L. B. KARP
50 BEALE ST.
SAN FRANCISCO, CA 94119

DAMES & MOORE

500 SANSOME STREET • SAN FRANCISCO, CALIFORNIA 94111 • (415) 433-0700
CABLE DAMEMORE TELEX 3 4242

August 27, 1971

Job No. 8865-001

Bechtel Corporation
50 Beale Street
San Francisco, California 94105

Attention: Mr. Melvin G. Lewis

Gentlemen:

With this letter we transmit 12 copies of our report "Foundation Investigation, Proposed Metropolitan Life Building, San Francisco, California." This report completes Phases II and III of the investigation as outlined in our April 1, 1971 proposal, as amended July 19, 1971 and presents our final foundation design recommendations.

During the course of this investigation, we have reviewed our findings and conclusions with your architects and engineers at Skidmore, Owings & Merrill. The planned foundation system utilizes 30-foot-long high capacity piles (115- and 135-ton design loads), predrilled and driven into an upper dense sand stratum. The system also calls for a dewatered subbasement floor slab at Elevation -25. This scheme is considered to be the most economical foundation system of several which were studied. However, a portion of the savings in floor slab costs are offset by higher pile costs due to a reduction in the bearing capacity of the sands caused by the upward seepage force toward the underdrained floor system.

The pile design loads are considerably higher than the 90-ton loads previously used for 18-inch-square prestressed piles at the new P.G. & E. Building for similar bearing capacity conditions in the upper sands. Thus, a pile load test program is necessary to verify safe pile loads and appropriate driving criteria. However, the current demolition and construction schedules limit the area available for initial test driving and load testing to the southerly third of the tower area. In order to minimize the changes in pile driving requirements in the balance of the site during production pile driving, more conservative pile sizes have been selected (20-inch-square for 115 tons and 24-inch-square for 135 tons) than would have been necessary if the entire site had been available for the test pile program. Driving of these piling with followers from the existing single-basement depth requires the use of heavy hammers and high-driving resistances to compensate for the temporarily high confinement in the bearing sands since about three fourths of the soil support is in end bearing. Recommendations for shoring design and information relating to seismic design have been presented in separate reports.

We enjoyed working on this project and we look forward to continued participation during bid selection and construction of the foundations. Please let us know if you have any questions concerning this report or if we can assist in any way during final design and construction.

Yours very truly,

DAMES & MOORE

William W. Moore
Senior Consulting Partner

WWM:RDD:RAB:is

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PROPOSED METROPOLITAN LIFE BUILDING

SAN FRANCISCO, CALIFORNIA

INTRODUCTION

This report presents the results of our foundation investigation for the proposed 36-story Metropolitan Life Building to be located at the corner of Market and Fremont Streets, San Francisco, California. The scope of this investigation was tailored to supplementing the considerable data in our files from nearby projects. Phase I of the investigation included evaluations of possible foundation schemes based on results of nearby subsurface information. The results of Phase I were informally presented. In this report we present the basic subsurface information upon which our recommendations were based and final design recommendations for the pile foundations. Also included are discussions on basement design considerations, lateral earth pressure recommendations, building settlement behavior, and our evaluation of the liquefaction potential of the bearing sands under seismic vibrations.

Preliminary studies were performed for shoring design, and for the analysis of the superstructure under anticipated seismic events. The results of these specialized studies were presented separately*. In

*1) Lateral Pressures for Preliminary Shoring Design, July 9, 1971.
2) Seismic Design Studies, Proposed High-Rise Structure, Market and Fremont Streets, San Francisco, California, for Metropolitan Life Insurance Company, August 6, 1971.

our report on shoring, we did not point out the importance of maintaining a watertight seal between the upper and lower aquifers. This is discussed in detail in the Conclusions and Recommendations paragraphs at the end of this report.

The location of the planned building with respect to test borings that we have drilled in the nearby downtown area is shown on Plates 1, 2 and 3. Contours of important subsurface strata as interpreted from the borings are also shown on these plates. The results of the two test borings and the associated laboratory testing performed specifically for this investigation are presented in Appendix A.

PROPOSED CONSTRUCTION

It is our understanding that the proposed tower will be 36 stories above the street and contain two basements approximately 30 feet deep. The tower, to be located at the north end of the property, will measure 130 by 200 feet to the center lines of the exterior columns. The basement walls will extend to the property lines which measure about 140 by 280 feet. The average dead plus live load of the building at the subbasement slab is estimated to be approximately 4,500 pounds per square foot. Columns are planned at 20 by 40 and 45-foot bays. Column loads will range from 1,300 tons for exterior columns to 2,300 tons for interior columns for dead plus live loads. It is planned to support the interior columns on two rows of 115-ton piles, 4 across and 4 feet on center, and the exterior columns on a double row of 135-ton piles 4 feet on center. Piles will be driven to end bearing in the underlying sand strata. It is expected that the piles will be driven with a follower prior to excavating for the basement in order to expedite construction. The lower basement slab-on-grade will be underlain by a drain blanket and subsurface drain system with gravity flow outlets that will eliminate hydrostatic uplift pressures beneath the slab. It is anticipated that the basement will be extended for future low-rise buildings on the adjacent property.

SITE CONDITIONS

The site is located on the bay side and immediately adjacent to the old shoreline of San Francisco Bay that in the early 1800's ran between First and Fremont Streets. The site was reclaimed by man-made fills consisting of sand and rubble placed more than 100 years ago. The sidewalk elevation is presently about Elevation +4, City of San Francisco Datum. The building area is now occupied by one single-story and two, three-story buildings, all with basements to the curbline. The existing adjacent buildings consist of three to six-story buildings. To the southwest on First Street (Lot 11), the two six-story buildings are known to be pile-supported. The three-story building to the southeast on Fremont Street (Lot 4) is thought to be supported on a timber mat; if so, this building will be underpinned. The adjacent building facing Market Street, on Lot 13, is scheduled for demolition prior to construction of the Metropolitan Life Building.

SUBSURFACE CONDITIONS

Based on the two test borings drilled for this investigation, as well as many borings drilled for nearby investigations, we believe that the subsurface conditions have been defined sufficiently to not require the additional borings that were initially contemplated. A summary of the soil conditions within the tower area as developed from straight line interpolation between borings, is tabulated below with a detailed description following. Variations of a few feet in the interpolated elevations of the various strata are likely to occur between borings.

LAYER	THICKNESS (Ft.)	TOP AND BOTTOM OF LAYERS: DEPTH BELOW STREET GRADE Min.	Max.	ELEVATION (S.F. Datum)	
		0	0	+4	+4
Fill (including basement)	18 - 26				
		18	26	-14	-22
Recent Bay Deposits	14 - 28				
		40	47	-36	-43
Bearing Sands	28 - 42				
		74	80	-70	-76
Upper Dessicated Old Bay Clay	13 - 20				
		84	97	-80	-93
Second Sand Layer	10 - 22				
		101	111	-97	-107
Thick Old Bay Clay	45 - 65				
		149	176	-145	-172
Intermixed Sands & Clays	88 - 94				
		258	264	-254	-260
Bedrock	-				

Below the basements of the existing buildings loose fills consisting of fine clean sand with some rubble are expected to extend to depths of 18 to 26 feet below street grade, with the deepest fill at the Market Street end of the site. Piling from existing and previous buildings may also be encountered. The piling is most likely to be wood piling extending to a depth of not more than 50 feet (Elevation -46 feet). The water level in the fill was, at the time of drilling, encountered at a depth of 14 feet (Elevation -10), which, we believe, is representative of the average water level throughout the year. Seasonal fluctuations are probably less than ±2 feet.

The fills are underlain by 14 to 28 feet of recent bay deposits consisting of soft to medium stiff silty clay intermixed with sand lenses. The thickest deposits were found at Boring No. 2 and the shallowest are expected to be at the diametrically opposite corner. The bottom of this stratum is shown by the full contour lines on Plate 1.

The recent bay deposits are underlain by 28 to 42 feet of dense to very dense sands termed "bearing sand." This is the uppermost layer from which major buildings can gain foundation support. The extent of this stratum is shown on Plate 1. The sands are primarily clean sand intermixed with silty and clayey sand, particularly near the top and bottom of the stratum. Occasional cementation was encountered. The water level in this stratum was measured in the observation well at about Elevation -30. Water table elevations are discussed in detail later.

The bearing sands are underlain by a 13 to 20-foot-thick stratum of very stiff marine clays, termed "Upper Dessicated Old Bay Clays." This portion of Old Bay Clay is highly consolidated (probably due to dessication) and will therefore not contribute appreciably to the settlement of the proposed building.

Beneath this layer is a sand stratum termed "Second Sand Layer," which would also offer high end bearing resistance for piles driven through the "bearing sand" stratum. The extent of this layer is shown on Plate 2.

The 45 to 65-foot-thick marine clay beneath, termed "Thick Old Bay Clay," is the major contributor to settlement for heavy buildings founded above. The top and bottom elevations are shown by the contours on Plate 3. It is not as highly over-consolidated as the Old Bay Clay layer above. For loads not exceeding the preconsolidation pressure it is only moderately compressible. However, under heavier loads it will compress similarly to soft Bay Mud.

Beneath the Old Bay Clays are very stiff clays intermixed with layers of very dense sands to bedrock. These layers will only compress slightly under the anticipated loads. Their properties would only be of interest if piling were to extend into them, a design condition that was considered but judged to be unnecessary and excessively expensive.

WATER LEVELS

The elevation of the groundwater table (piezometric level) is different in the sands above the recent bay deposits than in the sands below the recent bay deposits. The upper water table is normally about Elevation -10, San Francisco Datum, and has not undergone recent significant variations. It is controlled primarily by the mean tide level in San Francisco Bay. Based on observations taken at other downtown locations, the lower water table (piezometric level) in the bearing sands below the recent bay deposits has fluctuated from as high as the upper water table to as low as Elevation -36 to -40 feet. This lower water level has risen slowly over the last several months and is now at Elevation -28 as measured in observation well W-1 near Boring No. 1. It is very likely that the water level in these sands with time will rise to its earlier (pre-1969) high of about Elevation -10 feet.

When the water level in the bearing sand is below its normal Elevation -10, there is a downward seepage gradient in the recent bay deposits. The result is a tendency for the streets and fill-supported buildings to settle due to consolidation of the recent bay deposits. The interparticle stresses in the deeper soils below the bearing sands are also higher due to the reduction in buoyancy. This causes even the pile-supported buildings to settle due to consolidation of the underlying Old Bay Clays. During construction, the higher interparticle stresses in the bearing sands will cause piles to have higher capacity at the time of installation than later when the water table rises. The lower water pressure does have some offsetting benefits, however, as it reduces the lateral earth pressure on shoring, increases the passive resistance of soils below the bottom of the excavation, and reduces the tendencies for bottom heave.

CONCLUSIONS AND RECOMMENDATIONS

The presently proposed basement and foundation scheme utilizes 115 and 135-ton design capacity piles driven into the upper sand stratum, and a dewatered basement slab at Elevation -25. This scheme is considered to be satisfactory from support and settlement considerations, as well as being the most economically competitive type of foundation system. The bearing sands have adequate supporting capacity for the static and seismic building loads, and settlements of the underlying soils do not appear to be of such a magnitude as to cause excessive differential settlements within the tower.

In the remainder of this report we present a more detailed discussion of the design considerations for basement and piling installation, an explanation of our liquefaction studies, and the results of our settlement evaluations.

BASEMENT DESIGN AND CONSTRUCTION

Detailed recommendations were presented in our Progress Letter dated July 9, 1971 for the development of excavation procedures and design of shoring. Information on lateral pressures for design of the basement walls can be obtained from that letter. The basement walls should be designed with the groundwater considered to be at Elevation -10 even though the basement slab on the interior of the building will be drained. Although the basement lateral pressures will be influenced by the shoring details, a satisfactory design can be based on the active pressure diagrams for the shoring with certain precautions: 1) an allowance should be made for surcharge at the street sides of the building; 2) care should be exercised in placing backfill against basement walls not to develop high compaction pressures. Considering ordinary street traffic, an appropriate pressure diagram would be 200 pounds per square foot to a depth of 5 feet and then increasing at a rate of 35 pounds per square foot per foot to the water table at Elevation -10. From Elevation -10 to -20, the lateral pressure should be increased at the rate of 80 pounds per square foot and below -20 at the rate of 90 pounds per square foot.

It is our opinion that the subbasement slab can be satisfactorily supported on the recent bay deposits that are expected to extend about 10 to 20 feet below the bottom of the slab. To avoid designing the slab to resist about 15 feet of hydrostatuc pressure, a drain blanket of graded filter material will be placed beneath the slab with drain pipes into pumped sumps. The outlet elevations should be one to two feet below the slab. The slab should be well reinforced so that displacements do not develop at joints and wall connections resulting from the expected settlement of the pile caps and the slight swelling tendencies of the soils between the pile caps.

To avoid lowering the water table in the area surrounding the basement, which could lead to adverse behavior of the adjacent structures and/or troublesome legal problems, provisions must be included to avoid creating channels along the basement and foundation components that could conduct groundwater to the drain blanket from the sands that are both above and below the recent bay deposits.

As long as the water table in the bearing sands remains below the bottom of the excavation, there will be no tendency for the drain blanket to affect it. However, if the water table rises, as expected, to about Elevation -10, the upward hydraulic gradient in the clays between the drain blanket and the bearing sands will about balance the weight of the clays. This would produce a tendency for piping through any channels that may penetrate the clays. During excavation, any existing piling that are to be removed should be cut or broken off rather than pulled. The excavation should be inspected for old well casings and if found, these should be sealed with a heavy bentonite or cement grout. Any relief wells or dewatering wells to be used during construction should be sealed. It will be necessary that the backfill for the pile caps and elevator pits contain layers of impermeable material compacted tightly against the faces of the excavation and structures. Some methods of shoring could create channels for water to enter the drain blanket from one or both of the sand strata. Pervious working pads below the perimeter pile caps could also form potential seepage channels. Special attention should be given to these and similar details during design and construction to assure that the drain blanket is isolated from the water bearing sands.

We expect that seepage will be only a few gallons per day if the recommended measures are taken to seal openings that may be made through the recent bay deposits. Seepage water can probably be handled by the ordinary basement sumps and pumping equipment.

PILE CAPACITIES

The upward gradient in the groundwater within the clay soils below the basement drain blanket can be of sufficient magnitude to reduce the effective overburden pressure at the top of the bearing sands to practically zero for the highest anticipated groundwater condition. Such reductions in the effective overburden pressures directly effect pile capacities and the design condition for computations. It should therefore be appreciated that at least a portion of the savings in cost in designing the basement slab for zero hydrostatic uplift is lost due to added costs of larger pile sizes necessitated by these considerations.

With the above described groundwater condition, there is a relatively narrow range of pile penetrations to develop high end bearing capacities in the upper bearing sands. The percentage of ultimate pile capacity taken in end bearing is on the order of 70 to 80 percent. The bearing capacity of soils at shallower depths is limited by the relatively low effective confining pressures and at the greater depths it is limited by the strength of the underlying clays in resisting a punching failure. The following dead plus live load design capacities were developed from theoretical analyses and load test experience for 18-inch-square prestressed concrete piles used for the new P.G.&E. Building (see Appendix B). These capacities include a computed factor of safety of 2.0.

SQUARE PILE SIZE (In inches)	MAXIMUM CAPACITY (In tons)	PILE TIP PENETRATION BELOW SUBBASEMENT FLOOR SLAB — MINIMUM Penetration (Feet)	MAXIMUM Penetration (Feet)
18	115	36	41
20	135	36	40
22	160	36	39
24	190	36	38

Use of these computed maximum pile capacities for design involved considerable risk without extensive preliminary pile driving experience across the site and pile load tests. Also, previous experience at the

Bechtel and P.G.&E. building sites found variations in the upper sand stratum (during pile driving), which might cause problems in obtaining required bearing within the strict limits of pile penetrations indicated. Since present demolition schedules preclude wide coverage of the site with driving tests, and because of local variations in the supporting capacity of soils, more conservative pile designs have been selected. Foundation designs are based on 115 and 135-ton-capacity piles and call for 20 and 24-inch-square prestressed concrete piles to carry these loads. While the 22-inch-square pile would likely support the 135-ton design load, the 24-inch was selected on the basis of available sizes and the very high design load.

PILE DRIVING CONSIDERATIONS

Prestressed concrete piling driven prior to site excavations with long followers, are the only economically competitive pile type for the planned short, high-capacity bearing piles. The most efficient use of the prestressed piling requires accurate predetermination of tip elevations. This is simpler to achieve for friction piles than for the planned bearing piles in sand. In this latter case, assurance of satisfactory capacity of the individual piles is achieved by controlling field pile driving by an appropriate pile driving criteria. Within the zone of acceptable bearing capacity, significant variations in driving resistances are expected due to differences in relative density and cementation of the sands, and the densifying effect of previously driven piles. Consequently, the pile driving criteria should accommodate field adjustment in pile lengths when higher or lower driving resistances are encountered than specified at the design penetrations. Thus, a compromise is necessary between the desire for accuracy in predetermined pile lengths, simplicity in driving criteria, and the desirability of flexibility in actual length driven to meet field conditions. We believe the best solution is to use an estimated average pile length and a simplified driving criteria for bidding purposes and to modify the criteria as required after an initial pile driving program and pile load tests.

Pile driving criteria and design lengths for bidding purposes would be based upon the calculated pile lengths presented in the previ. section and analagous pile driving experience at the P.G.& E. Building. A summary of the P.G.& E. pile driving and load test data is presented in Appendix B. The major differences between the Metropolitan and P.G. & E. foundation conditions are that:

1. Higher pile loads are planned for the Metropolitan Building. (One-hundred and fifteen and 135 tons for 20- and 24-inch piles versus 90 tons for 18-inch piles at P.G. & E.).
2. The confining pressures in the bearing sands at the Metropolitan site are lower than at the P.G.& E., even though Metropolitan has a shallower basement (two stories versus three). This is caused by the upward seep. gradient through the bearing sands resulting from the underdrain system beneath the basement floor.

3. Only a limited portion of the Metropolitan site will be available for initial pile driving and load tests, and it will not be possible to develop early driving experience over most of the site. (At P.G.& E. comprehensive testing was performed at the cleared site prior to production driving).

This combination of higher pile loads, lower final effective pile confinement, and available area for initial testing, requires deeper penetration into the sand stratum (about 20 feet versus 10 feet at P.G. &E.),and larger pile sizes.Also,larger pile driving hammers will be needed to achieve the required pile bearing capacity for the condition of pile installation prior to site excavation. As was done at P.G. & E., the Metropolitan piles will be driven with followers before the excavation is complete. This, together with the temporary lower water table in the bearing sands, will require a bearing capacity at the time of pile installation, about two times greater than at the time of application of building loads. This factor requires that piling be driven to high driving resistance, and that pile load tests be carried to four (4) times their design load (factor of safety of 2).

Predrilling should be used to facilitate pile penetration to required depths. The predrilled hole should be no larger in diameter than the least dimension of the pile to minimize the reduction in friction support in the bearing sands. Because of caving in the sand strata, this will probably limit the drill bit size to 16 to 17 inches for the 20-inch-square piles. It is expected that predrilling to Elevation -55 (plus or minus) will be required to achieve design pile penetrations.

Predrilling is also advantageous in locating old wood pile obstructions and minimizing the effects of vibrations and noise related to pile driving.

INITIAL PILE DRIVING PROGRAM

We understand that the relative timing of demolition and pile driving operations are such that only a limited area in the southern half of the site will be available for initial pile driving and load tests. For the presently anticipated site conditions, approximately 8 piles should be driven at available pile locations. Four would be 20-inch in size and four 24 inches. These initial piles should be cast 5 feet longer than design lengths and should be driven with the identical equipment and followers planned for production piling. Variation in the depth of predrilling and driving resistance should be expected to meet the variable field conditions. Following the initial pile driving, an accessible location would be selected for load tests. Load tests on two 20-inch piles may be needed to verify the bearing capacities of soils. The load test frame and loading equipment should be sized for a 600-ton capacity to accommodate the effects of temporary confinement on the bearing sands at the time of testing and the desire to test the piling to soil yield or failure. It may be possible to test both piling under a single frame although the option for multiple load test setups should be provided for in the specifications. The piles would be installed in oversize predrilled holes to cutoff elevation, cased and backfilled with bentonite slurry to minimize frictional resistance in the upper soils. Additional predrilling into the sand stratum will probably be required to achieve the desired tip elevation and driving resistance for the load

For the P.G.& E. project, ultimate dynamic pile driving resistances as computed by wave equation analyses, correlated well with load test results for the pile of lower driving resistance (370 tons driving resistance versus 380-ton failure load), but over-estimated the capacity of the harder driven pile (450 versus 370 tons). Wave equation analyses have been performed for the 20-inch-pile and the type of follower employed at the P.G. & E. site. These analyses plus the P.G.& E. experience confirm that very heavy pile driving hammers will be required for the 115- and 135-ton capacity piles at the Metropolitan site. The wave equation analyses indicate that the 014 hammer (42,000 foot-pound-per-blow hammer utilized at the P.G.& E. site) can not drive the 135-ton piles to adequate bearing. Similarly, the 200-C, differential acting steam hammer (50,200 foot-pound-per-blow energy) would require higher pile driving resistances than desirable (over 200 blows per foot for 135-ton-pile). Consequently, we recommend that the pile specifications require pile driving hammers having the following minimum rated energies:

	Minimum Energy (Ft./Lb. per Blow)
Steam or Air	60,000
Diesel	80,000

Even for these hammers, high pile driving resistances are indicated for the 135-ton-pile(up to 20 blows per inch). This high driving resistance should be considered practical refusal with the above production pile driving equipment.

test pile(s), They would be driven full length without followers and extend a few feet above ground surface. Instrumentation of the load test piles with tell-tales and/or electric resistance strain gauges is desirable to evaluate the load distribution under test conditions. Such data would permit a less conservative evaluation of the reduction in pile capacities for final building conditions.

Based upon the results of the initial pile driving and load test(s), final pile lengths and driving criteria would be estabilished for the production pile driving. The finally selected pile lengths should permit a reasonably accurate predetermined tip elevation for the prestressed piling; however, it will be impossible to have anticipated all variations for foundation piling. Controlled predrilling depths and alternative driving resistance for lengths above and slightly below the planned tip elevations will provide the optimum flexibility to meet field conditions, while maintaining an efficient pile driving schedule. The most economical foundation system consistent with field conditions and satisfactory performance can only be obtained in this manner. If field changes were to be eliminated, a much more conservative pile design would be necessary.

PILE DRIVING CRITERIA

It is impractical to develop the detailed driving criteria to be used for the production pile driving prior to completion of the initial driving and load test program. The specifications should present a simplified criteria representative of the anticipated requirements and provide for modifications as the result of the initial test program.

Prior to gaining field experience with the actual equipment to be used for foundation piling, meaningful distinctions cannot be made between the required driving resistance for 20- and 24-inch piling. It is expected, however, that production pile driving criteria will require higher driving resistance for the 24-inch than 20-inch piles.

For the piling specifications we recommend that the following pile criteria be used for both piles:

1. 100 blows per foot at design pile tip elevations.
2. Within 2 feet of design tip elevation, 150 blows per foot.
3. Depth of predrilling to be varied to achieve design pile penetrations.

For design tip elevations, we recommend that the average between minimum and maximum calculated pile penetrations be used; that is, Elevation -62 for the piling in the deeper basement excavation. Because of the controlling influence of end bearing on pile capacity, a direct reduction in pile tip elevation can be made for the higher basement elevations. Thus, the design lengths of the piling throughout the building would be uniform at about 30 feet.

SETTLEMENTS

Tower settlements were estimated for several basement schemes in our Phase I studies. For the planned basement and foundation scheme, our Phase I settlement analyses would indicate maximum settlement of the central core on the order of two inches with respect to the adjacent streets and a maximum differential between interior and exterior columns of less than one inch. About half of this settlement would occur during the initial application of load. The remainder would occur over a period of several years and would be partially dependent on fluctuations in the water table that may occur following building construction.

As this analysis was performed prior to our field investigation, it was based on soil data obtained from adjacent borings. The soil profile now established reveals a greater thickness of the moderately compressible Old Bay Clay at the south end of the building than at the north end. Otherwise, the initially assumed soil properties were confirmed by the results of this investigation. If the building area were uniformly loaded, this would cause a slightly greater settlement of the south end and therefore a tilting of the tower. However, we believe that the unloading of the south end of the building area due to the deep basement excavation in the Plaza area, will counterbalance this tendency to tilting. Another cause of tilting due to differential settlement could be the possible deep basement excavations for future low-rise buildings to the west of the tower. Unloading of this area simultaneously with the excavation for this project could cause the tower to tilt towards Fremont Street on the order of 1 to 2,000. Delaying of the unloading several years with respect to the construction of the tower would essentially eliminate the tilting potential.

We have also performed preliminary settlement evaluations considering longer piles driven to end bearing in the second sand stratum. Although our analyses were not carried to completion because this scheme was judged as not necessary, it appeared that the higher stresses on the thick Old Bay Clay under the central column areas could cause more settlement than with piles driven no deeper than recommended for developing maximum capacity in the upper bearing sands.

SAND LIQUEFACTION EVALUATION

We have concluded that the bearing sands would not liquefy or lose strength should they be subjected to the most severe earthquake that was considered in the seismic analysis of the superstructure*. This earthquake, which is considered comparable to the 1906 'quake, had a peak bedrock acceleration of 0.40g, a duration of very strong shaking of 30 seconds, and a total duration of strong shaking of 60 seconds. The liquefaction potential of the bearing sands during a strong motion earthquake was evaluated using two separate methods. Both methods involve calculation of the cumulative effect of earthquake vibrations on the liquefaction potential of the sands as indicated by cyclic laboratory tests on sands of similar relative density and particle size distribution. The resulting cumulative damage number is comparable to that used in fatigue analyses of metals. If the cumulative damage is less than one, liquefaction is unlikely. The cumulative damage approach to liquefaction analyses was recently developed by Dr. Neville Donovan of our San Francisco office**.

The first method used the computed shear stresses in the sand deposits from the time histories of accelerations developed for the

*Seismic Design Studies, Proposed High-Rise Structure, Market and Fremont Streets, San Francisco, For Metropolitan Life Insurance Company (report dated August 6, 1971) (D&M Job No. 4243-011-03)

**"A Stochastic Approach to the Seismic Liquefaction Problem," by Neville C. Donovan; Conference on Applications of Statistics and Probability to Soil and Structural Engineering, 1971.

earthquake analysis of the structure. The second method used statistical relationships for the shear stress characteristics of the sand deposits based upon analyses of previous earthquake records but not directly utilizing the computed time histories of accelerations for this site. Comparable results were obtained from both methods.

These analyses indicate that liquefaction of the sands supporting the planned pile foundations would not occur during a 1906-type earthquake if the relative density of the sands were greater than 70 percent. As shown on Plate A5A in Appendix A, the minimum measured relative density of the bearing sands is 73 percent with all other values being in excess of 80 percent. A conservative design value of 80 percent relative density for the bearing sands is also consistent with the driving resistances encountered in sampling the sands at this site and elsewhere in downtown San Francisco. The water table used in the analyses was conservatively estimated to be at Elevation -10. The calculated cumulative damage for 80 percent relative density within the bearing sands is between 0.31 and 0.37, indicating an ample margin of safety against loss of pile support for the severe 1906-type earthquake.

On the other hand, liquefaction of the upper sandy fills during a severe earthquake is likely. For an estimated relative density of these sands of 50 percent, 7 feet below the water table, the calculated cumulative damage is 1.46, indicating the development of progressive liquefaction during the duration of strong shaking. Such a condition would not impair building performance, since support from these fill soils is not required for earthquake stability. Fill supported sidewalks and utilities servicing the building may be effected, however. For less severe earthquakes liquefaction of even the sand fills would not occur.

The calculated liquefaction of the sandy fills and satisfactory performance of the pile supporting materials for the severe earthquake is in agreement with the performance of buildings and streets during the 1906 earthquake. Whereas there was considerable ground displacement and settlement of the fills in the lower Market Street area in 1906, no adverse foundation behavior was evidenced by any pile-supported building. Thus, we are confident that there is a negligible probability of liquefaction of the bearing sands during a future earthquake.

The following Plates and Appendices are attached and complete this report.

Plate 1 - Estimated Contours of Bearing Sand Strata
Plate 2 - Estimated Contours of Top of Second Sand Strata
Plate 3 - Estimated Contours of Thick Old Bay Clay
Appendix A - Field Explorations and Laboratory Testing
Appendix B - P.G.& E. Building Load Tests

Respectfully submitted,

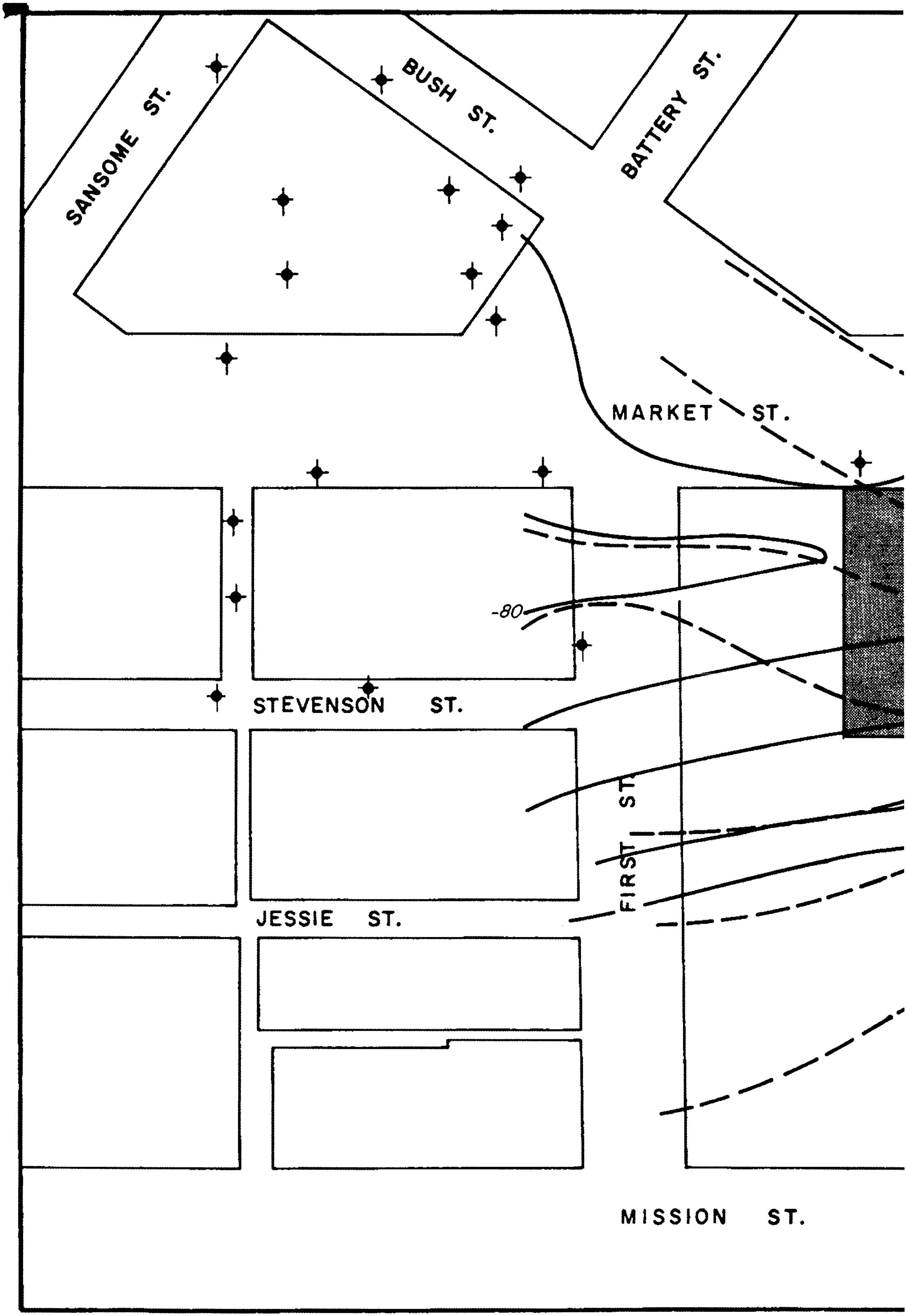
DAMES & MOORE

William W. Moore
Partner

Roy A. Bell
Project Engineer

August 27, 1971

Job No.8865-001-03



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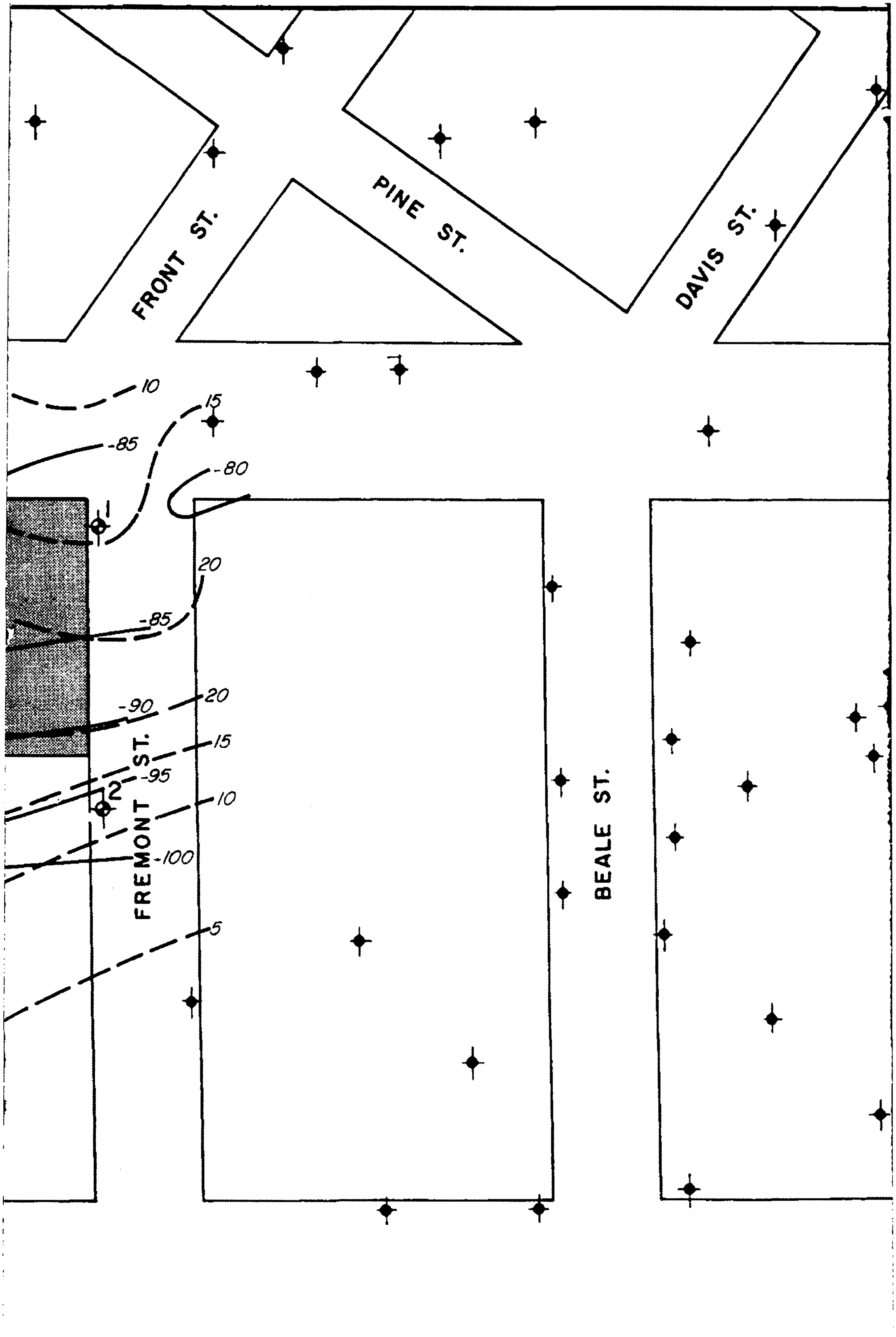
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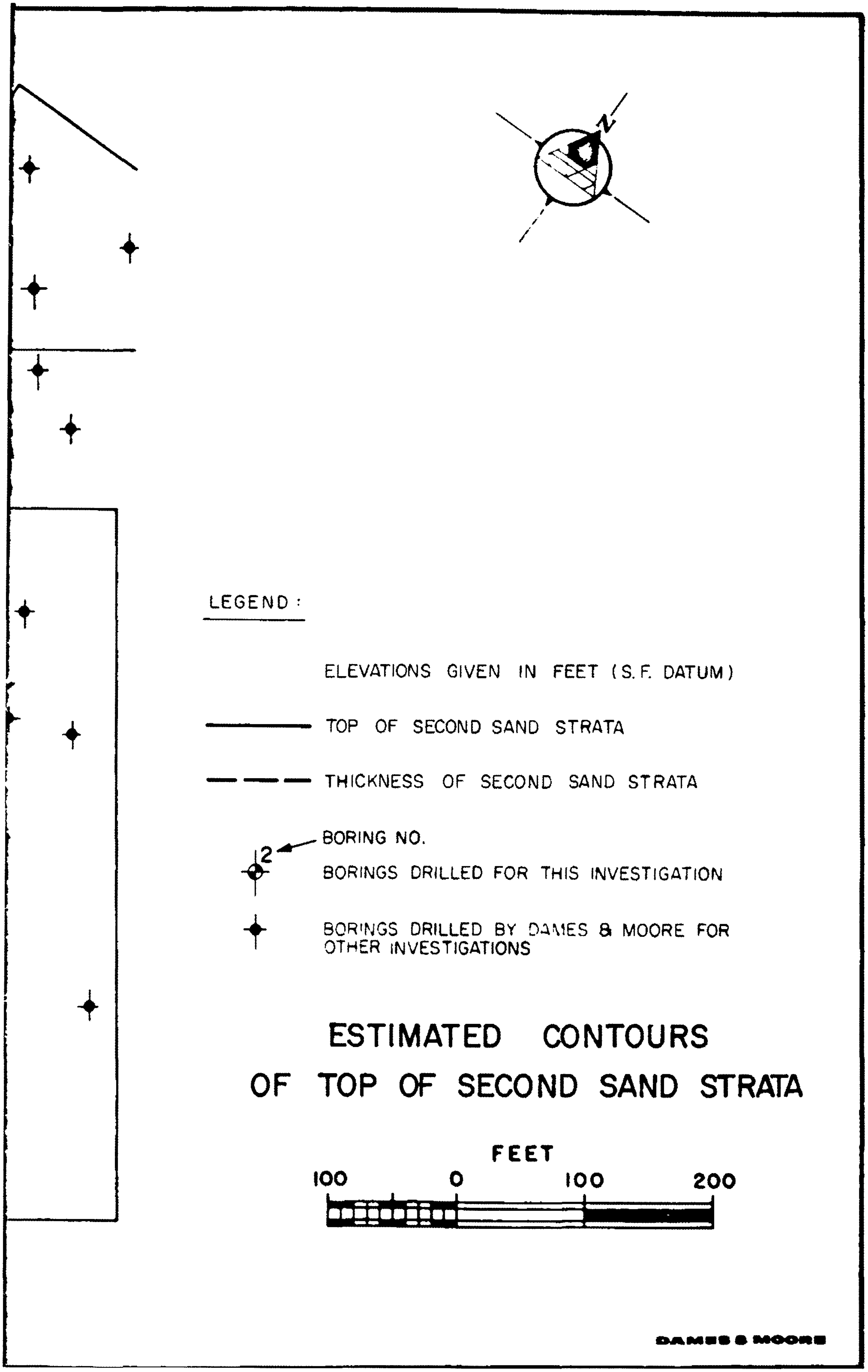
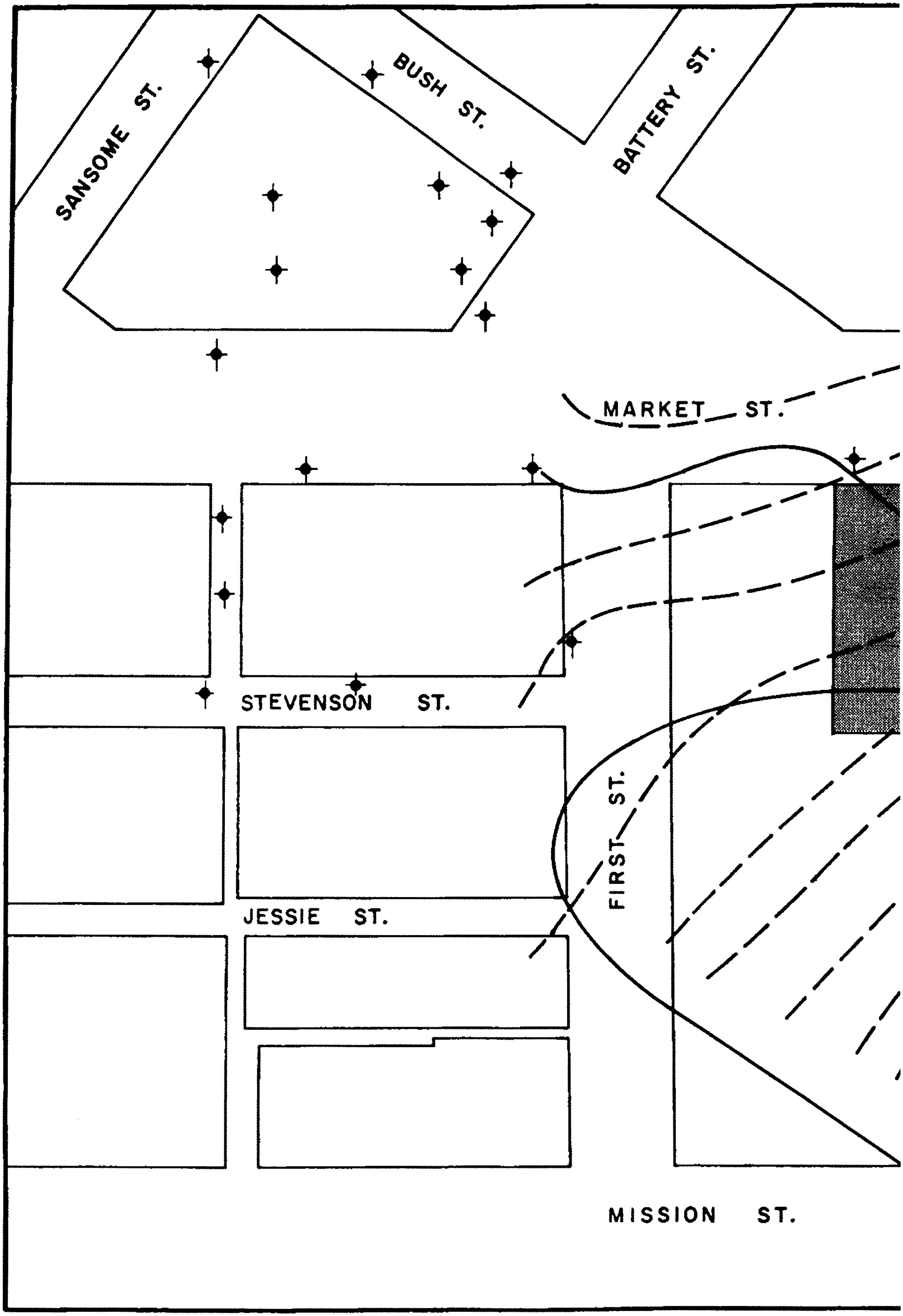
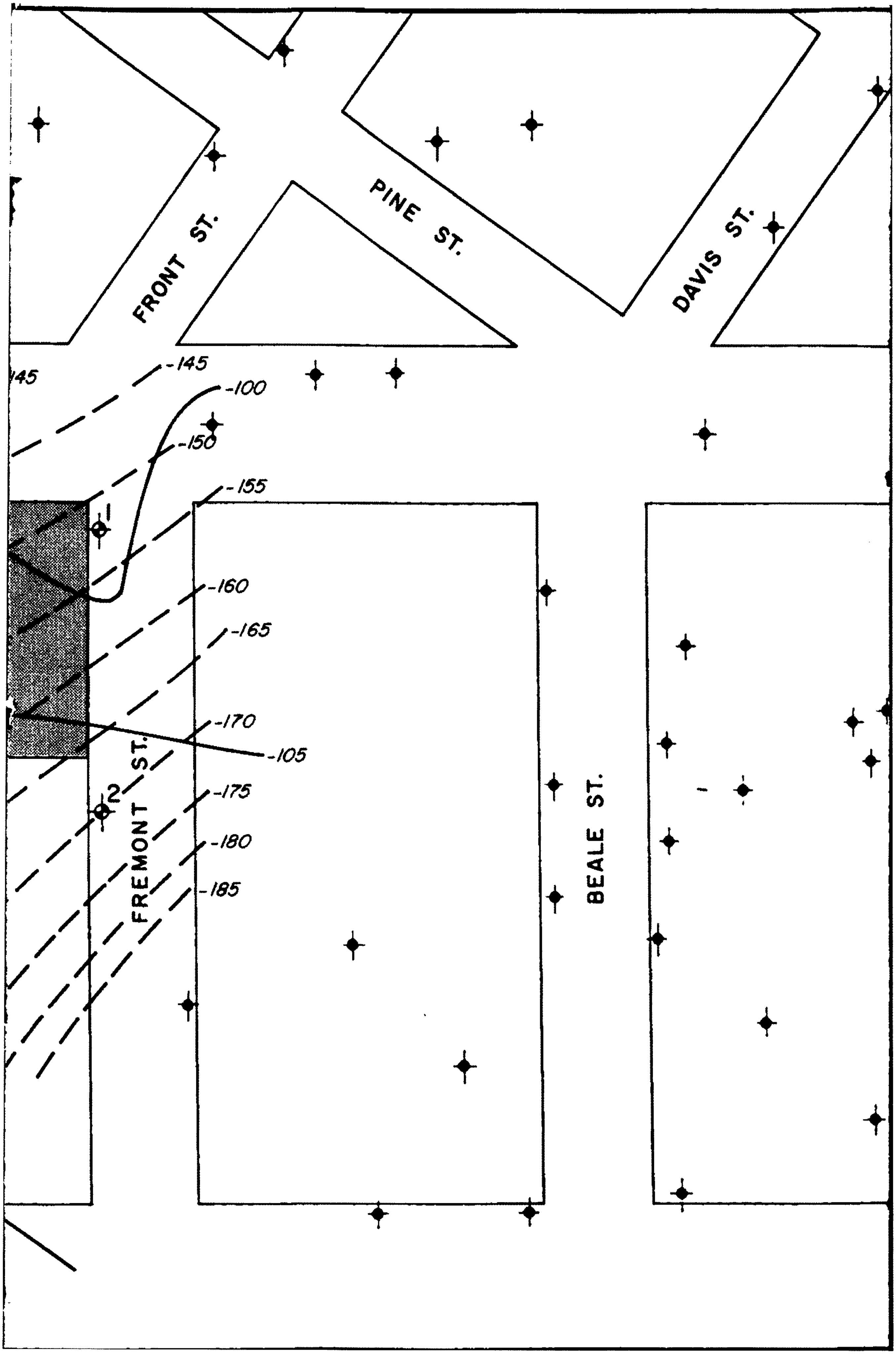
-100
5



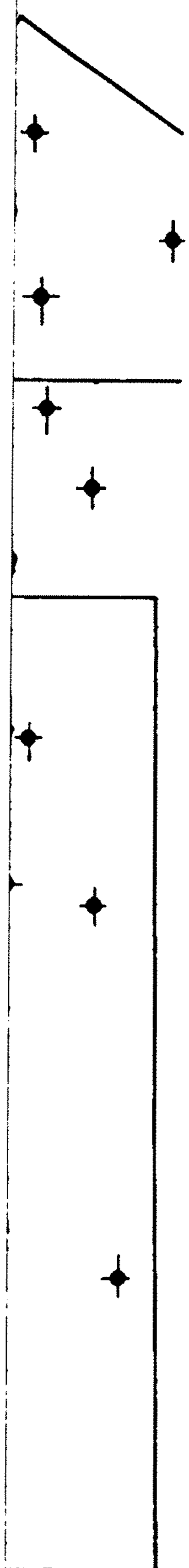
ESTIMATED CONTOURS OF TOP OF SECOND SAND STRATA

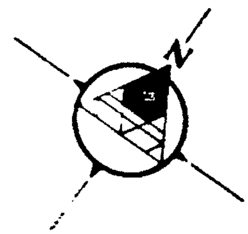
DAMES & MOORE


SANSOME ST.
BUSH ST.
BATTERY ST.
MARKET ST.
STEVENSON ST.
FIRST ST.
JESSIE ST.
MISSION ST.



FRONT ST.
PINE ST.
DAVIS ST.
FREMONT ST.
BEALE ST.
145
-145
-100
-150
-155
-160
-165
-170
-105
-175
-180
-185
1
2





LEGEND:

ELEVATIONS GIVEN IN FEET (S.F. DATUM)

——— TOP OF THICK OLD BAY CLAY STRATA.

— — — BOTTOM OF THICK OLD BAY CLAY STRATA.

2 BORING NO.

BORINGS DRILLED FOR THIS INVESTIGATION

BORINGS DRILLED BY DAMES & MOORE FOR OTHER INVESTIGATIONS

ESTIMATED CONTOURS OF THICK OLD BAY CLAY

FEET

100 0 100 200

DAMES & MOORE

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

The subsurface conditions at the site were explored by drilling two 5-inch-diameter test borings with rotary-wash equipment. The borings were drilled 3 to 4 feet into hard rock and terminated at 260 and 266 feet below street grade. The drilling and sampling was performed under the supervision of one of our engineers, who logged each boring and assisted in obtaining representative undisturbed soil samples for further observation and laboratory testing. The boring locations are shown on Plates I through 3. The logs are presented on Plates AIA and AIB. The soils are classified in accordance with the Unified Soil Classification System presented on Plate A2.

Three types of sampling tools were used, depending on the nature of the soil and the physical properties to be evaluated. Two of these, the Dames & Moore Underwater Sampler and Piston Sampler, are shown on Plates A6 and A7. In addition to these, the standard penetration sampler (ASTM Designation D-1586-67) was used to evaluate the relative densities and penetration resistance of the bearing sand stratum. The sampling resistance is shown on the boring logs and a key to the types of samplers and sample designations is presented on Plate A2.

A water level observation well, W-I, was installed in order to measure the water level in the bearing sands. W-I was installed in a 78-foot-deep, 5-inch-diameter boring located 9 feet southeast of Boring No. I. The well consists of a one-inch-diameter plastic casing with the lower 20 feet perforated. Fine gravel was placed around the lower 30 feet of the casing and the remainder was filled with a very thick bentonite slurry forming an impervious seal in the boring above the bearing sands.

LABORATORY TESTING

Representative soil samples were tested in our laboratory for shear strength, consolidation, and soil classification indices.

STRENGTH TESTS

Triaxial Compression - Two unconsolidated-undrained triaxial compression tests (TXUU), and four consolidated-undrained triaxial compression tests (TXCU) were performed. The peak shear stress and the confining pressures are tabulated adjacent to the appropriate samples on the boring logs. The method of performing the triaxial compression test is described on Plate A8.

Unconfined Compression - Twelve unconfined compression tests (UC) were conducted on clayey soil samples. The method of performing the test is explained on Plate A8 and the test results are presented on the Log of Borings.

Laboratory Vane Shear Tests - This test is conducted on clayey soils by introducing a 4-bladed vane into the soil sample and measuring the torque required to rotate the vane at a slow rate. The shear strength is computed from the torque required to shear the soil along the vertical and horizontal edges of the vane. The results of the 17 tests performed are presented on the Log of Borings.

CONSOLIDATION TESTS

Eight consolidation tests (C) were performed on the clayey soils that will contribute to the settlement behavior of the building. The consolidation test results are presented graphically on Plates A3A through A3D, and the method of performing consolidation tests is described on Plate A9. The initial portion of the tests was performed in a moist atmosphere, the remainder were performed with the samples inundated. The load at which the samples were inundated is noted for each test.

CLASSIFICATION TESTS

Moisture and Density - The moisture content and dry density was determined for all of the undisturbed samples in the top 200 feet to help assign foundation design parameters and to aid in correlation of soil layers between the borings at both this site and nearby sites. The tests were performed in accordance with ASTM Test Designation D-2216-66 and the results are tabulated adjacent to the samples on the Logs of Borings.

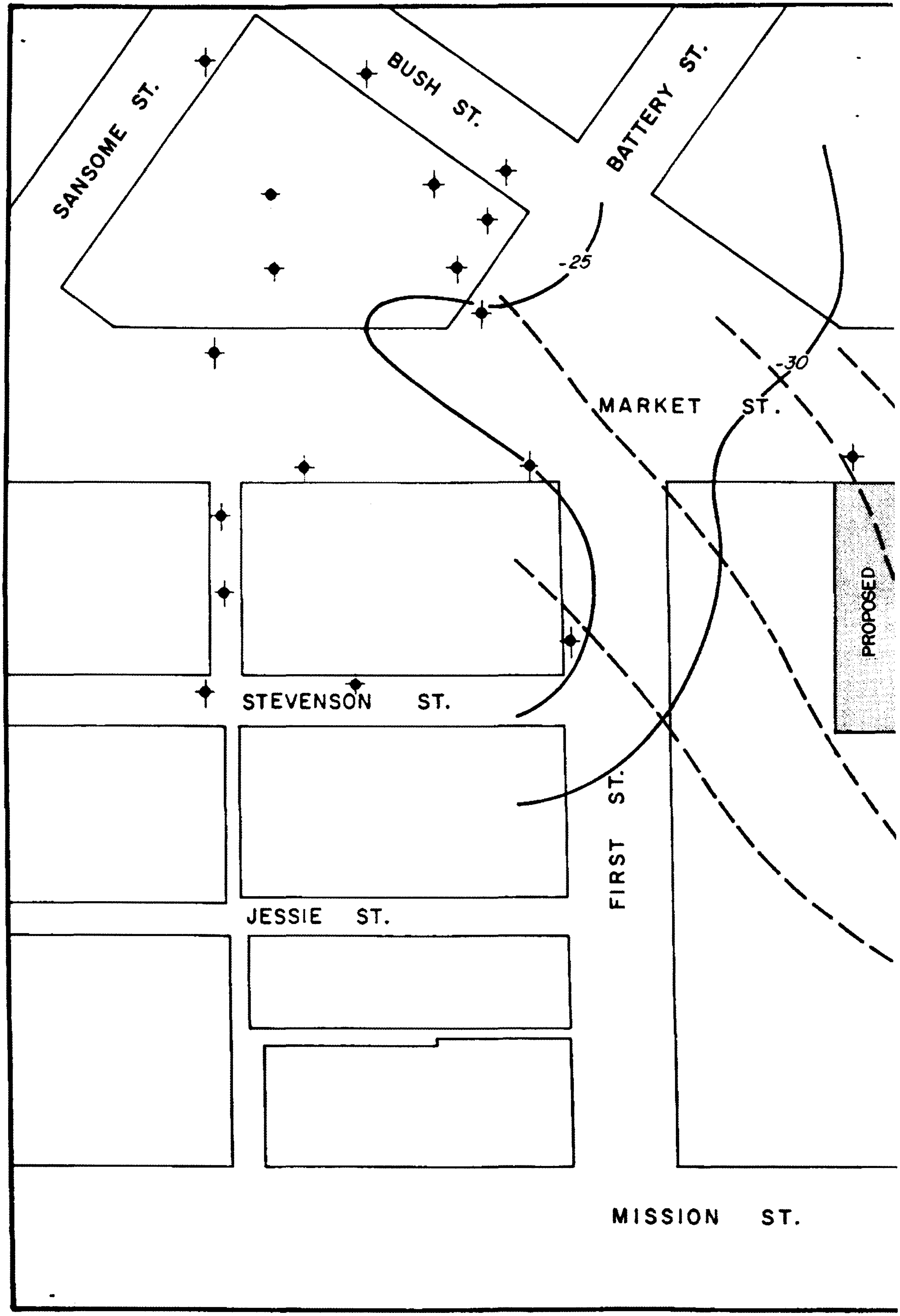
Atterberg Limits - As an aid to soil classification and evaluation of the consolidation tests, the liquid limit and plastic limit were determined for all consolidation samples and on one sample from Elevation -195 from Boring 1. The tests were performed in accordance with the ASTM Test Designations D-423-66 and D-424-59, and the liquid limit and plastic limit results are tabulated on the consolidation test data and on the Logs of Borings.

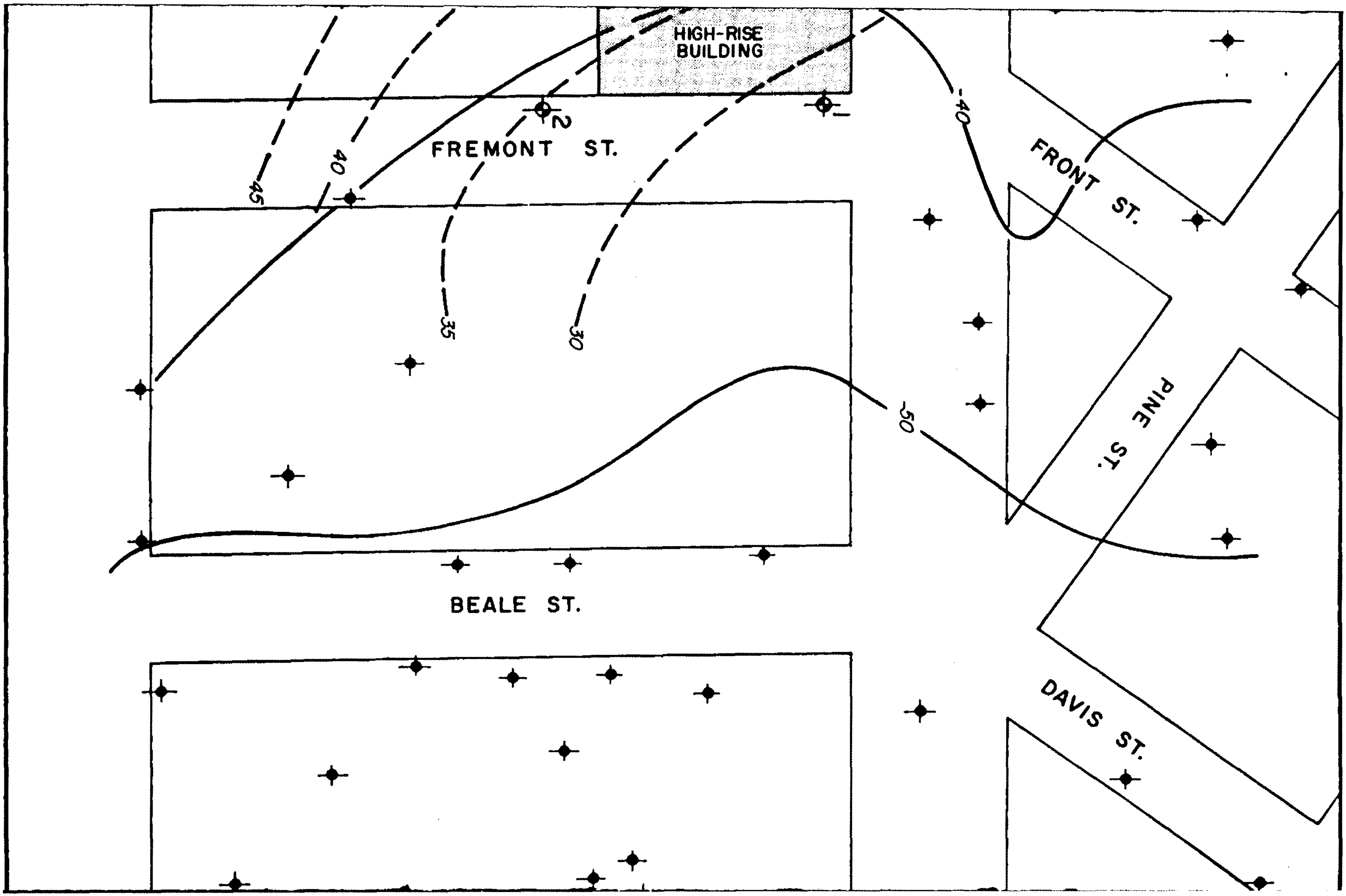
Grain-Size Distribution - Twelve sieve analyses were performed to determine the grain-size distribution of the potential pile bearing sands to assist in the evaluation of seismic liquefaction potential. The tests were performed in accordance with the ASTM Test Designation D-422-63. The results are presented on Plates A4A through A4E.

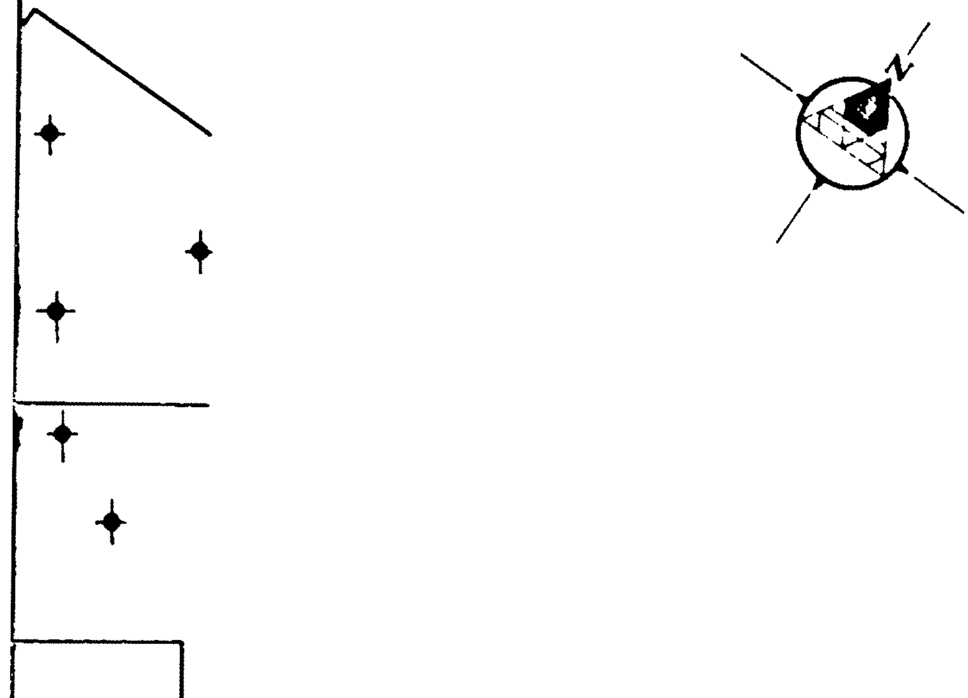
Relative Density Determinations - A maximum and minimum density test was performed for a combined sample consisting of sands from the bearing sand stratum. The test was performed in order to determine the relative densities of the sands in place. The test was performed in accordance with the ASTM Test Designation D-2049-69. A sieve analysis was also performed on the combined sample. The test results are shown on Plates A5A and A5B.

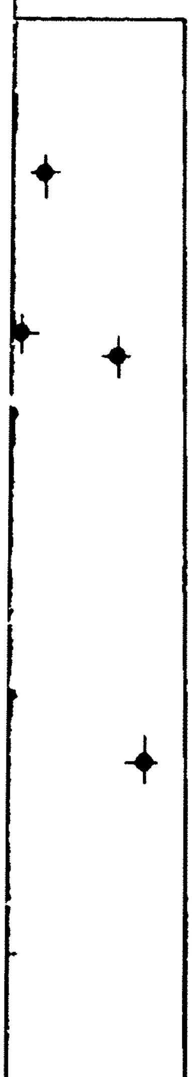
The following Plates are attached and complete this Appendix:

Plates A1A and A1B	- Log of Borings
Plate A2	- Soil Classification Chart and Key to Test Data
Plates A3A - A3D	- Consolidation Test Data
Plates A4A - A4E	- Sieve Analyses
Plates A5A and A5B	- Relative Density Test and Sieve Analysis of Combined Sample
Plate A6	- Dames & Moore U-Type Sampler
Plate A7	- Dames & Moore Piston Sampler
Plate A8	- Method of Performing Unconfined Compression and Triaxial Compression Tests
Plate A9	- Method of Performing Consolidation Tests


SANSOME ST.
BUSH ST.
BATTERY ST.
-25
-30
MARKET ST.
PROPOSED
STEVENSON ST.
FIRST ST.
JESSIE ST.
MISSION ST.


HIGH-RISE BUILDING
FREMONT ST.
FRONT ST.
PINE ST.
DAVIS ST.
BEALE ST.
1
2
-40
-50
30
35
40
45





LEGEND :

ELEVATIONS GIVEN IN FEET (S. F. DATUM)

TOP OF BEARING SAND & BOTTOM OF RECENT BAY DEPOSITS

THICKNESS OF BEARING SAND

2 BORING NO.

BORINGS DRILLED FOR THIS INVESTIGATION

BORINGS DRILLED BY DAMES & MOORE FOR OTHER INVESTIGATIONS

ESTIMATED CONTOURS OF BEARING SAND STRATA

FEET

100 0 100 200

DAMES & MOORE

BORING 1

DRILLED 4/3/71

ELEVATION IN FEET (S.F. DATUM): +10, 0, -10, -20, -30, -40, -50, -60, -70, -80, -90, -100, -110, -120, -130, -140, -150, -160, -170, -180, -190, -200, -210, -220, -230

MOISTURE CONTENT, %	DRY DENSITY P C F	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	TYPE OF STRENGTH TEST	SHEAR STRENGTH TEST RESULTS P S F: τ	SHEAR STRENGTH TEST RESULTS P S F: σ_3	TESTS REPORTED ELSEWHERE	TYPE OF SAMPLER	SAMPLING RESISTANCE
22	109							U	22
25	101			TXCU	1550	700		P	50
29	93			TXCU	1450	1500			
39	82			UC	1340			P	150
31	92			TXCU	510	3000			
21	109						SA	U	70/6"
								SPT	31
28	92						SA	U	30
							SA	SPT	84/6"
19	112			UC	1390		SA	U	35
33	90			UC	1830*			U	22
35	85			UC	1590*			P	400
43	78	46	24				C		
14	120			UC	3760			TW	68
								SPT	109
34	88			UC	2050*			TW	19
41	80	63	26				C		
48	74			UC	3090*			P	250
49	72	69	31				C		
26	99	38	20				C	TW	22
46	78	48	28				C	P	375
19	110							TW	17/2"
20	113							U	55/6"
20	110							U	90/6"
35	89	86	32					U	28/6"
								U	30/6"
18	112	30	18	TV			C	U	40/6"

ELEVATION +4'±

8" CONCRETE

BASEMENT SPACE

11" CONCRETE SLAB

FILL

SP — DARK GRAY FINE SAND (LOOSE)

(WATER LEVEL 4/3/71)

CH — BLACK SILTY CLAY WITH SOME SHELLS & SAND (SOFT) (BAY MUD)

(GRADING DARK GRAY)

CH/SM — INTERMIXED GRAY SILTY CLAY & SILTY FINE TO MEDIUM SAND (MEDIUM STIFF)

(GRADING SANDIER)

SC — LIGHT BROWN CLAYEY SAND (DENSE)

SP — BROWN FINE SAND (VERY DENSE)

SP/SC — (GRADING WITH LENSES OF CLAYEY SAND - DENSE)

(CEMENTED LENSES)

SM — GREENISH-GRAY SILTY FINE SAND WITH SOME BINDER

SM — BROWN SILTY FINE TO MEDIUM SAND (VERY DENSE)

SC — (GRADING TO CLAYEY SAND)

SC — GREENISH-GRAY CLAYEY FINE SAND (MEDIUM DENSE)

CH — GREENISH GRAY SILTY CLAY WITH TRACE DECAYED VEGETATION (VERY STIFF) (OLD BAY CLAY)

(THIN STREAKS OF BROWN FINE SAND)

SC — GREENISH GRAY CLAYEY FINE SAND WITH SOME CEMENTATION (VERY DENSE)

CH — GRAY SILTY CLAY (STIFF TO VERY STIFF) (OLD BAY CLAY)

(GRADING WITH OCCASIONAL SMALL GRAVEL & VERY STIFF)

(TRACE OF DECAYED VEGETATION)

(GRADING SLIGHTLY SANDY)

SM — GREENISH GRAY SILTY FINE TO MEDIUM SAND (VERY DENSE)

CH — GREENISH GRAY SILTY CLAY WITH SOME DECAYED VEGETATION

SM — GRAYISH BROWN SILTY FINE TO MEDIUM SAND (VERY DENSE)

SP — DARK GRAY FINE TO MEDIUM SAND (VERY DENSE)

CH — GREENISH GRAY SILTY CLAY WITH OCCASIONAL ANGULAR ROCK FRAGMENTS

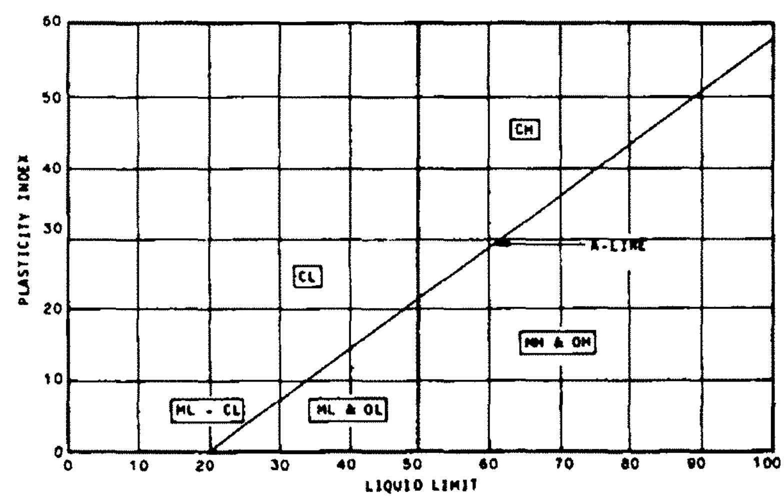
(LENSE OF GRAVEL)

CH/CL — (GRADING WITH INTERBEDDED SANDY CLAY LENSES & MORE SMALL ROCK FRAGMENTS)

Letter	Description	Major Divisions			
GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	CLEAN GRAVELS (little or no fines)	GRAVELS	MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	COARSE-GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE
GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES				
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	GRAVELS WITH FINES (appreciable amount of fines)			
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES				
SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES	CLEAN SANDS (little or no fines)	SANDS	MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE. For visual classification, the ¼" size may be used as equivalent to the no. 4 sieve size.	
SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES				
SM	SILTY SANDS, SAND-SILT MIXTURES	SANDS WITH FINES (appreciable amount of fines)			
SC	CLAYEY SANDS, SAND-CLAY MIXTURES				
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50			FINE-GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS				
OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY				
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50			
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS				
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS				
PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS			

the no. 200 sieve size is about the smallest particle visible to the naked eye

SOIL CLASSIFICATION CHART
UNIFIED SOIL CLASSIFICATION SYSTEM



PLASTICITY CHART

I CHART & KEY TO TEST DATA

KEY TO TEST DATA

TXCU	-	CONSOLIDATED UNDRAINED TRIAXIAL SHEAR TEST
TXUU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL SHEAR TEST
UC	-	UNCONFINED COMPRESSION SHEAR TEST
DSCU	-	CONSOLIDATED UNDRAINED DIRECT SHEAR TEST
TV	-	LABORATORY TORVANE TEST
LV	-	PRECISION LABORATORY VANE SHEAR TEST
SA	-	SIEVE ANALYSES
C	-	CONSOLIDATION TEST
τ	-	PEAK SHEAR STRESS, PSF
σ_1	-	MAJOR PRINCIPAL STRESS, PSF
σ_3	-	MINOR PRINCIPAL STRESS, PSF
σ_n	-	NORMAL STRESS ON DIRECT SHEAR SPECIMEN, PSF

KEY TO TYPE OF SAMPLER

U	-	DAMES & MOORE UNDERWATER SAMPLER
TW	-	DAMES & MOORE UNDERWATER SAMPLER WITH THINWALL EXTENSION
P	-	DAMES & MOORE PISTON SAMPLER
SPT	-	STANDARD PENETRATION TEST SAMPLER

KEY TO SAMPLES

INDICATES DEPTH OF UNDISTURBED SAMPLE

INDICATES DEPTH OF DISTURBED SAMPLE

INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY

INDICATES DEPTH OF STANDARD PENETRATION TEST

SOIL CLASSIFIC

BORING 2

DRILLED 4/10-11/71

MOISTURE CONTENT, %	DRY DENSITY P C F	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	TYPE OF STRENGTH TEST	SHEAR STRENGTH TEST RESULTS P S F: τ	SHEAR STRENGTH TEST RESULTS P S F: σ_s	TESTS REPORTED ELSEWHERE	TYPE OF SAMPLER	SAMPLING RESISTANCE
10	120							U	0
14	103							U	22
26	101			TXCU	720	1500		TW	5
53	69			UC LV	760 1360			P	50
35	86			TXUU LV	1010 580	3000		P	50
32	90			TXUU LV	820 1300	3500		P	75
22	106						SA	U	70/6"
							SA	SPT	90
21	112						SA	U	30/6"
19	114						SA	U	50/6"
19	112						SA	U	50/3"
20	114						SA	TW	18/5"
20	111						SA	U	42
33	90			UC LV	2300 3040			TW	19
40	79			UC LV	1540* 2360			P	300
37	85	61	26				C	P	250
21	109						SA	TW	20/4"
								U	50/5"
36	86			LV	2530			P	300
37	82			UC LV	2480 3150			TW	16
47 42	73 79	51	24	UC	3630		C	P	300
36	93			UC LV	2420 2610			TW	16
38	86			LV	3070			TW	18
40	83			LV	3180			TW	16
								TW	22/6"
23	106							U	22/6"
15	116			LV	3740			U	30
18	117							U	35/6"
37	86							U	70/6"
20	112							U	25/6"

ELEVATION IN FEET (S.F. DATUM)

+10, 0, -10, -20, -30, -40, -50, -60, -70, -80, -90, -100, -110, -120, -130, -140, -150, -160, -170, -180, -190, -200, -210, -220, -230

ELEVATION +4'±

4" CONCRETE
BASEMENT SPACE

CONCRETE SLAB

SP — BROWN FINE SAND WITH BRICK FRAGMENTS (LOOSE) — FILL
(GRADING DARK GRAY)
(WATER LEVEL 4/10/71)

SM/CL — INTERMIXED GRAY SILTY SAND & SILTY CLAY WITH SHELLS (SOFT) (BAY MUD)

CH — GRAY SILTY CLAY WITH SHELLS & OCCASIONAL SAND LENSES (SOFT) (BAY MUD)
(GRADING MEDIUM STIFF)
CH/SC — (GRADING WITH MORE SAND LENSES)
(LENSE OF SANDY GRAVEL)

SP — BROWN FINE TO MEDIUM SAND (VERY DENSE)
SM — (GRADING SILTY)
(CEMENTED LENSES)
SC — (LENSE OF BROWN CLAYEY SAND)
(DENSE WITH SOME CEMENTED LENSES)
(GRADING VERY DENSE)
SP — (GRADING TO FINE TO MEDIUM SAND)

SC/SP — GREENISH GRAY CLAYEY SAND WITH LENSES OF CLEAN SAND (MEDIUM DENSE)

CH — GREENISH GRAY SILTY CLAY WITH SOME WHITE CEMENTED NODULES & TRACE OF DECAYED VEGETATION (OLD BAY CLAY)

SM — GREENISH GRAY SILTY FINE TO MEDIUM SAND (DENSE)

CH — GRAY SILTY CLAY (STIFF TO VERY STIFF)(OLD BAY CLAY)
(TRACE OF DECAYED VEGETATION)

CL — GREENISH GRAY SANDY CLAY WITH LENSES OF SAND & WITH TRACE OF DECAYED VEGETATION (VERY STIFF)

CH — BROWNISH GRAY SILTY CLAY WITH DECAYED VEGETATION (VERY STIFF)

CL — BROWNISH GRAY SANDY CLAY WITH ROCK FRAGMENTS (VERY STIFF)
(GRADING MORE ROCK FRAGMENTS & GREENISH GRAY)

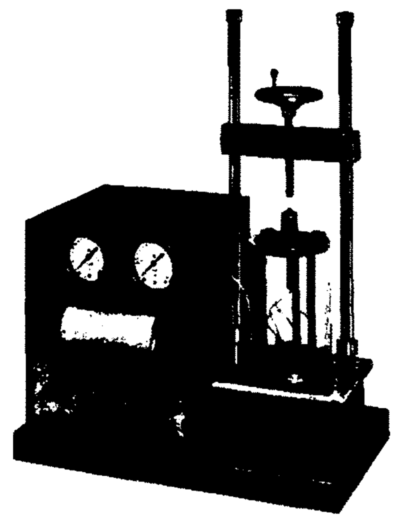
CH — GREENISH GRAY SILTY CLAY WITH BROWN SPOTS (VERY STIFF)

CH — MOTTLED GRAY & BROWN CLAY (VERY STIFF)

CL — GREENISH GRAY SANDY CLAY WITH SOME SMALL GRAVEL (VERY STIFF)

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

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



TRIAXIAL COMPRESSION TEST UNIT

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

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

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

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

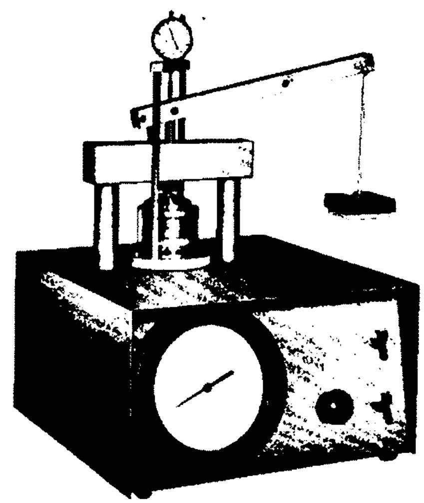
> UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.
>
> CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.
>
> DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

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

Method Of Performing Consolidation Tests

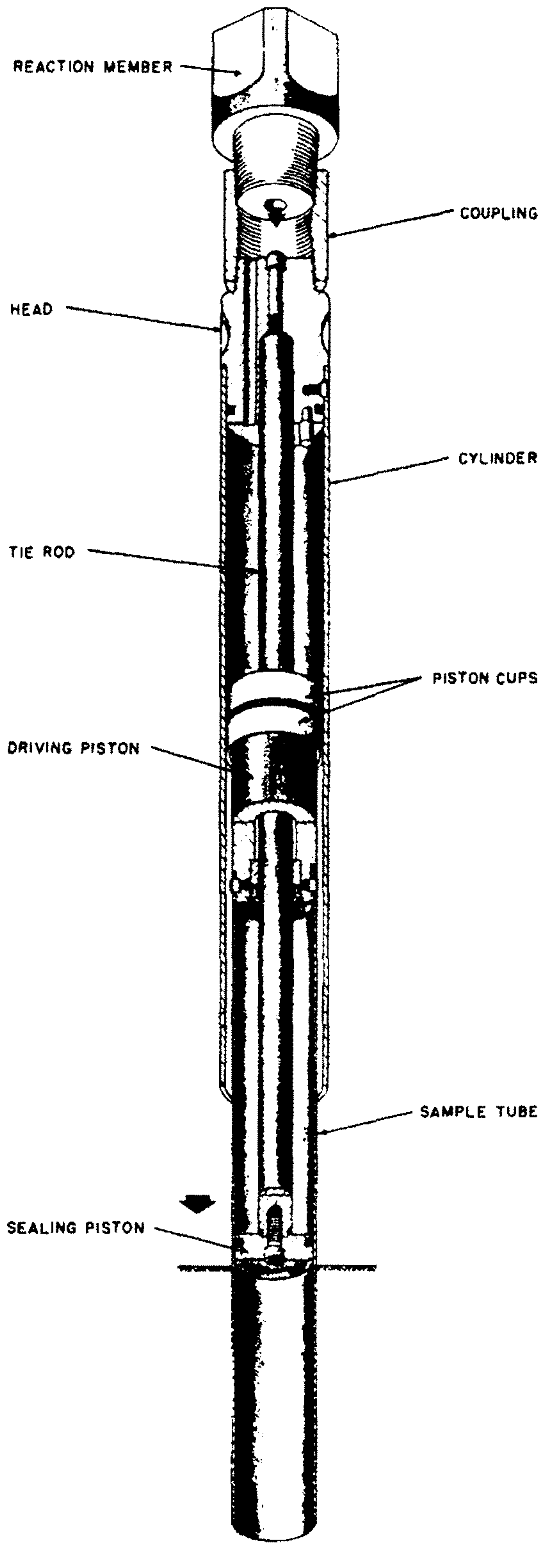
Consolidation tests are performed to evaluate the volume changes of soils subjected to increased loads. Time-consolidation and pressure-consolidation curves may be plotted from the data obtained in the tests. Engineering analyses based on these curves permit estimates to be made of the probable magnitude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

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



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

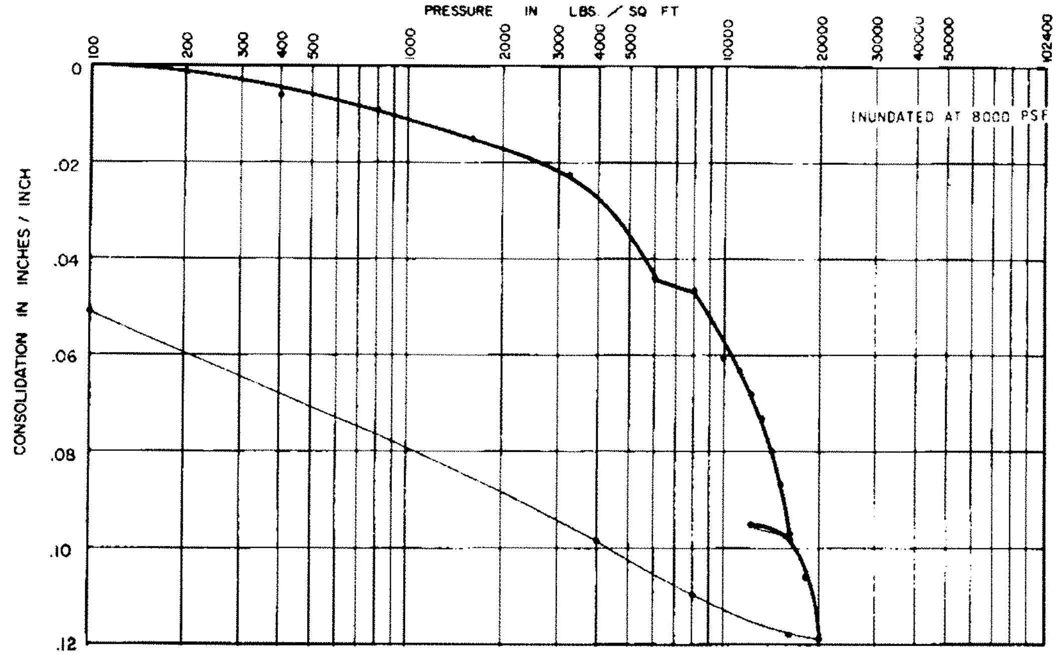
In testing, the sample is rigidly confined laterally by the brass ring. Axial loads are transmitted to the ends of the sample by porous disks. The disks allow drainage of the loaded sample. The axial compression or expansion of the sample is measured by a micrometer dial indicator at appropriate time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



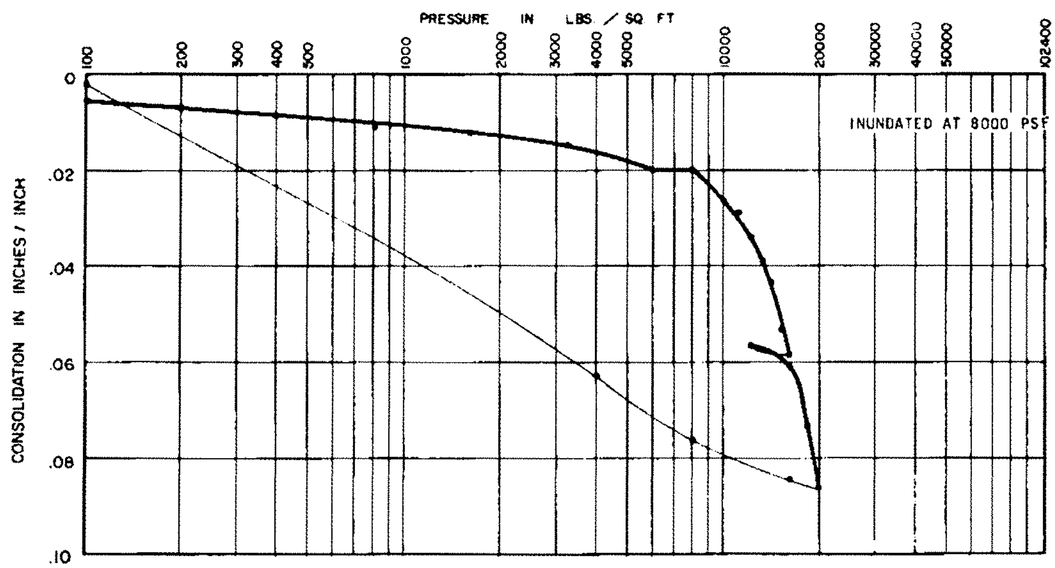
PISTON SAMPLER

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

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



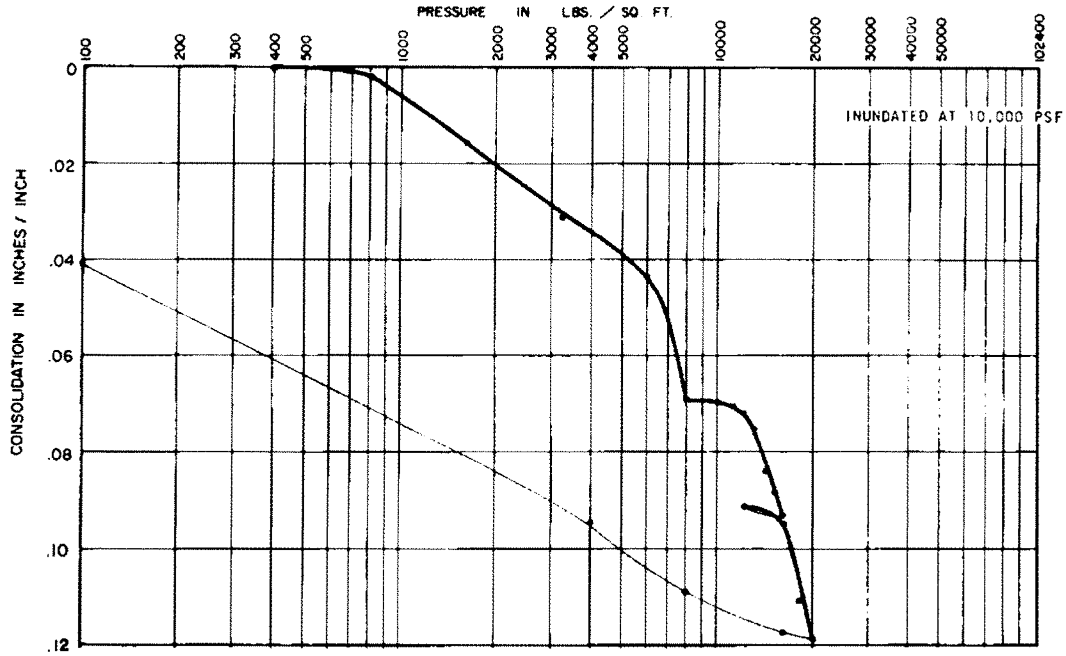
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	82	-78	10	CL	GREENISH GRAY SILTY CLAY	46	24	43	42	78	82



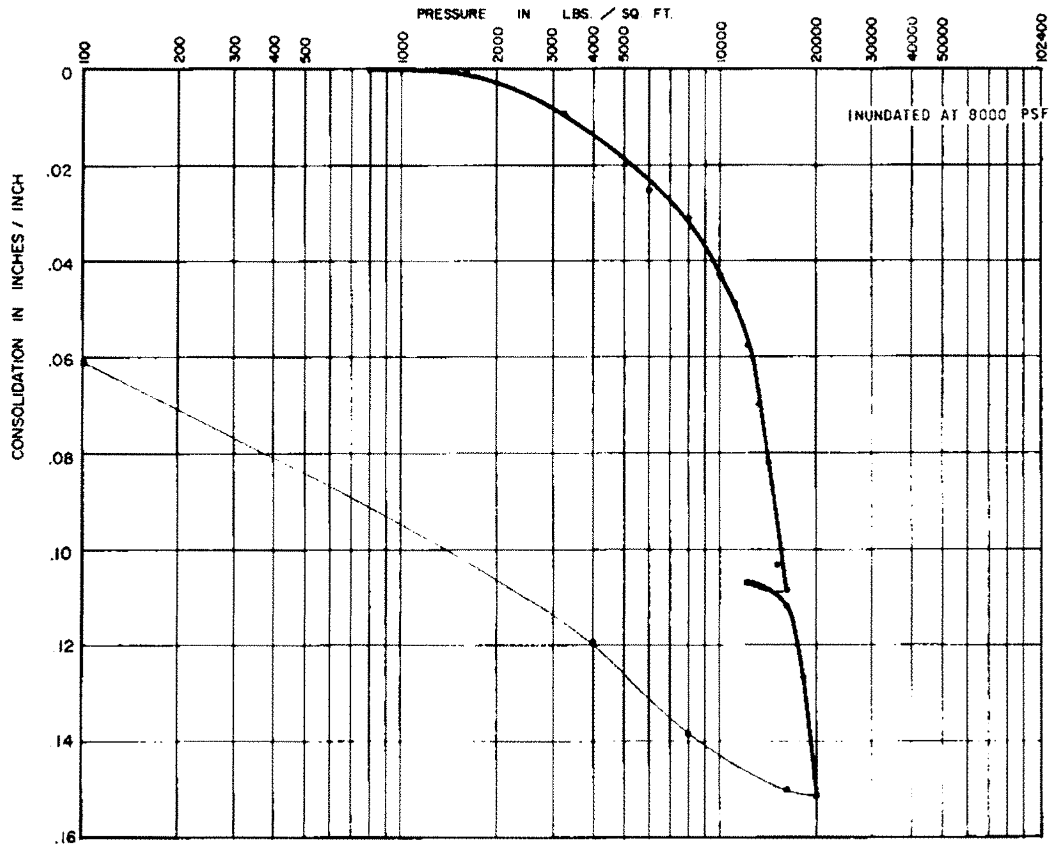
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
2	94	-90	16	CH	GREENISH GRAY SILTY CLAY	61	26	37	38	85	85

CONSOLIDATION TEST DATA

DAMES & MOORE



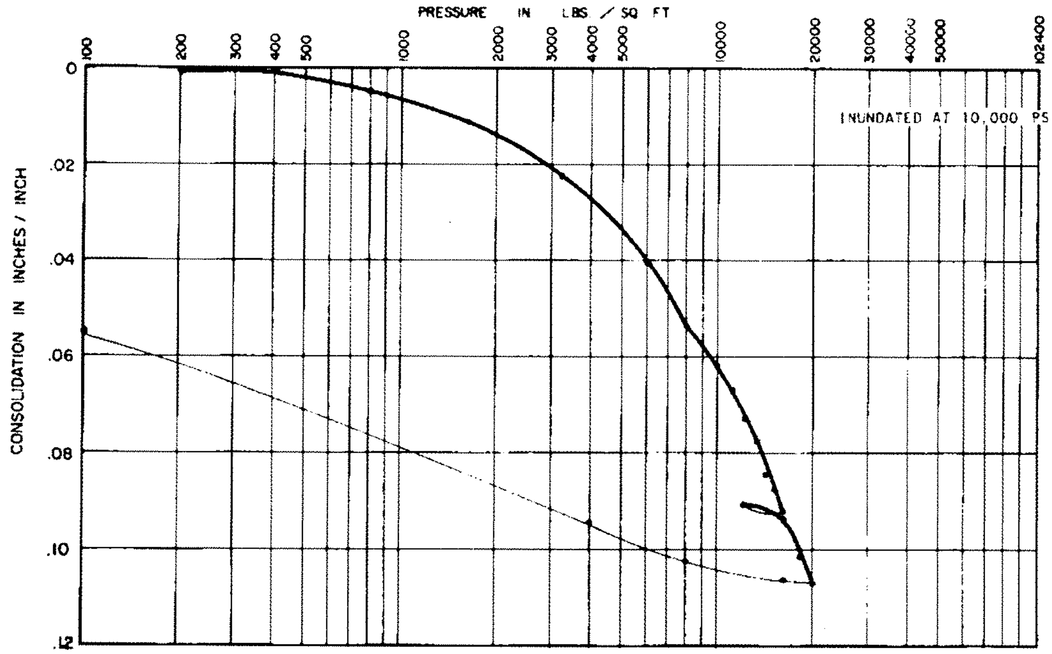
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	104	-100	12	CH	GRAY SILTY CLAY	63	26	41	39	80	83



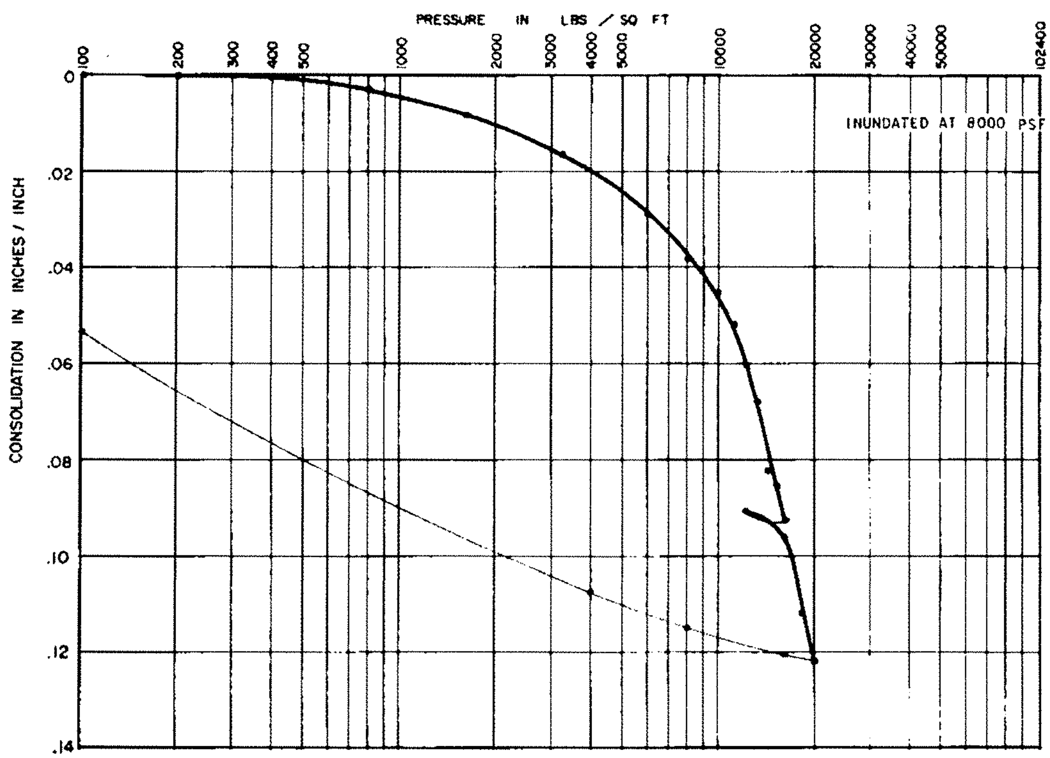
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	114	-110	13	CH	GRAY SILTY CLAY	69	31	49	47	72	76

CONSOLIDATION TEST DATA

DAMES & MOORE



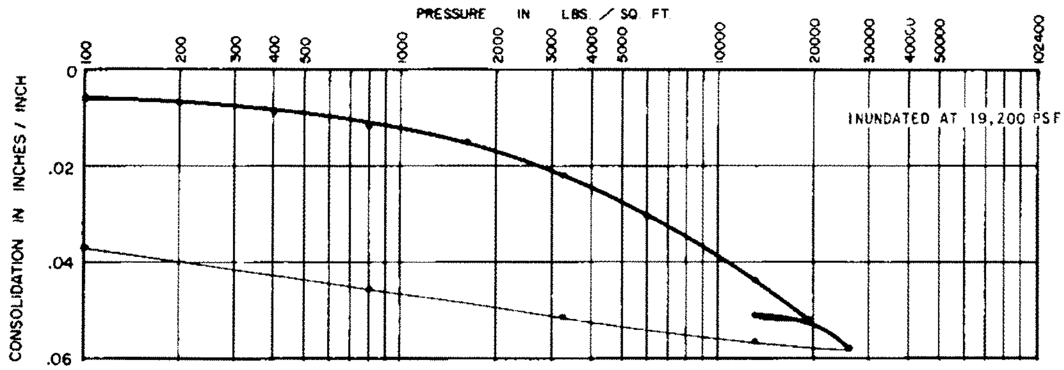
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	125	-121	14	CL	GRAY SILTY CLAY WITH SMALL GRAVEL	38	20	26	25	99	104



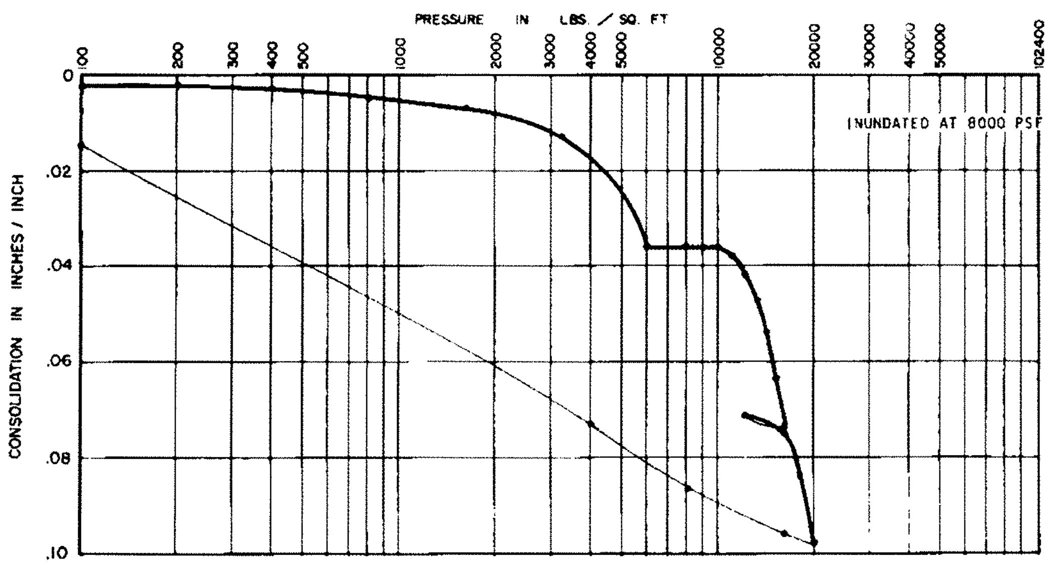
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	140	-136	15	ML	GRAY CLAYEY SILT	48	28	46	41	78	83

CONSOLIDATION TEST DATA

DAMES & MOORE



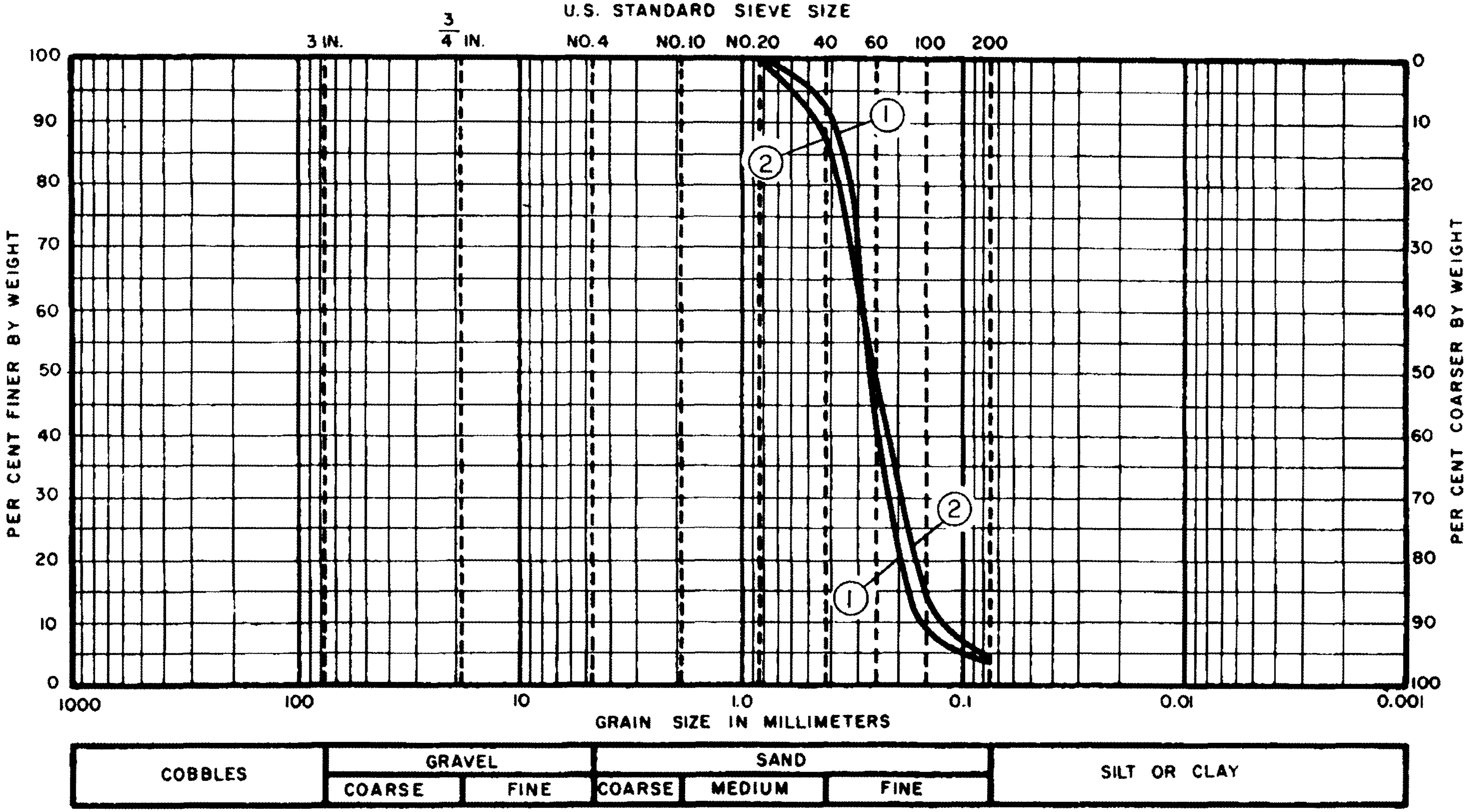
BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
1	230	-226	20	CL	GREENISH GRAY SANDY CLAY	30	18	18	16	112	117



BORING	DEPTH IN FEET	ELEVATION IN FEET	SAMPLE	SYMBOL	CLASSIFICATION	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %		DRY DENSITY PCF	
								BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
2	136	-132	21	ML	GRAY SILTY CLAY	51	24	47	46	73	74

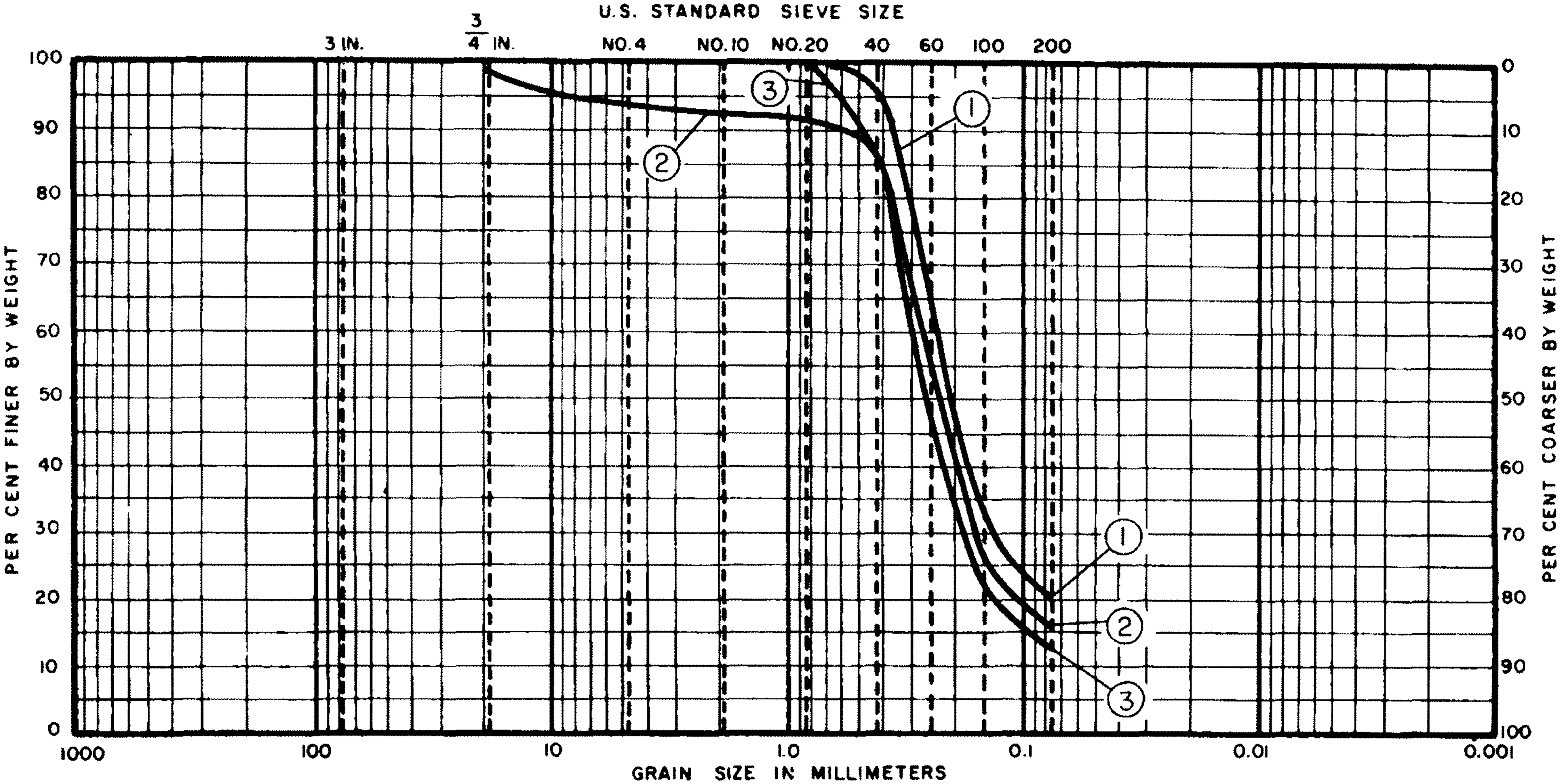
CONSOLIDATION TEST DATA

BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
1	51.5'	-47.5'	5			SP	BROWN FINE SAND	1
2	48.0'	-44.0'	7			SP	BROWN FINE SAND	2



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

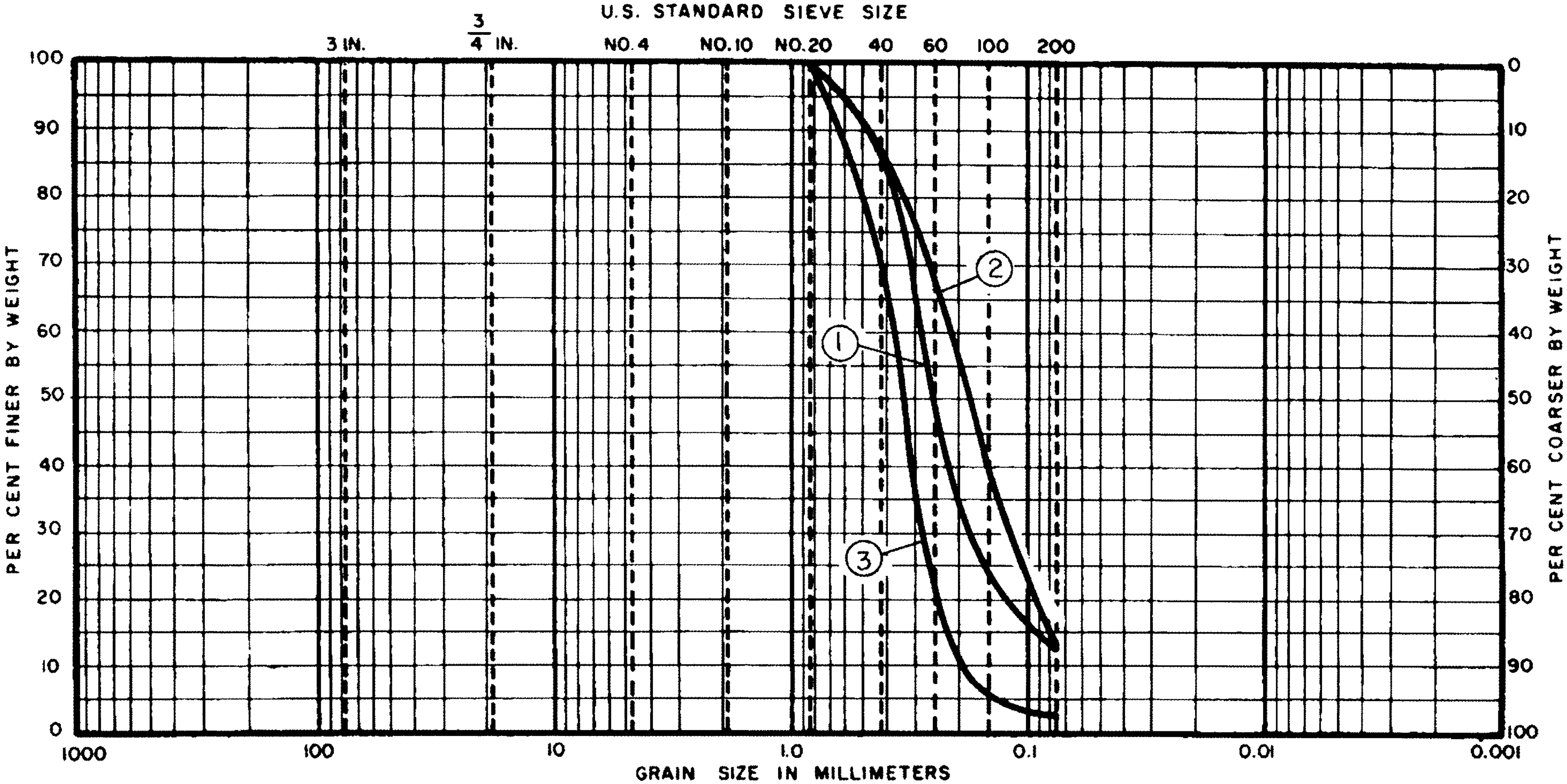
BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
1	60.5'	-56.5'	6 A			SM	GREENISH GRAY SILTY FINE SAND	1
2	52.5'	-48.5'	8			SM	BROWN SILTY FINE SAND	2
2	57.5'	-53.5'	9			SP/SM	BROWN SILTY FINE SAND	3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

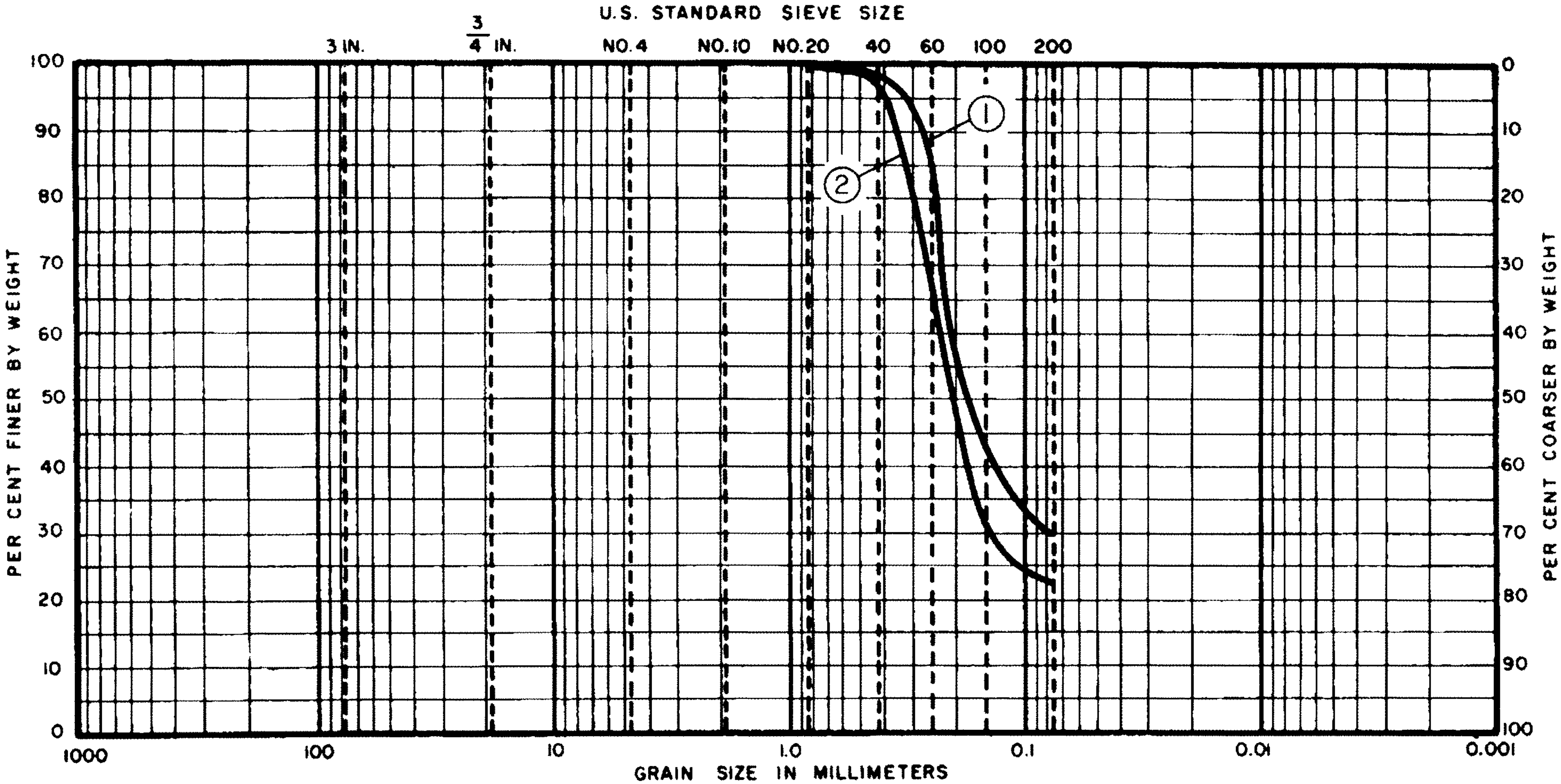
BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
1	65.5'	-61.5'	7			SM	BROWN SILTY FINE SAND	1
2	62'	-58.0'	10			SM	BROWN SILTY FINE SAND	2
2	68'	-64.0'	11			SP	BROWN FINE TO MEDIUM SAND	3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

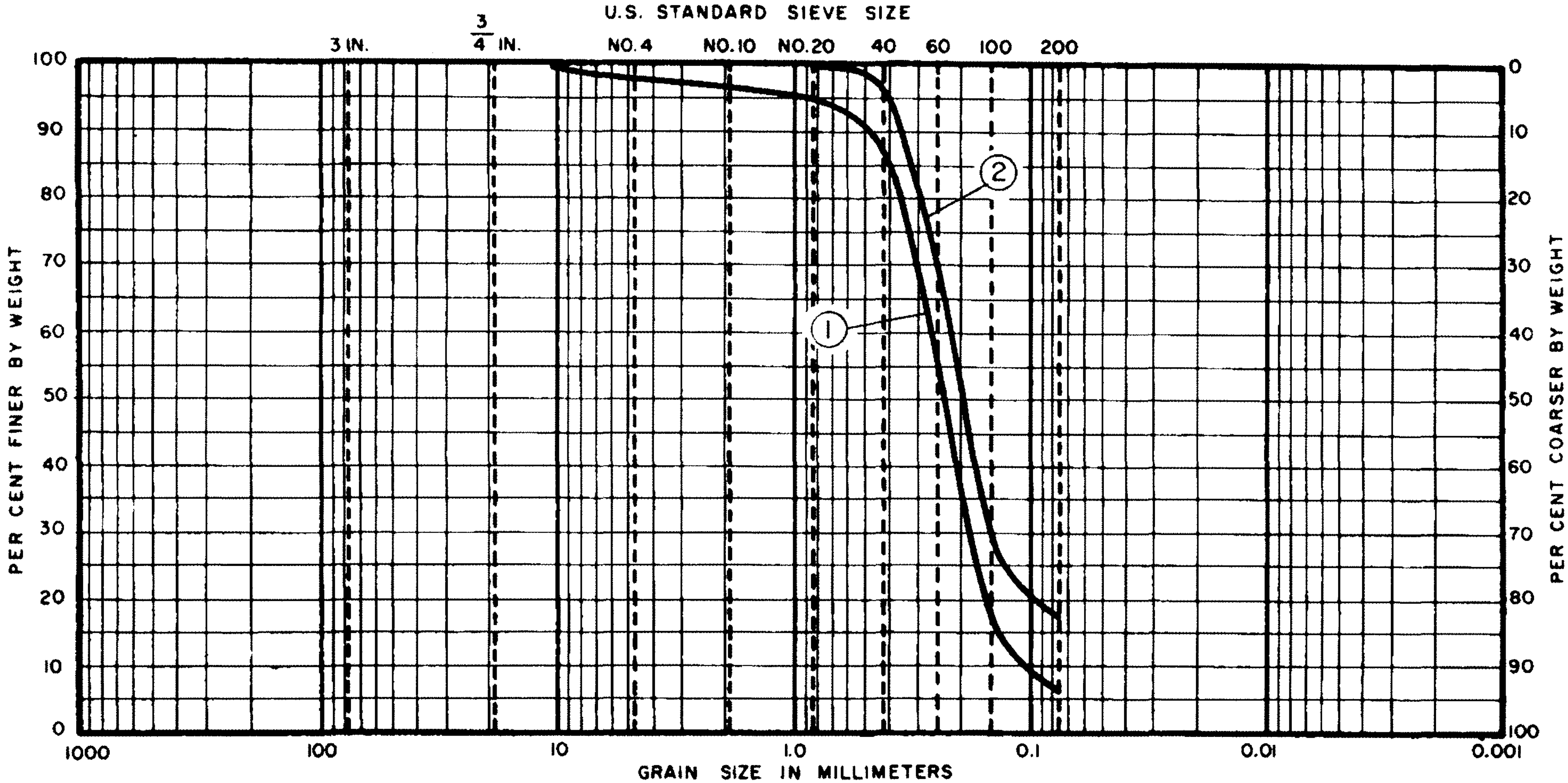
BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
1	72.5'	-68.5'	8			SC	GREENISH GRAY CLAYEY FINE SAND	1
2	73'	-69.0'	12 A			SM	GREENISH GRAY SILTY FINE SAND	2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

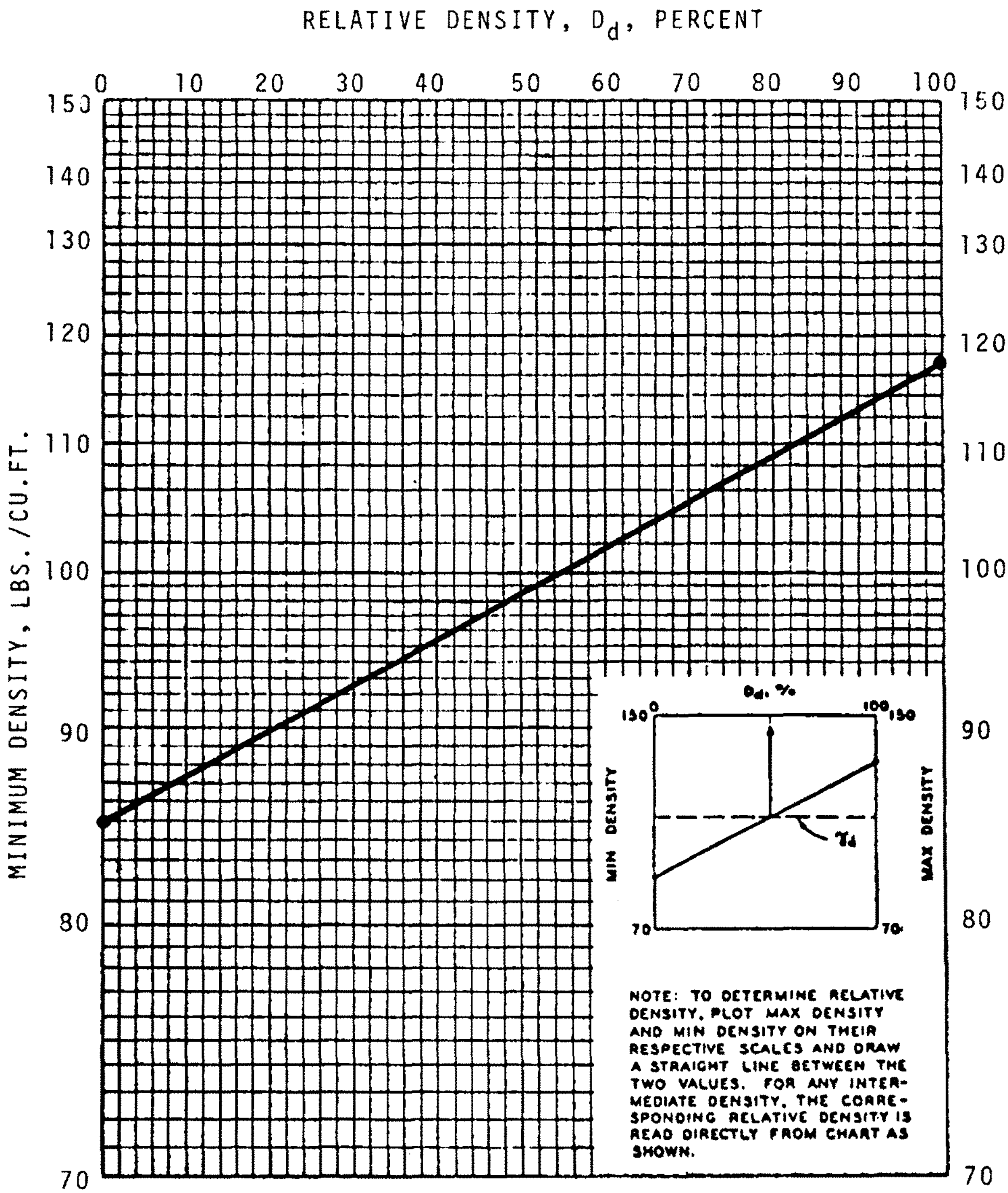
GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
2	79'	-75'	13			SP	GREENISH GRAY FINE SAND	1
2	101'	-95'	17 A			SM	GREENISH GRAY SILTY FINE SAND	2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)



SAMPLE:

COMBINED SAMPLE OF:

BORING 1, SAMPLE NO.5 FROM ELEVATION -48'

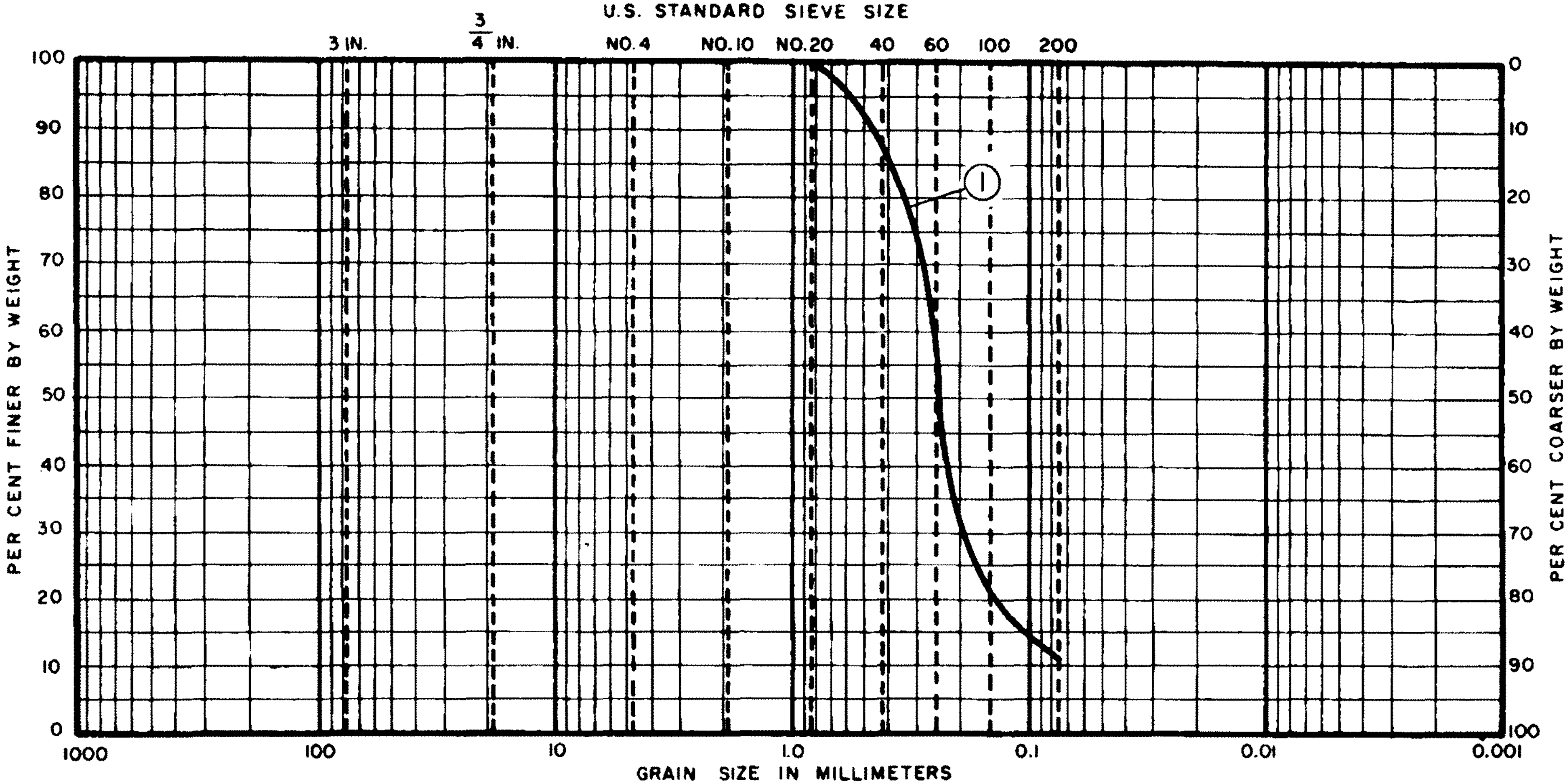
BORING 2, SAMPLES NO.7 THROUGH NO.11 FROM ELEVATIONS -44' TO -64'

CLASSIFICATION OF COMBINED SAMPLE: BROWN FINE SAND (SP-SM)

GRAPHICAL DETERMINATION OF RELATIVE DENSITY

DAMES & MOORE

BORING	DEPTH	ELEV.	SAMPLE	LIQUID LIMIT	PLASTIC LIMIT	SYMBOL	SOIL CLASSIFICATION	KEY
COMBINED SAMPLE:						SP/SM	BROWN FINE SAND	1
1		-48'	5					
2	-44' to-64'		7 through 11					



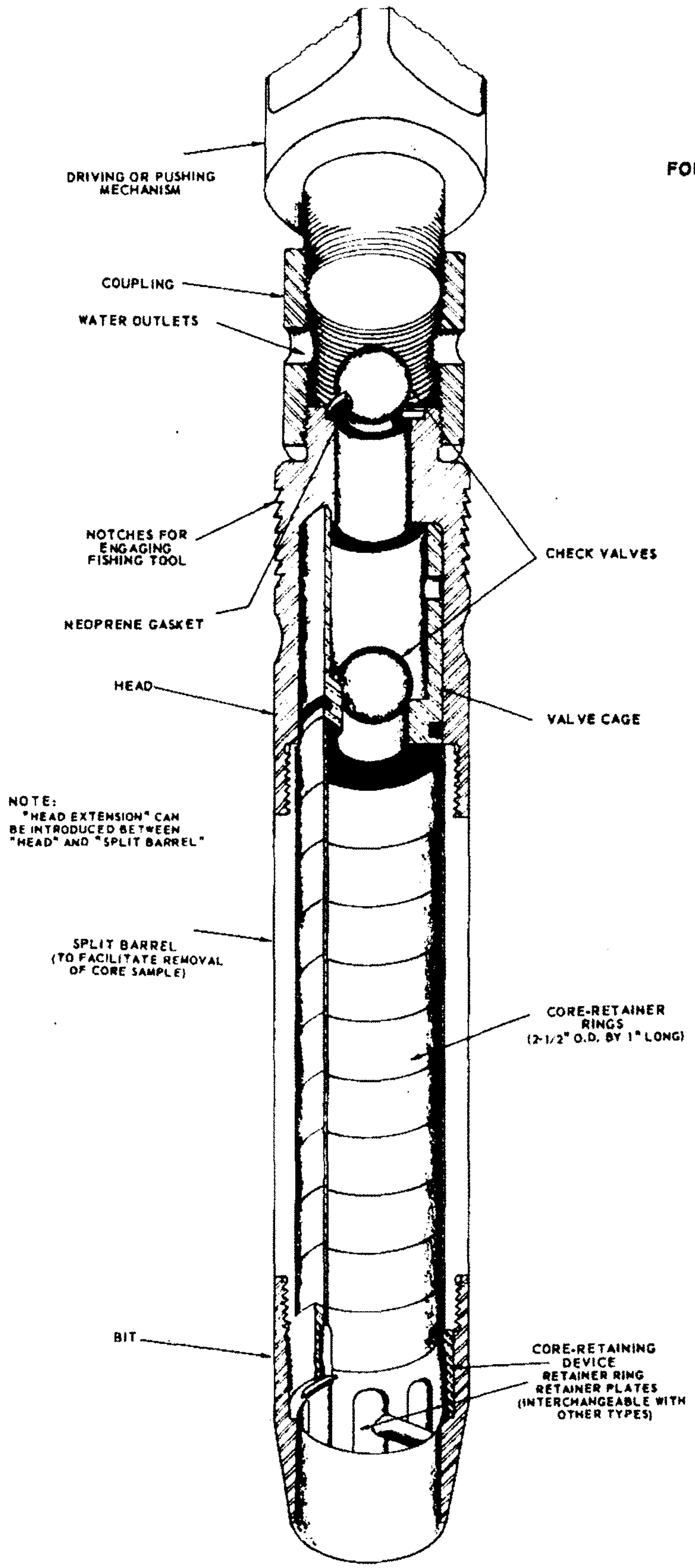
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

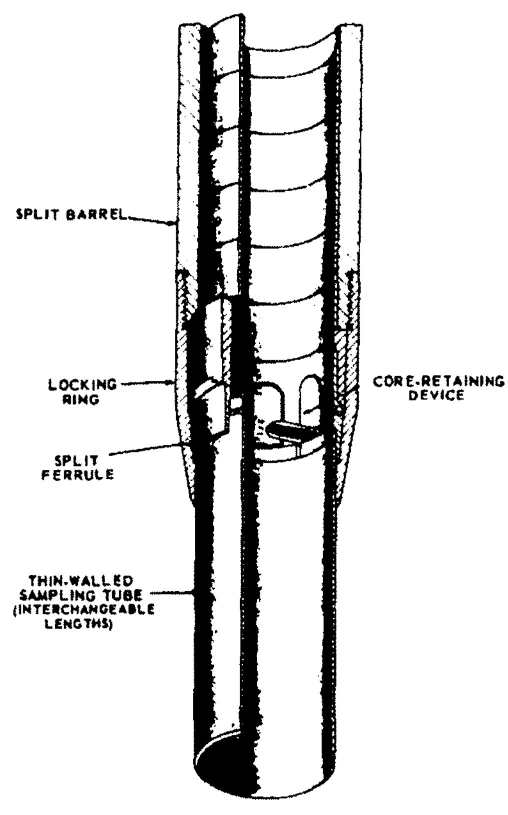
DAMES & MOORE

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



ALTERNATE ATTACHMENTS



417.9 (5-68)

APPENDIX B

P.G.&E. BUILDING LOAD TESTS

Eighteen-inch, prestressed concrete piles were used for support of this 34-story building. Dead plus live design loads were 90 tons. The piles averaged 20 feet in length and were driven with followers from a single basement elevation to penetrations of 4 to 10 feet into the bearing sands which underlie the bay mud about 60 feet below street level. Piles were driven with an 014 (42,000 foot-pounds per blow) single-acting steam hammer. At design tip elevation, a driving resistance of 100 blows per foot was required. Piles were permitted to stop within one foot of the bottom of predrilled holes if refusal driving was encountered (35 blows per inch). If specified driving resistance was not encountered at the design tip elevation, piles were overdriven one foot and stopped, regardless of blow count. Piles which did not meet driving criteria were assigned a reduced capacity. Column footings were evaluated on the basis of total capacity, and extra piles were added only if total capacities were below building column loads.

Prior to production pile driving, piles were driven at 12 column locations throughout the building area. Twelve piles were driven with followers as planned for production piles and 12 without to evaluate the relative effect of the follower on pile driving resistance. Two locations were chosen for pile load tests; one of high driving resistance and one of relatively easy driving. An additional load test pile was driven at each of these locations. Temporary support in the fill and bay mud to design pile cutoff was eliminated by predrilling an oversized hole to that depth prior to pile driving. In the hard driving area, the load test pile was driven to a final resistance of 21 blows per inch, 6½ feet into the bearing sands. The other load test pile, which encountered easier driving, penetrated 10 feet into the sands with driving resistances of about 10 blows per inch for the last 5 feet. Prior to excavating the two additional basements, load tests on these piling indicated a failure capacity for the hard driving pile of 370 tons and 380 tons for the easier driving. Basement excavation would later remove about one-half of the overburden pressure in the bearing sands. It was conservatively estimated that this would have a comparable reduction in the pile capacity. This condition required a load test capacity of 360 tons for a 90-ton design load pile. This factor of four (4) was to satisfy the requirement of the San Francisco Building Code for a factor of safety of two (2) on pile design loads, times two (2) for reduction in bearing capacity of the sands. After excavation to pile cutoff elevation (about 50 feet below street level), the easy driving test pile (failure load initial test 380 tons) was cut off and retested to a failure load of 250 tons. This 250-ton failure capacity was greater than the evaluated failure capacity of about 220 tons, for the condition where the pile had not been previously tested. This test indicated that the reduction in pile capacity was less than the one-half indicated by conventional soil mechanics theory. This difference is partially attributed to a maximum or limiting effective overburden pressure for the initial pile load test condition and partially to some slight cementation in the sand strata which is not lost by reduction in overburden pressure.