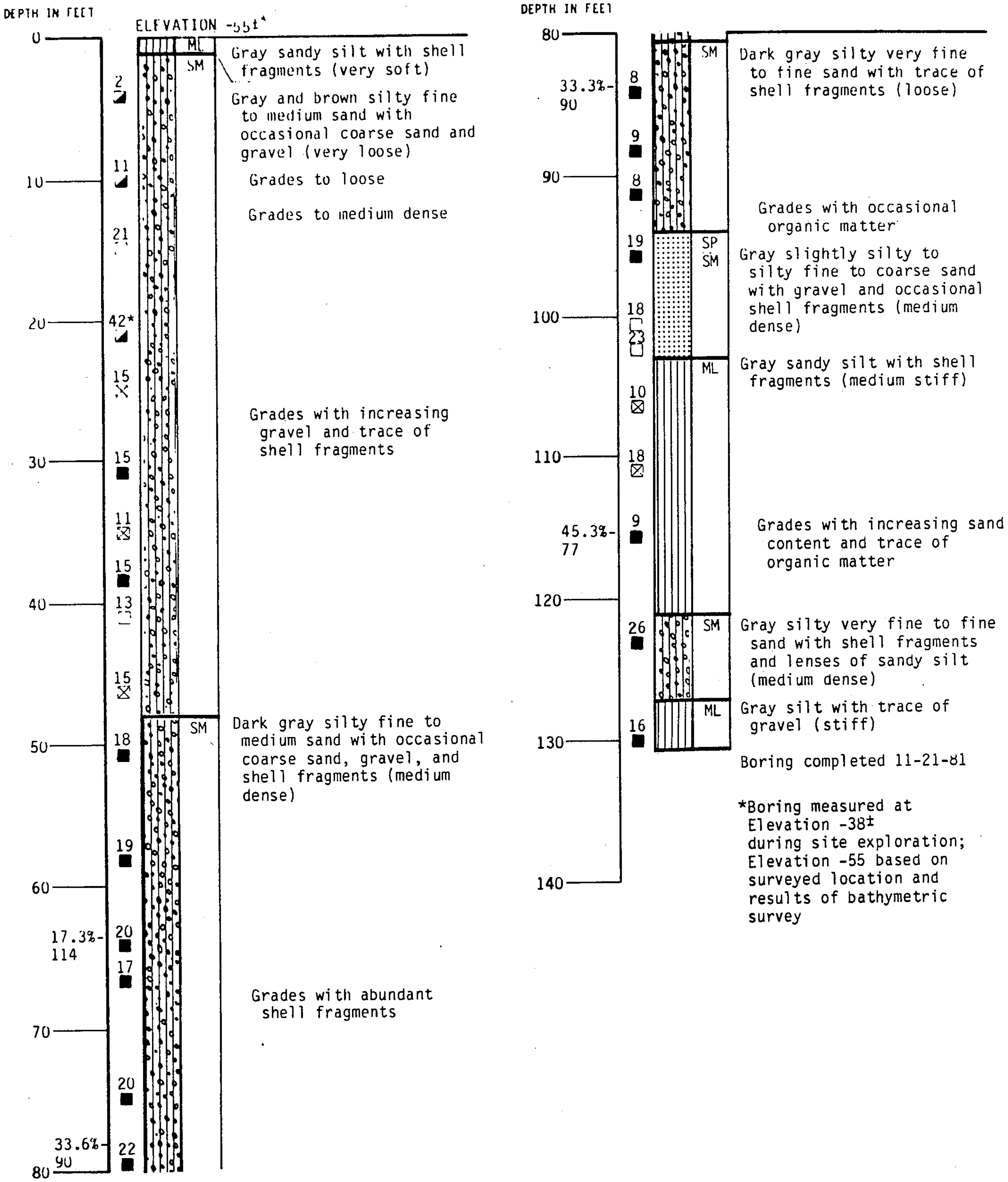
Boring SB-5-81



LOG OF BORINGS

Dames & Moore

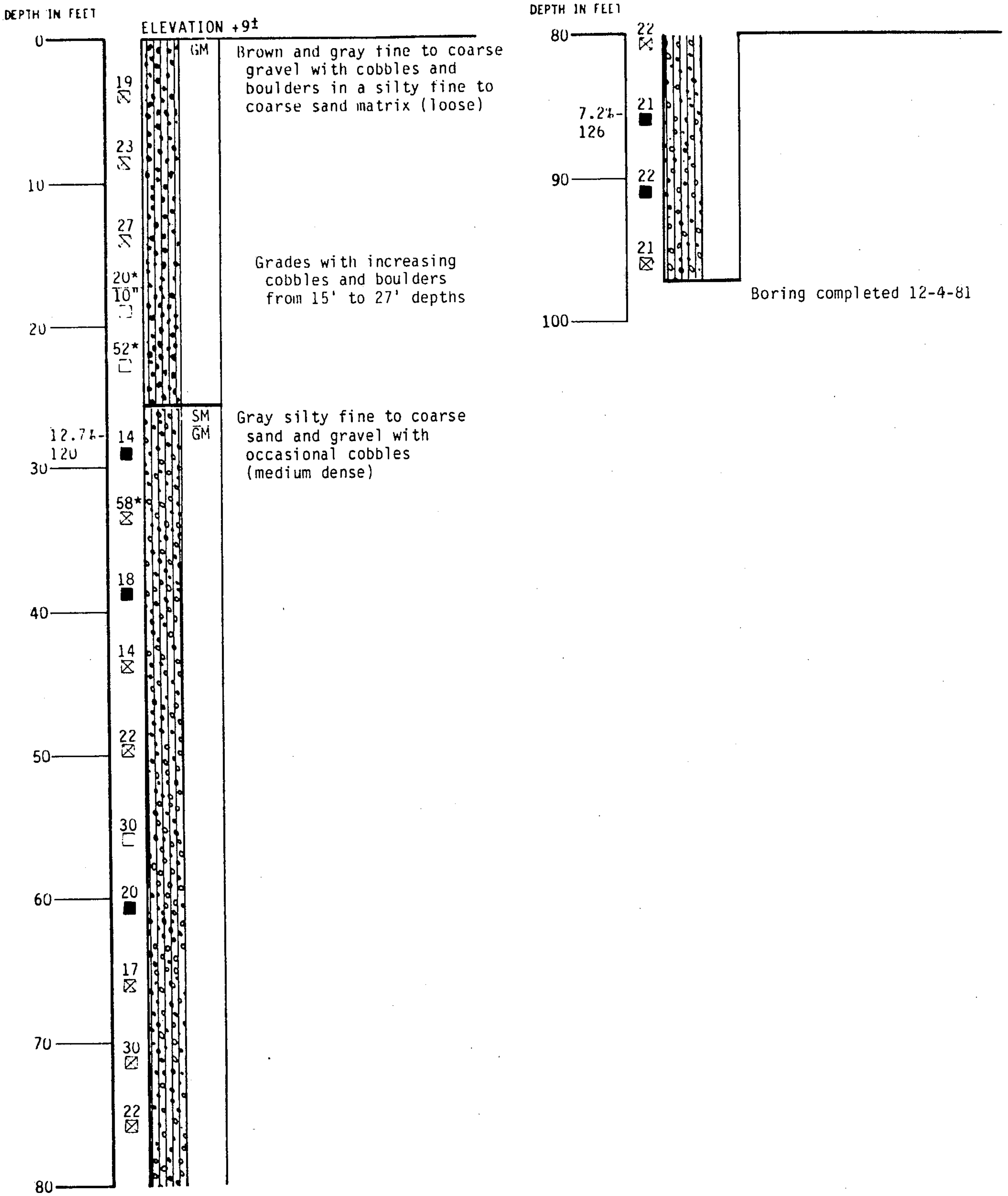
Boring SB-6-81



LOG OF BORINGS

Dames & Moore

Boring SB-7-81

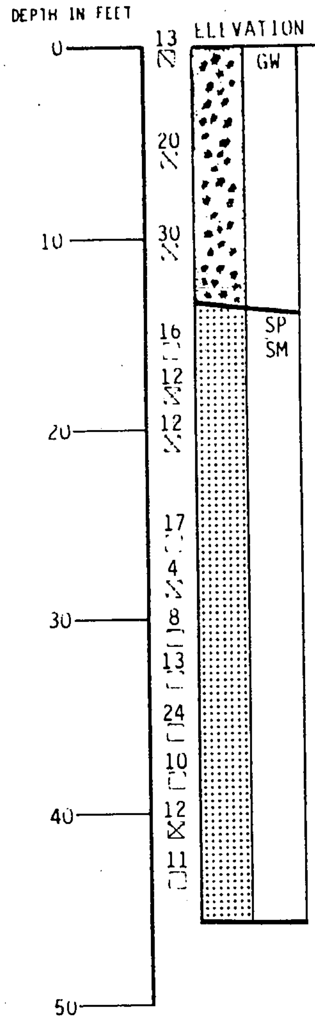


LOG OF BORINGS

Dames & Moore

Boring SB-8-81

ELEVATION +2±



Dark gray fine to coarse gravel in a silty fine to coarse sand matrix (medium dense)

Grades with lenses of silty fine to medium sand

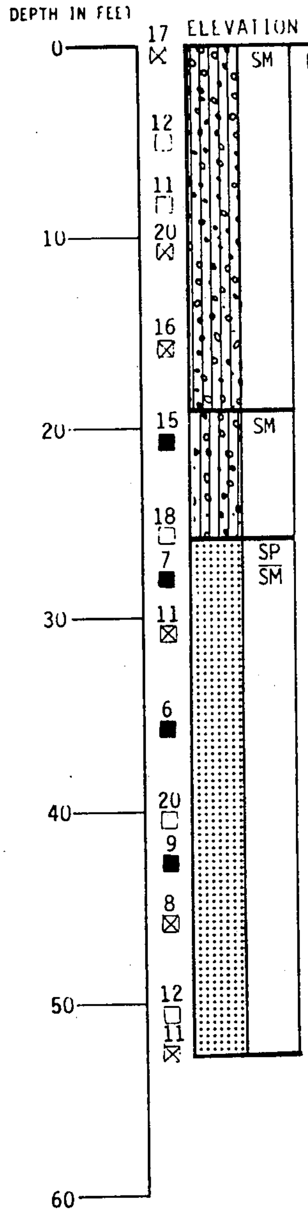
Brown and gray silty fine to coarse sand with gravel (medium dense)

Grades with decreasing coarse sand

Boring completed 11-12-81

Boring SB-9-81

ELEVATION +6±



Brown and gray silty fine to coarse sand with gravel (medium dense)

Grades with increasing gravel

Dark gray silty very fine to fine sand with some shell fragments (medium dense)

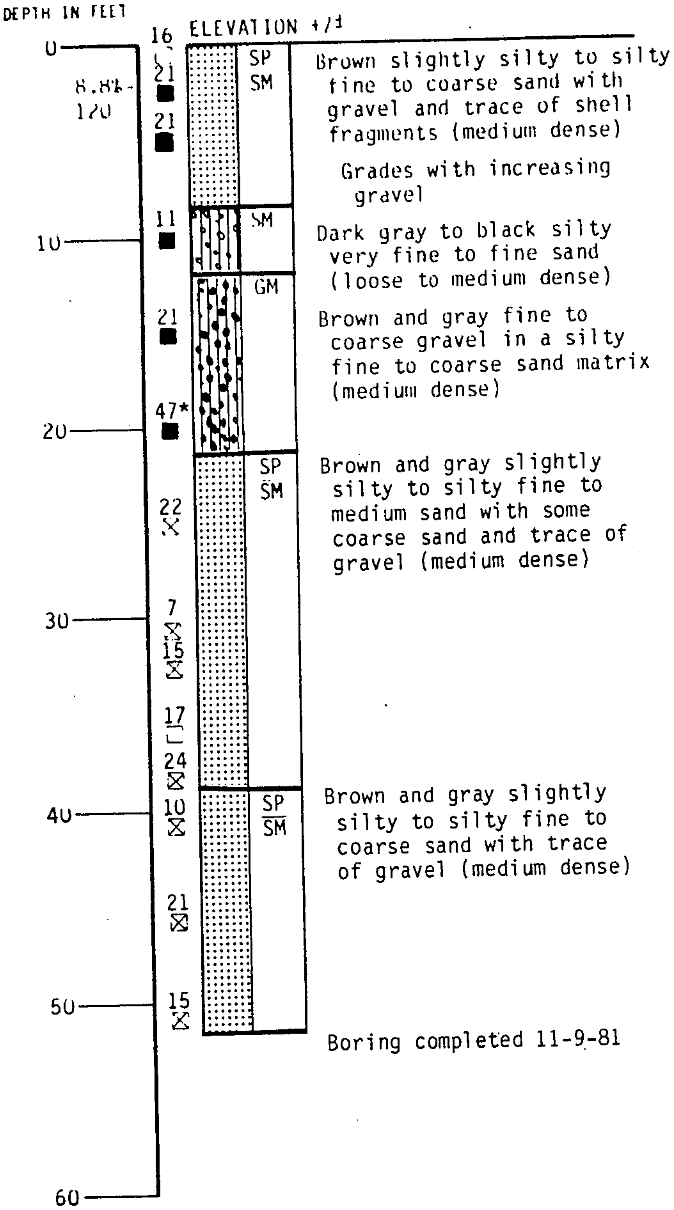
Dark gray to black slightly silty to silty fine to medium sand with some coarse sand and gravel (loose)

Boring completed 11-10-81

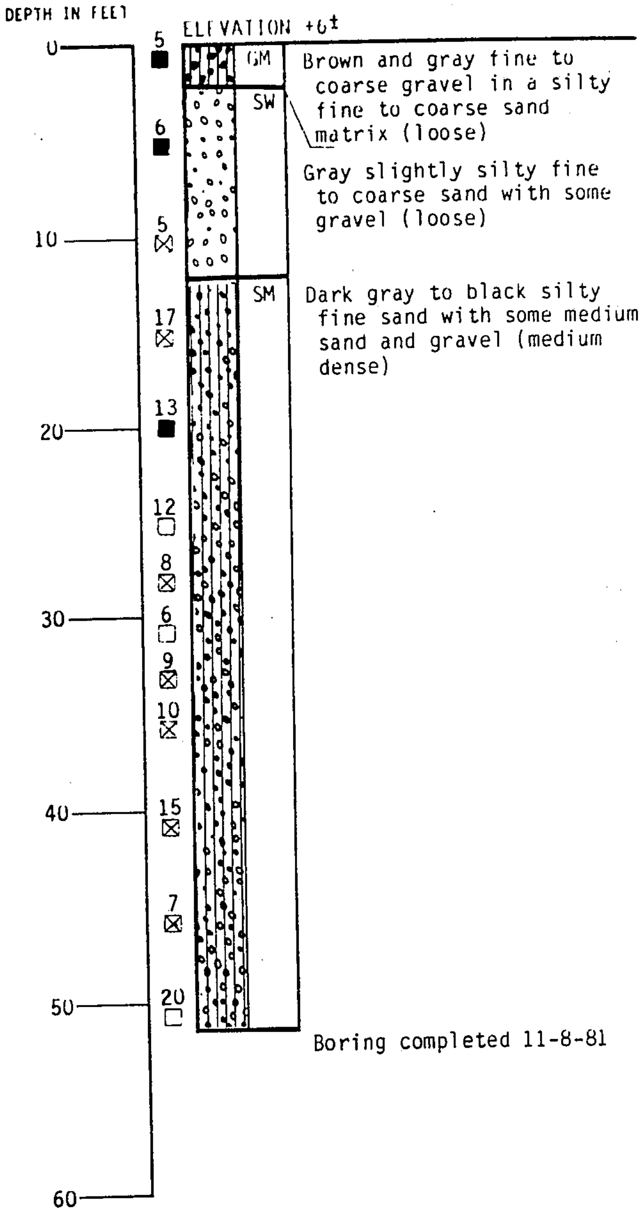
LOG OF BORINGS

Dames & Moore

Boring SB-10-81



Boring SB-11-81



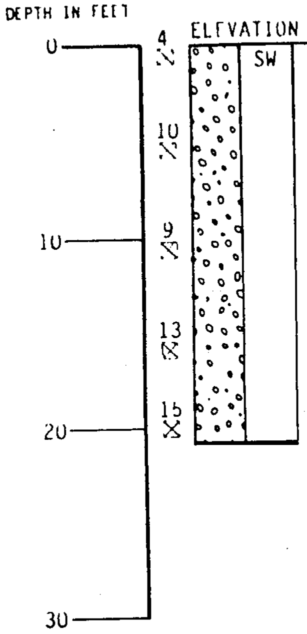
LOG OF BORINGS

Dames & Moore

Job No. 6842-003-20

Boring BB-2-81

ELEVATION +5±



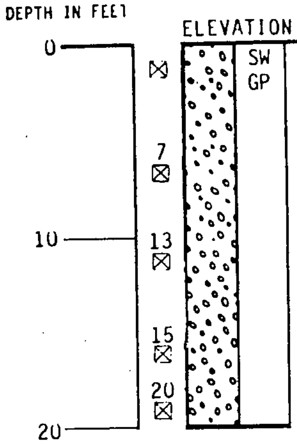
Dark gray to black slightly silty fine to coarse sand with some gravel and shell fragments (loose)

Grades to medium dense with increasing silt and gravel content

Boring completed 11-17-81

Boring BB-3-81

ELEVATION +6±



Brown and gray slightly silty fine to coarse sand and gravel with organic matter and abundant shell fragments (very loose to loose)

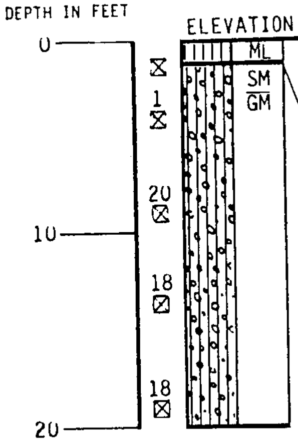
Grades with decreasing shell fragments

Grades to medium dense

Boring completed 12-21-81

Boring BB-4-81

ELEVATION +5±



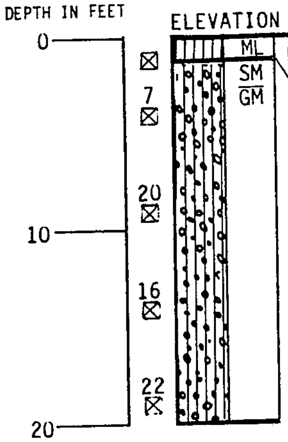
Black silt with organic matter, shell fragments, and occasional gravel (very soft)

Brown silty fine to coarse sand and gravel with occasional organic matter and shell fragments (loose)

Boring completed 12-20-81

Boring BB-5-81

ELEVATION +5±



Black sandy silt with shell fragments (very soft)

Brown silty fine to coarse sand and gravel with occasional shell fragments (loose)

Grades to medium dense with occasional cobbles

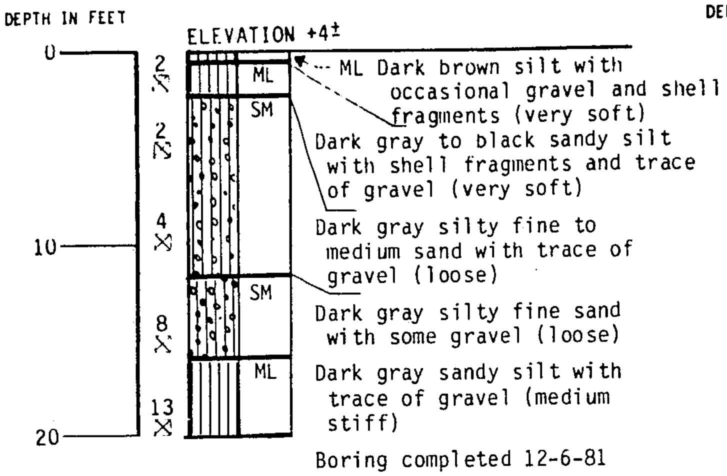
Grades with increasing sand content

Boring completed 12-20-81

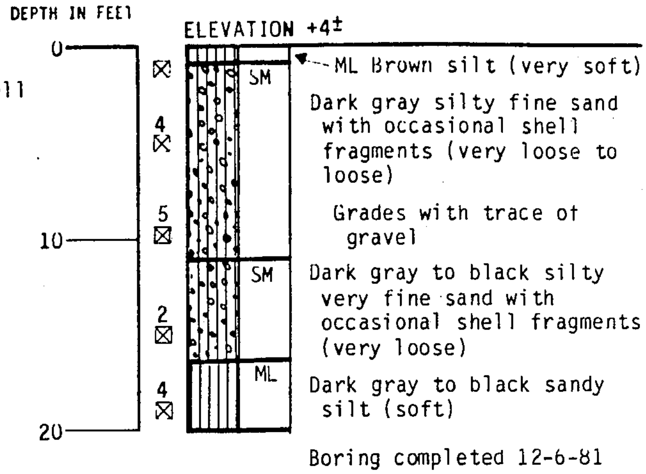
LOG OF BORINGS

Dames & Moore

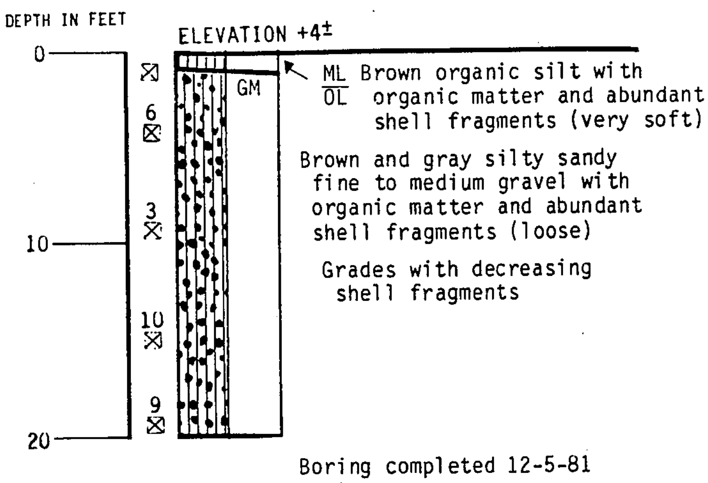
Boring BB-6-81



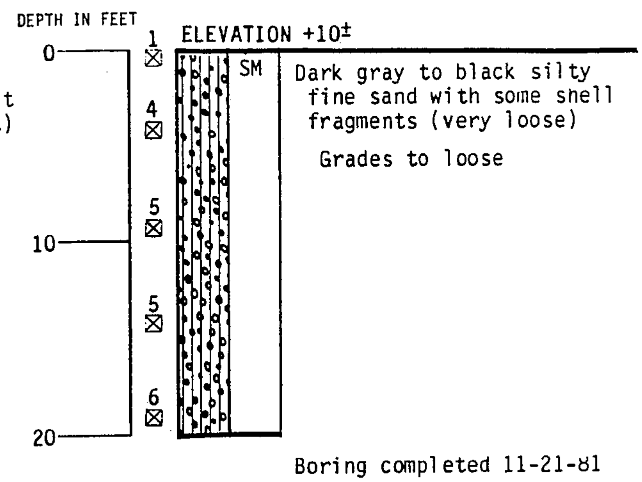
Boring BB-7-81



Boring BB-8-81



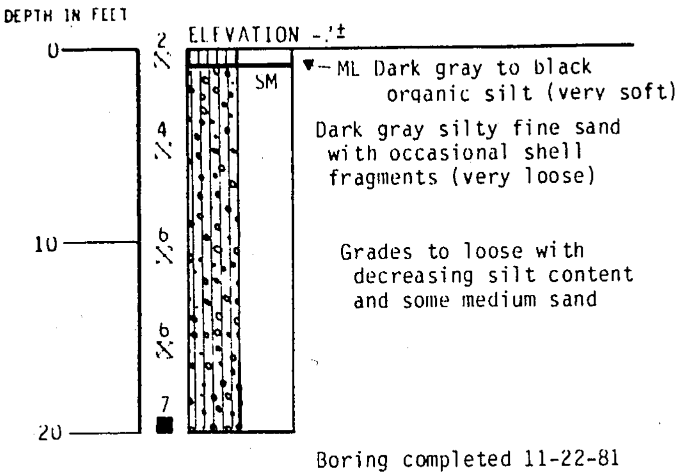
Boring BB-9-81



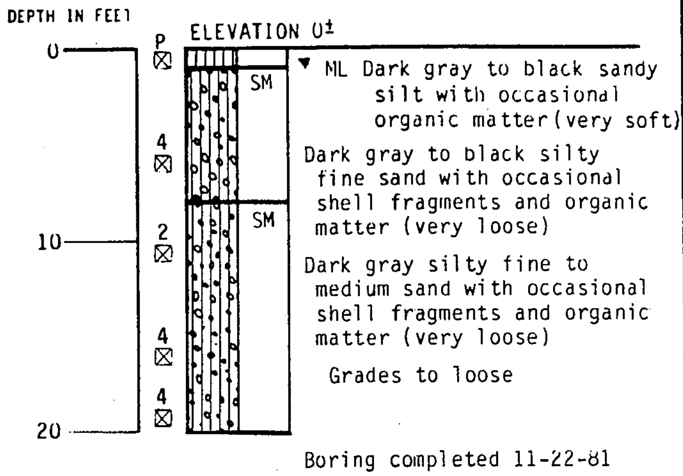
LOG OF BORINGS

Dames & Moore

Boring BB-10-81



Boring BB-11-81



LOG OF BORINGS

Dames & Moore

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
Coarse Grained Soils More than 50% of material is LARGER than No. 200 sieve size.	Gravel and Gravelly Soils More than 50% of coarse fraction RETAINED on No. 4 sieve.	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.
		Gravels with Fines (Appreciable amount of fines)		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clayey gravels, gravel-sand-clay mixtures.
	Sand and Sandy Soils More than 50% of coarse fraction PASSING No. 4 sieve.	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.
		Sands with Fines (Appreciable amount of fines)		SM	Silty sands, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures.
Fine Grained Soils More than 50% of material is SMALLER than No. 200 sieve size.	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 silt and organic silty clays of low plasticity.
	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.

Note: Dual symbols are used to indicate borderline soil classifications.

Unified Soil Classification System

Dames & Moore

BORING	DEPTH	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU. FT.	NORMAL PRESSURE LBS./SQ. FT.	PEAK SHEAR STRENGTH LBS./SQ. FT.	ULTIMATE SHEAR STRENGTH LBS./SQ. FT.
SB1	75'	Silty fine to coarse sand with gravel	20.7	105	1500	1400	1000
SB1	93'	Slightly silty fine to coarse sand	11.9	122			
			30.3	93	1500	1340	940
SB1	108'	Silty fine to medium sand			3000	2640	1850
					4500	3360	2350
SB2	4'	Silty fine to medium sand	25.4	91	300	470	330
					600	960	670
SB2	28'	Silty very fine sand and sandy silt	28.1	95	400	360	250
					800	700	500
					1200	1200	830
SB2	74'	Silty very fine to fine sand	32.1	88	2500	2100	1470
					3500	2760	1930
					4500	3420	2390
SB2	104'	Silty very fine sand	37.1	83	3000	2460	1720
					4000	3060	2140
					5000	3780	2640
SB3	34'	Slightly silty to silty fine to coarse sand	17.8	111	1500	1900	1330
					3000	2940	2060

SUMMARY OF DIRECT SHEAR TEST DATA

Job No. 6842-003-20

Dames & Moore

BORING	DEPTH	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU.FT.	NORMAL PRESSURE LBS./SQ.FT.	PEAK SHEAR STRENGTH LBS./SQ.FT.	ULTIMATE SHEAR STRENGTH LBS./SQ.FT.
SB3	76'	Silty fine to medium sand	29.9	92	3500 4500 5500	2940 3900 4380	2050 2730 3060
SB3	94'	Silty very fine to fine sand and sandy silt	27.5	97	4000 5000	3300 3960	2310 2770
SB5	68'	Silty fine to coarse sand	19.5	107	3000 3500 4000	2400 3120 3660	1680 2180 2560
SB6	64'	Silty fine to medium sand	17.3	114	2000 3000 4000	1740 2460 2820	1220 1720 1970

SUMMARY OF DIRECT SHEAR TEST DATA

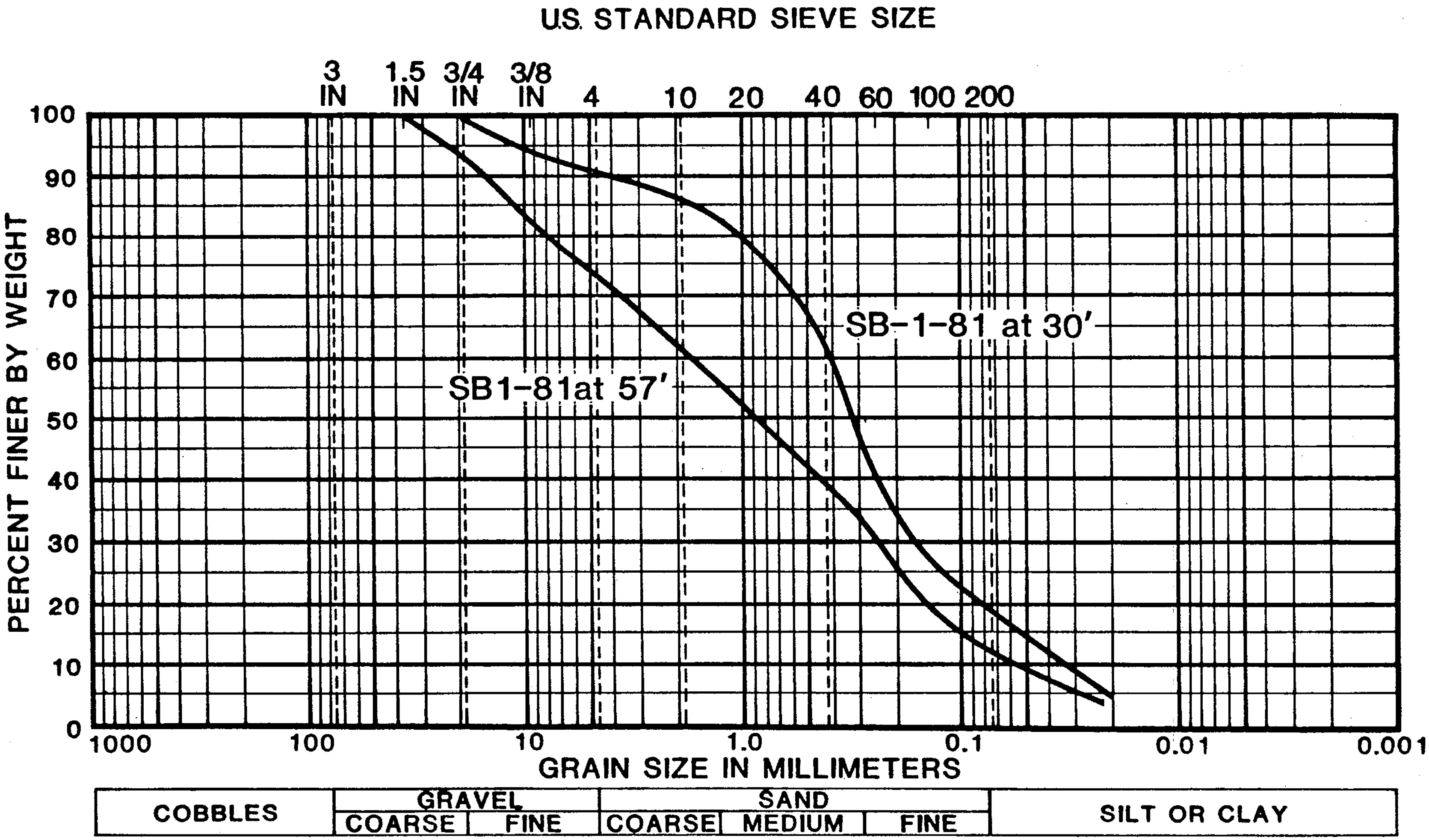
BORING	DEPTH	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU.FT.	CELL PRESSURE IN LBS./SQ.FT.	DEVIATOR STRESS* IN LBS./SQ.FT.
SB-1-81	25½'	SILTY FINE TO COARSE SAND	16.5 -	- -	1000 1000	1630** 6800**
SB-6-81	50½'	SILTY FINE TO COARSE SAND	12.4 -	- -	2000 2000	1400** 14400**
SB-6-81	58'	SILTY FINE TO COARSE SAND	18.1 -	- -	3000 3000	10900** 14000
		NOTE: Dry density not evaluated due to method of testing.				

*STRESS AT YIELD POINT
**SAMPLE REMOLDED

SUMMARY OF TRIAXIAL TEST DATA

Job No. 6842-003-20

Job No. 6842-003-20

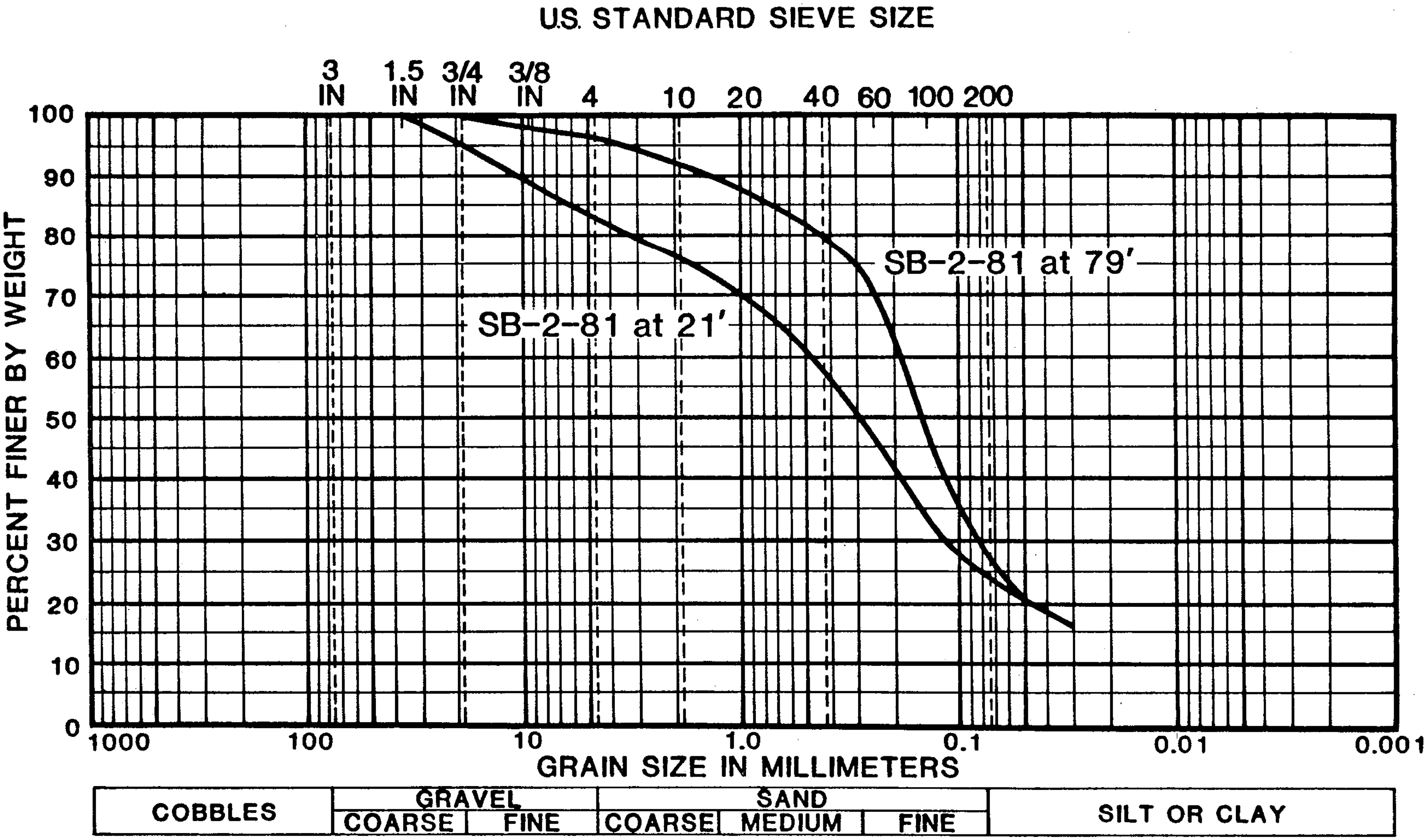


BORING	DEPTH	DESCRIPTION
SB-1-81	30'	Silty fine to coarse sand with occasional gravel
SB-1-81	57'	Silty fine to coarse sand with some gravel

GRADATION CURVE

Dames & Moore

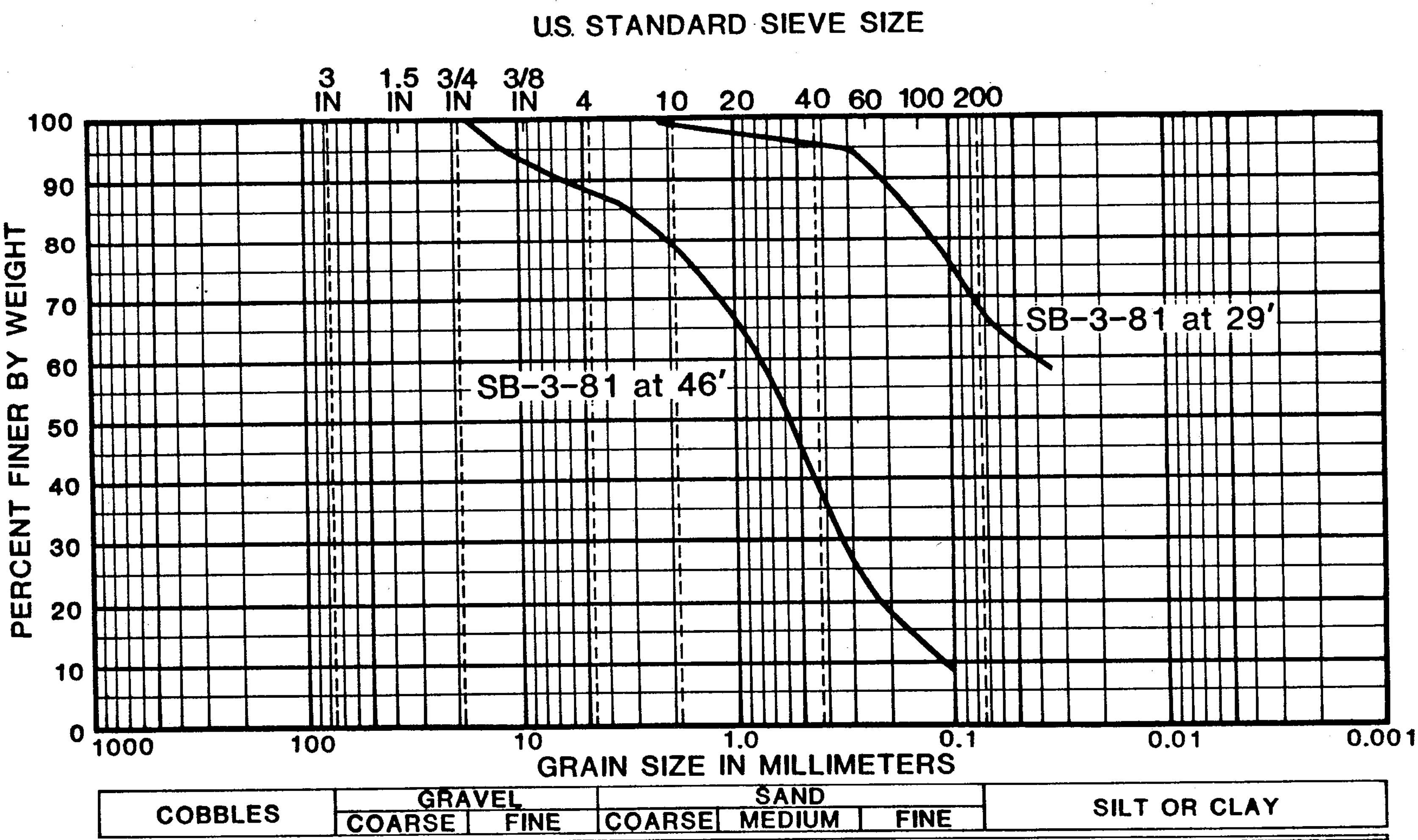
Plate B-17



BORING	DEPTH	DESCRIPTION
SB-2-81	21'	Silty fine to medium sand with occasional gravel
SB-2-81	79'	Silty very fine to fine sand with a trace of coarse sand and gravel

GRADATION CURVE

Job No. 6842-003-20

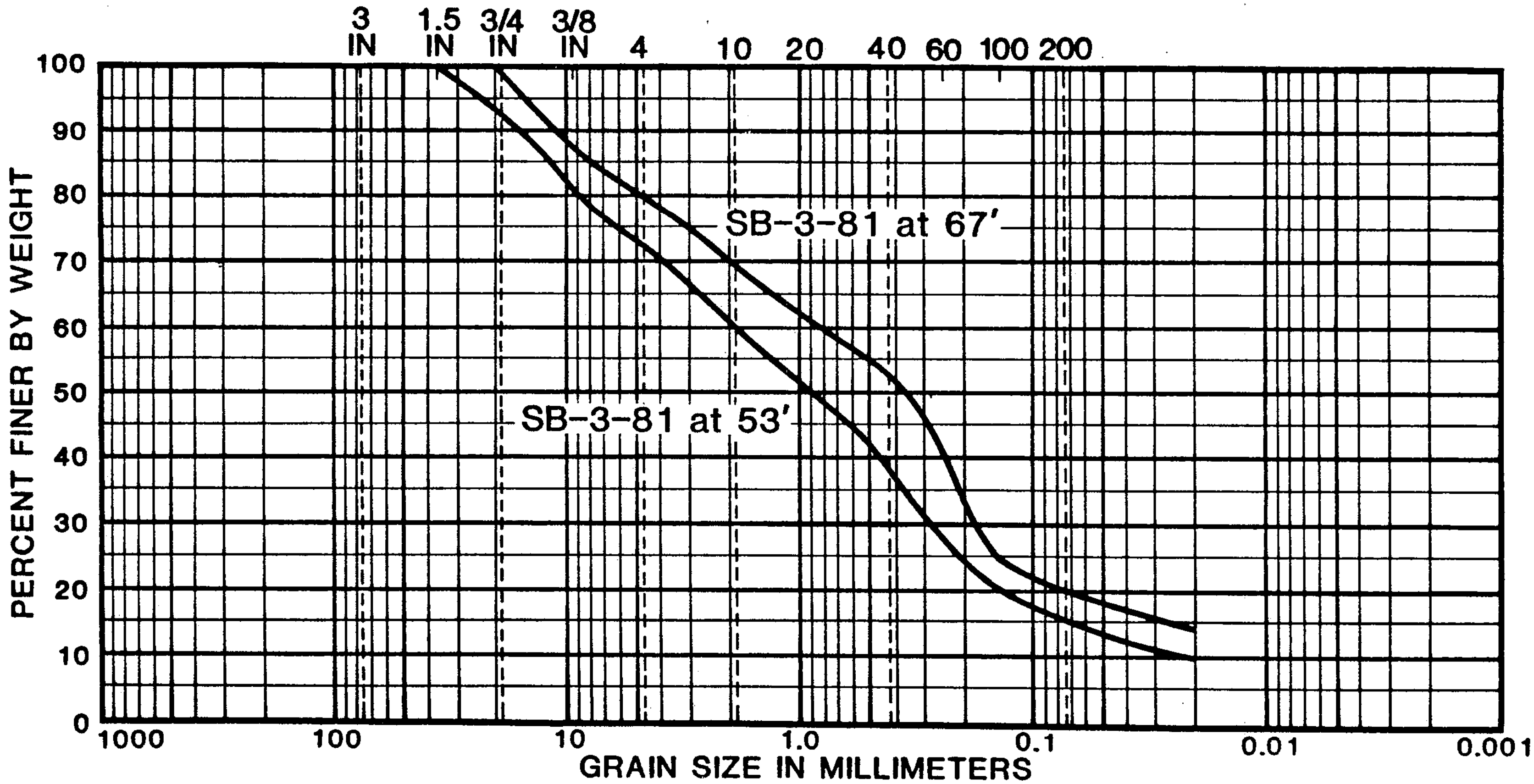


BORING	DEPTH	DESCRIPTION
SB-3-81	29'	Sandy silt
SB-3-81	46'	Silty fine to coarse sand with occasional gravel

GRADATION CURVE

Dames & Moore

Plate B-19

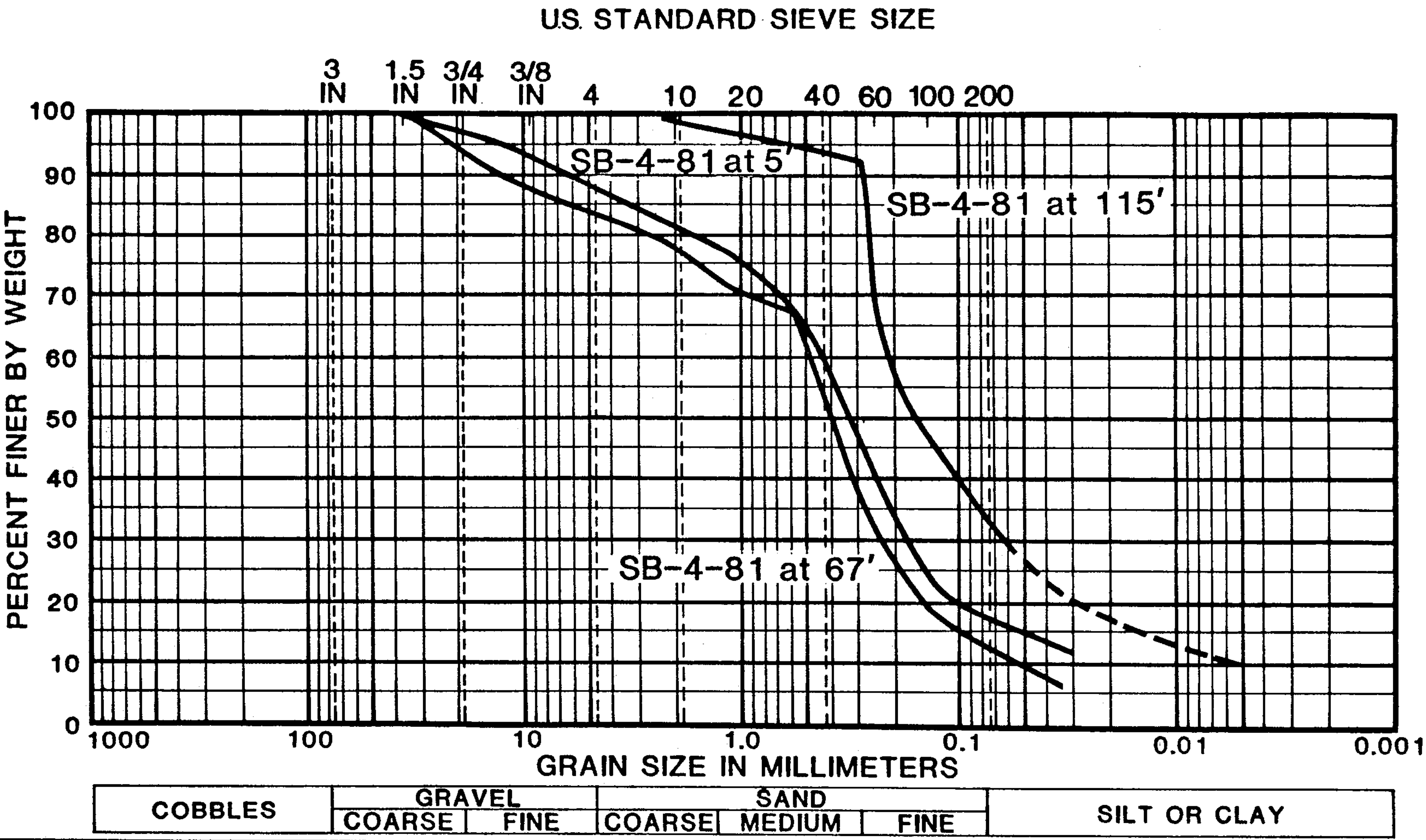


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
SB-3-81	53'	Silty fine to coarse sand with occasional gravel
SB-3-81	67'	Silty fine to coarse sand with occasional gravel

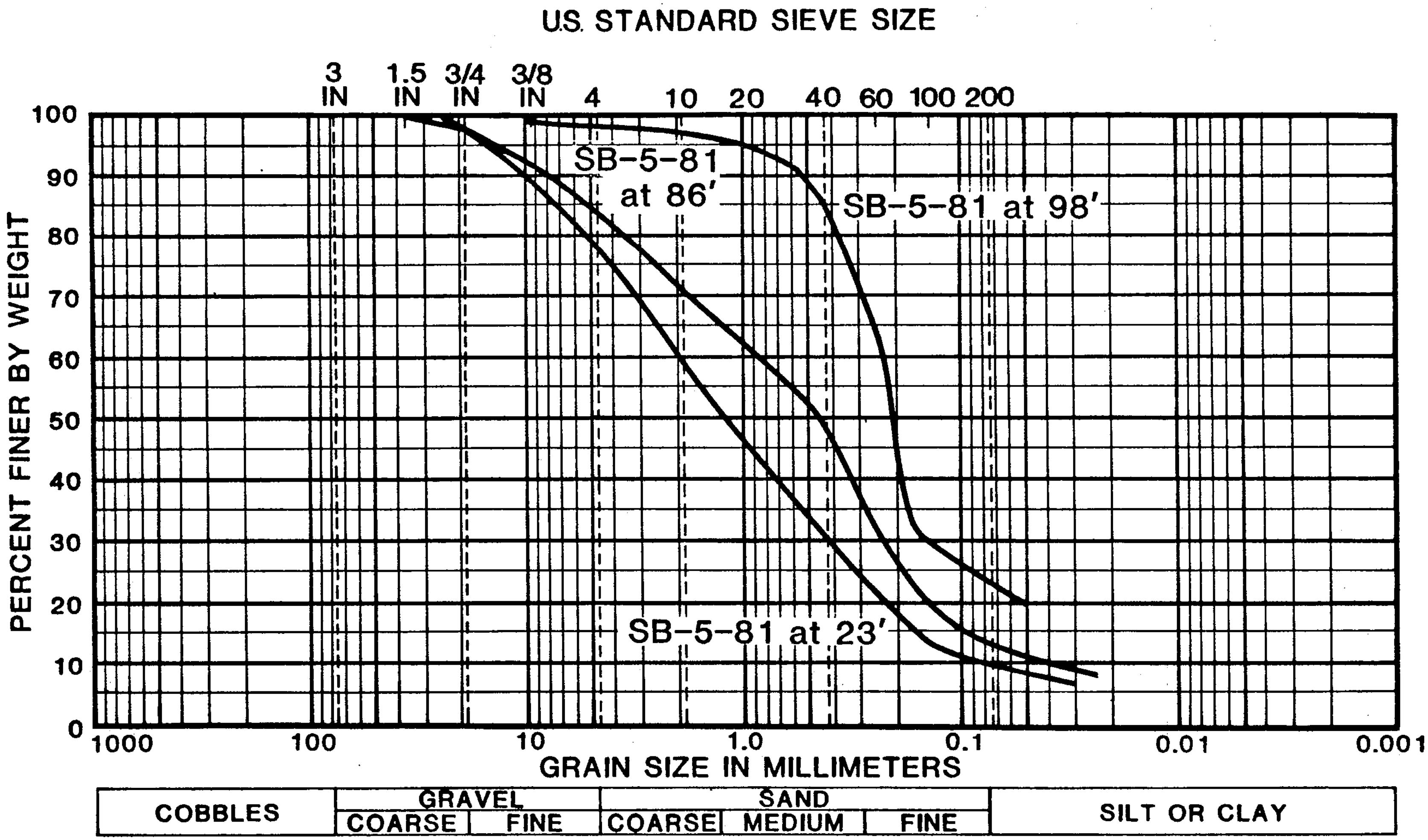
Dames & Moore

GRADATION CURVE



BORING	DEPTH	DESCRIPTION
SB-4-81	5'	Silty very fine to medium sand with gravel
SB-4-81	67'	Silty fine to coarse sand with some gravel
SB-4-81	115'	Silty very fine to fine sand

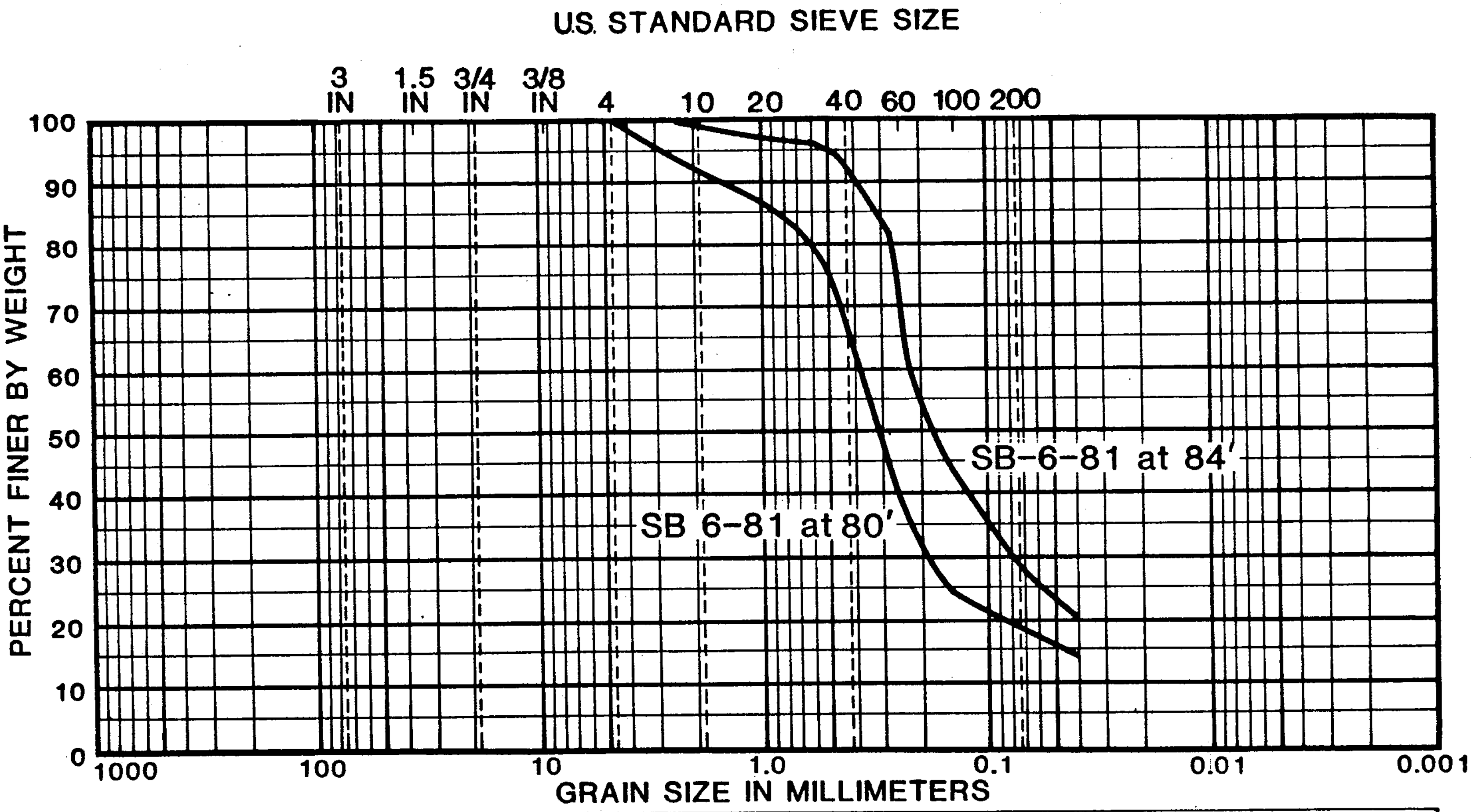
GRADATION CURVE



BORING	DEPTH	DESCRIPTION
SB-5-81	23'	Slightly silty fine to coarse sand with gravel
SB-5-81	86'	Silty fine to coarse sand with some gravel
SB-5-81	98'	Silty sand with a trace of gravel

Dames & Moore

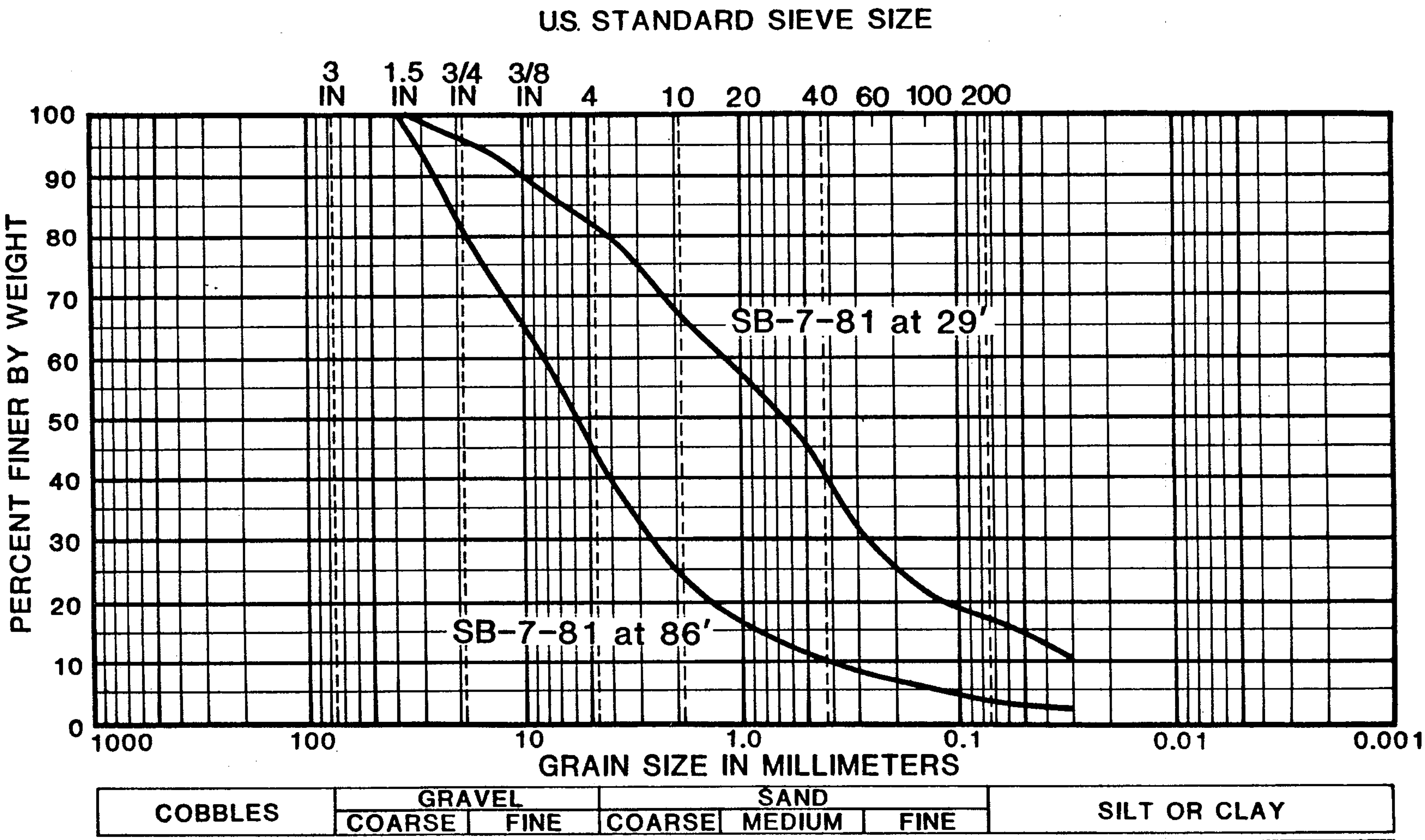
GRADATION CURVE



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
SB-6-81	80'	Silty fine to medium sand with occasional coarse sand
SB-6-81	84'	Silty very fine to fine sand

GRADATION CURVE



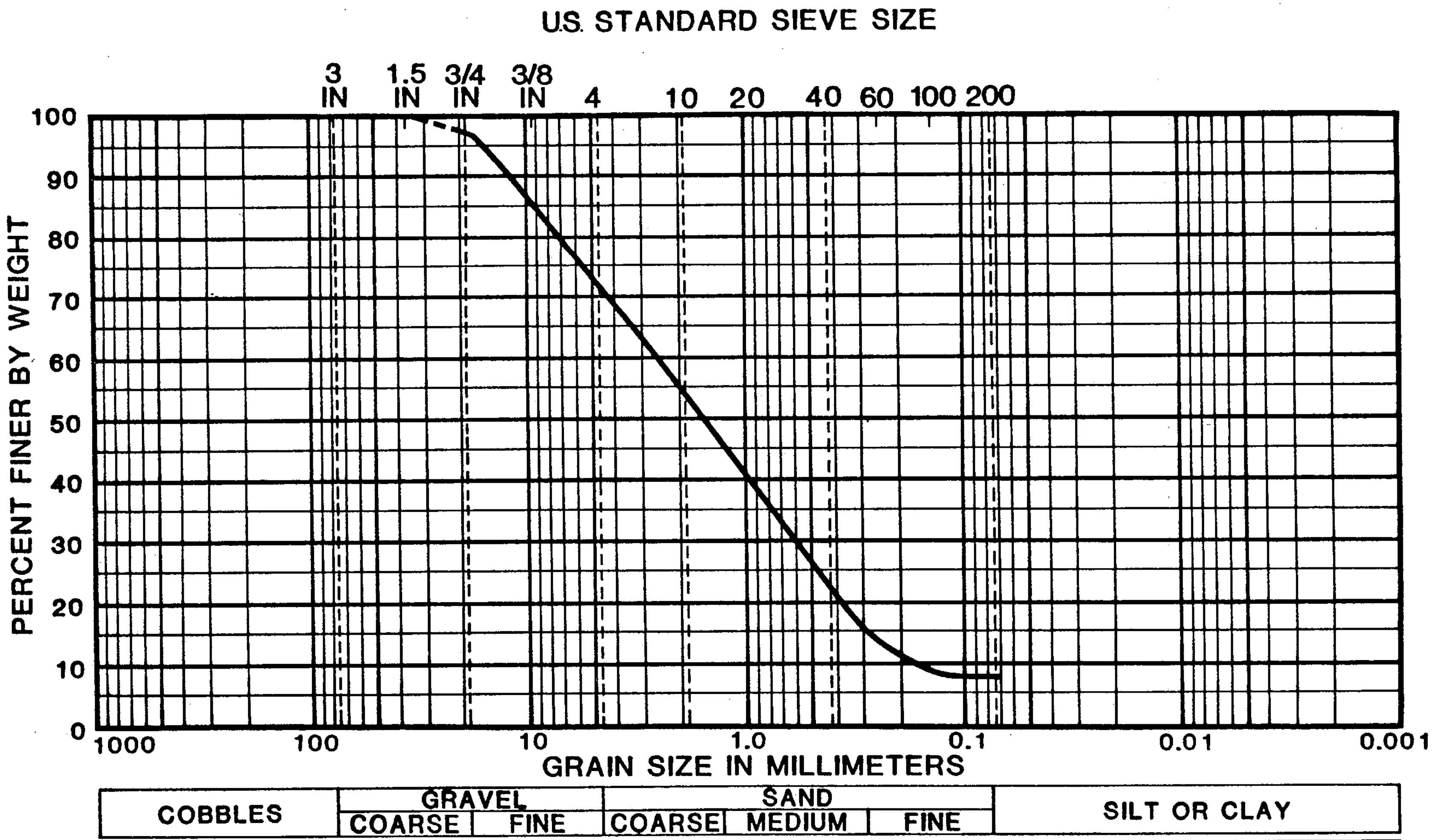
BORING	DEPTH	DESCRIPTION
SB-7-81	29'	Silty fine to coarse sand and gravel
SB-7-81	86'	Slightly silty fine to coarse sand and gravel

GRADATION CURVE

Job No. 6842-003-20

Dames & Moore

Plate B-24

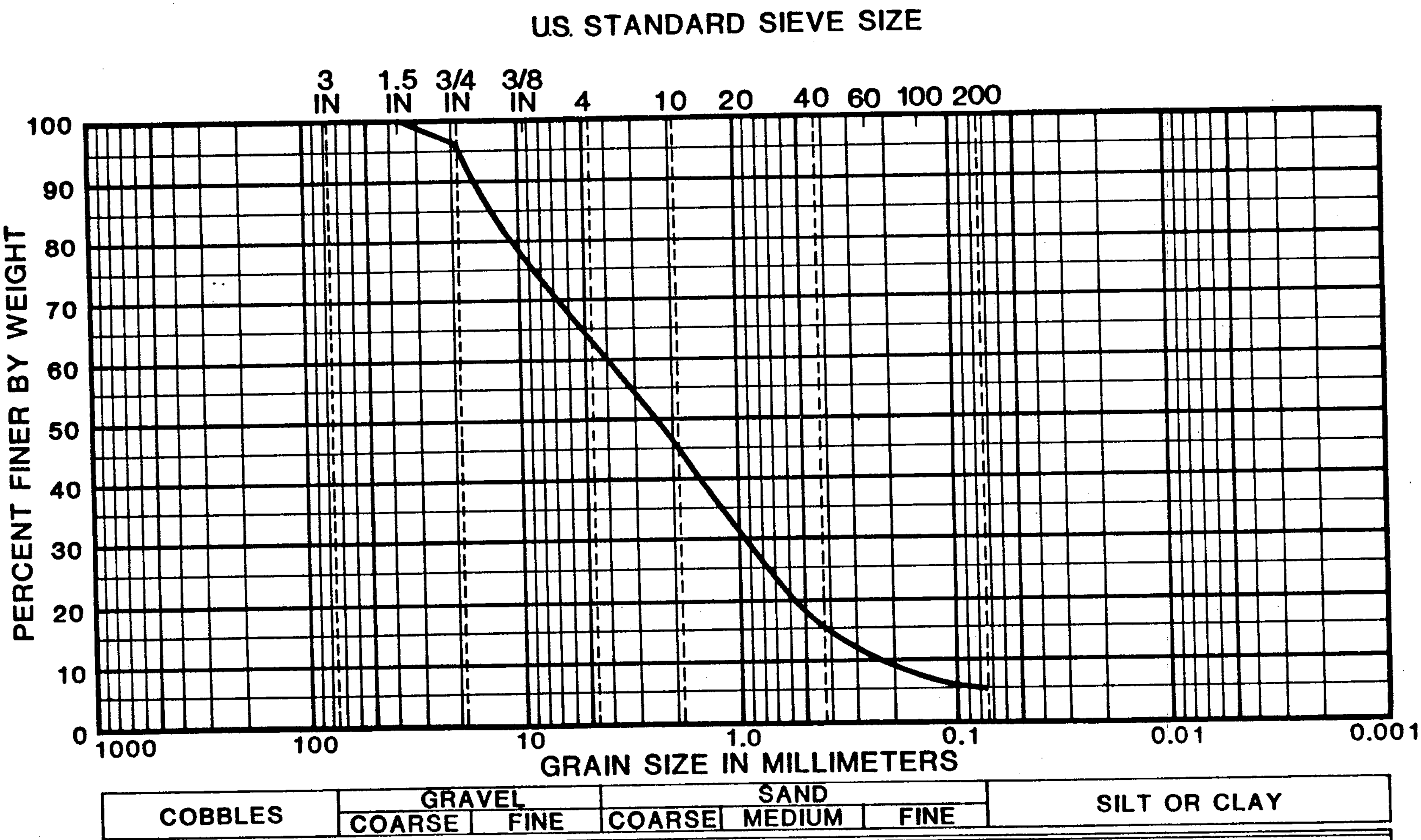


BORING	DEPTH	DESCRIPTION
SB-10-81	2'	Slightly silty fine to coarse sand with gravel

GRADATION CURVE

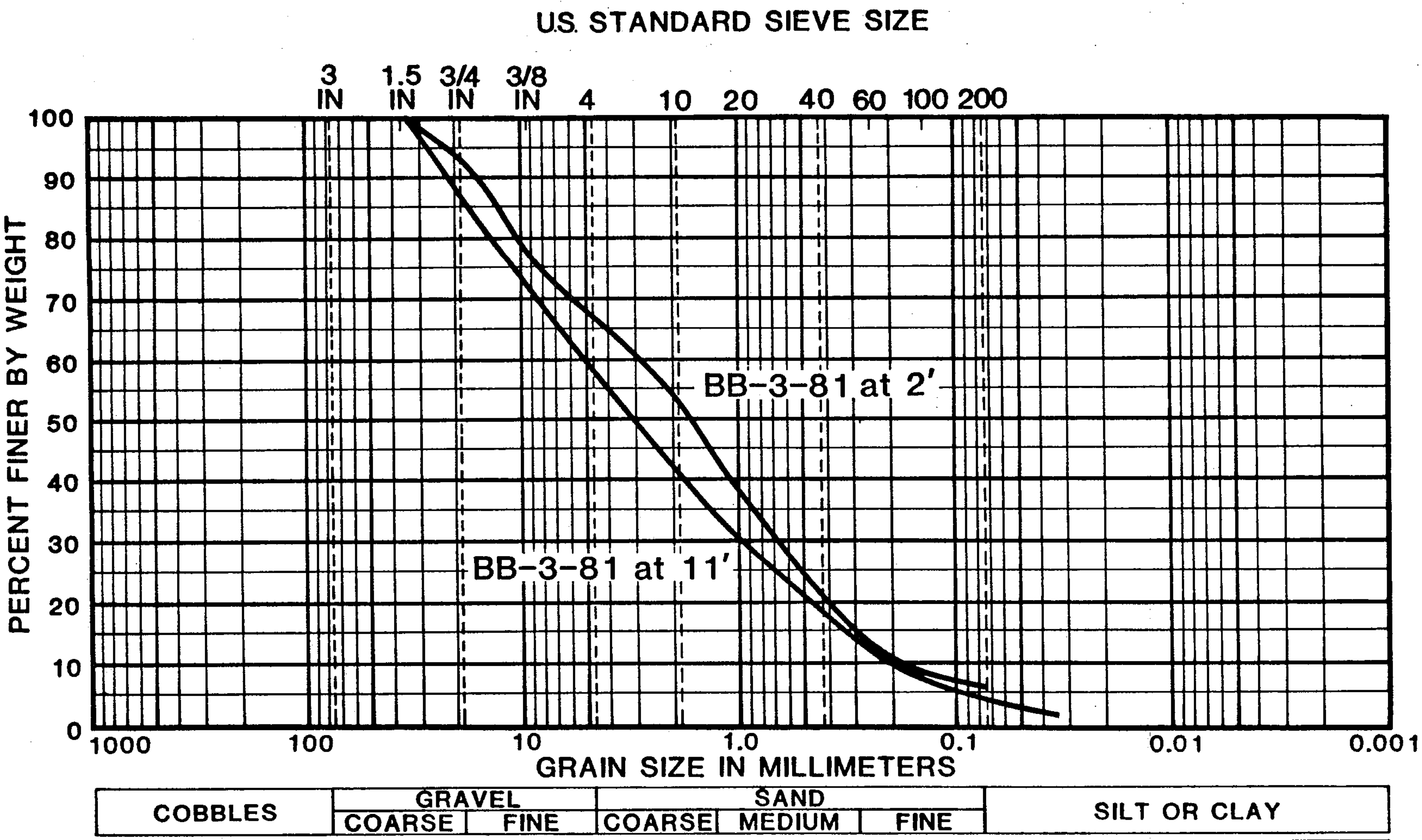
Job No. 6842-003-20

Dames & Moore



BORING	DEPTH	DESCRIPTION
BB-2-81	1'& 5'	Slightly silty fine to coarse sand with gravel

GRADATION CURVE



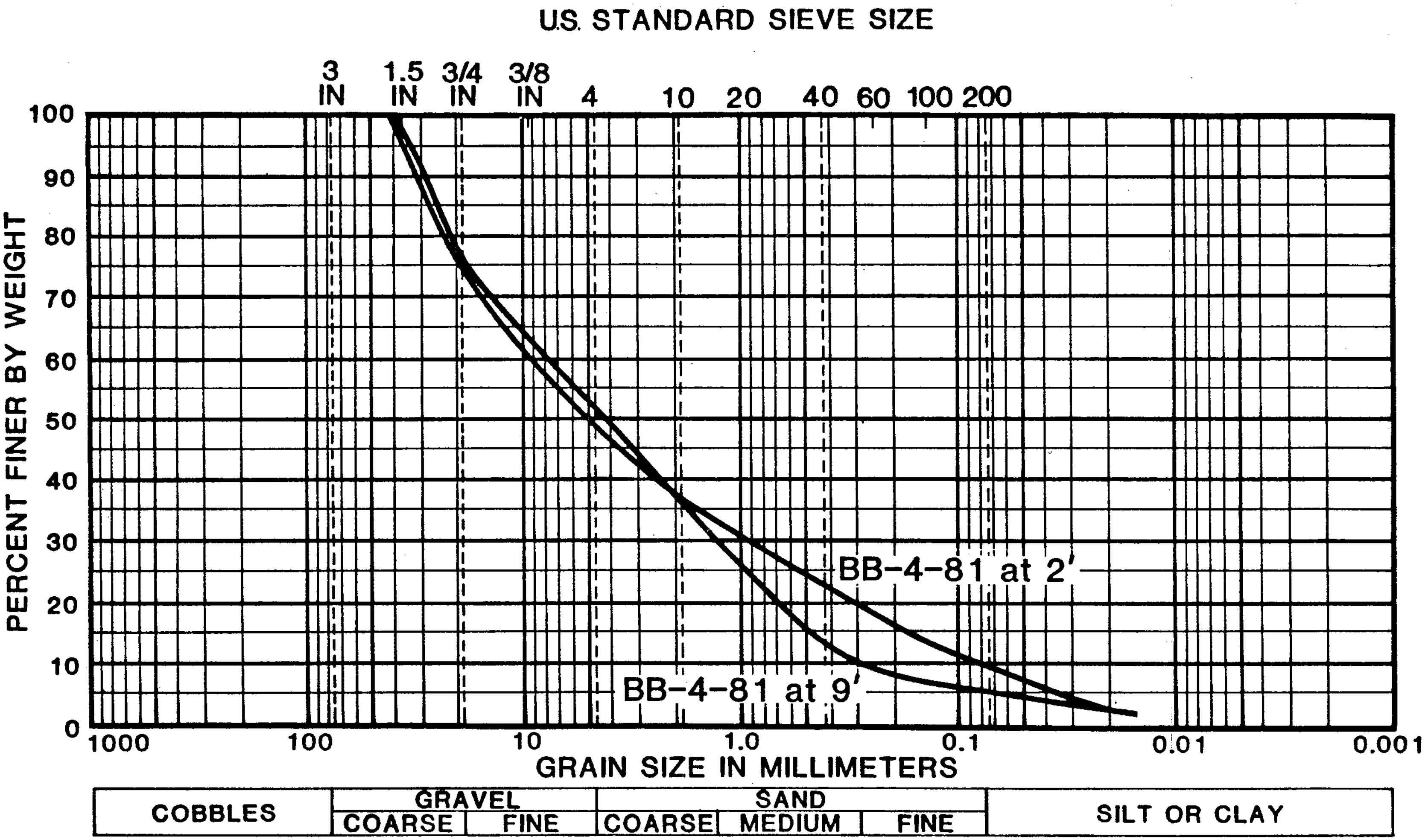
BORING	DEPTH	DESCRIPTION
BB-3-81	2'	Slightly silty fine to coarse sand with gravel
BB-3-81	11'	Slightly silty fine to coarse sand and gravel

GRADATION CURVE

Job No. 6842-003-20

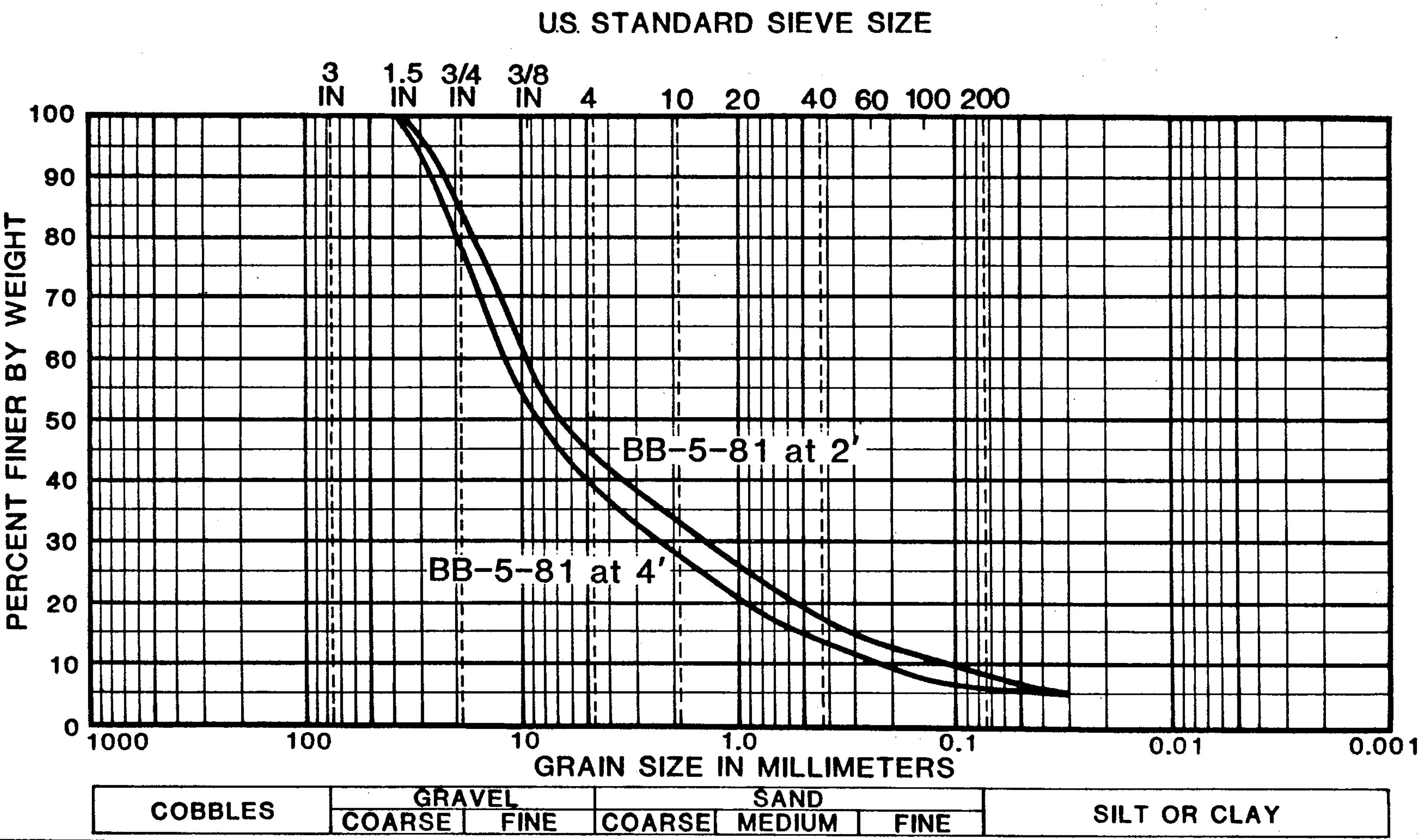
Dames & Moore

Plate B-27



BORING	DEPTH	DESCRIPTION
BB-4-81	2'	Silty fine to coarse sand and gravel
BB-4-81	9'	Slightly silty fine to coarse sand and gravel

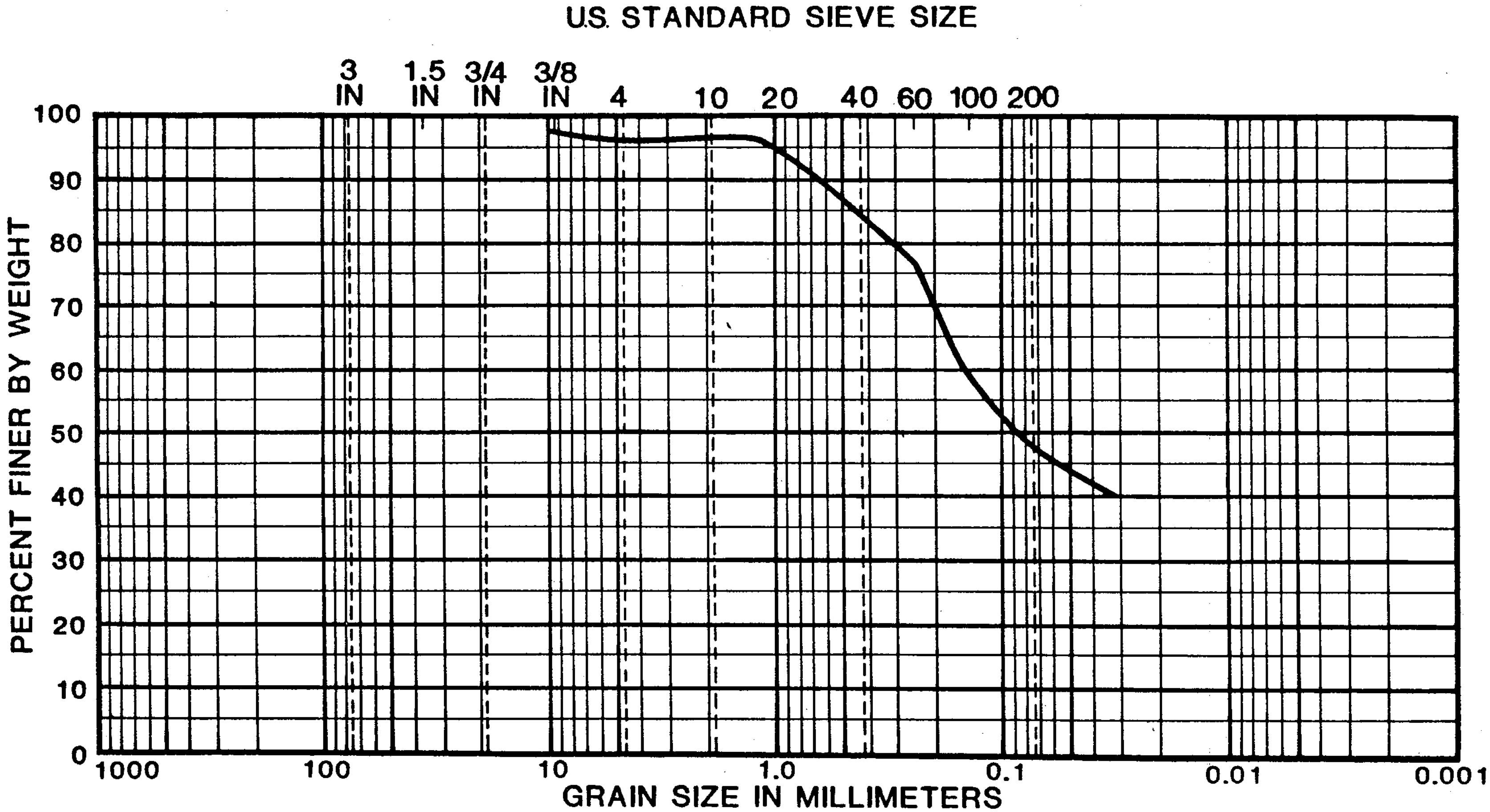
GRADATION CURVE



BORING	DEPTH	DESCRIPTION
BB-5-81	2'	Silty fine to coarse sand and gravel
BB-5-81	4'	Slightly silty fine to coarse sand and gravel

GRADATION CURVE

Job No. 6842-003-20

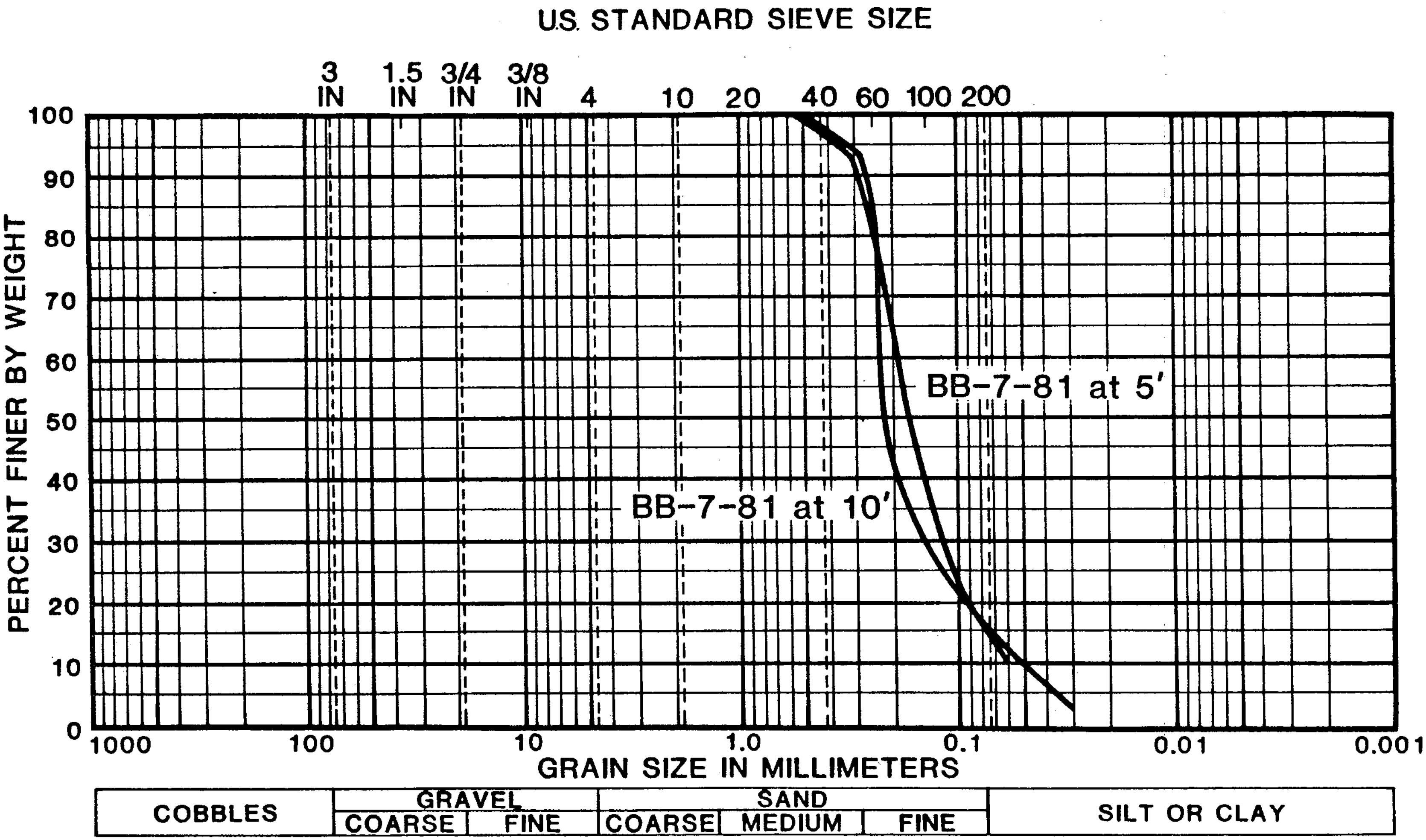


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
BB-6-81	5'	Silty fine to medium sand with a trace of gravel

Dames & Moore

GRADATION CURVE



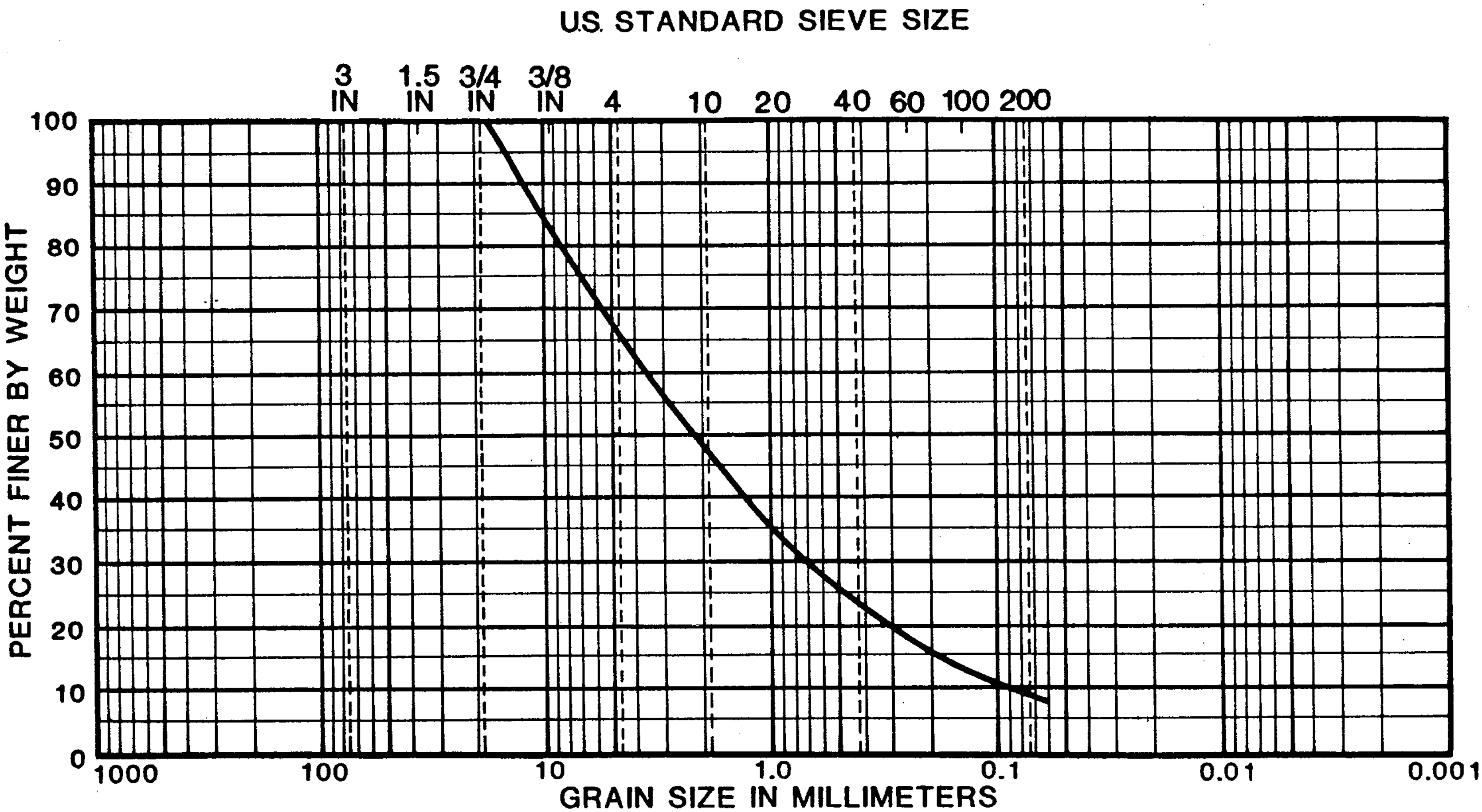
BORING	DEPTH	DESCRIPTION
BB-7-81	5'	Silty fine sand
BB-7-81	10'	Silty fine sand

GRADATION CURVE

Job No. 6842-003-20

Dames & Moore

Job No. 6842-003-20

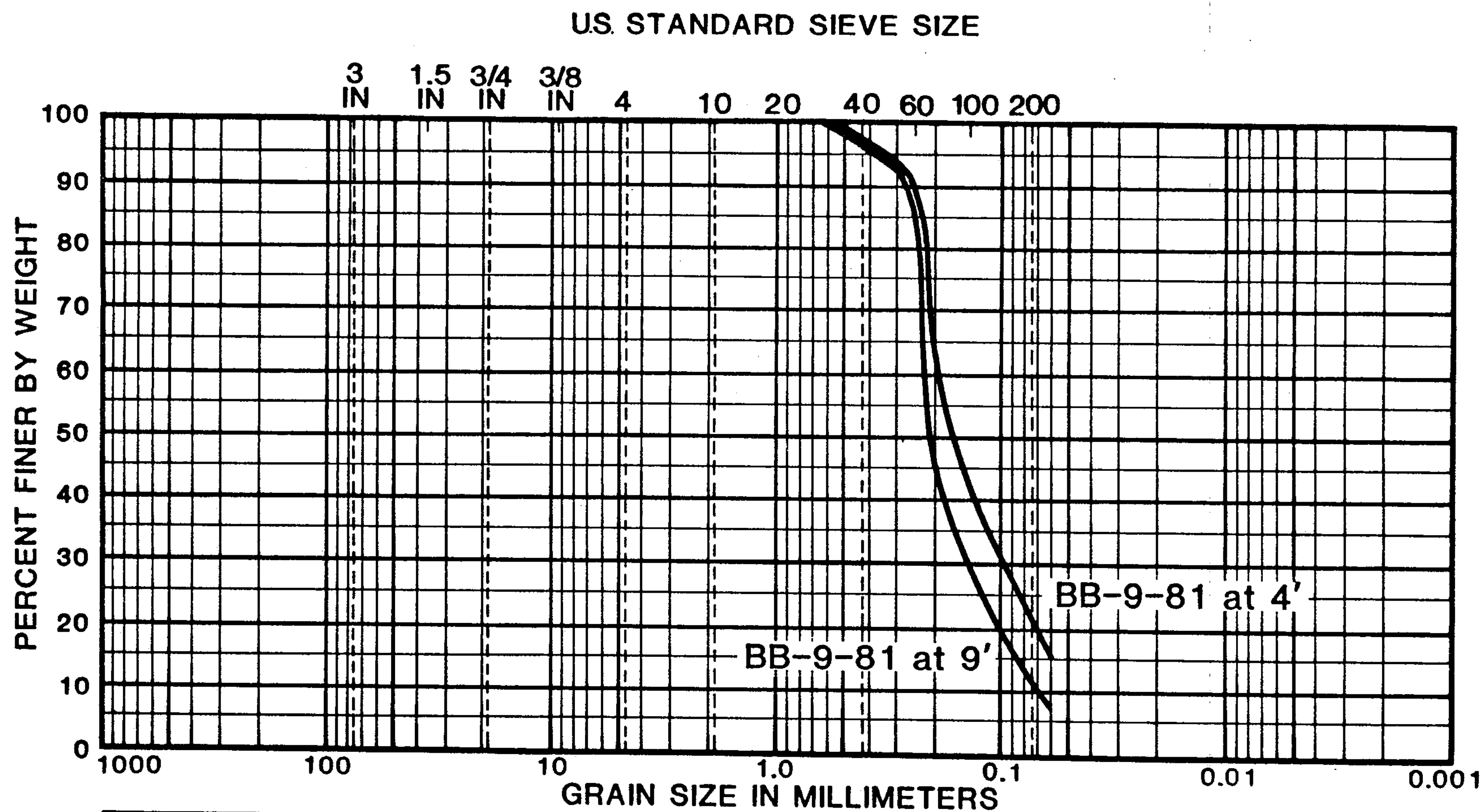


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
BB-8-81	9'	Silty sandy fine to medium gravel

GRADATION CURVE

Dames & Moore

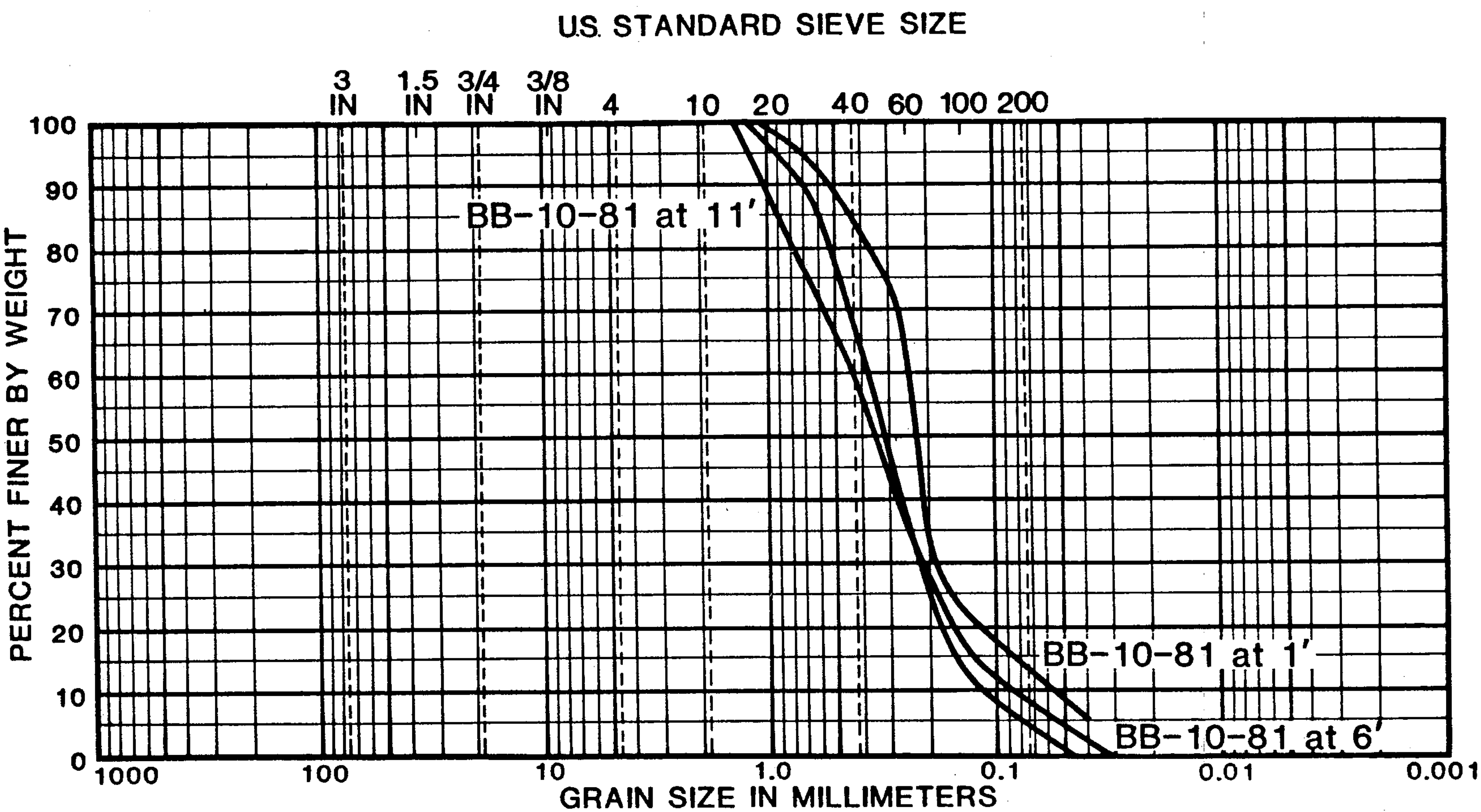


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
BB-9-81	4'	Silty fine sand
BB-9-81	9'	Silty fine sand

GRADATION CURVE

Dames & Moore



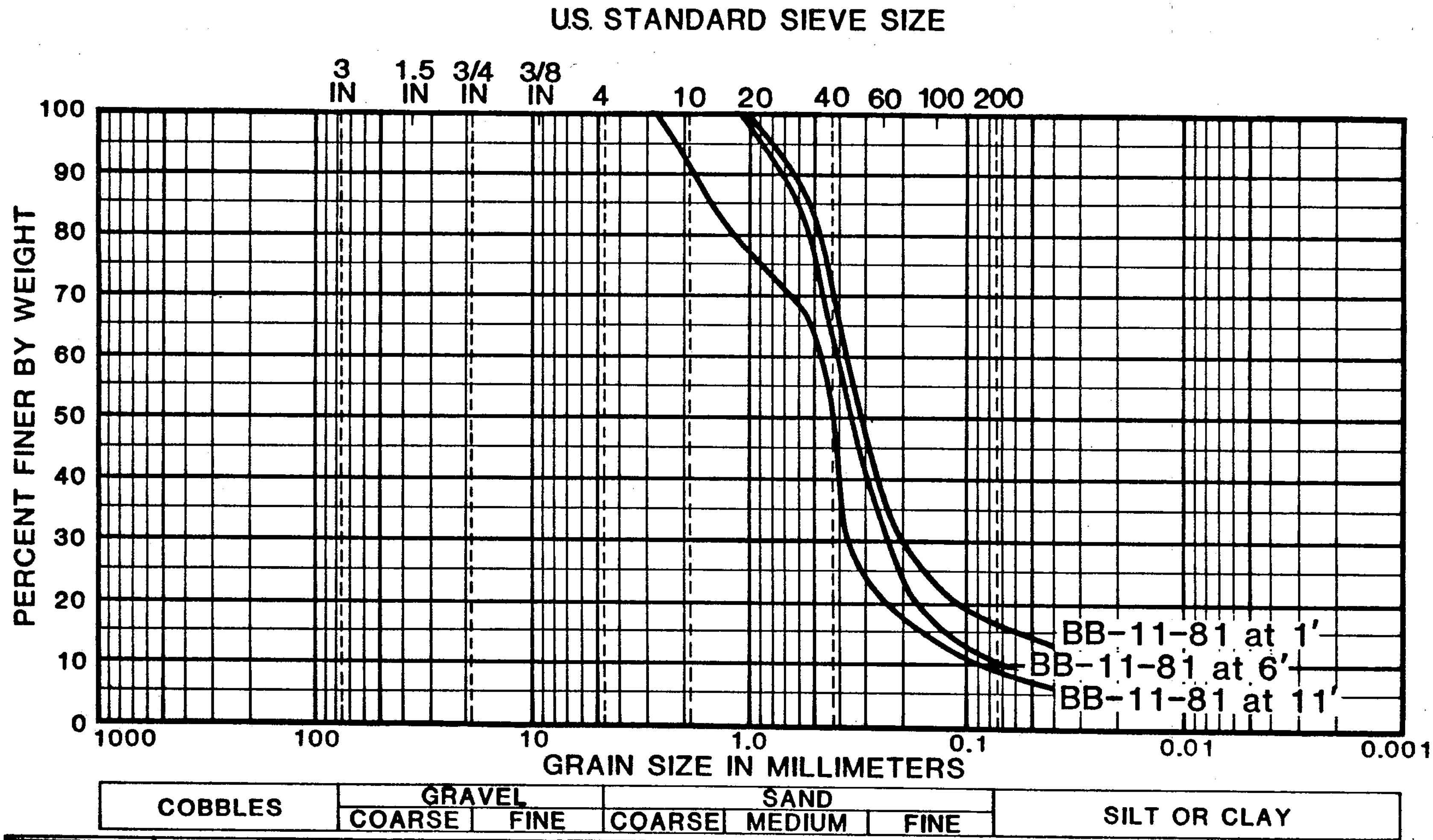
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
BB-10-81	1'	Silty fine sand
BB-10-81	6'	Slightly silty fine sand with some medium sand
BB-10-81	11'	Silty fine sand with some medium sand

GRADATION CURVE

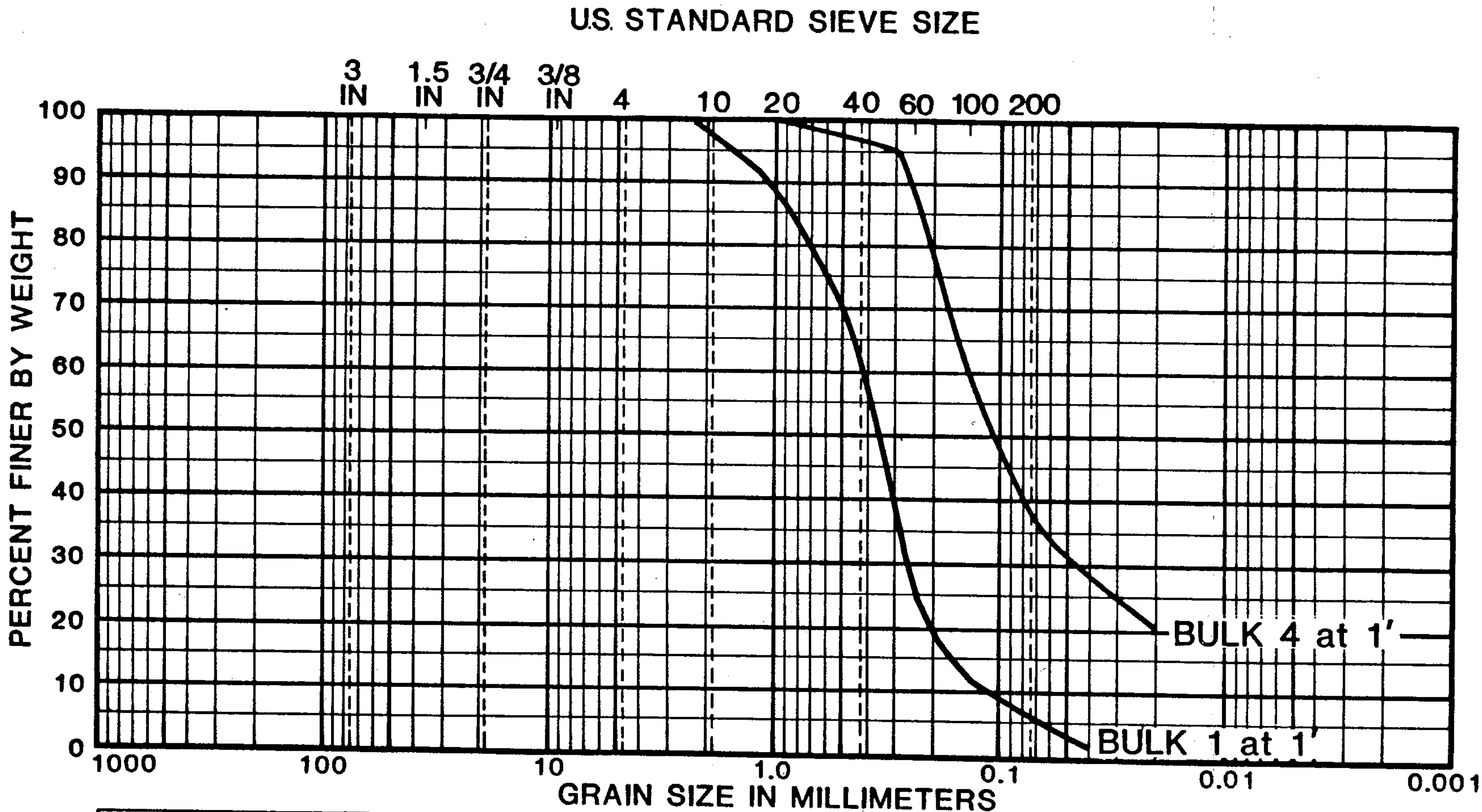
Job No. 6842-003-20

Dames & Moore



BORING	DEPTH	DESCRIPTION
BB-11-81	1'	Silty fine sand
BB-11-81	6'	Silty fine to medium sand
BB-11-81	11'	Silty fine sand with medium sand

GRADATION CURVE



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

BORING	DEPTH	DESCRIPTION
BULK 1	1'	Slightly silty fine to medium sand
BULK 4	1'	Silty fine sand

GRADATION CURVE

SAMPLE NO. Bulk 1 DEPTH 0-1' ELEVATION

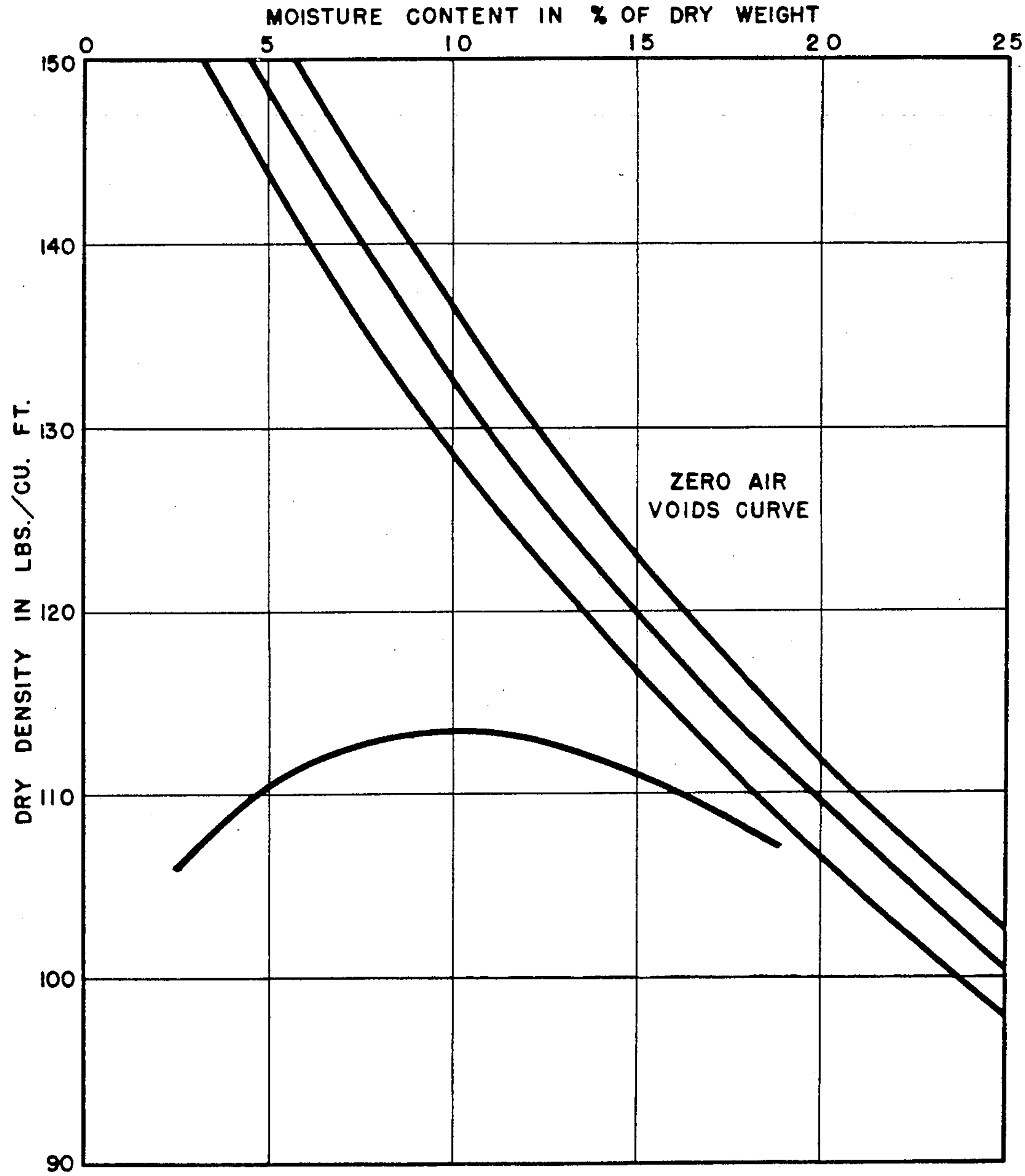
SOIL Slightly silty fine to medium sand

LOCATION

OPTIMUM MOISTURE CONTENT

MAXIMUM DRY DENSITY

METHOD OF COMPACTION



COMPACTION TEST DATA

DAMES & MOORE

SAMPLE NO. Bulk 1 DEPTH 1-2' ELEVATION

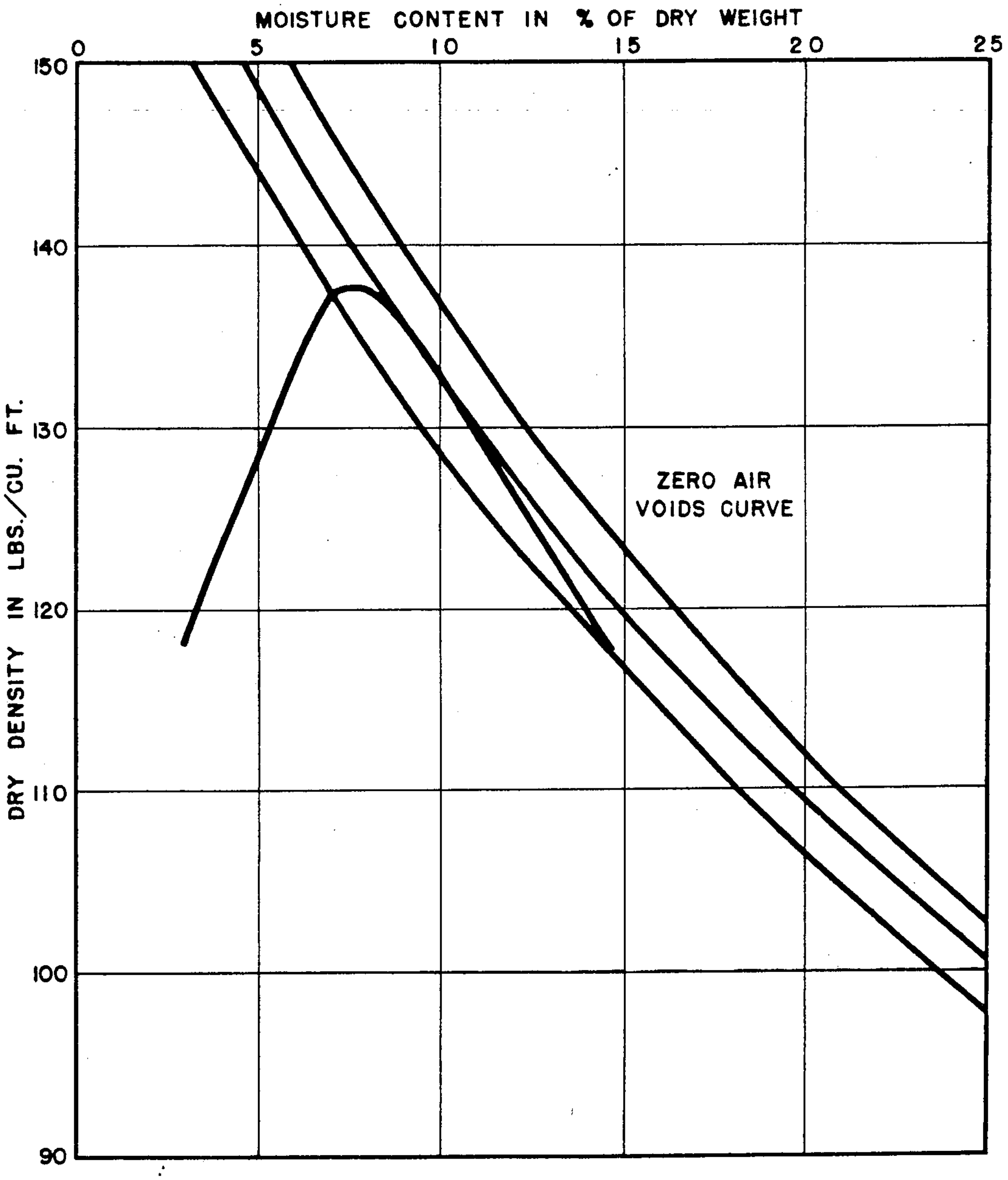
SOIL Fine to coarse sand and gravel

LOCATION

OPTIMUM MOISTURE CONTENT

MAXIMUM DRY DENSITY

METHOD OF COMPACTION



COMPACTION TEST DATA

DAMES & MOORE

SAMPLE NO. Bulk 3 DEPTH 1-2' ELEVATION

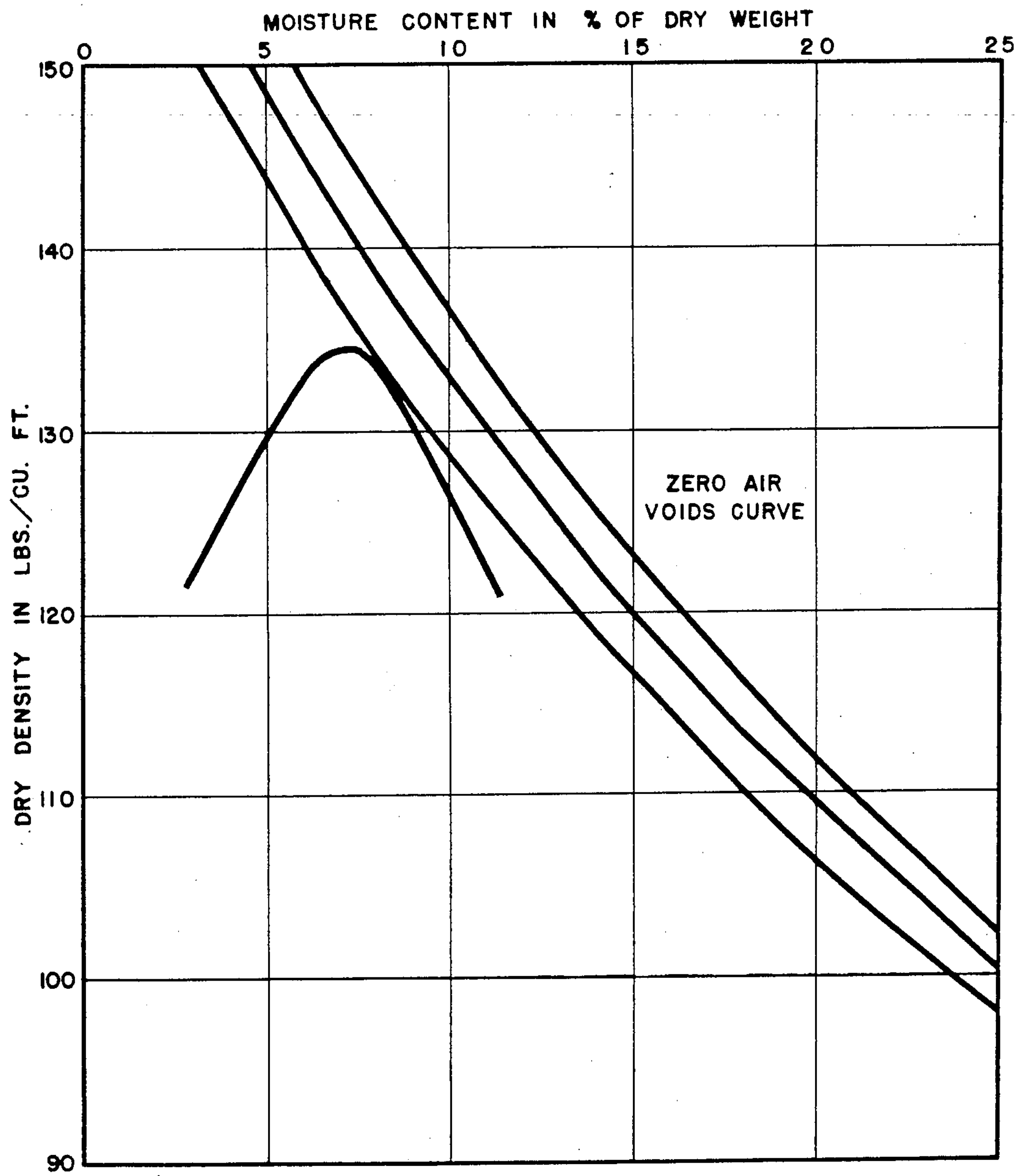
SOIL Silty fine to coarse sand and gravel

LOCATION

OPTIMUM MOISTURE CONTENT

MAXIMUM DRY DENSITY

METHOD OF COMPACTION



COMPACTION TEST DATA

DAMES & MOORE

SAMPLE NO. Bulk 4 DEPTH 1' ELEVATION

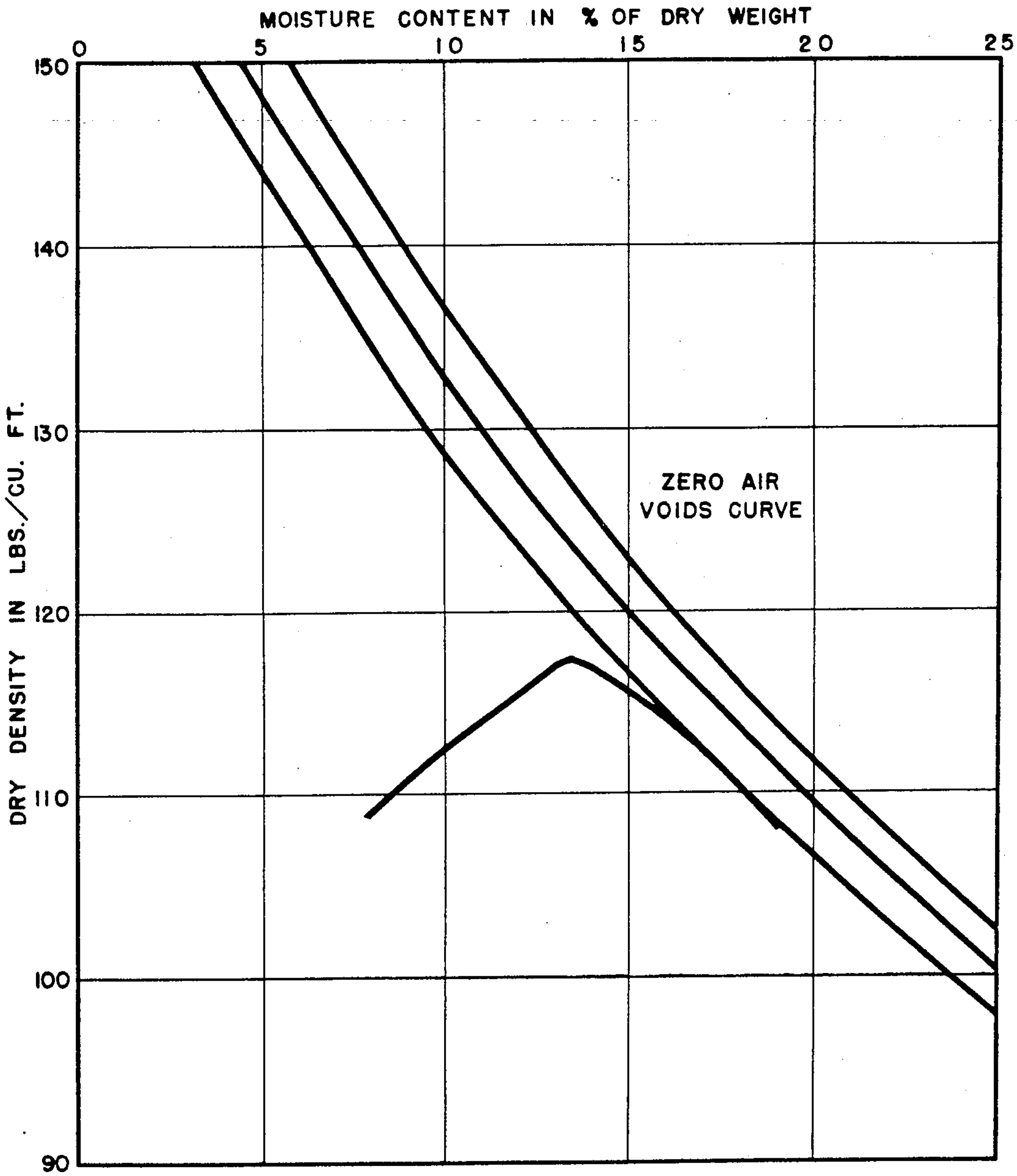
SOIL Silty fine sand

LOCATION

OPTIMUM MOISTURE CONTENT

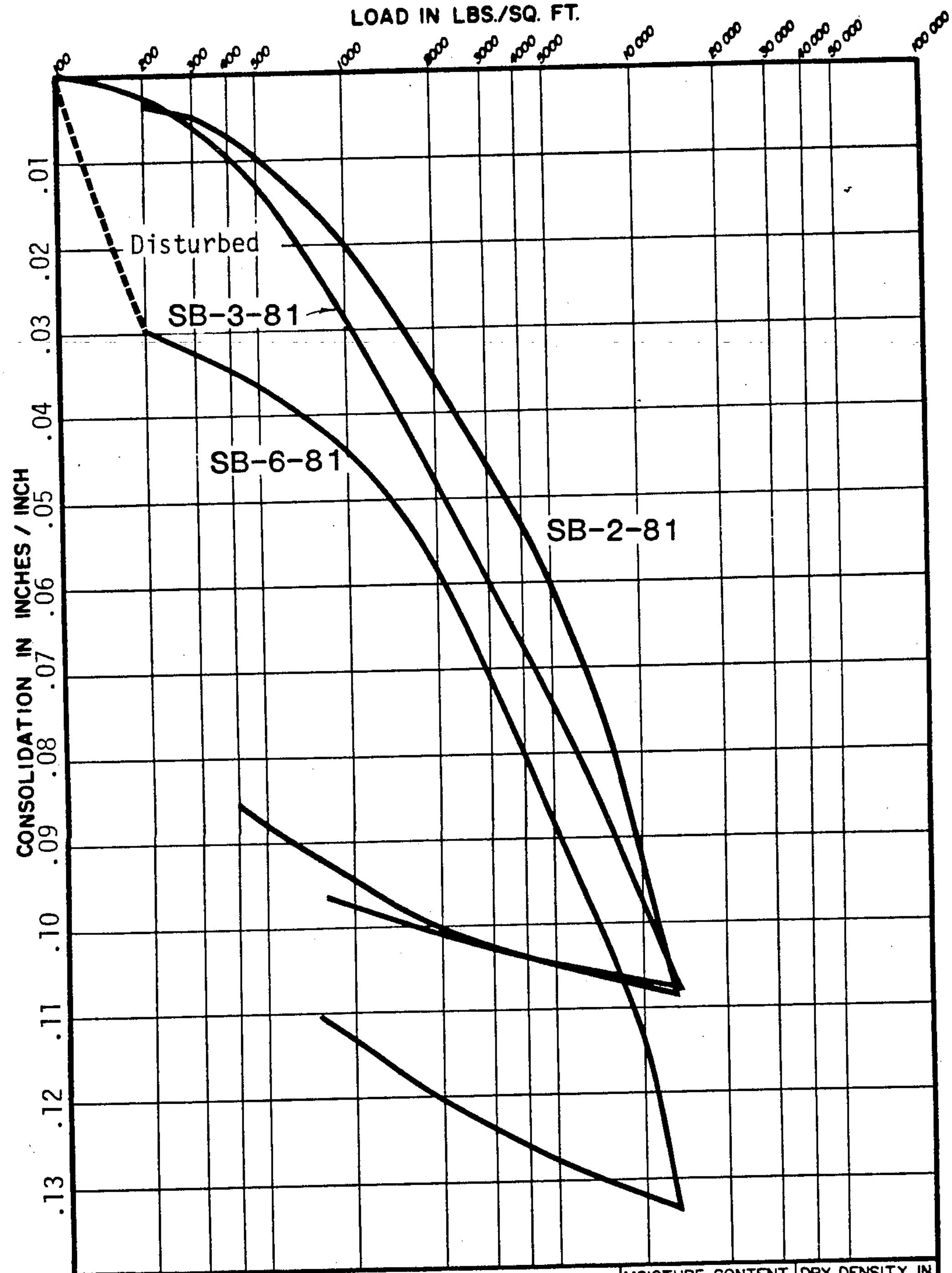
MAXIMUM DRY DENSITY

METHOD OF COMPACTION



COMPACTION TEST DATA

DAMES & MOORE



BORING	DEPTH	SOIL TYPE	MOISTURE CONTENT BEFORE	MOISTURE CONTENT AFTER	DRY DENSITY IN LBS./CU. FT.
SB-2-81	89'	Silty very fine sand & sandy silt	42.2%	36.7%	80
SB-3-81	124'	Sandy silt	57.0%	42.2%	64
SB-6-81	117'	Sandy silt	43.3%	40.0%	77

CONSOLIDATION TEST DATA

Dames & Moore

APPENDIX C

DESIGN PARAMETERS AND PROCEDURES FOR LIQUEFACTION STUDY

SEISMIC DESIGN PARAMETERS

Seismic History

Earthquakes which have occurred in the vicinity of Juneau are thought to be the result of movement along the Fairweather fault, which is located approximately 90 miles west of Juneau. This fault is one of five located within about 100 miles of the project area. Each of the four remaining faults have shown no apparent movement during the last one million years and are considered quiescent with the possible exception of the Lynn Canal-Chatham Strait fault which is located about 25 miles from Juneau.

The Fairweather fault is known to be responsible for earthquakes with Richter magnitudes of 8 or more. A study completed by the Alaska Department of Transportation and Public Facilities (DOTPF) for the new Gastineau Channel Bridge concludes that an earthquake with a maximum magnitude of 8.5 has a significantly high probability of occurrence along the Fairweather fault during a 50-year project life. The report concluded also that the recurrence interval of that earthquake is on the order 255 years. For the Gastineau Channel Bridge, this event was termed the "operating earthquake."

The Lynn Canal-Chatham Strait fault, which is located approximately 25 miles from Juneau, has been attributed by some researchers to the active Denali Fault system. It has been postulated by researchers that this fault may be active and capable of inducing moderate earthquakes with a Richter magnitude on the order of 6.5 at a distance of about 25 miles from Juneau. This event with a recurrence interval of about 500 years was termed in the DOTPF report as the "contingency earthquake."

Ground Acceleration and Earthquake Magnitude

An estimate of the horizontal ground acceleration induced by an earthquake which may occur along a bedrock fault is required for stability analysis, evaluation of liquefaction potential, and site period analysis. Normally, horizontal ground acceleration at a specific site is associated with the proximity of the active bedrock fault system, magnitude of the event which may occur along that system, and probability of occurrence. For a given magnitude earthquake, the horizontal ground acceleration typically is greater as the distance from the site to the causative fault decreases.

The Nuclear Regulatory Commission (NRC) has defined three levels of earthquake hazard which are used during design and operation of nuclear reactors. Each of the three levels is associated with a probability of occurrence and severity of horizontal ground acceleration at the site. Design of major structures and facilities now routinely incorporates the definition of earthquake hazard established by the NRC.

Generally, the "operating earthquake" (maximum probable) is related to the highest probability of occurrence but least severity, the "contingency earthquake" (maximum credible) is associated with a lower probability of occurrence but greater severity, and the "probabilistic earthquake" (maximum conceivable) with the lowest probability of occurrence and greatest severity. The earthquake used for design of a structure may be either the operating earthquake or the contingency earthquake, depending on the importance and cost of the structure, value of ground acceleration related to each level of earthquake hazard, and other factors. The horizontal ground acceleration for the operating earthquake is usually lower than that of the contingency earthquake. The horizontal ground acceleration is based on the earthquake magnitude (generally expressed as a value ranging from 1 to about 9 on the Richter scale), distance of the site from the causative fault, and attenuation of the soil mass which underlies the site.

Site-specific earthquake design criteria were not available for the Gold Creek Reclamation Project site. However, we have reviewed studies completed by the DOTPF and their consultants during design of the new Gastineau Channel Bridge. The parameters which we have considered are tabulated below:

	Horizontal Ground Acceleration (percent g)[(a)]		Earthquake Magnitude Richter Scale	
Source	Operating Earthquake	Contingency Earthquake	Operating Earthquake	Contingency Earthquake
Alaska DOTPF report dated December 1976	0.19	0.23	8.5	6.5
"Evaluation of Liquefaction Potential, Gastineau Channel Bridge No. 740" by Woodward-Clyde Consultants June 13, 1977	0.15	0.18	8.5	6.5

(a) g = the acceleration due to gravity
(32.174 ft/sec/sec = 1.0 g.)

The ground acceleration values presented by Woodward-Clyde Consultants in their report were determined using more current procedures than those outlined in the DOTPF report. Accordingly, we have adopted for our study the values of 0.15 and 0.18 g.

The value of horizontal ground acceleration controls slope stability analysis while horizontal ground acceleration combined with the duration of ground shaking (which generally increases as the magnitude of the earthquake increases) generally controls liquefaction studies. It is important to note that the contingency earthquake results in a higher ground acceleration than that of the operating earthquake due to distance from the site. However, the duration of ground shaking will be greater for the operating earthquake as a result of the greater magnitude.

We have used a horizontal ground acceleration (a) of 0.15 g (highest probability occurrence) for the slope stability analyses and have considered a = 0.18 g (lower probability of occurrence). The liquefaction studies have been based on a = 0.15 g (highest probability of occurrence, long duration). The contingency earthquake, a = 0.18 g, was not considered because the duration of ground shaking and, therefore, influence on liquefaction potential would be less than that of the operating earthquake.

The basis for values of ground acceleration used during our analyses is the seismic evaluation completed by the DOTPF. That study incorporated deterministic and probabilistic procedures to assess the probability, magnitude, and location relative to the bridge site of seismic events. The study defined the operating earthquake and contingency level earthquakes for the bridge site. However, the results of the study infer that the events selected are both contingency level events.

Site Period

Determination of the UBC seismic site response coefficient (S) is dependent on the fundamental period of the structure (T) and the site period (T_S). We have evaluated the site period using dynamic properties of the soil materials obtained from empirical relationships based on soil void ratio and our knowledge of seismic parameters for similar soil conditions. As previously noted, soil conditions consist of very loose to medium dense sand. These soils are expected to overlie very dense glacial till, which in turn overlies bedrock at depths of approximately 170 to 210 feet below current grades.

The multi-layer method specified by the International Conference of Building Officials (1976) was used to calculate the site period for the Gold Creek Reclamation Project area. Based on our analyses, we estimate that the site period (in seconds) is in the range of 0.9 to 1.1. The range of site period should be evaluated during structural design with the more restrictive result being utilized. We wish to emphasize, however, that the uncertainty of the calculation to arrive at a value for site period should be recognized. Variations in soil properties can result in an error of ± 15 percent in the estimate of site period. The computational methods introduce additional uncertainties such that use of the site period ranging from 20 percent less to 20 percent greater than the calculated value may be appropriate.

SOIL PARAMETERS

Soil unit weights were selected from laboratory test results for natural soils underlying the project area. The soil strength parameter required for input into the slope stability computer program is the angle of internal friction. The sampling methods implemented during site exploration, in addition to transport of samples from site to laboratory, resulted in significant sample disturbance. In general, loose soil samples become more dense and the structure of dense soils is disturbed. This type of sample disturbance may significantly influence the laboratory test results.

Another measure of soil strength and unit weight is the number of blows, N, required to drive the sampling apparatus a distance of 1 foot into the undisturbed soil mass. This approach has been the subject of extensive research by geotechnical engineers for a number of years. Although many factors affect the N value, several relationships between N, angle of internal friction, and relative density have been developed. An illustration of the variation of friction angle, and relative density for different N values is presented on Plate C-1, Relationship Between Standard Penetration Resistance, Angle of Internal Friction and Relative Density.

Based on the results of our site exploration, laboratory testing, and experience with similar granular soil deposits, it is our conclusion that the appropriate range of friction angle which best represents the very loose to medium dense sand soils which underlie the periphery of the project area is between 30° and 34°. Our conclusions are based also on current slope inclinations, assumption of static safety factors, and back calculation of friction angle by infinite slope analysis.

PROCEDURES FOR LIQUEFACTION STUDY

Empirical Analysis

We have evaluated the liquefaction potential of the soils underlying the project area utilizing the empirical procedure of Seed (1979). A peak ground surface acceleration of 0.15 g which corresponds to the operating earthquake was used in the analysis. The operating earthquake controls for this case since the duration of ground shaking and, therefore, the number of significant stress cycles which the soil mass will be subjected to are greater than for the contingency earthquake. The two procedures implemented during our study are based on the most critical factors which affect soil liquefaction, including soil type, relative density, initial confining pressure, and intensity/duration of ground shaking.

In general, the cyclic stress condition which may be induced in the soil deposit by the design earthquake is evaluated and compared with the stresses which are required to cause liquefaction based on published data for similar soil conditions or laboratory test results. The cyclic shear stress ratio (τ_h/σ_o') induced by the design earthquake at any depth may be estimated using the design ground acceleration in the following empirical equation:

$$\tau_h/\sigma_o' = 0.65 \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

where a_{max} = 0.15 g for this study

σ_o = total overburden pressure at depth of calculation

σ_o' = effective overburden pressure at depth of calculation

r_d = a stress reduction factor which varies from 1.0 at the ground surface to 0.6 at 100-foot depth.

The primary basis of comparison between the project area and other sites where liquefaction is known to have occurred is the value of N_1, which is the blow count (N) value corrected to an overburden pressure of 1 ton per square foot. N_1 can be determined using the relationship

$N_1 = C_n \cdot N$

Where C_n is a correction factor (after Seed 1979), based on studies completed by Bieganousky and Marcuson (1977). C_n was applied to values of N which inferred a relative density greatern than about 50 percent.

The cyclic shear stress ratio required to initiate liquefaction may be estimated, then, using values of N_1 and correlation curves based on performance of other soil deposits during seismic events. Plate C-2 illustrates data compiled by Seed (1979) for various earthquake intensities which are a function of the cyclic stress ratio required to cause liquefaction and values of N_1. The cyclic stress ratio at a given depth is calculated and may be compared with available corrected blow count data to evaluate the susceptibility of the deposit to liquefaction. As an example, the stress ratio at a depth of 25 feet below existing site grade has been plotted on Plate C-2. The corrected value of N_1 required to limit the potential for liquefaction during an appropriate 8-1/4 magnitude earthquake is about 19.

A graphic representation aids in the comparison of the earthquake-induced shear stress and shear stress ratio required to cause liquefaction at various depths in a soil deposit. Where the stress required to cause liquefaction is less than that induced by the design earthquake, liquefaction is likely to occur. Plates C-3 through C-5 illustrate the average values of N_1 at 10-foot increments of depth for various borings at the site and the corresponding cyclic stress ratio required to cause liquefaction. Included on each plot is the stress ratio induced by the design earthquake for the current conditions and subsequent to fill placement to about Elevation 25. Based on these data, it appears that the penetration resistance of the sand soils underlying the project area increases from the south side of the site northward toward the existing shoreline. The implication is that the potential for liquefaction is lower near the current shoreline.

Analytical Analysis

The second procedure proposed by Seed and Idriss (1971) compares the earthquake-induced shear stress to the stresses causing liquefaction as a function of relative density and grain size. For the purposes of this analysis, our estimates of soil relative density were based on N values as noted on Plate C-1.

A range of mean-effective grain size and relative density was selected and evaluated for liquefaction potential. The result of our study is illustrated on Plate C-6. On this basis, we conclude that for sand soils which have a mean-effective grain size less than about 0.7 millimeters (mm) and a relative density of less than about 60 percent, the susceptibility to liquefaction is moderate to high.

The following plates are attached and complete this appendix:

Plate C-1 - Relationship Between Standard Penetration Resistance Angle of Internal Friction and Relative Density

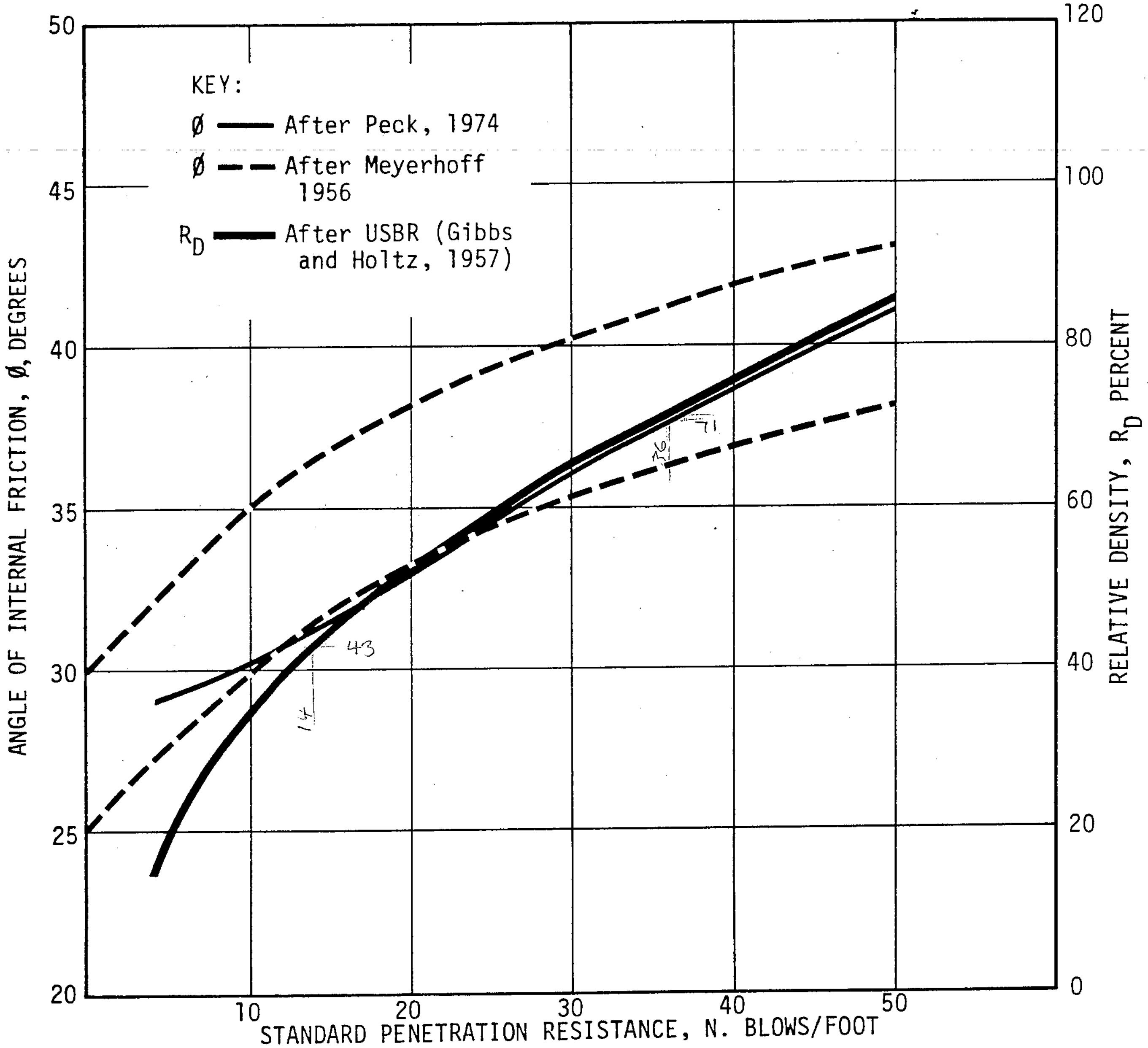
Plate C-2 - Correlation Between Liquefaction and Modified Penetration Resistance

Plate C-3 - Results of Liquefaction Study Based on Modified Penetration Resistance

Plate C-4 - Results of Liquefaction Study Based on Modified Penetration Resistance

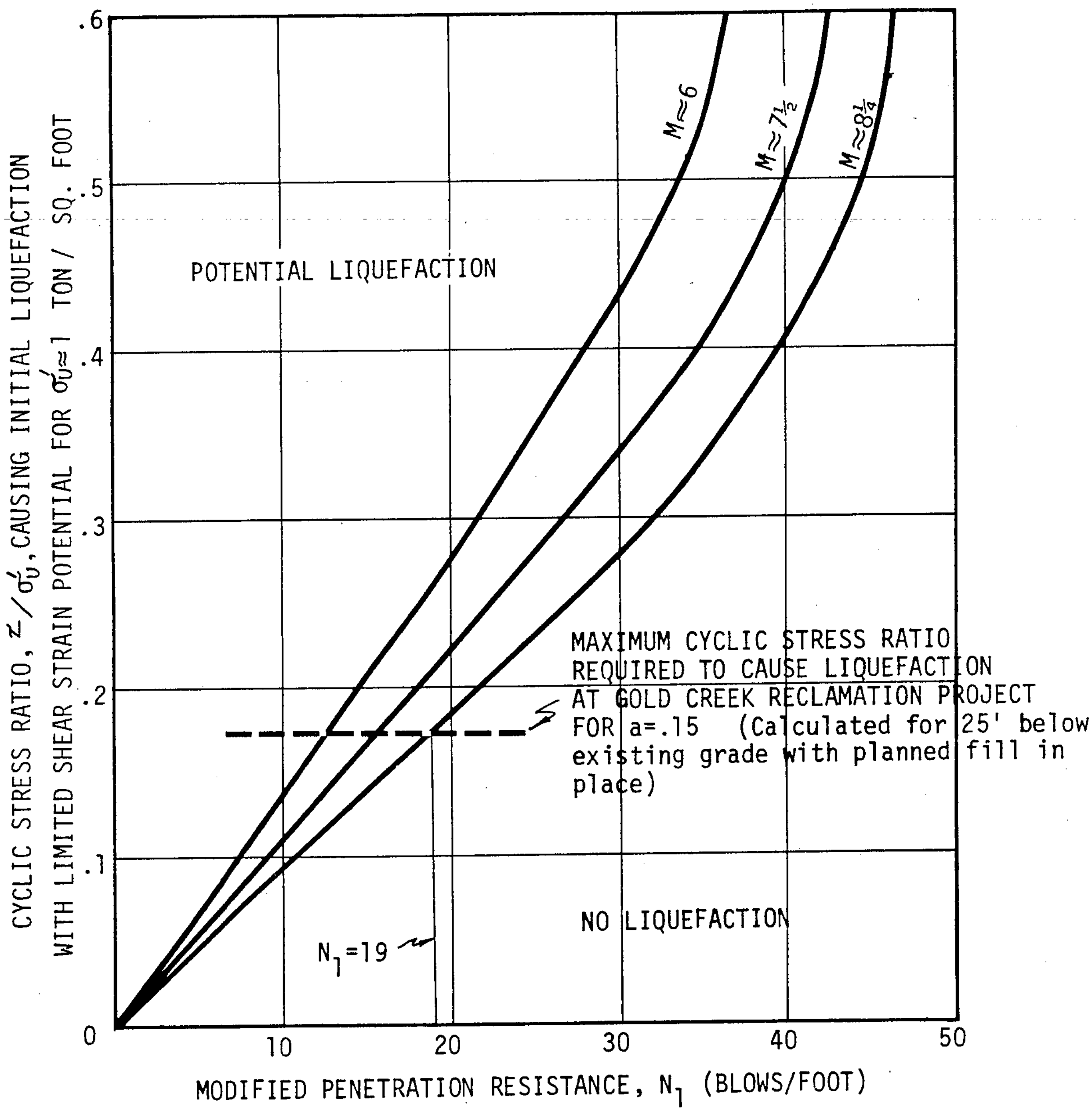
Plate C-5 - Results of Liquefaction Study Based on Modified Penetration Resistance

Plate C-6 - Results of Liquefaction Study Based on Relative Density and Mean Grain Size



Relationship Between Standard Penetration Resistance, Angle of Internal Friction and Relative Density

Dames & Moore

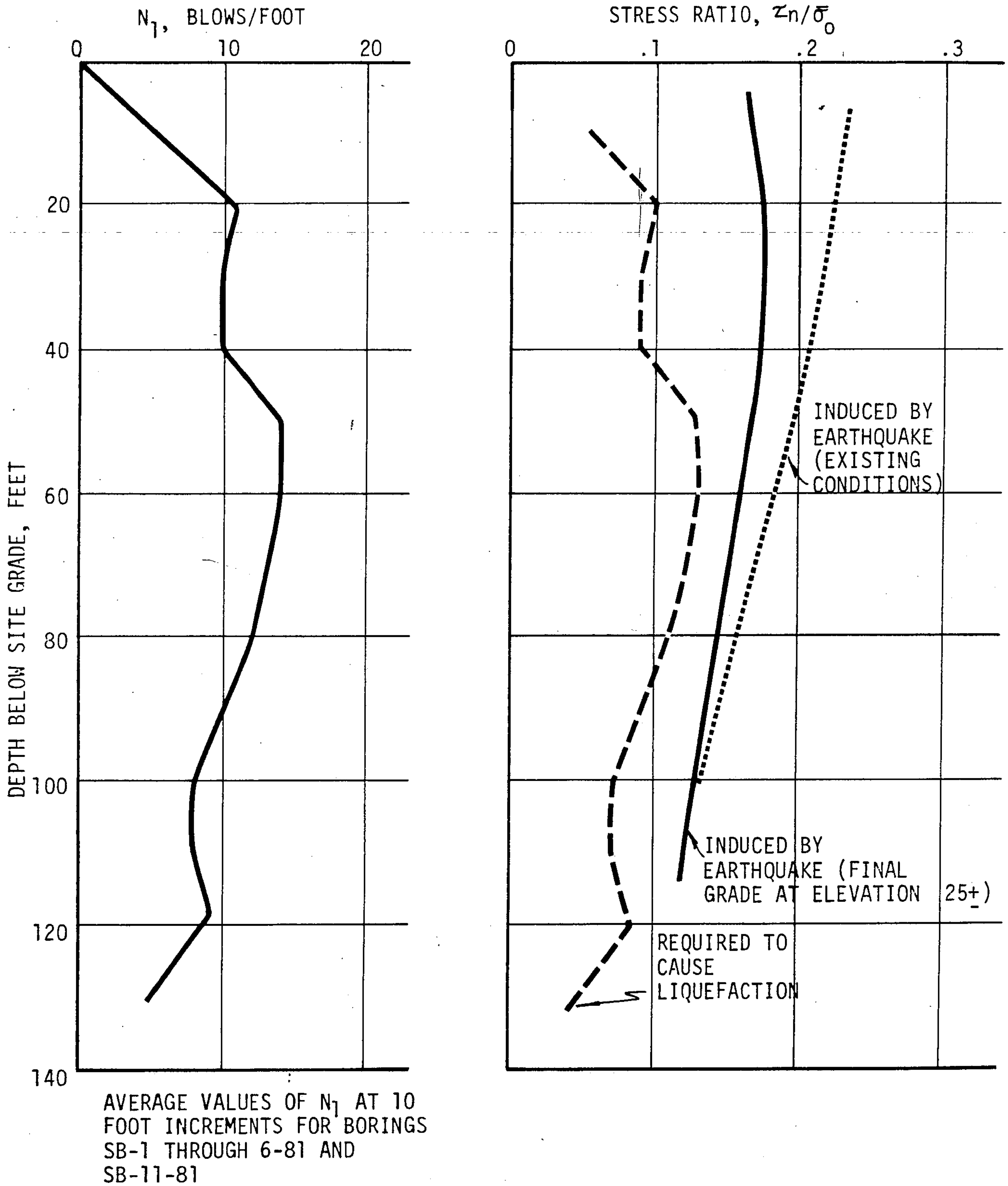


NOTES:

1. M denotes approximate earthquake intensity based on Richter scale
2. Curves based on results of site studies and large scale laboratory tests (After Seed, 1979)

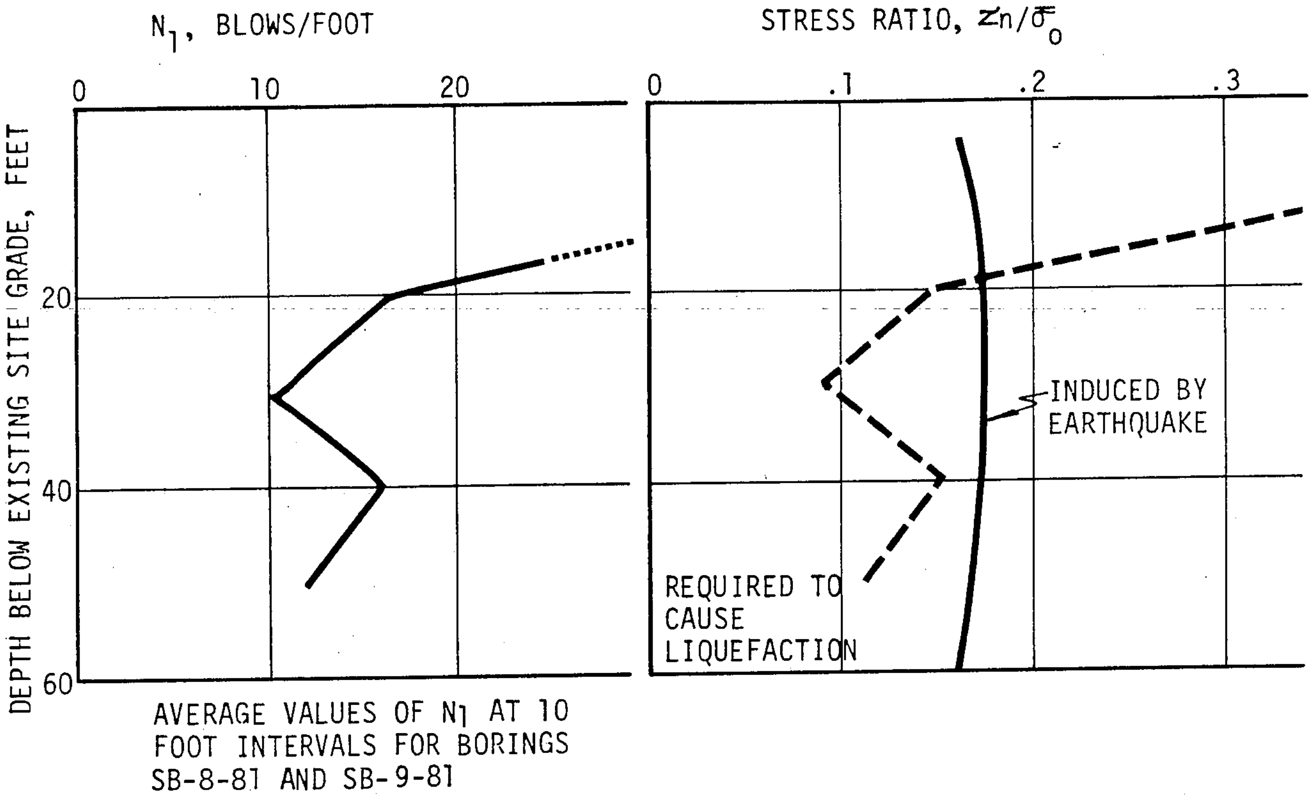
Correlation Between Liquefaction and Modified Penetration Resistance

Dames & Moore



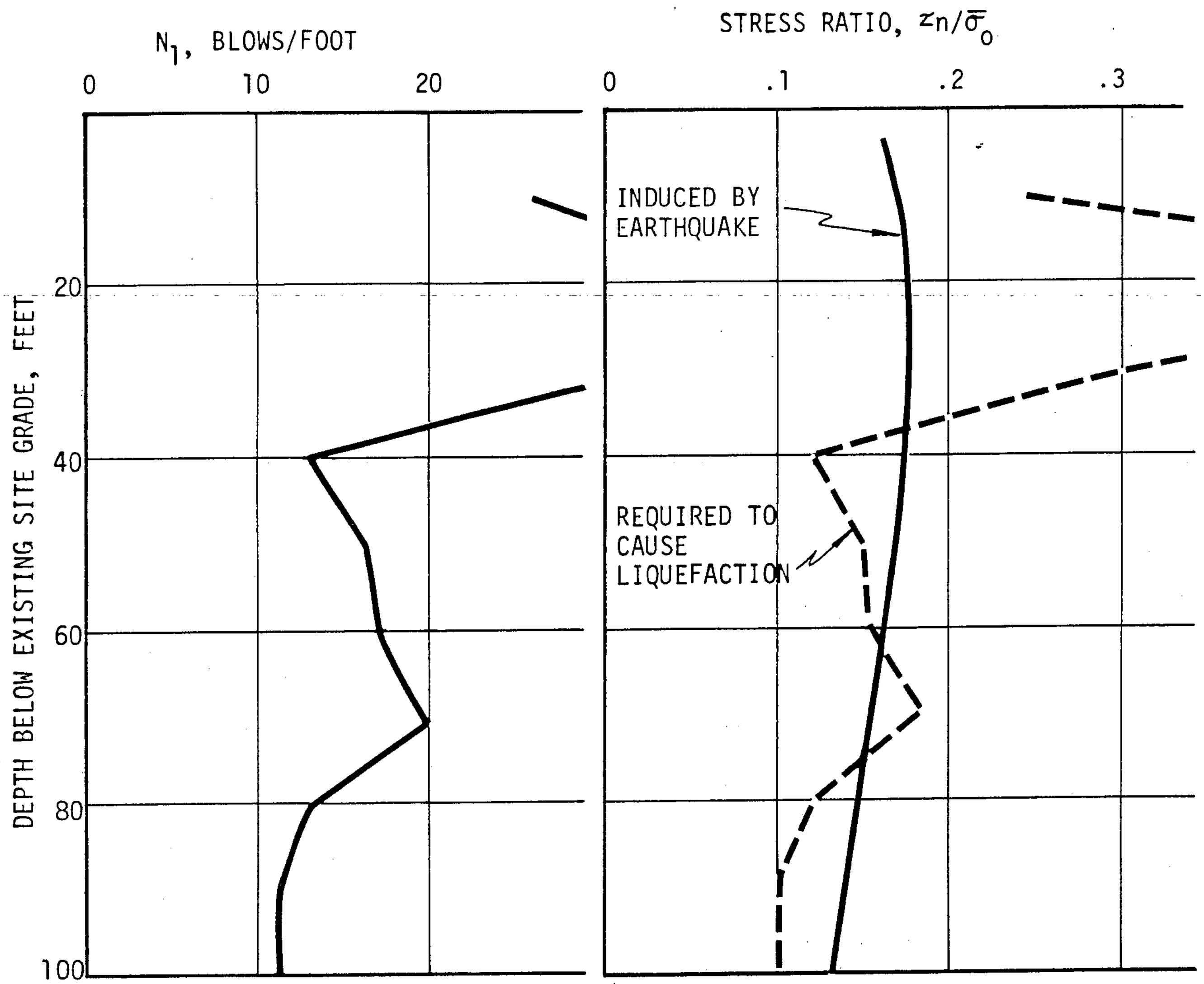
Results of Liquefaction Study Based on Modified Penetration Resistance

Dames & Moore



Results of Liquefaction Study Based on Modified Penetration Resistance

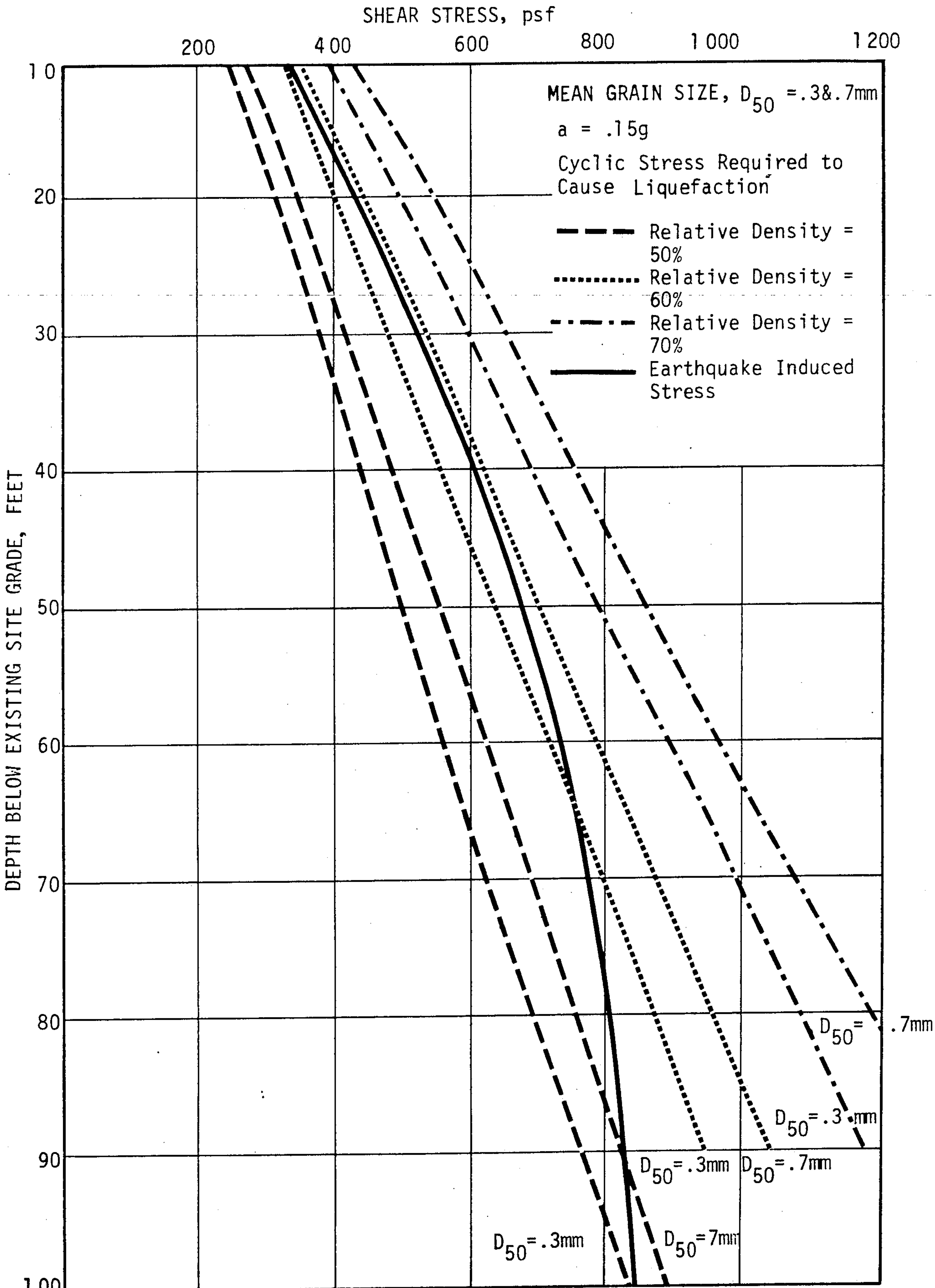
Dames & Moore



AVERAGE VALUES OF N_1 AT 10 FOOT INCREMENTS FOR BORINGS SB-7-81 AND SB-10-81 (0 to 50') AND SB-7-81 (50' to 100')

Results of Liquefaction Study Based on Modified Penetration Resistance

Dames & Moore



Results of Liquefaction Study Based on Relative Density and Mean Grain Size

Dames & Moore