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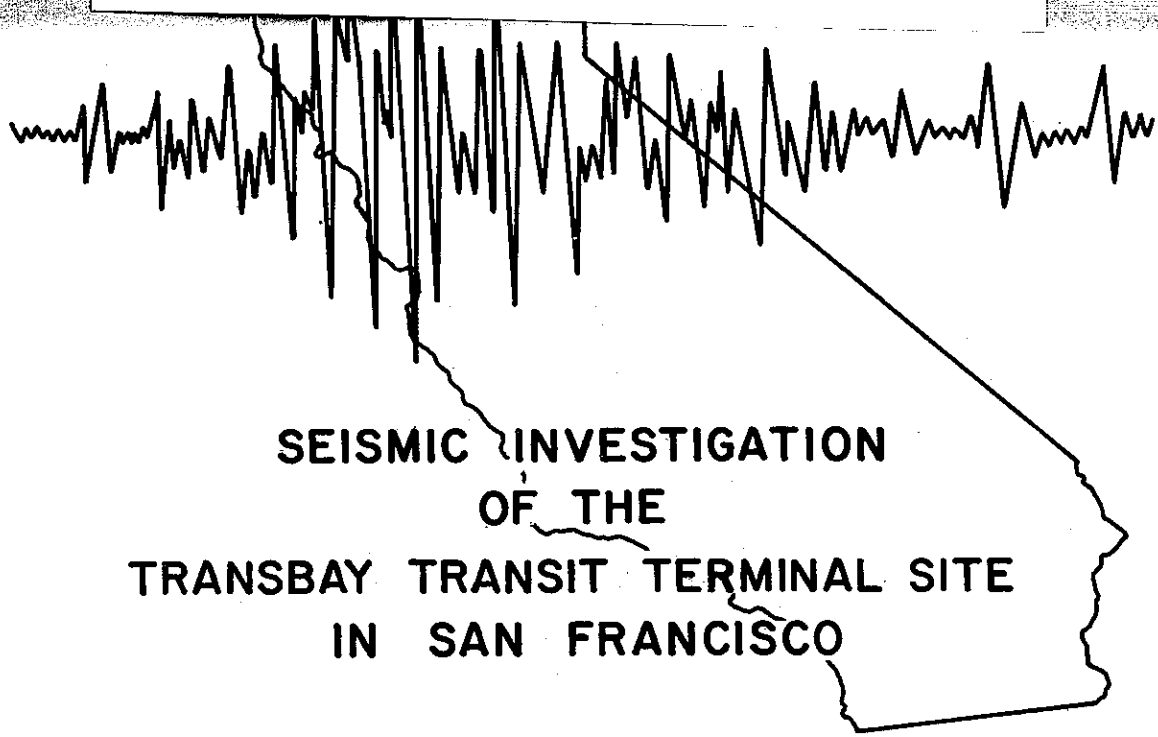

SEISMIC INVESTIGATION OF THE TRANSBAY TRANSIT TERMINAL SITE IN SAN FRANCISCO

TRANSLAB

CALIFORNIA DEPARTMENT OF TRANSPORTATION

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State of California

Business and Transportation Agency

Memorandum

To : All District Directors

Date: December 17, 1973

File :

From : DEPARTMENT OF TRANSPORTATION — Division of Highways
Transportation Laboratory

Subject:

The Transportation Laboratory has acquired the capability of analytically evaluating the seismic ground response of proposed construction sites. The attached report entitled "Seismic Investigation of the Transbay Terminal Site in San Francisco" presents the results of our first such study conducted entirely in-house.

The purpose of the investigation was to evaluate in somewhat general terms a rather large site to aid in assessing the feasibility of proposed development schemes. The approach and methods used in the study represent the current state-of-the-art and therefore may be of interest to your personnel who are involved in aseismic design activities.

John L. Beaton

JOHN L. BEATON
Chief, Transportation Laboratory

RHP:kk

Attachment

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State of California
Department of Transportation
Division of Highways
Transportation Laboratory

November 19, 1973

Transbay Transit
Terminal Project
(Lab Auth 662564)

Mr. L. B. Steedman
Chief, Division of
Administration Services
Sacramento, CA

Dear Sir:

Submitted herewith is the report:

SEISMIC INVESTIGATION

OF

THE TRANSBAY TRANSIT TERMINAL SITE

IN

SAN FRANCISCO

Study made by . . . . . Foundation Section
Under general direction of . . . . . Raymond A. Forsyth
Work supervised by . . . . . Donald L. Durr
Project Director . . . . . R. H. Prysock
Project Engineer . . . . . Kenneth A. Jackura
Project Assistant . . . . . D. J. Kuhl
Report by . . . . . Kenneth A. Jackura

Very truly yours,

JOHN L. BEATON
Chief, Transportation Laboratory

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TECHNICAL REPORT STANDARD TITLE PAGE

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16. ABSTRACT

The findings of a seismic investigation of the Transbay Transit Terminal site are presented, along with data describing geophysical characteristics of the underlying soils. Ground response analyses at critical locations within the boundaries of the site for earthquakes ranging in magnitude from 5.5 to 8+ are reported. Structural response spectra have been included with a discussion of their implications concerning the height of future structures. Liquefaction potential of saturated sand layers is evaluated and deformation of critical subsurface strata computed.

17. KEY WORDS Earthquake analysis, seismicity, geophysical surveys, San Francisco Geology, ground response studies.	18. DISTRIBUTION STATEMENT

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INTRODUCTION

This report presents the results of a geologic and seismic study of the Transbay Transit Terminal site in San Francisco. The purpose of the study was to provide sufficient ground response information to allow a general evaluation, with respect to earthquake and geologic hazards, of the proposed plans for the terminal property. Results of this study will be included in the Environmental Impact Statement (to be prepared by District 04) for the Transbay Terminal Future Utilization Study. The work reported herein was authorized by L. B. Steedman, Chief, Division of Administrative Services, on March 9, 1973, in a memorandum to J. L. Beaton, Chief, Transportation Laboratory.

The scope of this investigation was outlined in a memorandum dated February 13, 1973, from J. L. Beaton to L. B. Steedman, and included the following:

1. A review of pertinent literature and available soil engineering data. Information thus gained would serve as background and source material for describing known geologic and seismic conditions attendant to earthquake engineering problems likely to be encountered at the site.

2. A field investigation to: (a) secure soil samples for laboratory static and dynamic testing; (b) establish a suitable soil profile between ground surface and bedrock; and (c) conduct cross-hole seismic surveys to determine soil parameters for use in analytical work.

3. Laboratory testing as required, the extent of which would be dependent upon the quality of the field seismic survey results.

4. Computer analyses, using developed information as input, to estimate ground response in terms of acceleration and deformation.

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5. An evaluation, primarily in qualitative terms, of the effects of the estimated site ground response on the types of structures likely to be considered in fulfilling the recommended plans for the Terminal Project.

It should be emphasized that this study was not intended to provide complete information required for a detailed design effort. Consequently, the design of any structure for a particular site within the bounds of the Terminal Property will require further soil and foundation investigation, planned and executed in accordance with specific needs.

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SUMMARY

The purpose of this investigation was a determination of ground response at the transbay terminal site to potential earthquakes in the San Francisco Bay area. The "site", as used herein, included that area bounded by Main, Mission, Second, and Folsom Streets in San Francisco. Six representative areas within the site bounds were selected for detailed analysis. They collectively represent the soil and geological conditions of the entire terminal site and were selected based upon the four most favorable redevelopment schemes as outlined in the Future Utilization Study (1). This method of selecting areas for detailed study ensured that individual land parcels common to all four utilization schemes would be evaluated for seismic response.

Seismicity at the site was studied by considering the earthquake record within central coastal California encompassing the northern section of the San Andreas Fault. The study area, approximately 55 miles wide, extended from a line about 40 miles north of San Francisco, southward approximately 200 miles. Earthquakes within this region were found to be generated at a frequency level of about 3 times greater than California as a whole.

Four earthquakes encompassing the range of event severity that would be expected to affect the subject site were selected as input source motions for analyzing ground response.

Earthquakes selected were as follows:

A. Local Events

1. Small magnitude (5.5) - Golden Gage Park Quake of 1957.

2. Intermediate magnitude (7.0) - San Fernando Quake of 1971 (Castaic Record, Modified).

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3. Large magnitude (8+) - Berkeley Quake (artificial).

B. Distant Event

1. Large magnitude (8+) - San Fernando Quake of 1971 (Castaic Record, Modified).

Base rock motions caused by the above earthquakes were transformed to the soils overlying base rock at the site by use of the SHAKE-3 computer program (2). Soil profiles and properties for use in the program were determined in the field by drilling, sampling, and up-hole and cross-hole seismic surveys. Laboratory tests were also conducted for supplemental and supporting information. Data from the three local events were used in evaluating ground response at all six of the small study areas. The effects of the one distant event were evaluated at two of the small areas.

Expected acceleration levels for potential structures at the site were estimated by constructing response spectra which depict the magnitude of generated seismic forces for a range of structural periods. A structural damping value of 5% was used. These response spectra illustrate critical structural heights most affected by the input source motion. Typical results show that short period buildings (5 stories) would develop maximum acceleration levels when located on the shallow soil deposits characteristic of the sandier eastern sector of the site; longer period structures (40 stories) would experience the greatest acceleration levels when founded on the deeper, long period deposits characteristic of the western sector.

Liquefaction potential of the fill and underlying natural sand deposits was evaluated by utilizing simplified, empirically validated procedures. Liquefaction potential was considered to be negligible for all but the closely centered M = 8+

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earthquakes. Saturated fills and certain saturated relatively shallow native sand deposits, however, were found likely to liquefy if subjected to this large event. Further field and laboratory testing will be necessary to identify more completely the extent of potentially critical liquefaction areas.

Sub-surface displacements were also studied. Analyzed soil deposits, composed partially of soft bay mud, developed as much as 14 inches of maximum relative displacement, with individual results dependent on severity of event and bay mud thickness. However, relative displacement values were negligible for locations where bay mud was not present. It therefore appears that pile foundations should be used and designed to accomodate relative displacements of the magnitude indicated above.

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TRANSBAY DEVELOPMENTAL AND INVESTIGATIVE PLAN

The Transbay Transit Terminal in San Francisco is the western terminus for existing Transbay commuter services. The use of this terminal will be eliminated upon completion of the Bay Area Rapid Transit System in 1973. To continue maximum utilization of the land area of the terminal, redevelopment schemes for part or all the complex have been proposed.

A Transbay Terminal future utilization study (1) conducted in 1971 resulted in classifying various developmental proposals into four concepts: 2, 5, 10 and 20 (see Figure 1). Each concept was concerned with developing various parcels within the Transbay complex.

The existing Terminal Building site, the westerly sector which includes 5 land parcels, is proposed for redevelopment in all concepts with certain surrounding parcels comprising additional development features for concepts 10 and 20. Since the utilization study recommended adoption of concept 5, followed in preferential order by concepts 10, 20 and 2, the five westerly parcels were thus assured of redevelopment. As a consequence, field investigative efforts concentrated solely on the westerly parcels. Similarly, the subsequent theoretical earthquake analysis was primarily concerned with a seismic analysis of the westerly sector although other pertinent parcels were also evaluated. Information so gleaned would then be sufficient for a general seismic assessment of the entire Transbay complex.

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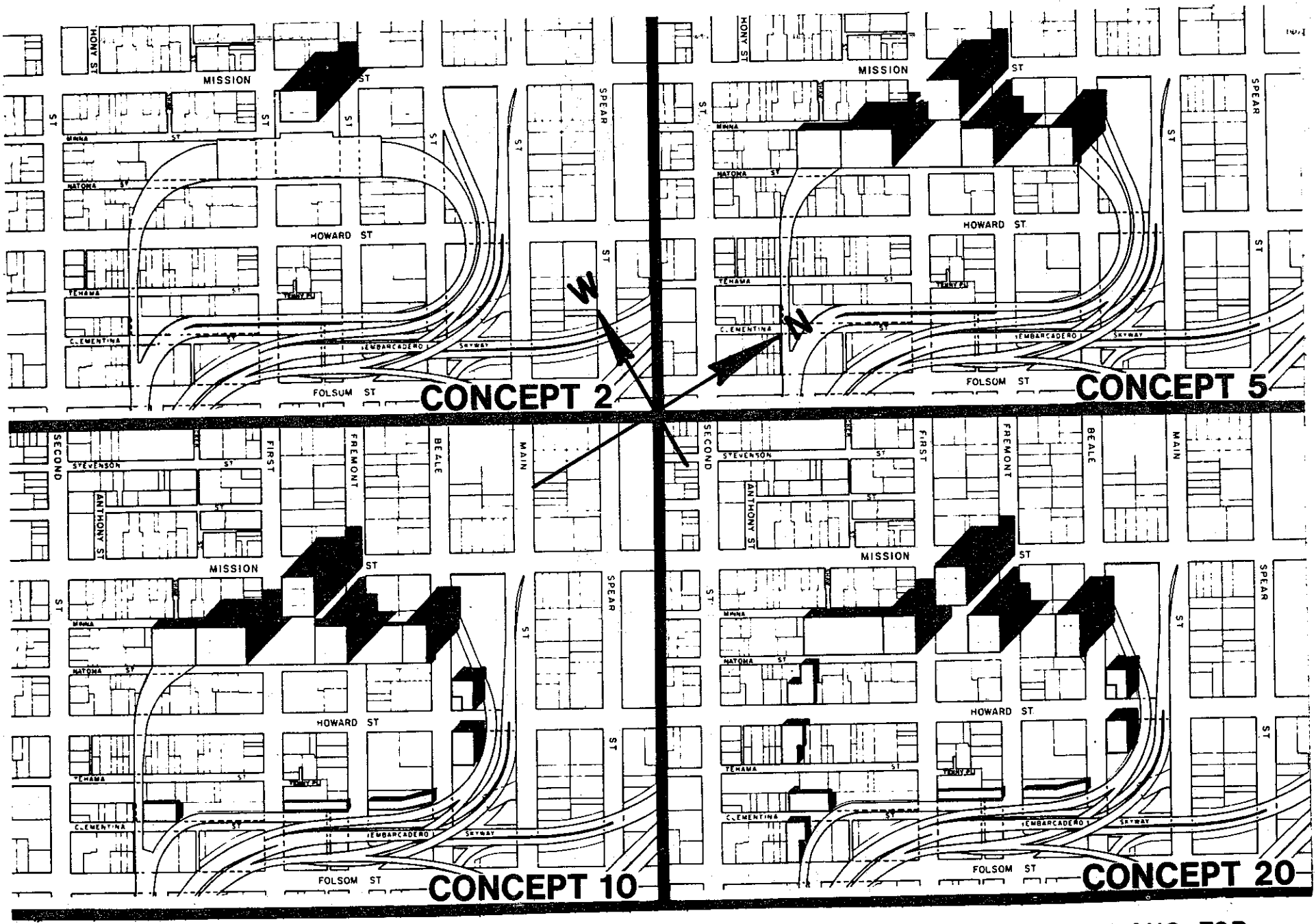

Fig. 1 VIEW OF TRANSBAY SITE ILLUSTRATING PARCEL DEVELOPMENTAL PLANS FOR 4 DIFFERENT ULTILIZATION CONCEPTS. (1)

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GEOLOGY

The San Francisco Bay Region has been a locus of earth movement for the past 60 million years. The Bay, itself, is a drowned river system which occupies a depression bounded by two highs, the Berkeley Hills on the east and the Montara Mountains on the west. An active fault parallels each of the uplifted areas; the San Andreas on the west and the Hayward on the east. The area between, within which the terminal building is located, is underlain by rocks of the Franciscan Formation of Jurassic age.

The terminal building is at the location of the former Yerba Buena Cove, which was filled in over the years to provide flat land for the city's growth. The shore line of 1849 was at about the middle of the present terminal building. By the year 1852 the shore line had been moved beyond the northern end of the building. This resulted in about 15 feet of fill at the north end of the terminal site. Beneath this fill is about 175 feet of younger bay mud, clay and sand deposits, which overlie the Franciscan bedrock of shale and sandstone. The bedrock surface has considerable relief. Yerba Buena Cove was itself a steep walled submerged valley that had been filled in by sediments.

Most of the northeast portion of the San Francisco Peninsula is covered by man-made structures such as streets and buildings, so that geologic observations are limited to a few scattered outcrops and information from excavations and borings. Hand samples of rock specimens collected from outcrops at Rincon Hill, Nob Hill and Telegraph Hill were identified as Franciscan sandstone. Bedding plane attitudes were measured at each of these outcrops, and used in conjunction with those shown on the U.S. Geological Survey map (3). The bedrock structure, as evidenced by attitudes on Telegraph, Russian and Nob Hills, seems

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to indicate a north plunging syncline with the axis along Columbus Avenue and continuing through the Terminal Building site. This is also the interpretation of Schlocker, Bonilla and Radbruch of the Geological Survey (3).

The NW-SE trend of the syncline is also the general orientation of most faults in this area. Faulting in the Franciscan Formation is usually marked by the presence of serpentine. A number of boreholes have been drilled in this area over the past several decades. Some of the drill logs identify bedrock in the borings as serpentine or serpentine gouge. Other borings, which should have encountered the same material, identify bedrock as sandstone, composed of assorted sand grains and chloritized greenstone fragments. Some surface evidence of a fault exists throughout this area. All of the outcrops contain small shear zones although most are of limited extent and exhibit no displacement. The valley between Telegraph Hill and Nob and Russian Hills has certain features that suggest a lineation. However, at this time, the plunging syncline seems to offer the best explanation of the structure.

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FIELD AND LABORATORY INVESTIGATIONS

General

Field investigations consisted of soil borings and geophysical surveys. The borings were used to delineate soil strata and to obtain soil samples for testing. Geophysical surveys were conducted to establish field dynamic properties.

Borings

During the period June 15 to July 8, 1973, four borings were made to depths of approximately -210 feet. Two borings were located in a parking lot on the northwest corner of Beale and Howard Streets and an additional two were located in a parking lot between Natoma and Minna Streets, at the southern end of the Transbay Terminal site. Locations of these borings were determined primarily by access requirements.

One boring at each location was utilized for delineating subsurface soil types, obtaining undisturbed 2.8-inch diameter tube samples, and conducting Standard Penetration Tests at select depths. The second hole at each location was established solely for the purpose of conducting cross-hole geophysical surveys.

Information obtained from the new borings was coordinated with data from previous borings of the Terminal site conducted in the 1930's by the Transportation Laboratory. These older borings precluded the need for a large number of new borings. Subsurface profiles of the site, developed from all collected information, are shown as six typical profiles; five transverse to the terminal building, Figures 2A through 2E, and one along the

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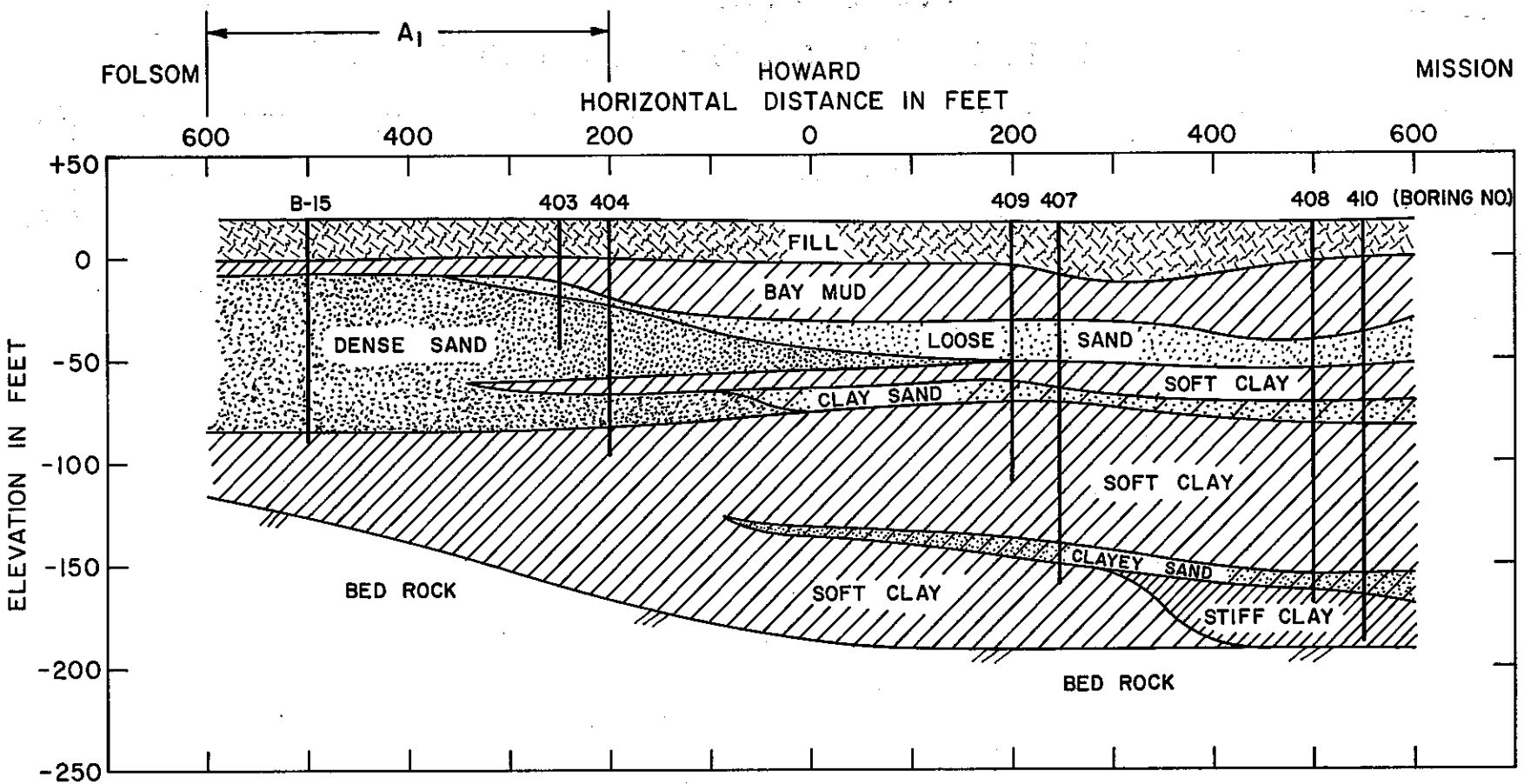

Fig. 2A SOIL SECTION PROFILE "A-A"
(ON BEALE STREET)

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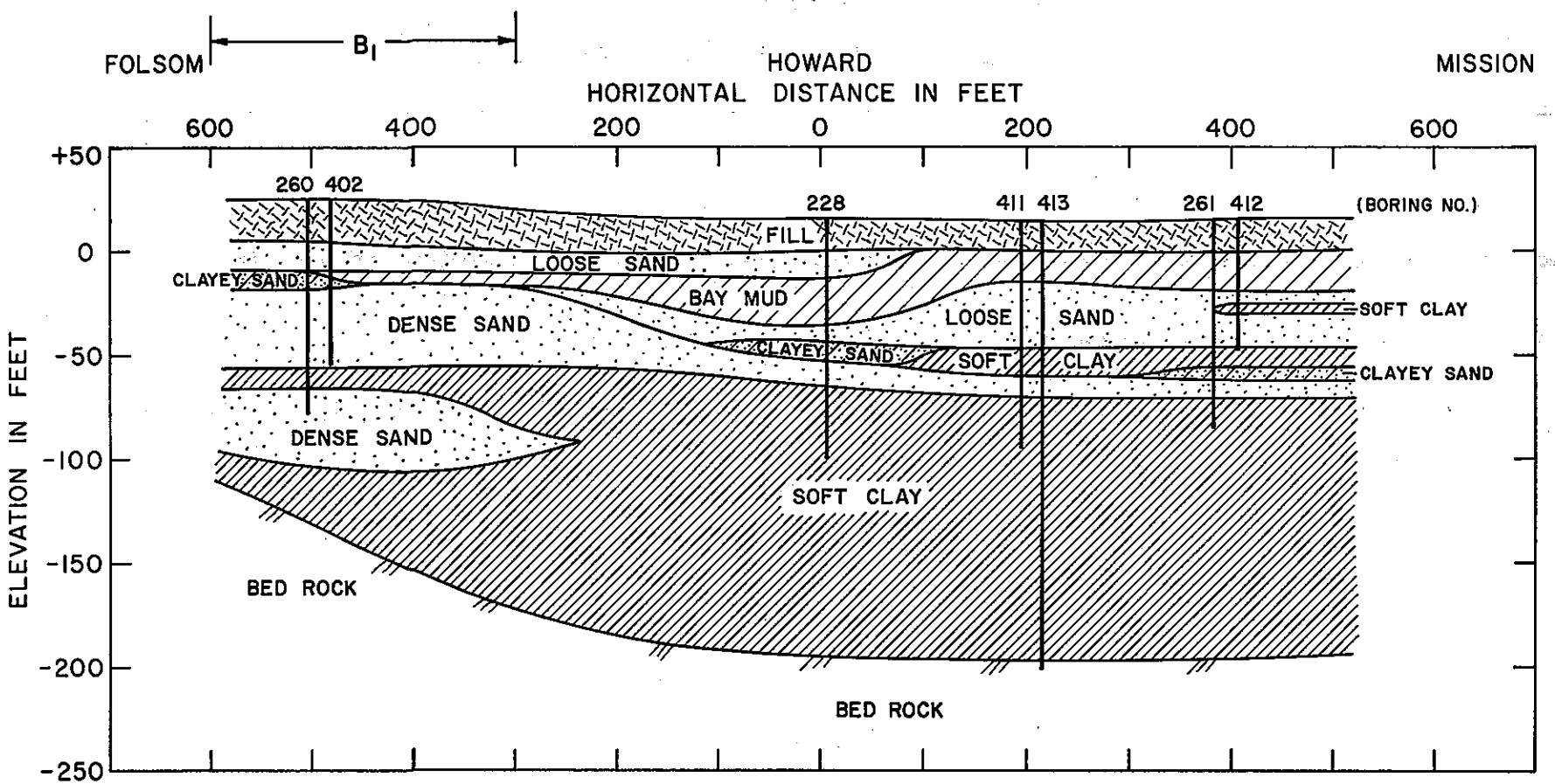

Fig. 2B SOIL SECTION PROFILE "B-B"
(ON FREMONT ST.)

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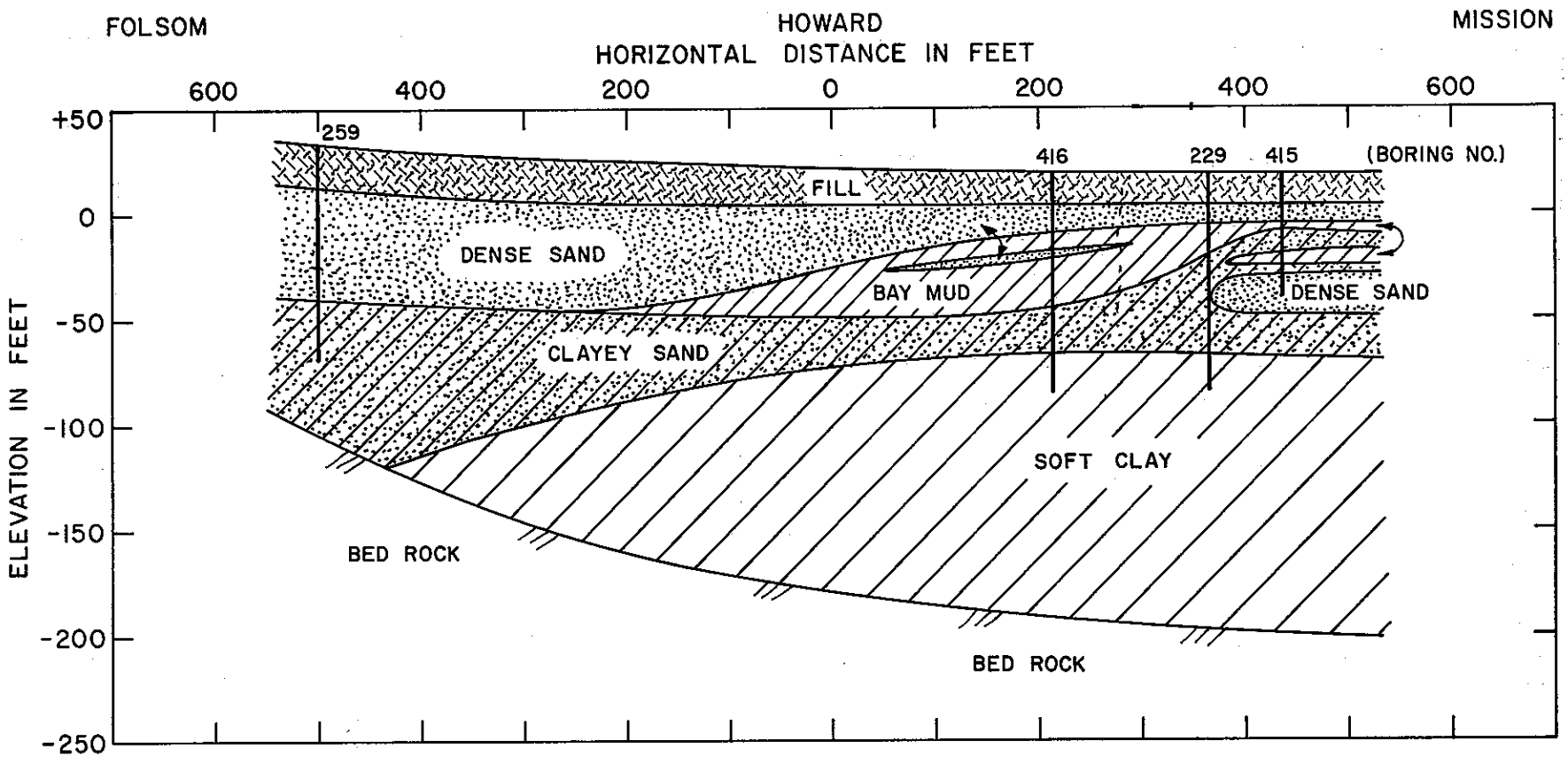

Fig. 2C SOIL SECTION PROFILE "C-C"
(ON FIRST STREET)

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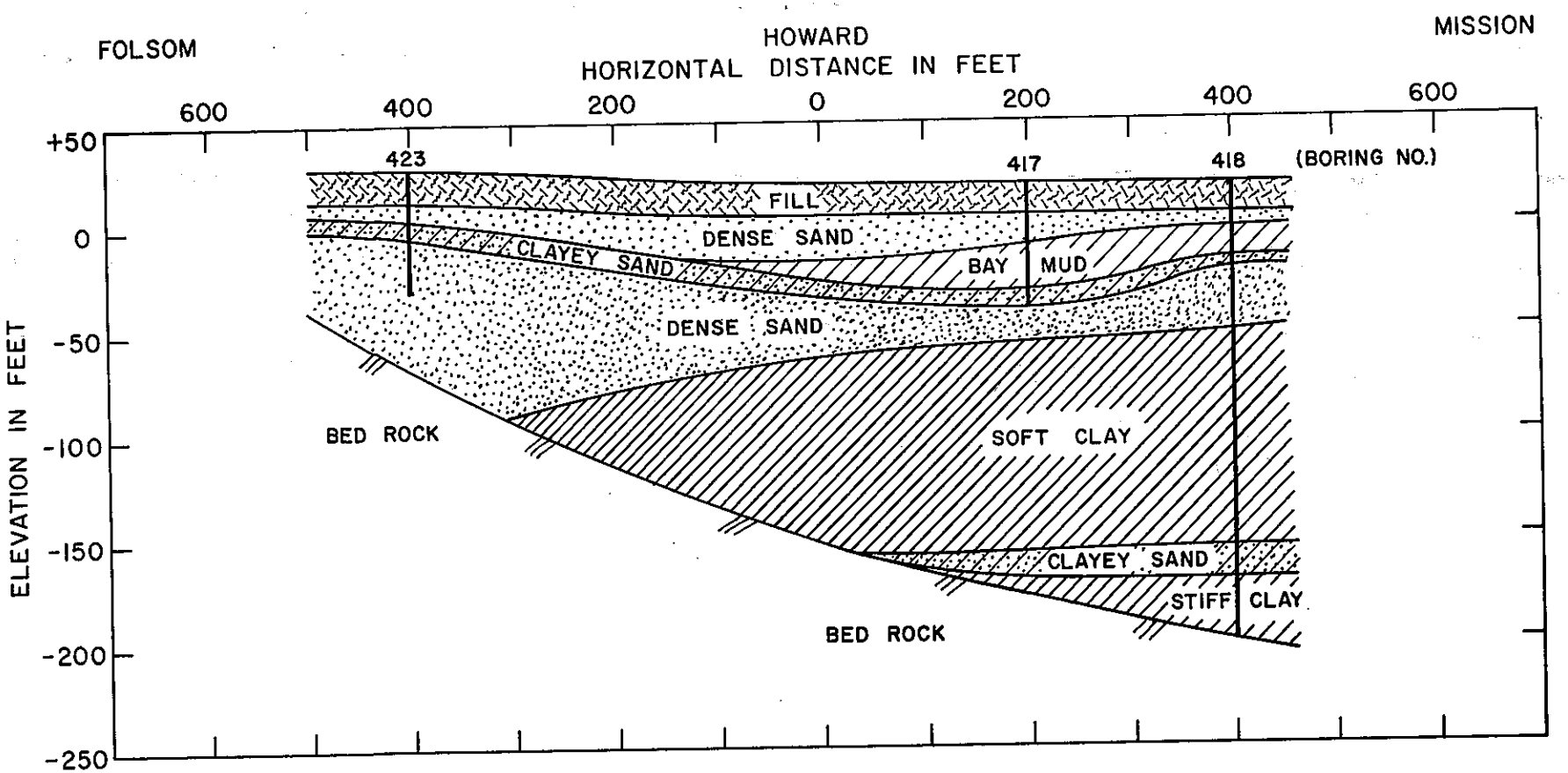

Fig. 2D SOIL SECTION PROFILE "D-D"
(BETWEEN 1st & 2nd STREET)

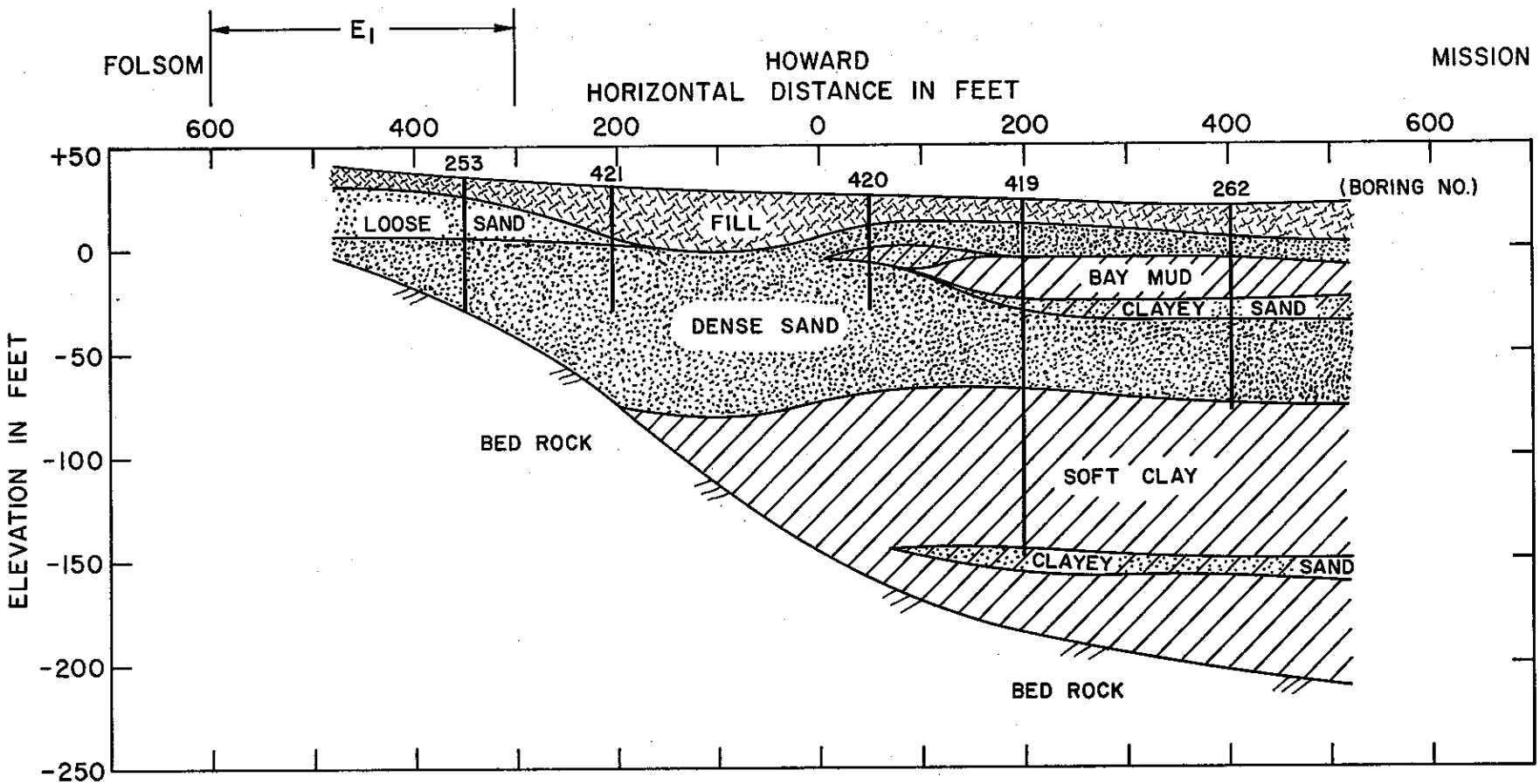

Fig. 2E SOIL SECTION PROFILE "E-E"
(BETWEEN 1st & 2nd STREETS)

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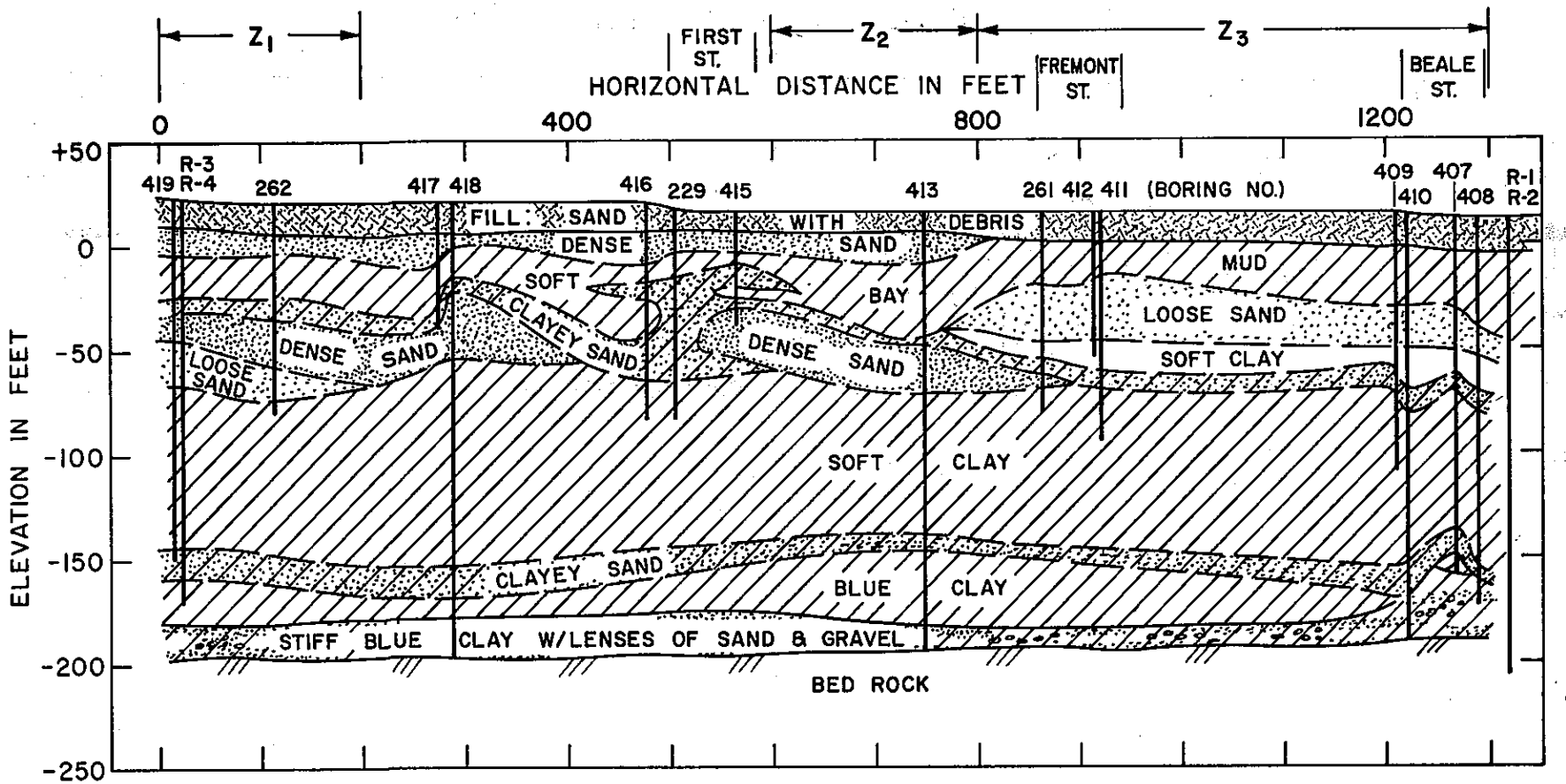

Fig. 2Z SOIL SECTION PROFILE "Z-Z"
(ALONG TRANSBAY TERMINAL BUILDING)

building, Figure 2Z (down the roadway centerline). The locations of these sections are shown on a plan map of the area, Figure 3, together with the locations of both old and new borings.

Retrieved undisturbed soft clay soil samples were subjected to static and dynamic laboratory tests. Other tests included grain size analysis of these specimens as well as grain size analysis of the sand strata encountered at shallow (less than 70 feet) depths. Static and dynamic tests were conducted to provide supplemental information to the geophysical surveys and as a check on observed field data. Laboratory and field data are included in the appendices.

Geophysical Investigations

Several geophysical methods were employed to determine the shear wave velocities of each of the different soil layers. Seismic measurements were obtained for each soil layer as well as bedrock at two locations, shown on Figure 3 as Locations 1 and 2. Knowledge of the in situ material wet densities and the shear wave velocities permitted determination of initial shear moduli values for each layer. Inclusion of these predetermined values into a "wave propagation" computerized program allows analysis of ground movement caused by earthquakes.

All borings penetrated to bedrock and each was cased with 4-inch PVC pipe to maintain an "open hole." Hole spacing was approximately 20 feet. Cross-hole and down-hole surveys were employed at Location 1 (northwest corner, Figure 3) while cross-hole surveys only were employed at Location 2 (southwest corner, Figure 3).

Blasting caps, detonated at various depth intervals, were used to generate seismic waves for the cross-hole surveys. The

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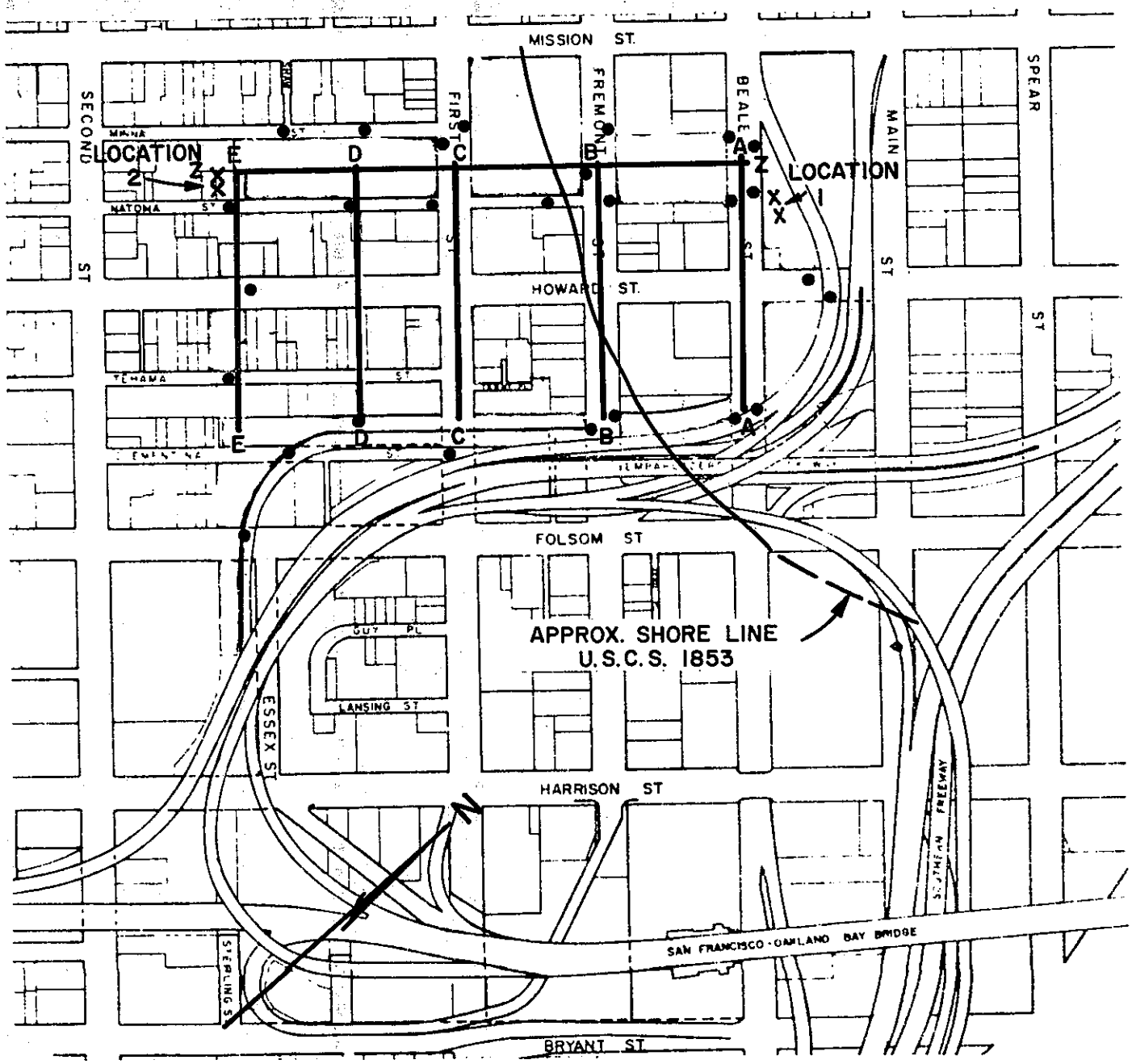

Fig. 3 PLAN MAP OF TRANS BAY SITE AND VACINITY SHOWING LOCATION OF 1936 BORINGS (•), 1973 BORINGS (x) AND ALIGNMENT OF THE SIX TYPICAL PROFILES A-A, B-B, C-C, D-D, E-E AND Z-Z.

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waves were detected by a 3-component geophone installed in the second hole at a parallel level. An inflatable neoprene packer positioned the phone against the casing wall. An Electro-Tech M4E amplifier with camera was utilized as the recording instrument.

Down-hole surveys at Location 1 were conducted for the upper 35 feet of material. The geophone was positioned at various depth intervals and energy waves were created at the surface by striking a wooden plank horizontally. Pure shear waves were thus created to enable clearer delineation of the shear wave velocities in the upper material strata.

Results of the geophysical tests are presented on Figure 4 and also compared to other investigators' seismic surveys of similar materials in the Bay Area. Variations between the two seismic survey sites for similar material strata are noted. Since there is some subjective interpretation of the exact shear wave arrival time, small differences can be expected. Coupling of the casing to the surrounding soil was assumed to be accomplished by the natural squeezing process of the soil strata, but which may not be truly effective in all cases. However, the distinguishing seismic differences, mainly noticed in the upper 50 feet or so, were attributed to geologic variations. Location 1 was on the north or bay side of the 1853 shore line while Location 2 lies south or inland of the old shore line; hence, depositional or geological differences could account for the variations.

Site Description Based on Boring Records

The following soil description is derived principally from the borings of 1936, supplemented by additional information developed during this investigation.

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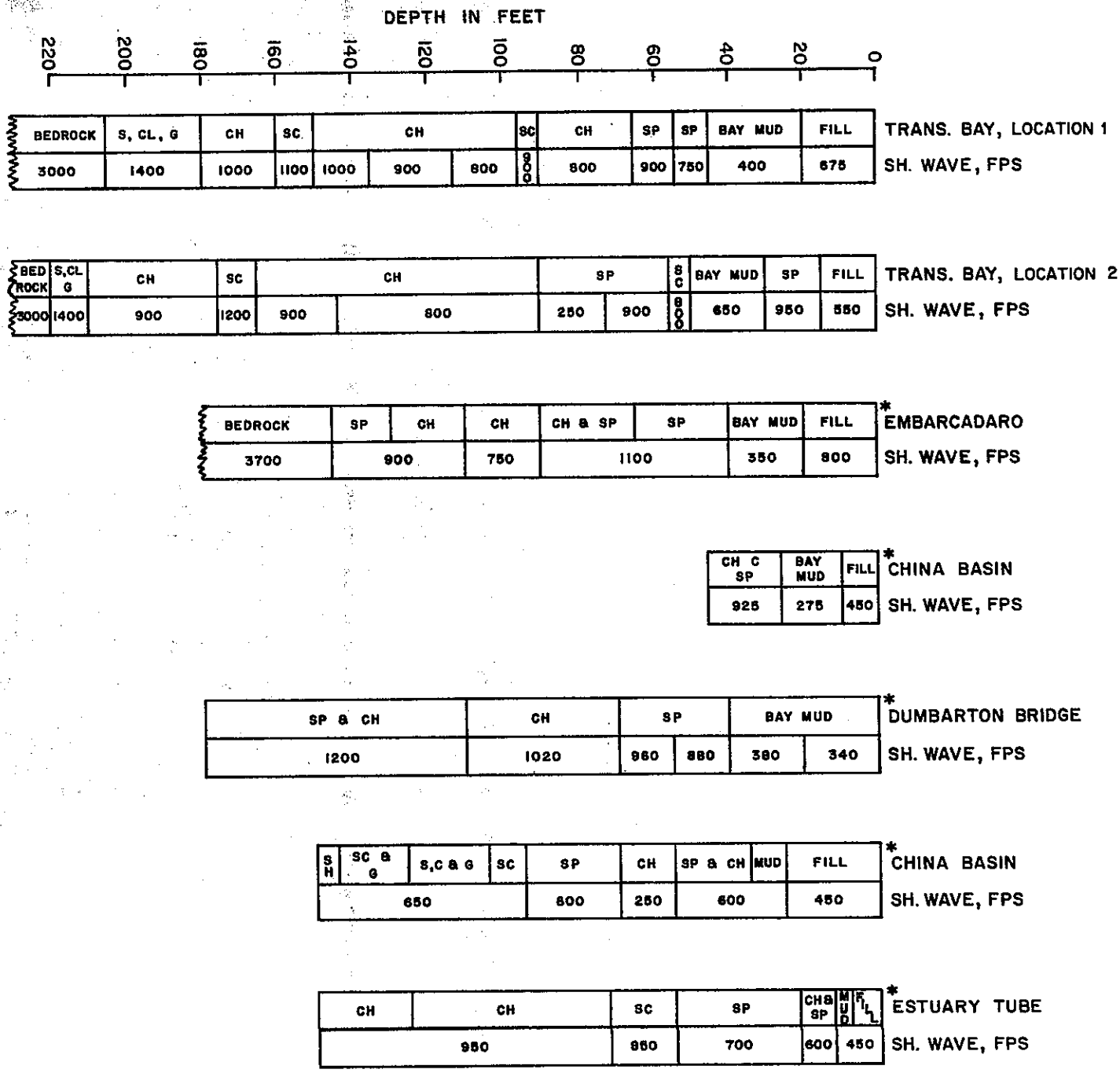

*From Shannon & Wilson

Fig. 4 COMPARISON OF IN-SITU SHEAR WAVE VELOCITIES FOR DIFFERENT LOCALITIES IN AND AROUND SAN FRANCISCO

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The northerly Transbay section (water side 1853 shore line shown on Figure 3) and in general, the surrounding area north to the waterfront, is covered with uncompacted fill material that varies from 10 to 30 feet in thickness. The fill consists of dune sand and rubbish; the rubbish including building rubble, old ship hulls, etc. The fill rests upon soft bay mud which exhibits wide variations in engineering properties.

The upper sand or fill in the southerly section (land side 1853 shore line) comprises some 10 to 20 feet of loose dune sand, containing some building rubble. This sand overlies a dense 5- to 10-foot thick natural sand stratum which, in turn, overlies the bay mud.

The bay mud is an extremely soft, slightly organic, silty clay containing large amounts of sea shells and occasional thin seams or lenses of silt and sand. The bay mud under the fill varies from 0 to 30 feet in thickness with the thickest portions on the water side of the 1853 shore line. This poorly consolidated mud exhibits high compressibility and low shear strengths.

The Pleistocene sands underlying the bay mud are fairly dense (generally greater than 90% relative density) and are interspersed with clay and sand-clay mixtures. Stratum thicknesses are varied. The sandy clays and clays overlying the rock at the site are moderately stiff. The clay directly above the rock is extremely stiff and contains lenses of sands and gravels.

Bedrock at the site consists of highly fractured sandstones and shales of the Franciscan Formation.

The bedrock contour map, Figure 5, illustrates the wide variation in depth to bedrock at the site. The westerly sector (existing Terminal Building area) is located approximately 200 feet above

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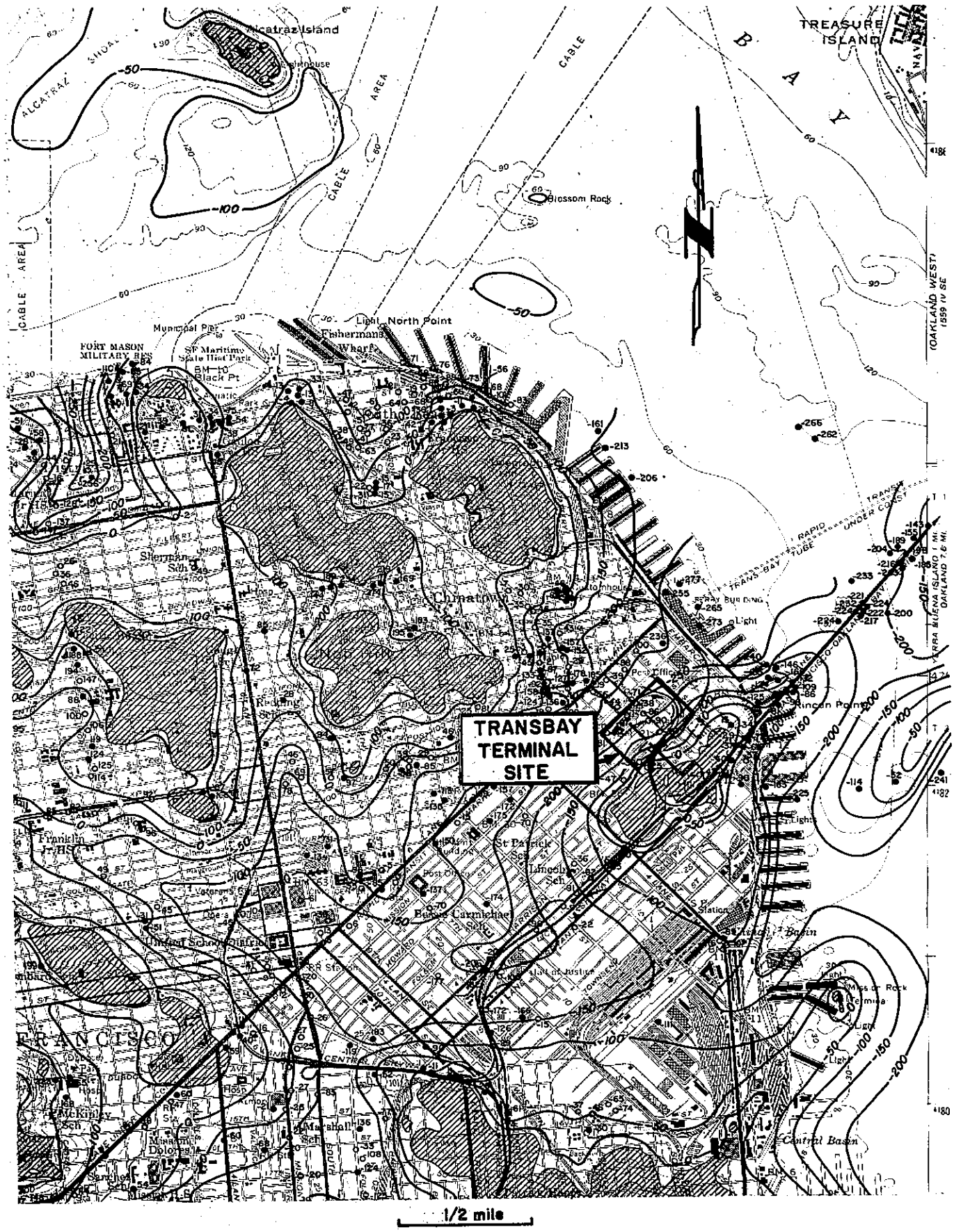

Fig.5 BEDROCK CONTOURS IN FEET OF A PORTION OF SAN FRANCISCO NORTHERN PENINSULA

BY JULIUS SCHLOCKER, USGS 1961 (3)

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bedrock. This bedrock valley is quite steep and is exposed at the ground surface, as massive sandstone, approximately 1300 feet to the east.

Soil Properties From Lab and Field Data

In general, the soil properties required in the evaluation of the dynamic ground response include: field unit weights, strengths (cohesive soils) or relative densities (granular soils), shear modulus and damping factors. These properties can be obtained by laboratory testing methods on undisturbed samples. However, sample disturbance is an uncontrollable problem that limits the usefulness of defining certain field dynamic characteristics by laboratory testing procedures, hence field testing of the essential soil properties (relative density and shear modulus) should be conducted whenever possible.

In-situ shear modulus values can be determined from seismic wave velocity measurements and relative density by field standard penetration tests.

Due to relatively constant properties exhibited for sands (Seed and Idriss (4)) it was found that the shear modulus for granular materials could be adequately defined by relating it to the confining pressure as follows;

$$G = 1000\ K_2\ (\bar{\sigma}_m)^{1/2} \qquad (1)$$

where $\bar{\sigma}_m$ is the effective mean stress and K_2 a measure of the influence of void ratio (relative density) and strain amplitude. For saturated clays, establishment of simplifying shear moduli relationships using undisturbed samples was complicated by the

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large effects of strain amplitude and sample disturbance. In-situ measurements eliminates the problems resulting from sample disturbance, but to date, techniques have not been developed for inducing large controlled strain amplitudes in natural deposits. Thus, field moduli can be obtained only at small strain levels. Ultimately, using laboratory testing techniques, it was recognized that variations in clay characteristics could be taken into account with a reasonable degree of accuracy by normalizing the shear modulus, G, with respect to the undrained shear strength, S_u, and expressing the relationship G/S_u as a function of shear strain. Reasonable estimates of the shear modulus versus shear strain can then be obtained by determining the in-situ shear modulus value (at very small strains) using geophysical survey methods. Field shear wave velocities so obtained are related to the shear modulus by the relationship:

$$V_s = (G \times \frac{g}{\gamma_T})^{1/2} \qquad (2)$$

where V_s = shear wave velocity

g = acceleration of gravity

γ_T = total unit weight of the investigated layer

Thus, utilization of equation 2 and the appropriate empirically derived information relating shear modulus to shear strain for cohesionless and cohesive soils will adequately define the field shear modulus for a range of strain levels.

Results of the field seismic surveys for this project were previously illustrated on Figure 4. Compared to these are results of seismic surveys conducted on nearby projects by other investi-

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gators. Variation in shear wave velocities for similar material strata at the different sites is minor. Consequently, laboratory dynamic testing to determine shear modulus was conducted only as a check on the field data. Laboratory dynamic tests were determined on bay mud and soft clay samples retrieved from -37 foot and -120 foot depths, respectively. The samples were tested in a consolidated undrained state and shear moduli values for low strain levels determined at various effective confining pressures using a "resonant column" test apparatus. These samples are shown as Nos. R1-5-6 and R1-13-1 on Table I, a summary of laboratory test results. Triaxial compression tests were conducted on similar soft clay and bay mud samples (Sample Nos. R1-6-1 and R1-13-7, Table I) to check shear modulus by static test methods. These undisturbed specimens were consolidated to the effective field overburden pressure and sheared to failure in an undrained state. The following empirical relationship for clays (Seed and Idriss (4))

$$G = 2300\ S_u \qquad (3)$$

was used to obtain the initial shear modulus value by utilizing the undrained static shear strength (S_u). For comparison, field shear modulus values computed from the seismic surveys at the corresponding laboratory sample depths are presented. Complete laboratory test records are included in the appendix.

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TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Sample Ft.	Depth PSI	Field Shear Modulus, KSF	LABORATORY DATA Shear Strength (S_u), KSF	Shear Modulus (G), KSF	Damping Ratio, % Obtained	Damping Ratio, % Used	Shear Strain %	Consolidation Pressure, PSI
R1-5-6 (Bay Mud)	37	16	500*	-	860+	46+	2	0.0001	20
R1-6-1 (Bay Mud)	38	17	-	1.5	1380◊	-	2	-	17
R1-13-1 (Soft Clay)	120	46	2800*	-	1900+	24+	3	0.0006	50
R1-13-7 (Soft Clay)	121	47	-	3.9	3600◊	-	3	-	49

* Field shear modulus based on seismic survey at indicated depth.

\+ Shear and damping ratio's computed from resonant column test at indicated shear strain

◊ Shear modulus computed from consolidated- undrained triax compression test (Su)

Soil Profiles Selected for Analytical Study

To facilitate analysis, 6 sections within the site bounds were selected for ground motion studies with each section selected on the basis of its soil profile, depth to bedrock, and developmental importance. These 6 sections are shown on Figure 2 as A_1, B_1, E_1, Z_1, Z_2, and Z_3. A plan map, Figure 6, illustrates assumed sub-areas representative of the studied soil sections. The sub-areas (blocked areas on Figure 6) are denoted by the symbols Z_1, Z_2, Z_3, A_1, B_1, and E_1. Sub-areas Z_1 and E_1 are considered to be wholly within or on the landside of the 1853 shore line. Sub-areas Z_2, Z_3, A_1 and B_1 are considered to be on the sea side of this line.

Section Z-Z (Figure 2Z), a longitudinal soil profile of the westerly sector (along the existing Terminal Building), represents the material profile of those parcels considered to be of prime redevelopmental potential. Sections A-A through E-E (Figures 2A through 2E) are soil profiles transverse to the Terminal Building and bound those parcels considered to be of secondary developmental importance. The selective "sectioning" method of analysis simplified the theoretical analysis and provided information about structural response to earthquake loading for all parcels within the bounds of the terminal site.

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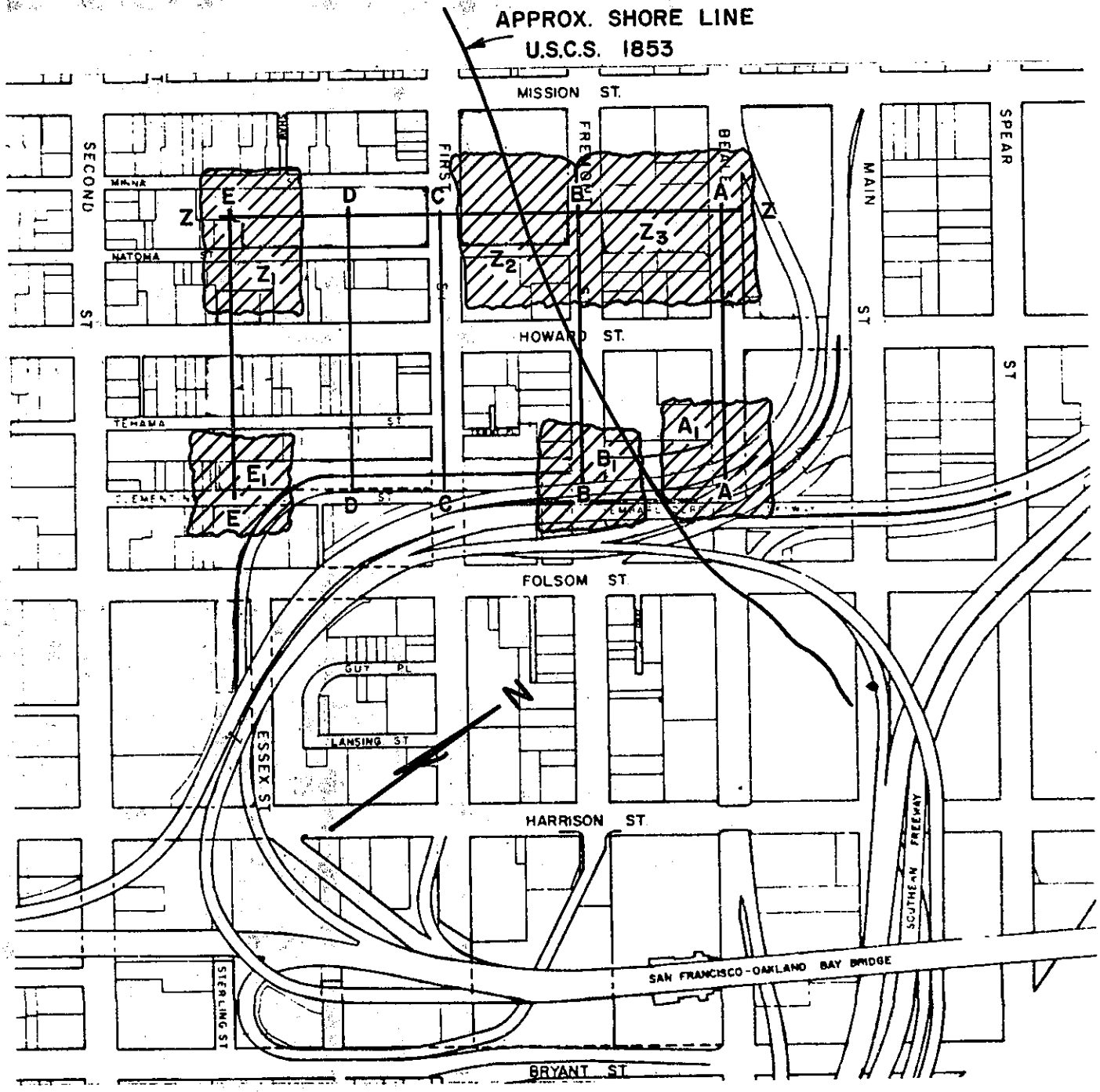

Fig. 6 PLAN MAP OF TRANS BAY SITE SHOWING 6 AREAS SELECTED FOR SEISMIC STUDY. ALSO SHOWN ARE THE ALIGNMENT OF THE SOIL PROFILES DEPICTED IN FIG. 2.

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SITE SEISMICITY

To determine earthquake - induced ground motion at any site, various geophysical conditions and events must be known. Such information must include (1) knowledge of the soil type to bedrock and its response to dynamic loading, (2) surface and bedrock topography, and (3) characteristics of bedrock motion for appropriate earthquakes. Items (1) and (2) pose little difficulty since field exploration can provide the information for classifying subsurface soil conditions and dynamic characteristics. Item (3), however, is somewhat more involved and demanding of engineering judgement. For example, site bedrock motion is dependent upon the activity of, and the nearness of the site to, fault systems. Also, the problem of selecting one or more design earthquakes appropriate for the design life of a project arises.

Earthquake Magnitude

Within the continental United States and adjacent parts of Mexico enough data have been collected to show a positive correlation between the length of historic surface fault rupture (often called causative fault) and maximum expected earthquake magnitude, (Bonilla (5)). This correlation is useful to the engineer since knowledge of the faulting distance enables rational assessment of an earthquake's severity.

Additional research by numerous investigators ultimately resulted in relationships correlating bedrock accelerations to earthquake magnitude and distance to the causative fault, thus providing the necessary information for eventual evaluation of the earthquake's energy wave propagation through the overlying alluvium.

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Relationships illustrating the correlation between length of fault, magnitude, intensity, rock accelerations and distance to the fault are shown on Figures 7, 8 and 9.

The information contained in these figures is based solely on earthquakes recorded in Western California; hence, knowledge of the proximity of the Transbay Terminal site to known active faults will allow a reasonable assessment of its rock base motion.

Earthquake Probability

In general, probability of occurrence of strong ground motion is best selected by historical evaluation. Recorded earthquakes of some arbitrary minimum magnitude are plotted against frequency and a resulting statistical relationship established. The frequency of earthquakes is estimated by Gutenberg and Richter (6), with the equation:

$$N = AN_o \, e^{-M/B} \tag{4a}$$

$$n = \frac{dN}{dM} = \frac{1}{B} AN_o \, e^{-M/B} \tag{4b}$$

where N is the number of shallow earthquakes per year having magnitudes equal to or greater than M, in area A, and n the rate of change of number of earthquakes with respect to magnitude. Coefficients for equations 4a and 4b are well defined on a global basis since an adequate number of large earthquakes have been recorded. However, th is not the case with respect to California due to a paucity of historical data. Hence, for planning purposes, it is reasonable to assume that an

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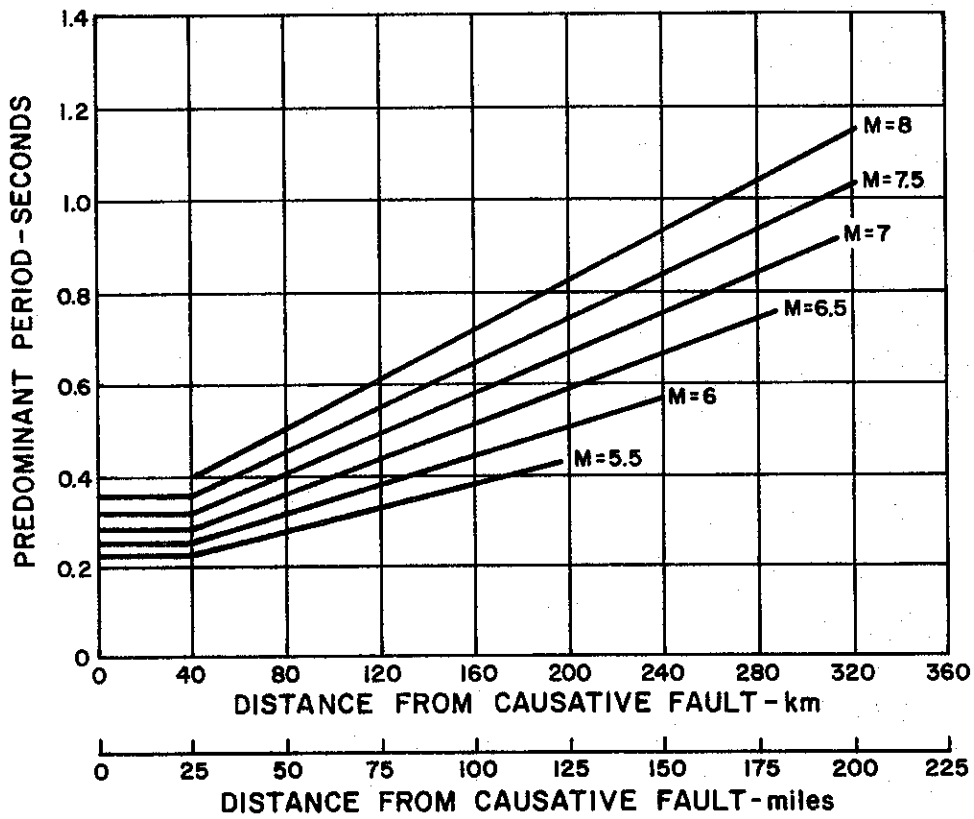

Fig. 7 PREDOMINANT PERIODS FOR MAXIMUM ACCELERATIONS IN ROCK (13)

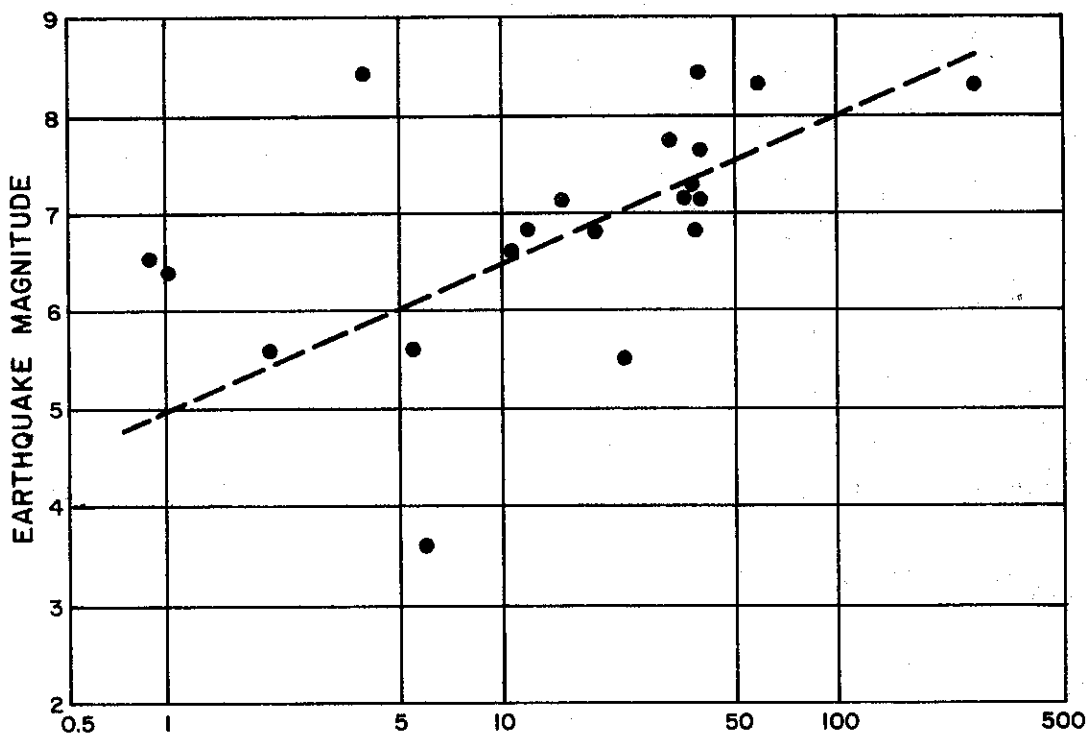

Fig. 8 LENGTH OF SURFACE RUPTURE, MAIN FAULT - miles (13)

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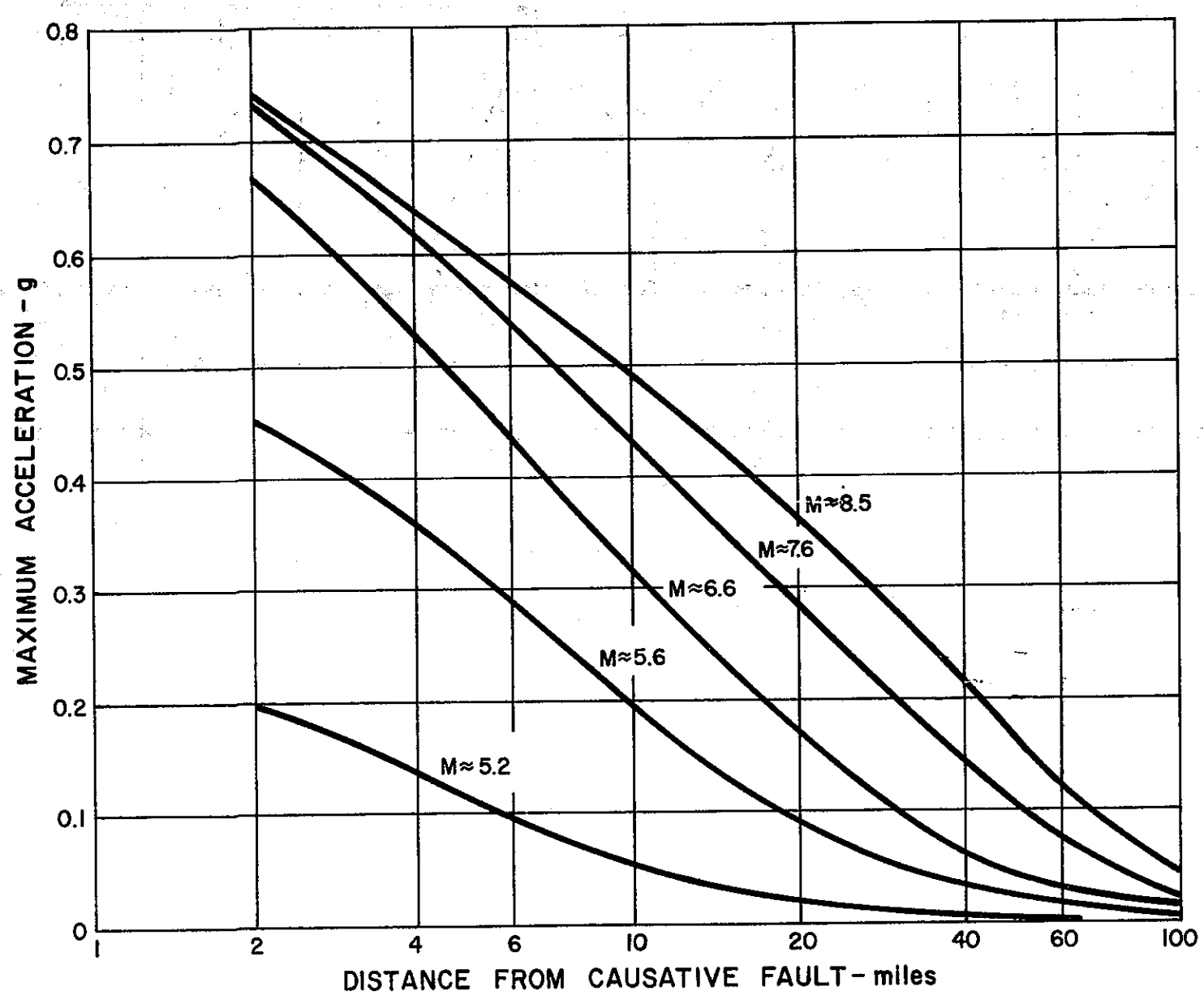

Fig. 9 AVERAGE VALUES OF MAXIMUM ACCELERATIONS IN ROCK (13)

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8+ credible earthquake will affect the coastal area once every 100 years. Lesser magnitude earthquakes would naturally be generated with greater frequency.

Probability concepts are utilized to provide an understandable basis for rationally assessing the seismic potential of any area. The results of such a study are not necessarily indicative of what may or may not happen, but provide an estimate of the potential of certain seismic events occurring within a given time period. Housner (7) presented an equation that enables calculation of the probability that a specified area (in California) will experience a particular magnitude of ground shaking given certain empirical parameters. Using Housner's equation, the probability that a given area, a, will experience a given shock in a large area, A, is

$$P = 1-e^{-a/A} \qquad (5)$$

Using probability concepts to determine the expected number ,N, of bedrock acceleration excitations at the site for any range of earthquake magnitudes and any time periods, the following empirical relationship evolves:

$$N = C_1 C_2 T(1-e^{-a/A}) \qquad (6)$$

where C_1 = number of earthquakes per year per 1000 square miles for any magnitude (Figure 11)

C_2 = study area/1000 mi^2 = 11

T = time in years (taken as 100)

a = area of influence of specific bedrock acceleration level for a given magnitude (Table IV) and

A = study area = 11,000 mi^2 (Central California Coast)

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Table II lists all recorded earthquakes equal to or greater than Magnitude 4 occurring in the 11,000 mi^2 study area (Figure 10) after 1934. Also included are those events with Magnitude 7 or greater occurring after the year 1800 (8). The information on Table II is plotted on Figure 11 and is compared to the seismic frequency interval of California as a whole. The seismic frequency of the studied area, represented by a dashed line, is about 3 times greater than that of California and is used as a basis of selection for the coefficient C_1 in equation 3. Table III lists the theoretical area, a, that would experience the given bedrock acceleration levels or larger at the given earthquake magnitude. Table IV lists the number of individual excitations, N, for the damaging range of any earthquake. The bottom line (i.e., total from all earthquakes) is a sum total of all expected excitations of the noted bedrock acceleration levels the terminal site will experience in a 100 year period. In other words, the site can statistically expect about eleven shocks in 100 years at a bedrock acceleration level of 0.1g and about two shocks in 100 years at a bedrock acceleration level of 0.3 g's. This study analyzed four earthquakes that had site bedrock acceleration levels of approximately 0.07, 0.1, 0.3 and 0.5 g's. The results of this analysis are presented in a later section of this report.

Design Earthquakes

Seismically active faults located in California that could be damaging in the San Francisco area were taken from a California Division of Mines and Geology report (9). The report lists all historically active faults and all faults with known or suspected Holocene (last 10,000 year) offset.

Maximum probable earthquake base rock acceleration, duration, and period were determined using the empirical relationships established in Figures 7, 8 and 9. Table V lists those faults that would be most damaging to the Bay area, and specifically, the Transbay Terminal site. Also included are the characteristics of the motion at the Terminal site as determined from the same figures.

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TABLE II

Summary of Study Area Seismicity

Study Area = 11,000 mi.^{2}

Richter Magnitude	≥ 4	≥ 5	≥ 6	≥ 7	≥ 8
Study Period	1934-1971	1934-1971	1934-1971	1800-1971	1800-1971
No. of epicenters in study area	268	61	1	4	1
Ave. No. of Epicenters in study area/year/1000 mi.^{2}	.658	.150	.0025	.0021	.0005

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Fig. 10 SEISMIC STUDY AREA

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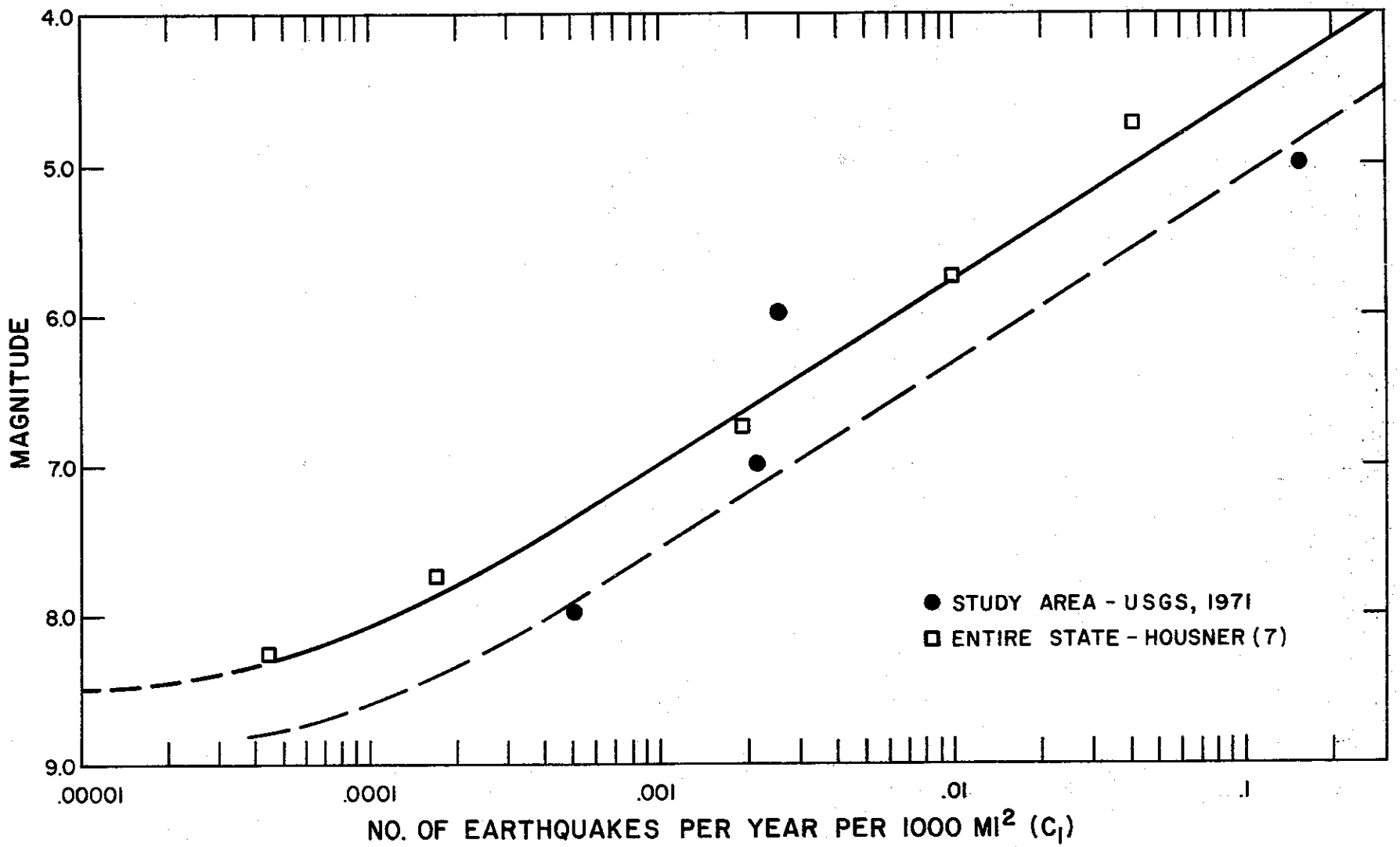

Fig. 11 COMPARISON OF STUDIED AREA SEISMICITY FREQUENCY INTERVAL TO THAT OF THE ENTIRE STATE

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TABLE III

BEDROCK ACCELERATION LEVEL
INFLUENCE AREAS (sq. mi.) = a

Earthquake Magnitude	Bedrock Acceleration Level (g) 0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.70
5.0	40	17	0	0	0	0	0	0
5.5	1750	700	175	70	17	0	0	0
6.0	4500	2000	650	250	120	45	0	0
6.5	6500	3200	1200	550	275	140	80	0
7.0	12,000	7000	2500	1300	750	400	275	120
7.5	22,000	12,000	5500	2700	1600	750	450	270
8.0	45,000	25,000	13,000	7000	4000	2100	1150	550
8.5	80,000	50,000	28,000	16,000	9500	5000	2750	1400

TABLE IV

Average Number of Bedrock Accelerations Equal to or Greater Than Those Indicated Which Are Likely to Occur in a 100 Year Period (n)

Magnitude	Bedrock Acceleration (g) .05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
5.0	.48	0.20						
5.5	7.3	3.1	0.8	0.30	0.07			
6.0	6.6	3.3	1.1	0.44	0.21	0.08		
6.5	3.5	2.0	0.8	0.37	0.20	0.10	0.06	
7.0	2.0	1.5	0.63	0.34	0.20	0.11	0.06	0.03
7.5	1.0	0.8	0.48	0.26	0.16	0.08	0.05	0.03
8.0	0.49	0.44	0.34	0.23	0.15	0.09	0.05	0.02
8.5	0.14	0.14	0.13	0.11	0.08	0.05	0.03	0.017
Total from all Earthquakes	21.5	11.48	4.28	2.05	1.07	0.51	0.25	0.097

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These three faults are considered to be the most active and would produce the largest base rock accelerations at the terminal site. For design purposes, two magnitude 8+ earthquakes, one near and one distant, were evaluated as were two earthquakes of lesser magnitude (M=5.5 and 7.0). Table VI lists the characteristics of those earthquakes investigated and the proximity of the site to the assumed epicenter.

Table V

Active Fault	Magnitude	Duration	Period	Dist. to Fault	Rock Acc.
San Andreas	8.25	35 Sec	0.35 Sec	9+ mi.	0.49 g's
Hayward	7.6	30 Sec	0.35 Sec	9+ mi.	0.44 g's
Calaveras	7.6	30 Sec	0.35 Sec	20 mi.	0.28 g's

Table VI

Earthquake	Magnitude	Distance to Epicenter	Bedrock Acc. Levels @ Site	Period
Golden Gate	5.5	9 mi	0.13 g	0.12 Sec
Castaic record of 1971 San Fernando Quake (Modified)	7.0	9 mi.	0.25 g	.35 Sec
Berkeley A-1	8+	9 mi.	0.49 g	.35 Sec
Castaic record of 1971 San Fernando Quake (Modified)	8+	200 mi.	0.07 g	0.50 Sec

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GROUND MOTION ANALYSIS

Earthquake motions or shock waves originate in bedrock as a result of crustal rupture, and in turn are transmitted through any overlying alluvium to the earth's surface. For convenience, the shock waves creating the surface disturbances are assumed to be primarily an upward propagation of shear waves emenating from bedrock. This assumption simplifies the analysis since the problem can be reduced to that of a one-dimensional shear wave if the ground surface, the rock surface, or the boundaries between different soil layers are essentially horizontal. However, if these boundaries, individually or collectively, are inclined, accurate soil deposit responses can only be made by techniques such as the finite element method.

The terminal site subsurface investigation indicated bedrock to be inclined in the eastern sector; rising from a depth of -200 feet to ground surface within a horizontal distance of approximately 1,300 feet. Therefore, any analyses of this area using a one-dimensional mathematical solution that assumes horizontal boundaries could be expected to differ from a two-dimensional finite element solution that accounts for irregular boundaries. However, the uncertainties involved in properly assessing the governing soil parameters, the variability of soil conditions from place to place, and the inability to accurately predict the wave form of several design earthquakes, appear to be as significant as the method of analyses. Hence, utilization of a one-dimensional solution appears justifiable for determining the general nature of the ground motion.

For the westerly sector (existing Terminal Building) the underlying bedrock and soil layers are, for all practical purposes, horizontal. Consequently, very little, if any, difference could be expected in the computed ground motion using either the one-

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or two-dimensional solution. In the eastern sector (those parcels actually of minor developmental concern) some differences in computed ground motion may occur between these two methods.

Seed and Idriss(10) conducted a study of ground motions within the San Francisco area using the one-dimensional approach as a solution to the problem. The results indicated close agreement to observed movements for the Alexander Building during the 1957 earthquake. The Alexander Building (located at Pine St. off Market) is situated 200 feet above fairly steeply inclining bedrock, hence, conditions of horizontal boundaries are not strictly met. Based upon these observations the one-dimensional model was used for analyzing the response of the horizontally depicted soil deposits. The program used in the analysis was developed by Schnabel, Lysmer and Seed (2) and is based on the continuous solution to the wave-equation (Kanai, (11)). The nonlinearity of the shear modulus and damping of the soil deposits are accounted for by the use of equivalent linear soil properties. Thus an iterative procedure is used to obtain values for modulus and damping compatible with the effective strains in each soil layer.

In general, the analytic procedure involves the following steps:

1. Select an accelerogram with those characteristics that would most likely occur in bedrock underlying the site; i.e., the maximum acceleration, predominant period, and effective duration of the design earthquakes (taken from recorded earthquake motions).

2. Determine the dynamic properties of the soil deposits (field and lab testing).

3. Compute the response of the soil deposit to the baserock motions (analytic procedure involving computer work).

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RESULTS OF GROUND RESPONSE STUDIES

The varied developmental and subsurface conditions existing at the terminal site required evaluating representative soil sections to facilitate the seismic response analysis. These soil sections labeled Z_1, Z_2, Z_3 and A_1, B_1, E_1 represent soil conditions of the site's westerly sector (existing Terminal Building Area) and the easterly sector respectively. The site's westerly sector is composed of deeply deposited aluvium, predominately clayey in texture while the eastern sector is sandier and more shallow.

Response Spectra

Acceleration and velocity response spectra were obtained for the six soil sections (Figure 2) utilizing three of the four earthquake motion records previously discussed. The distant M = 8+ record was used on only two soil sections, Z_2 and B_1, which represented the average deep clay and shallow sandy sections respectively.

Figures 12 and 13 illustrate the acceleration and velocity response spectra for 5% structural damping, along with the maximum ground surface accelerations and the section location within the Transbay site. Only one structural damping factor (5%) was used in order to reduce the volume of computer data; also, evaluating other structural damping values would provide little additional information for the purposes of this study.

Deformation of Subsurface Strata

In reviewing the displacement profiles for several soil sections, maximum relative displacement was noticed to always occur in the layers bounding the soft bay mud. Consequently, as an expeditious

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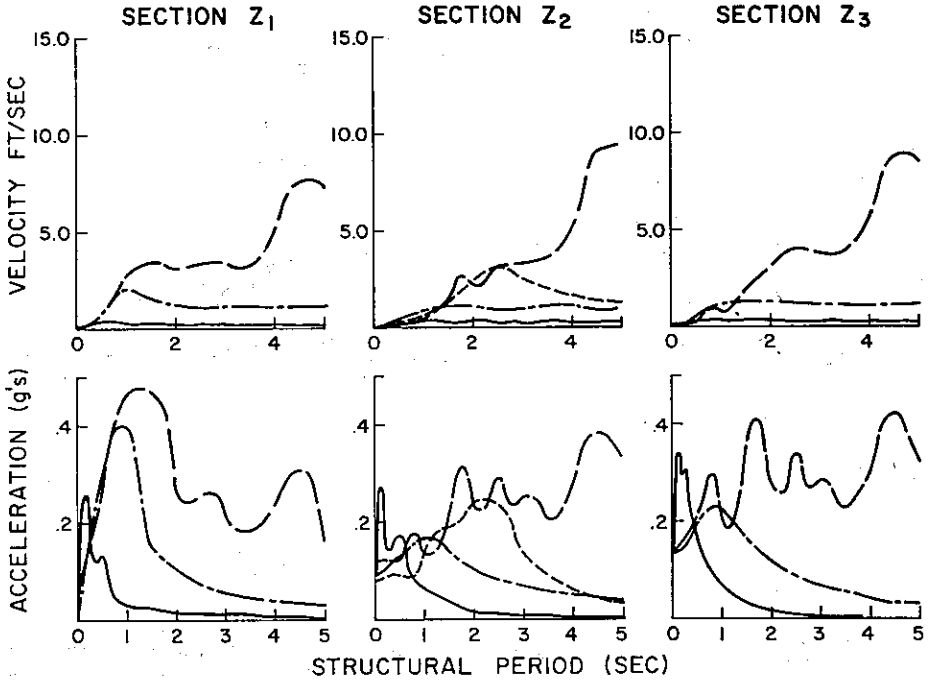

RESPONSE SPECTRA FOR 5% STRUCTURAL DAMPING

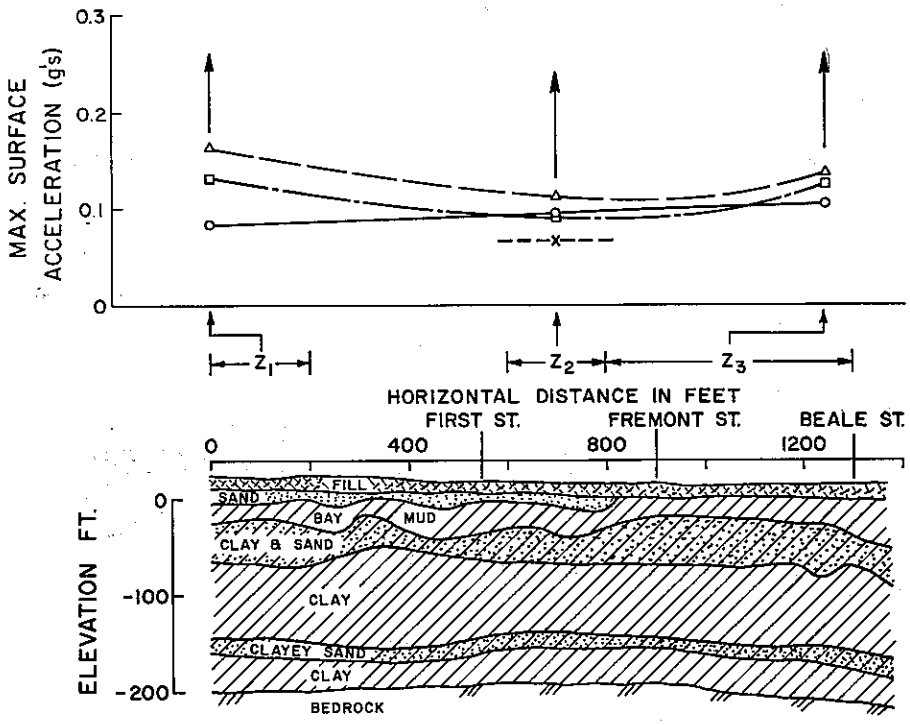

Fig. 12 ACCELERATION AND VELOCITY RESPONSE SPECTRA AND MAXIMUM GROUND SURFACE ACCELERATIONS FOR THE NOTED SECTIONS AT THE TRANSBAY SITES WESTERN SECTOR (EXISTING TERMINAL BUILDING AREA)

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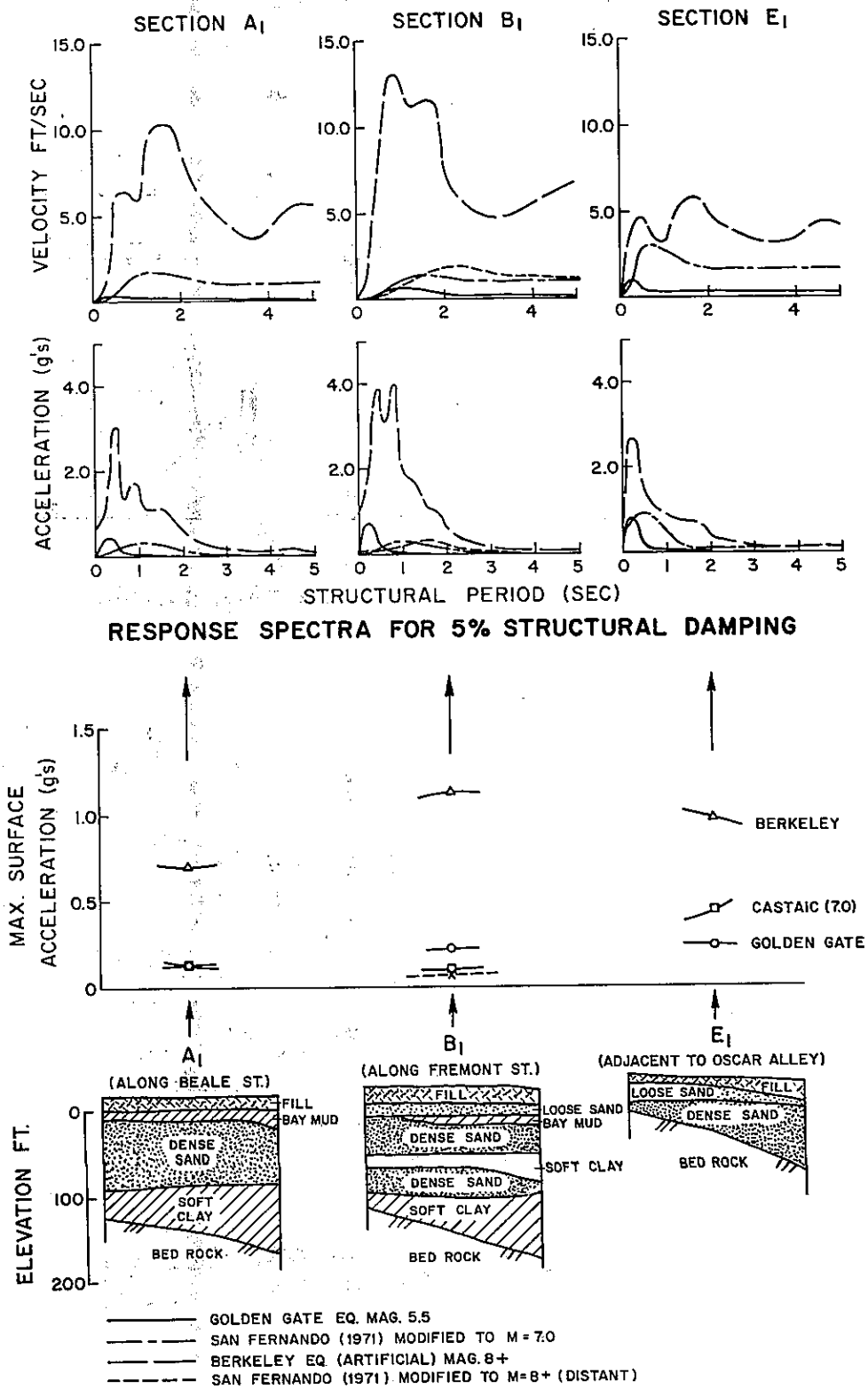

Fig.13 ACCELERATION AND VELOCITY RESPONSE SPECTRA AND MAXIMUM GROUND SURFACE ACCELERATIONS FOR THE NOTED SECTIONS AT THE TRANSBAY SITES EASTERN SECTOR.

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move, the maximum relative displacements for several earthquakes were tabulated for those soil sections which contained bay mud. The results are shown on Table VII for the M = 8+ and M = 5.5 earthquakes, along with the bay mud thicknesses and total depth to bedrock. For descriptive purposes, a deflection curve of the Z_2 soil section, Figure 14, was plotted to illustrate the deformed shape of an initially straight member after subsurface movement had reached peak relative displacement values.

Liquefaction Potential

Liquefaction is a phenomenon in which a saturated cohesionless soil loses strength during an earthquake due to the development of high pore water pressures. This strength loss generally results in large permanent shear deformations taking place in the affected stratum and possibly the overlying strata.

The terminal site is covered with a layer of loose, sometimes partially submerged, granular fill material and is underlain to various degrees with rather fine-grained medium-dense to very dense sands. Since liquefaction potential depends on the density and granular nature of the soil, the initial soil stresses, and the earthquake characteristics, these factors were investigated in evaluating the liquefaction potential of the site. The liquefaction potential was investigated using an empirically validated procedure proposed by Seed and Idriss (12).

The results of the study are presented on Figure 15. The dashed lines represent the required shear stress level necessary to initiate liquefaction while the solid lines indicate shear stress levels generated by the earthquakes. Potential zones of liquefaction occur when the earthquake induced stress levels (solid line) exceeds the failure stress level (dashed line).

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TABLE VII

Profile	Max.Rel.Displ. of Layers Bounding Bay Mud, Inches M= 8+	M= 5.5	Thickness Bay Mud Layer, Feet	Ave. Depth to Bedrock, Feet
A_1	1.4	0.1	15	150
B_1	2.2	0.1	5	160
E_1	~ 0	~ 0	*0	35
Z_1	2.2	-	15	210
Z_2	14.2	0.2	30	210
Z_3	13.3	0.2	25	210

* No Bay Mud Present : Relative Displacements taken between fill and dense sand

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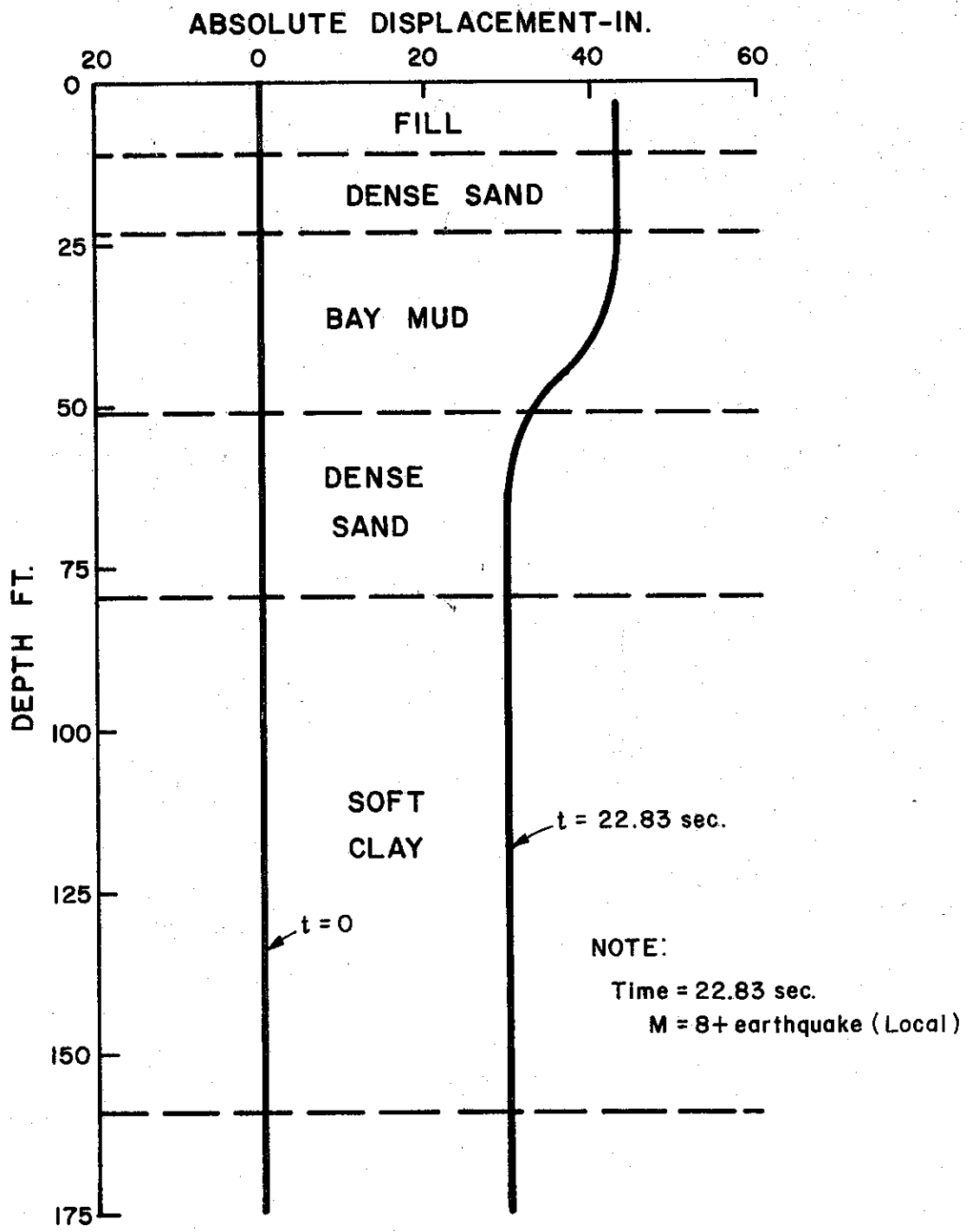

Fig.14 MAXIMUM RELATIVE DISPLACEMENT PROFILE FOR SECTION Z_2

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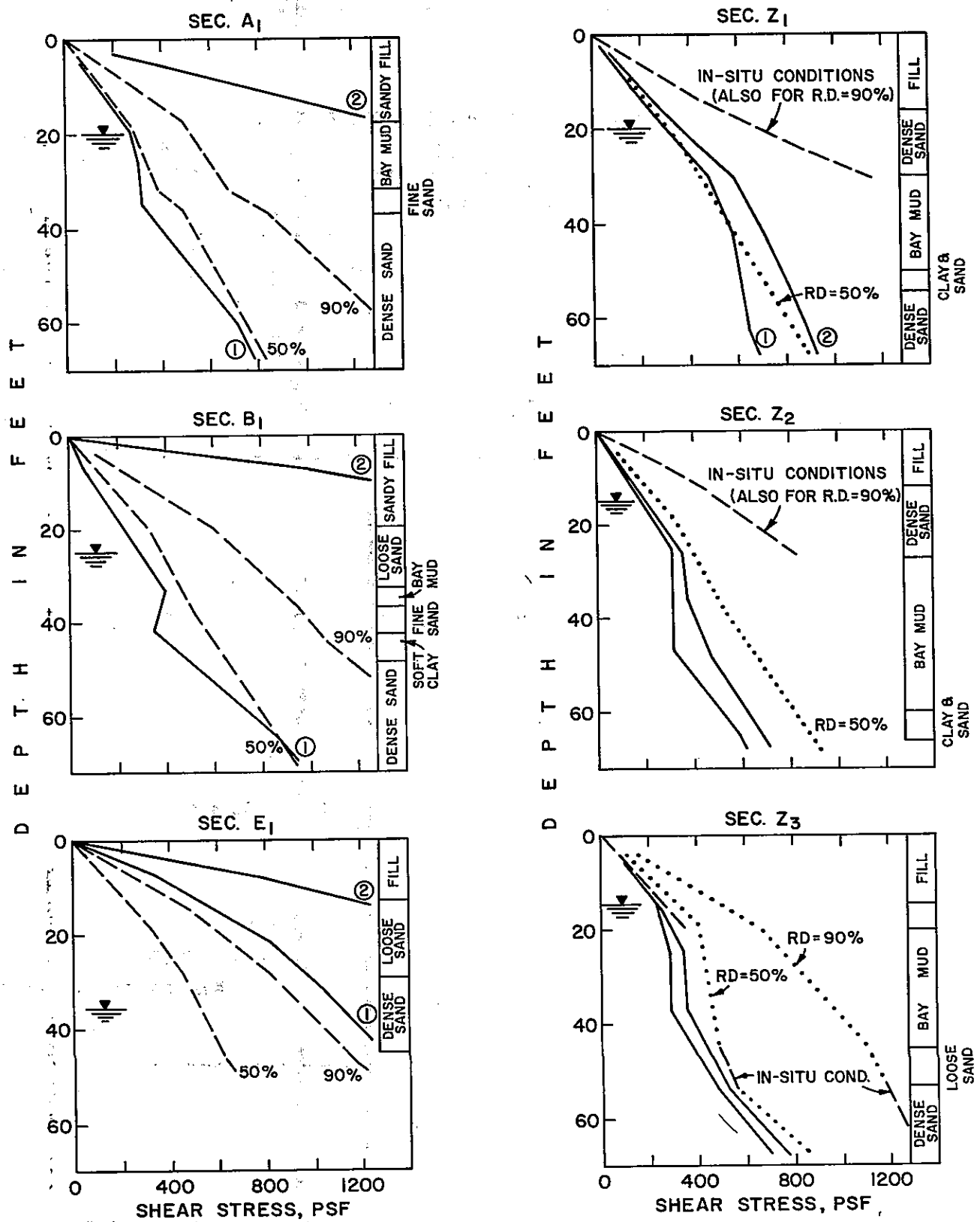

① SAN FERNANDO (1971) MODIFIED TO M=7.0

② BERKELEY (ARTIFICIAL) MAG. 8.+

DASHED LINES INDICATE DYNAMIC SHEAR STRESS LEVEL REQUIRED TO INITIATE LIQUIFACTION. SOLID LINES ARE THE EARTHQUAKE INDUCED SHEAR STRESS LEVELS.

Fig.15 RESULTS OF LIQUEFACTION STUDY

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The dashed lines are representative of cyclic durations generated by earthquakes with an approximate magnitude of 7 or greater. Since earthquakes much smaller than M = 7 do not have the required number of significant stress pulses to promote failure, only the M = 7 and greater earthquake induced shear stress levels are plotted.

Due to the unavailability of relative density data for the majority of the upper sands at the site, liquefaction calculations were based on relative density values of 50% and 90% for most of the soil sections. Reasonable assumptions of actual liquefaction potential can then be made by studying the figures where actual field data was obtained and calculated. These are Sections Z_1 and Z_3 with interpolation for Z_2. The dashed line on these sections are shown discontinuous; thus indicating that material of differing relative densities were encountered. Relative density values of 50% and 90% are also shown (dotted lines) to add significance to the existing soil conditions.

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DISCUSSION OF RESULTS

Small magnitude (5.5) event -

The Golden Gate earthquake record of 1957 was used "as is" to typify future, small magnitude quakes having a probable recurrence interval of once in 10 years. A short cyclic period is characteristic of this event and resulted in rather high peak acceleration response levels within the shallower, sandier soil sections A_1, B_1 and E_1. The 0.7g+ acceleration level for structures of about 2 or 3 stories within these sections appears quite severe and was not substantiated by the small recorded damage during 1957. However, for the deeper, predominately clay sections analyzed (Z_1, Z_2, Z_3), acceleration levels appeared reasonable and were consistent with observed results. Liquefaction and relative displacements are considered negligible.

Intermediate magnitude (7.0) event -

A magnitude 6.6 Castaic earthquake record of the 1971 San Fernando earthquake was modified to a magnitude 7.0 with a probable recurrence interval of once in 30 years. Developed ground surface acceleration levels, with the exception of soil section E_1, were 0.13g or less, and peak structural forces were 0.4g or less; critical structure height was 10 stories. However, the shallow and sandy section E_1 experienced peak ground surface and structural acceleration levels of 0.44g and 0.9g, respectively; critical structure height is 5 stories. Liquefaction is possible below ground water level for areas in and around the vicinity of section E_1 only. Maximum relative pile displacements are considered negligible for all areas.

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Large Magnitude (8+) event -

A Berkeley 8+ artificial earthquake simulated all closely centered large magnitude events having a probable recurrence interval of once in 200 years. The deep soil sections (Z_1, Z_2, Z_3) of western sector experienced relatively small ground surface and structural acceleration levels considering the earthquake's severity; peak values registered were 0.16g and 0.48g, respectively. The most critical building height is 40 stories; however, no distinct critical structure height existed due to the acceleration response spectra peaking over a fairly broad range of structural periods.

The eastern section (A_1, B_1, E_1), presumably because of its shorter natural period, exhibited extremely large ground surface and structural acceleration levels. Peak ground surface acceleration was 1.2g and peak structure acceleration of 3.8g developed for structure heights in the 5 story range. The severity of the generated seismic forces warrants a critical look at any structure intended for construction within these eastern bounds.

Liquefaction is not likely to occur in the Z or western sections; however, potential liquefaction is likely in the eastern sector for sandy soil deposits below the water table.

Relative pile displacements are considered moderately large for the western sections only. Large displacement occurs exclusively within the bay mud layer and is on the order of 14 inches of lateral displacement per 30 feet of (mud) depth, thus, pile design should consider accomodating developed deflections.

Distant large magnitude (8+) event -

This event, a modified Castaic record of the San Fernando quake, typified an 8+ earthquake occurring approximately 200 miles

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from the site with a probable recurrence interval of once in 200 years. Associated long natural periods, common to distant quakes, suggested potentially dangerous acceleration levels due to increased "tuning" between soil column and earthquake period, even though site bedrock acceleration would be small (0.07g). Sections Z_2 and B_1 were analyzed and represented the average deep clay and shallow sandy sections, respectively. Developed ground surface accelerations were small, 0.08g, but as anticipated, spectral acceleration levels were strong. Peak acceleration values of 0.35g were developed for structure heights in the 20 story range. General appearance indicated that damage potential of this distant quake may be greater than the local 7.0 event. Liquefaction is not expected and pile displacements are considered to be negligible.

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APPENDIX A

BORING LOGS

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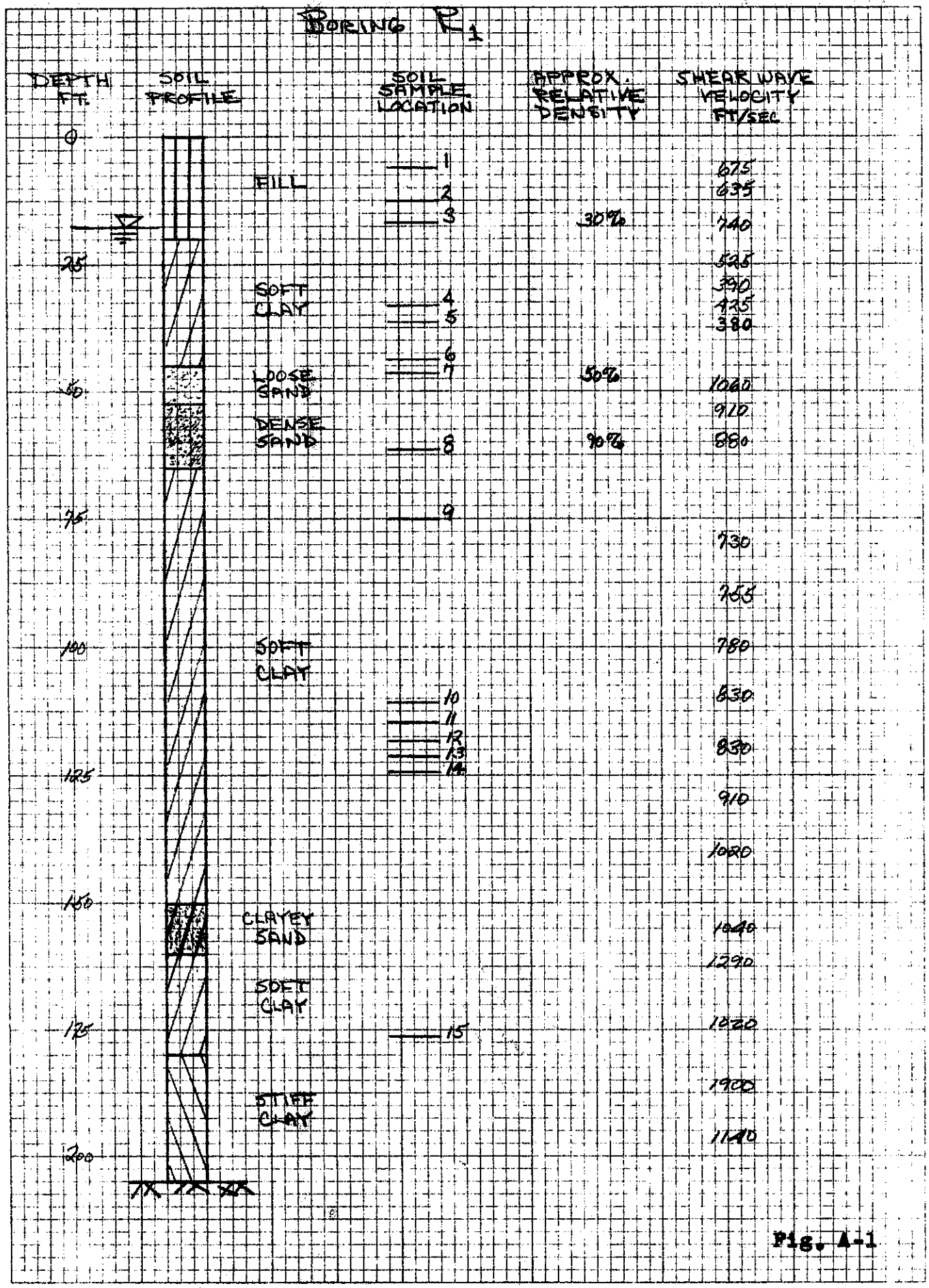

Fig. A-1

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BORING R3

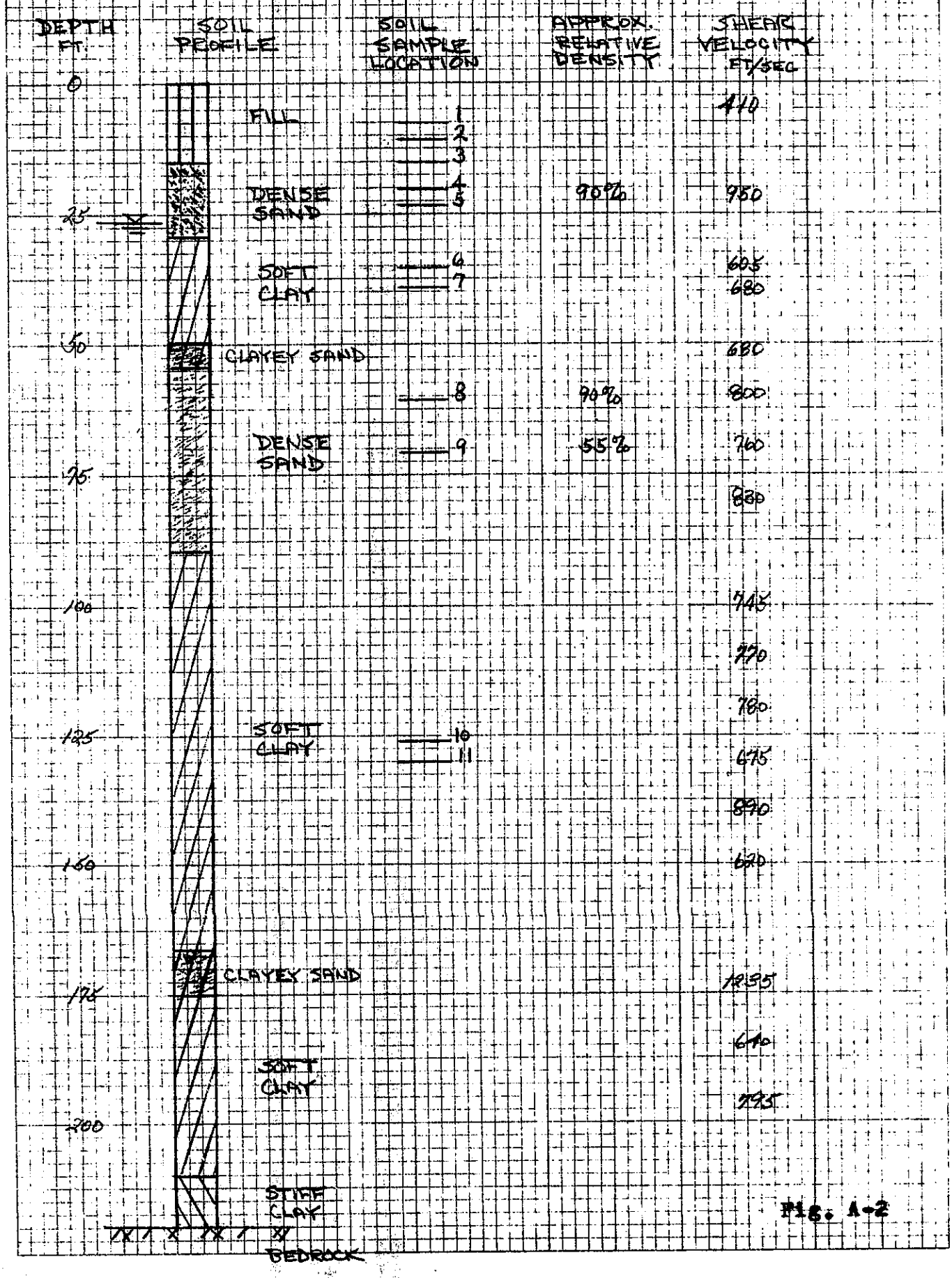

FIG. A-2

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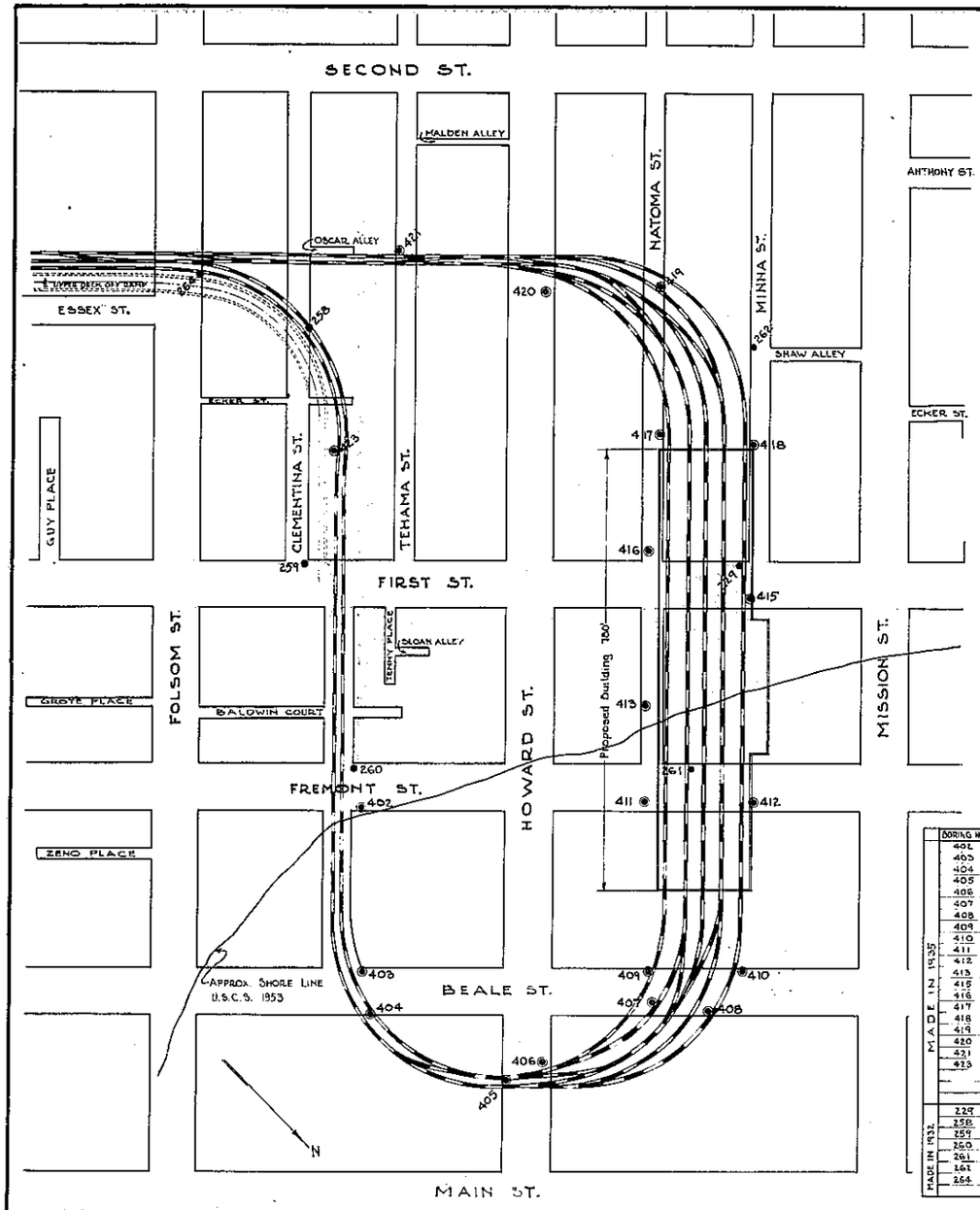
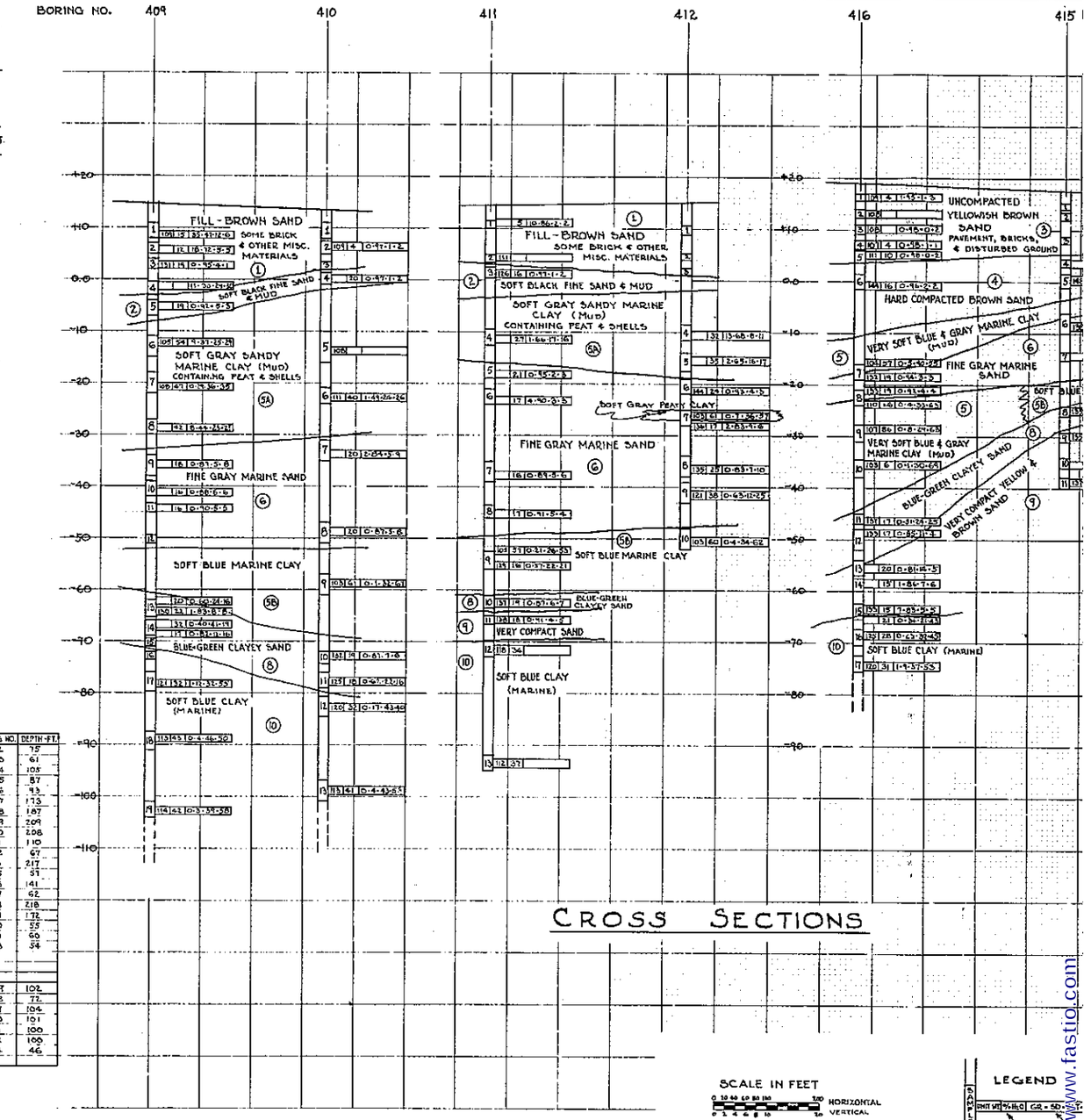


	BORING NO.	DEPTH-FT.
MADE IN 1935	402	75
	403	61
	404	105
	405	87
	406	93
	407	173
	408	187
	409	209
	410	208
	411	110
	412	67
	413	217
	415	51
	416	141
	417	62
	418	218
	419	172
	420	55
	421	60
	423	54
MADE IN 1932	229	102
	258	72
	259	104
	260	101
	261	100
	261	100
	264	46



CROSS SECTIONS

SCALE IN FEET

Bridge Interurban Tracks

● Borings made 1932

◉ Borings made 1935

PLAN OF TEST BORINGS

SEE SHEET No. 2 FOR SOIL STRATA THRU BORINGS 406-408-410-412-415-229-418-262-419-420

SEE SHEET No. 3 FOR SOIL STRATA THRU BORINGS 405-406 407-409-411-261-413-416-417-419-420-421

SEE SHEET No. 4 FOR SOIL STRATA THRU BORINGS 264-258-423-259-260-402-403-404-405

SCALE IN FEET

HORIZONTAL

VERTICAL

LEGEND

ClibPDF - www.fastio.com

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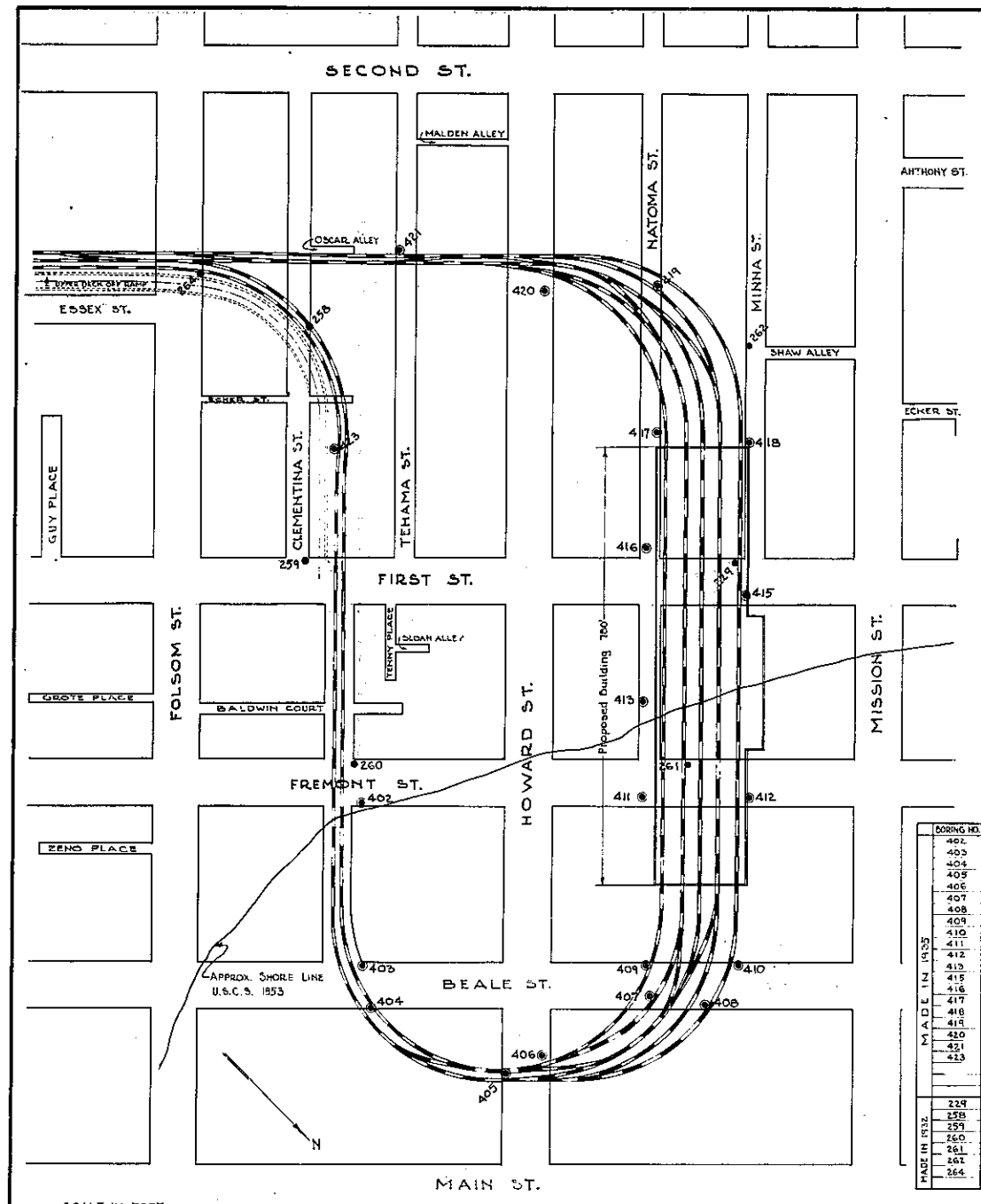
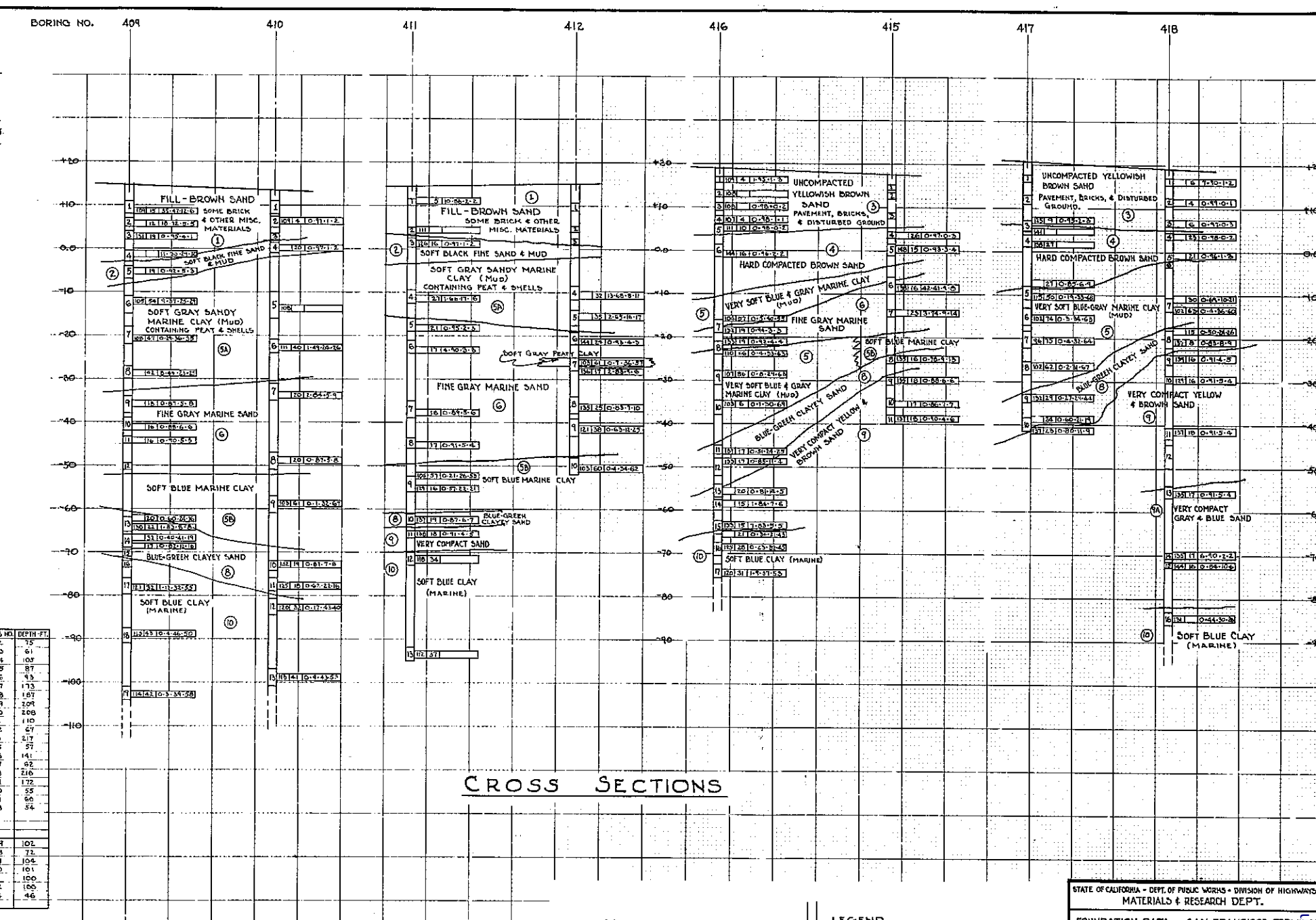


BORING NO.	DEPTH-FT.
MADE IN 1935	
402	75
403	61
404	103
405	87
406	93
407	173
408	157
409	209
410	208
411	110
412	67
413	217
415	57
416	141
417	62
418	216
419	172
420	55
421	60
423	54
MADE IN 1932	
229	102
258	72
259	104
260	101
261	100
262	100
264	46

PLAN OF TEST BORINGS

SEE SHEET No. 2 FOR SOIL STRATA THRU BORINGS 406-408-410-412-415-229-418-262-419-420
SEE SHEET No. 3 FOR SOIL STRATA THRU BORINGS 405-406 407-409-411-261-413-416-417-419-420-421
SEE SHEET No. 4 FOR SOIL STRATA THRU BORINGS 264-258-423-259-260-402-403-404-405



STATE OF CALIFORNIA - DEPT. OF PUBLIC WORKS - DIVISION OF HIGHWAYS
MATERIALS & RESEARCH DEPT.

FOUNDATION DATA - SAN FRANCISCO TERMINAL
SAN FRANCISCO-OAKLAND BAY BRIDGE

PLAN OF TEST BORINGS UNDER OVERHEAD VIADUCTS & TERMINAL BUILDING

CROSS-SECTIONS OF SOIL STRATA

DIST. CO. RT. SEC. | CORRECT | APPROVED | DATE | SHEET NO.

Fig. A-3

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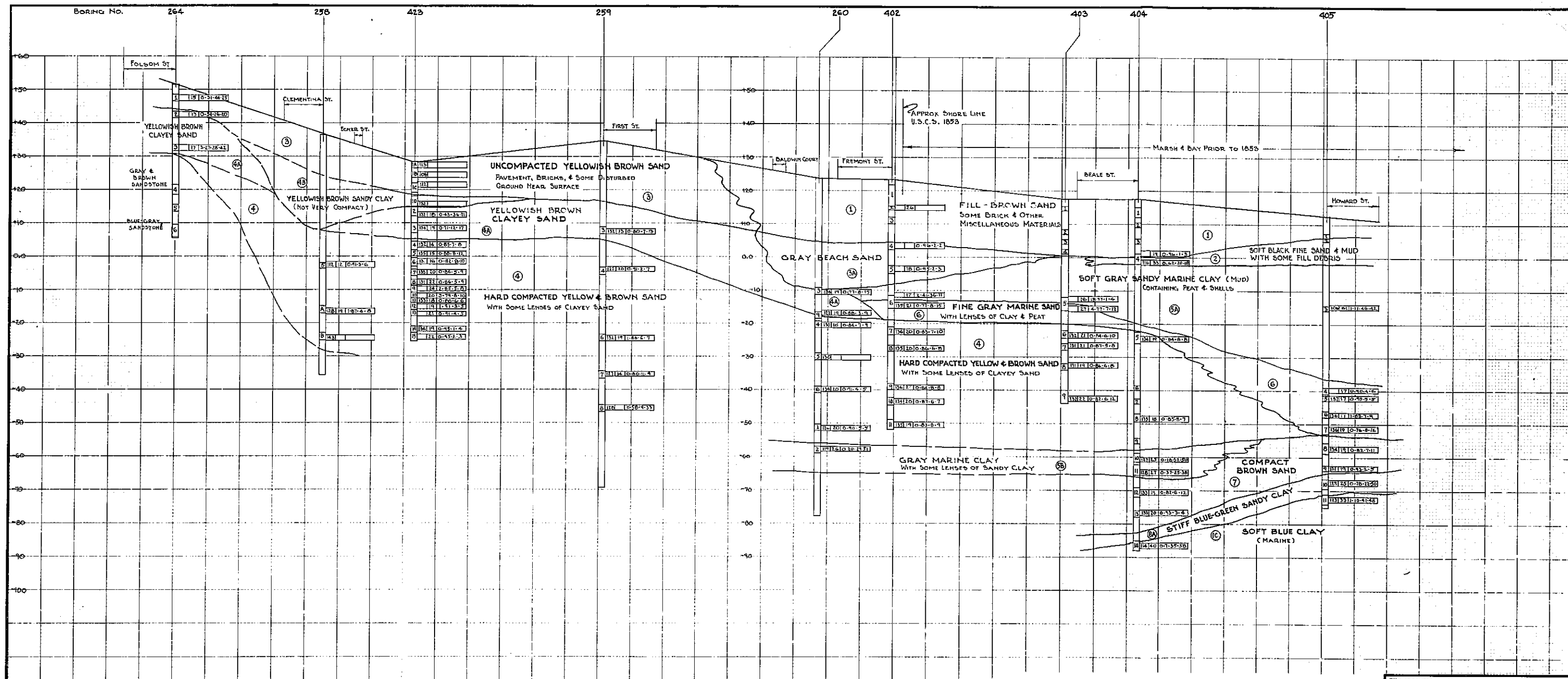

SEE PLAN ON SHEET 1 FOR LOCATION OF BORINGS.

STATE OF CALIFORNIA - DEPT. OF PUBLIC WORKS - DIVISION OF HIGHWAYS
MATERIALS & RESEARCH DEPT.

FOUNDATION DATA - SAN FRANCISCO TERMINAL
SAN FRANCISCO - OAKLAND BAY BRIDGE

SOIL STRATA UNDER SOUTHEAST OVERHEAD VIADUCT FROM FOLSOM ST. NEAR ESSEX TO HOWARD ST. NEAR BEALE ST.

DIST. CO. RT. SEC. | CORRECT | APPROVED Thos. E. Stanton Jr. | DATE | SHEET NO.

Fig. A

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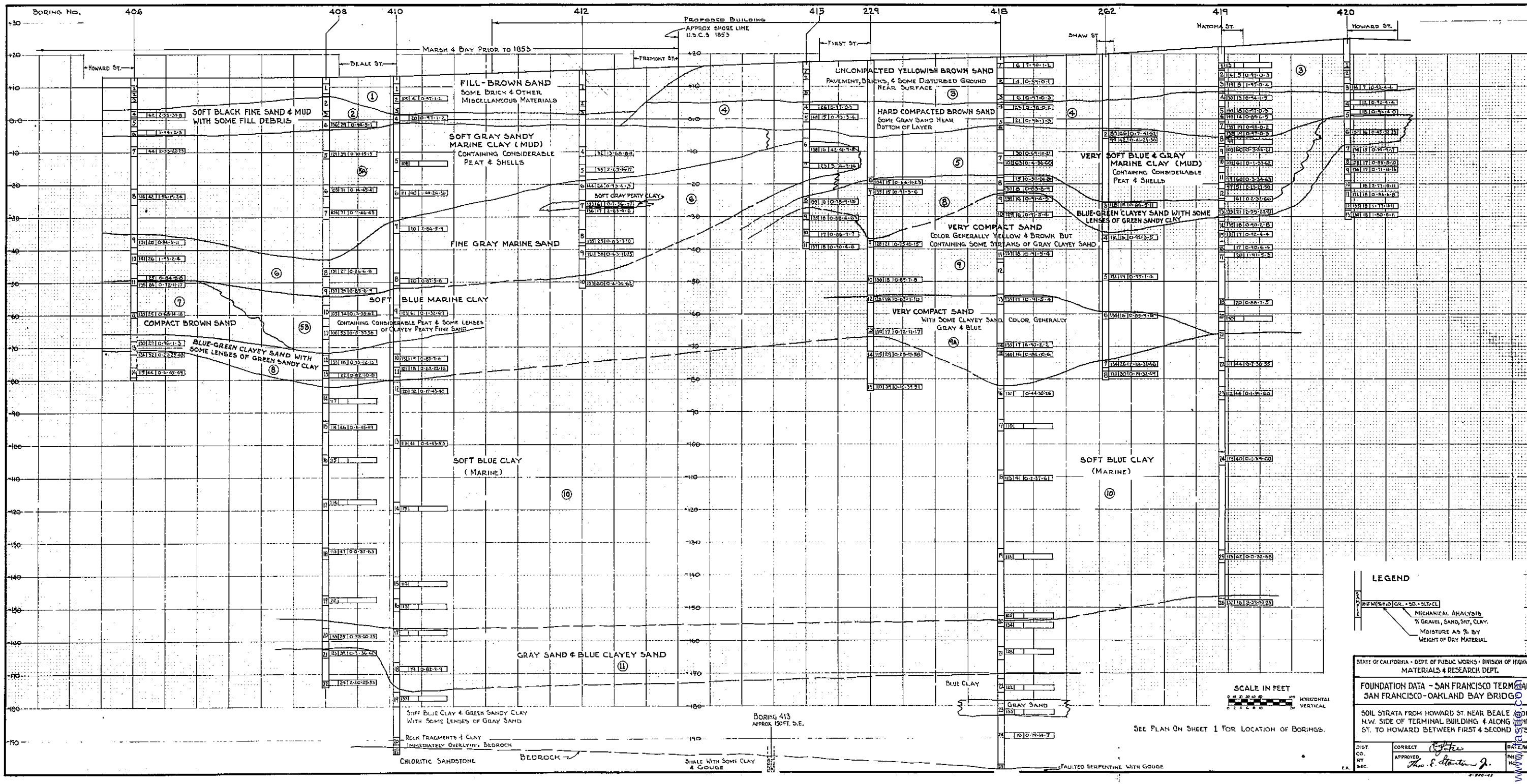

SEE PLAN ON SHEET 1 FOR LOCATION OF BORINGS.

Fig. A-

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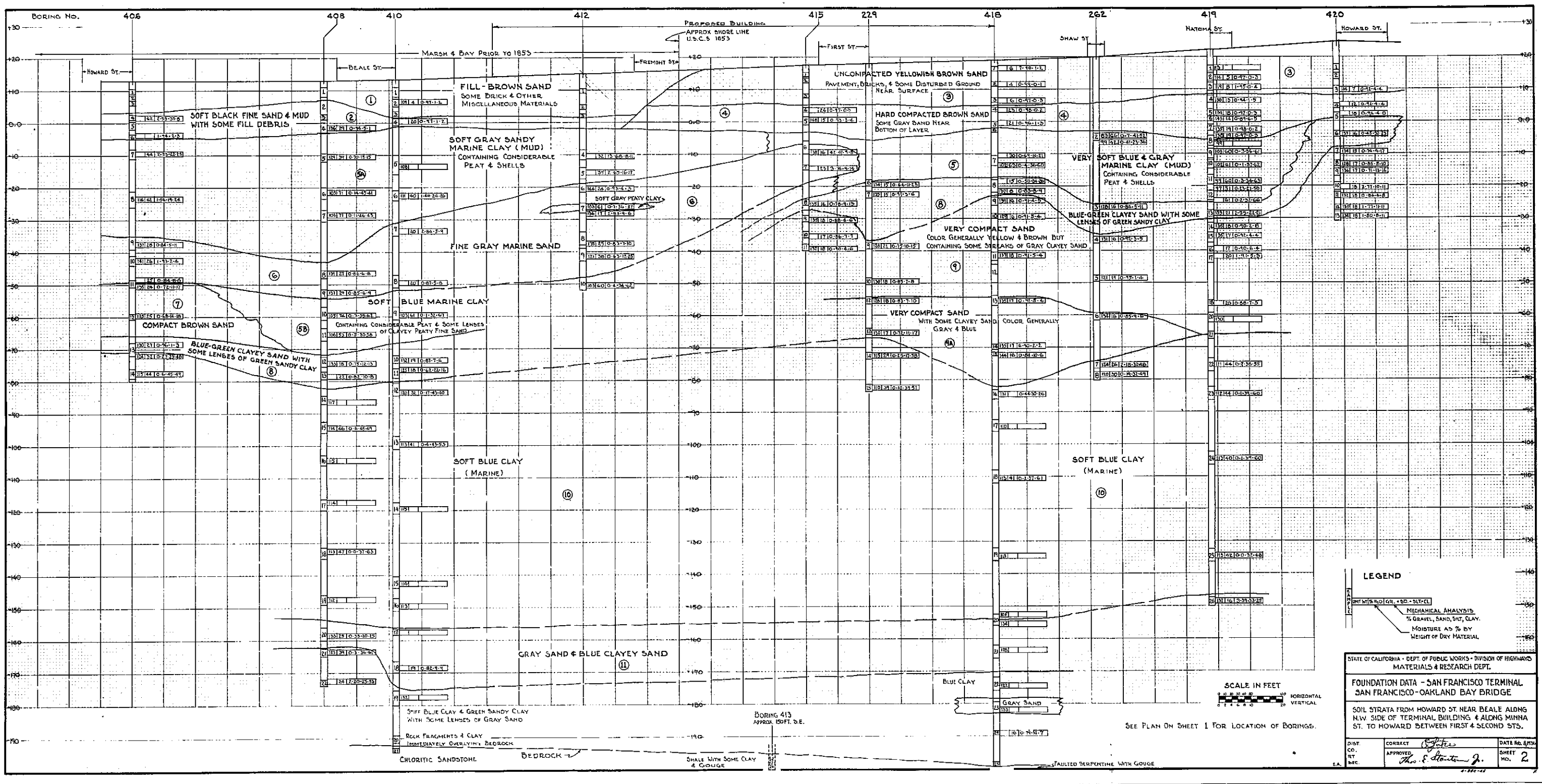

Fig. A-4

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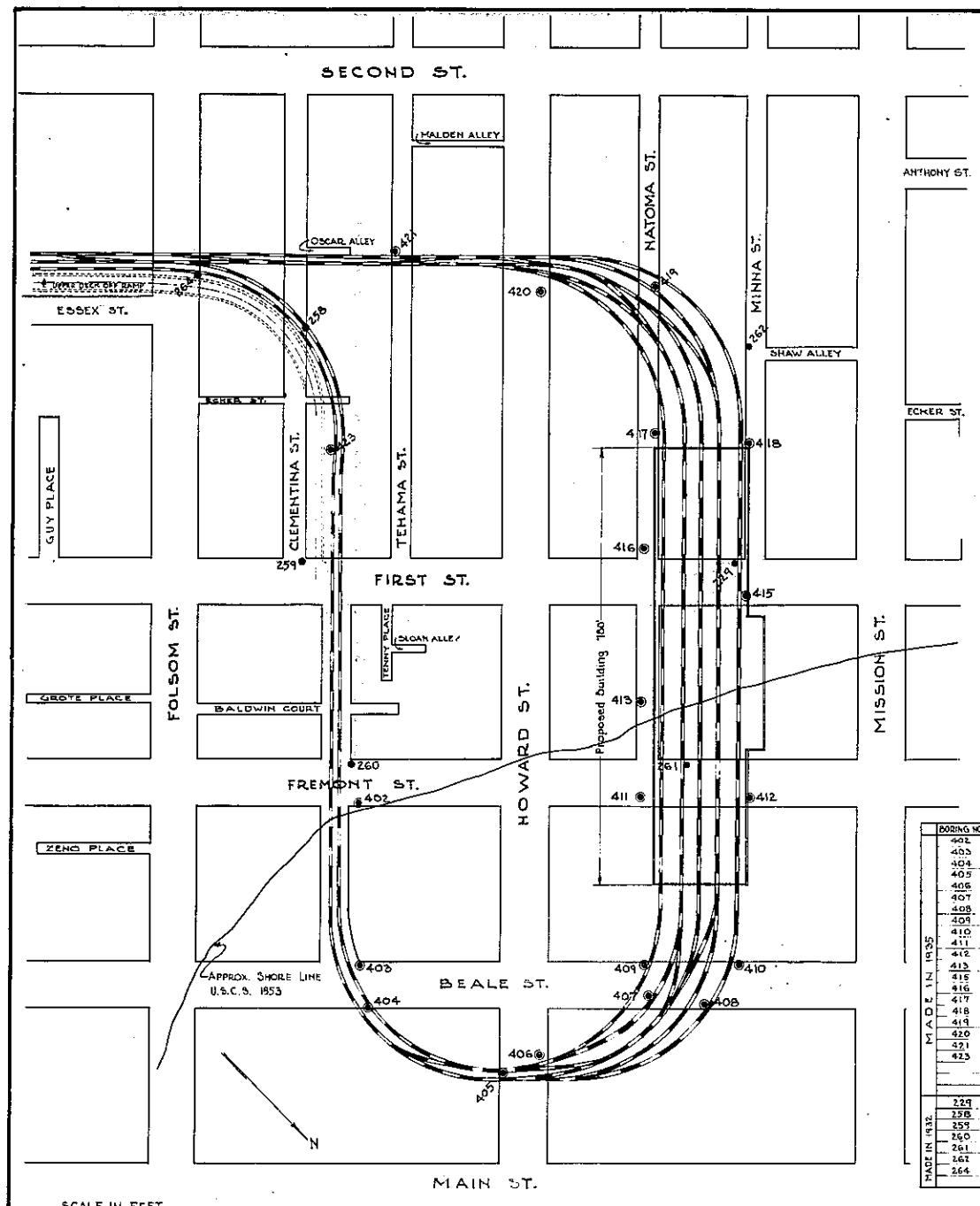
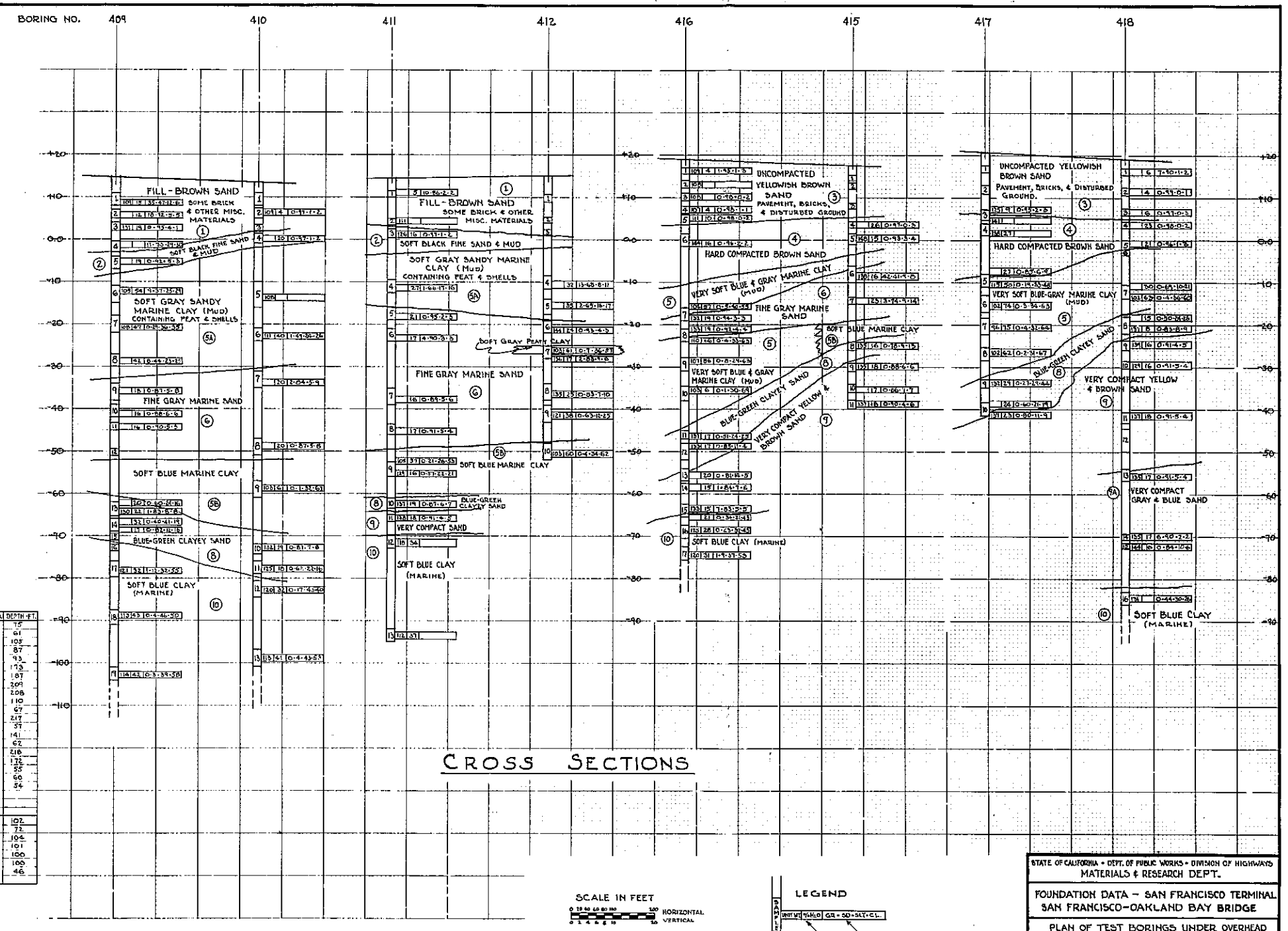


	BORING NO.	DEPTH-FT.
MADE IN 1935	402	75
	403	61
	404	105
	405	87
	406	93
	407	173
	408	187
	409	209
	410	208
	411	110
	412	67
	413	217
	415	57
	416	141
	417	62
	418	218
	419	172
	420	55
	421	60
	423	54
MADE IN 1932	229	102
	258	72
	259	104
	260	101
	261	100
	262	100
	264	46

PLAN OF TEST BORINGS

SEE SHEET NO. 2 FOR SOIL STRATA THRU BORINGS 406-408-410-412-415-229-418-262-419-420
SEE SHEET NO. 3 FOR SOIL STRATA THRU BORINGS 405-406 407-409-411-261-413-416-417-419-420-421
SEE SHEET NO. 4 FOR SOIL STRATA THRU BORINGS 264-258-423-259-260-402-403-404-405



CROSS SECTIONS

SCALE IN FEET
HORIZONTAL
VERTICAL

LEGEND
SAMPLE
UNIT WT. | % MUD | GR-SD-SILT-CL.
MECHANICAL ANALYSIS
% GRAVEL, SAND, SILT, CLAY.
MOISTURE AS % BY WT. OF DRY MATERIAL.

STATE OF CALIFORNIA - DEPT. OF PUBLIC WORKS - DIVISION OF HIGHWAYS
MATERIALS & RESEARCH DEPT.

FOUNDATION DATA - SAN FRANCISCO TERMINAL
SAN FRANCISCO-OAKLAND BAY BRIDGE

PLAN OF TEST BORINGS UNDER OVERHEAD VIADUCTS & TERMINAL BUILDING

CROSS-SECTIONS OF SOIL STRATA

DIST. CO. RT. SEC. | CORRECT | APPROVED Thos. E. Stanton Jr. | DATE Feb. 8, 1938 | SHEET NO. 1

Fig. A-3

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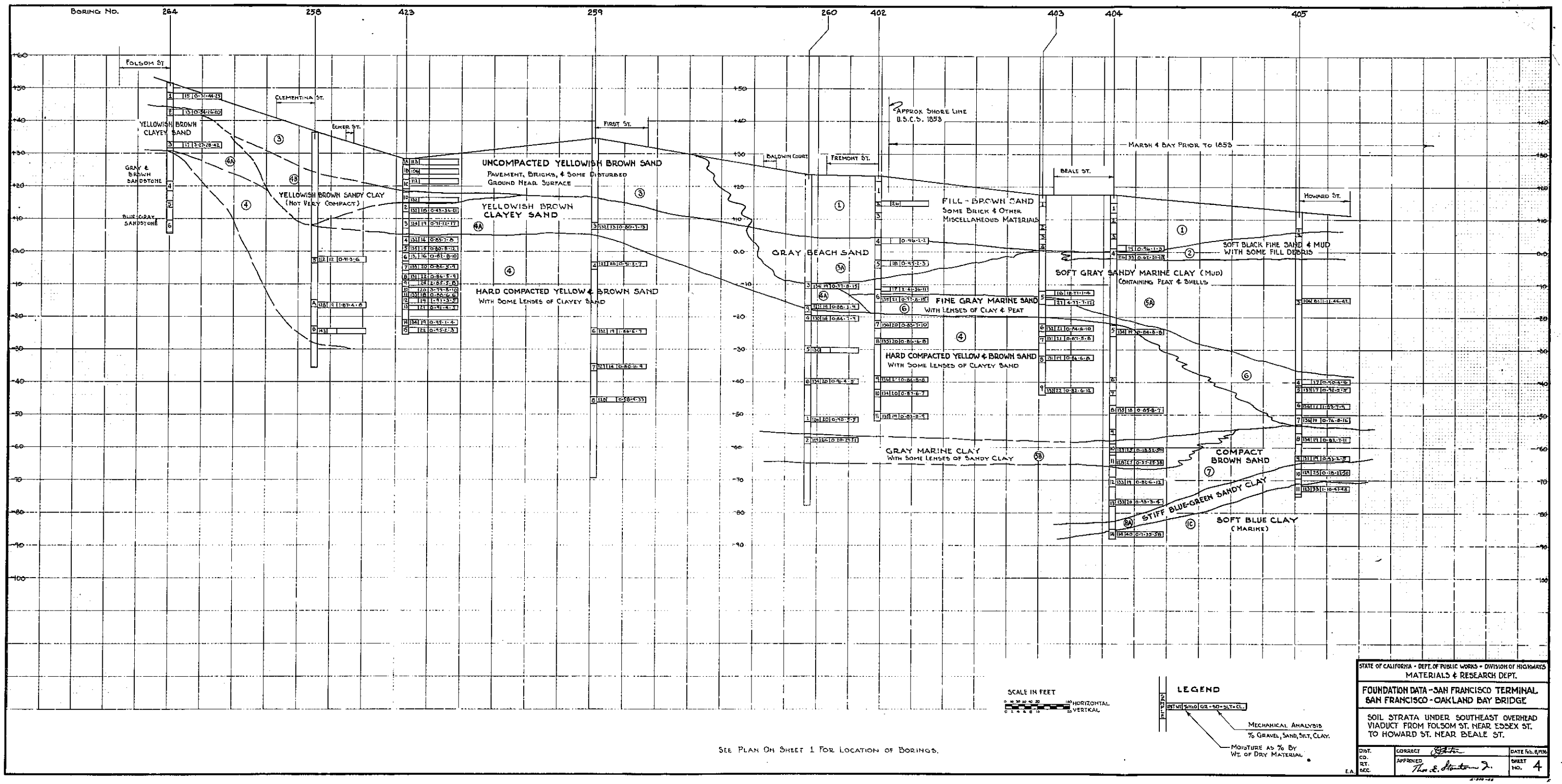

Fig. A-6

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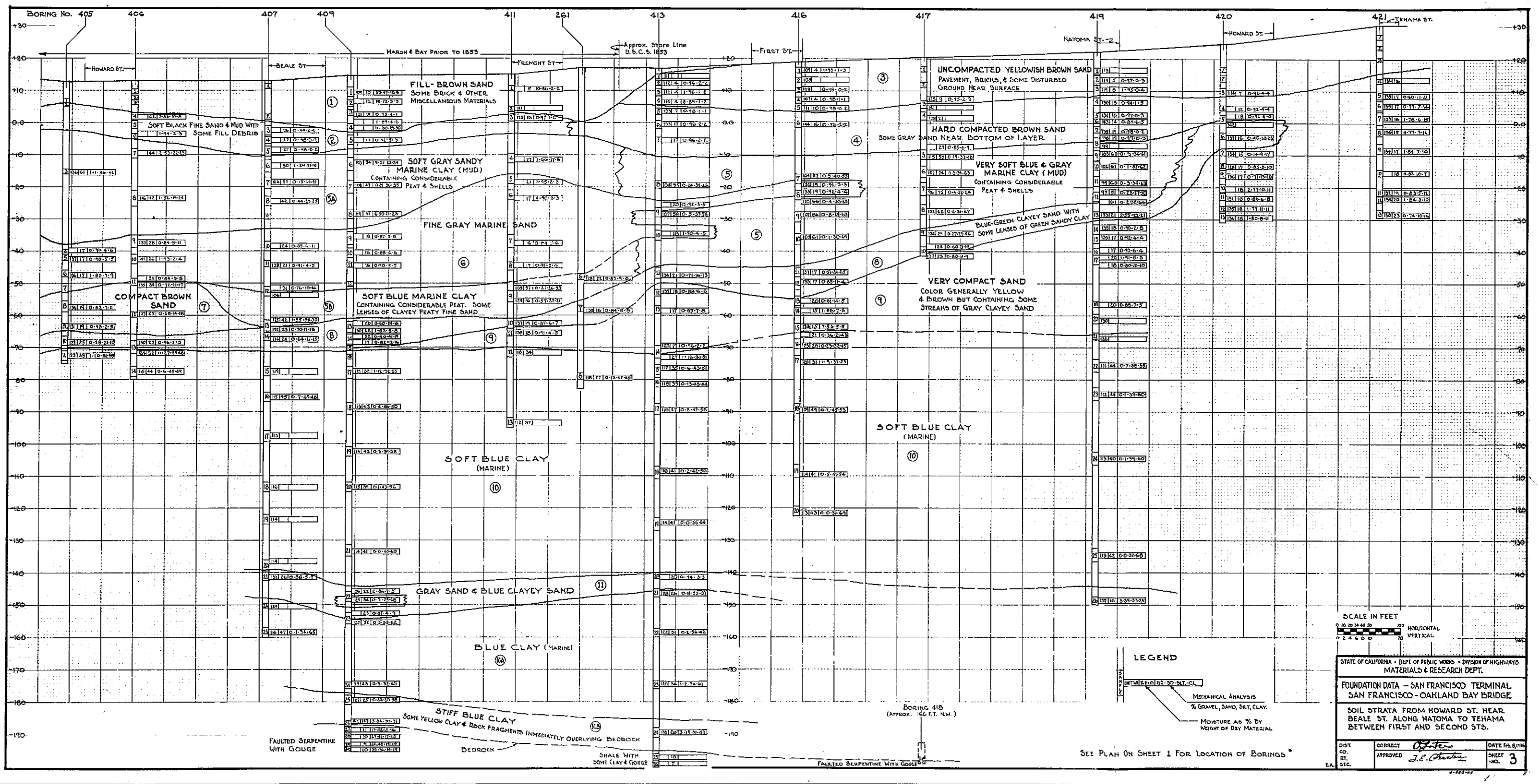

Fig. A-5

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APPENDIX B

STATIC AND DYNAMIC TRIAXIAL TEST RESULTS

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TRIAXIAL SHEAR TEST
JACKURA
SAMPLE NO. R1-6-1 ~~R1-5-3~~

JOB NO. 602-562564
SPECIMEN NO. 1

CONFINING PRESSURE: 1.25 TSF PROVING RING 450

CORRECT STRAIN	DEVIATOR STRESS	MID. PORE PRES.	VOL. CHANGE PCT	TOTAL MAJ.PRIN. STRESS	EFFECTIVE MAJ.PRIN. STRESS	EFFECTIVE MIN.PRIN. STRESS	PRINCIPAL STRESS RATIO
.000	.00	.00	.0	1.25	1.25	1.25	1.00
.245	.60	.00	.0	1.86	1.86	1.25	1.48
.491	.80	.00	.0	2.06	2.06	1.25	1.64
.736	1.06	.00	.0	2.31	2.31	1.25	1.84
.982	1.17	.00	.0	2.42	2.42	1.25	1.93
1.472	1.30	.00	.0	2.55	2.55	1.25	2.04
1.963	1.37	.00	.0	2.62	2.62	1.25	2.09
2.454	1.42	.00	.0	2.67	2.67	1.25	2.13
2.945	1.44	.00	.0	2.70	2.70	1.25	2.15
3.436	1.47	.00	.0	2.72	2.72	1.25	2.17
3.927	1.48	.00	.0	2.73	2.73	1.25	2.18
4.417	1.48	.00	.0	2.73	2.73	1.25	2.18
4.908	1.48	.00	.0	2.73	2.73	1.25	2.18
5.890	1.47	.00	.0	2.72	2.72	1.25	2.17
6.871	1.46	.00	.0	2.71	2.71	1.25	2.17
7.853	1.44	.00	.0	2.69	2.69	1.25	2.15
8.835	1.42	.00	.0	2.68	2.68	1.25	2.14
9.816	1.39	.00	.0	2.65	2.65	1.25	2.11
11.780	1.28	.00	.0	2.54	2.54	1.25	2.02
13.743	1.19	.00	.0	2.44	2.44	1.25	1.95
15.706	1.17	.00	.0	2.42	2.42	1.25	1.94
17.669	1.14	.00	.0	2.40	2.40	1.25	1.91
19.633	1.11	.00	.0	2.36	2.36	1.25	1.89

PCT. MOISTURE BEFORE TEST= 39.40
PCT. MOISTURE AFTER TEST= 39.40
WET U.W. BEFORE TEST= 109.47
DRY UNIT WEIGHT= 78.53

Fig.
B-1

ClibPDF - www.fastio.com

TRIAXIAL SHEAR TEST
JACKURA 121-6-1
SAMPLE NO. ~~81-5-6~~

JOB NO. 602-662564
SPECIMEN NO. 1

CONFINING PRESSURE: 1.25 TSF PROVING RING 450

CONSOLIDATED-UNDRAINED

$\bar{\sigma}_3 \simeq \bar{\sigma}_v$

SOFT BAY MUD SAMPLE; DEPTH -37'

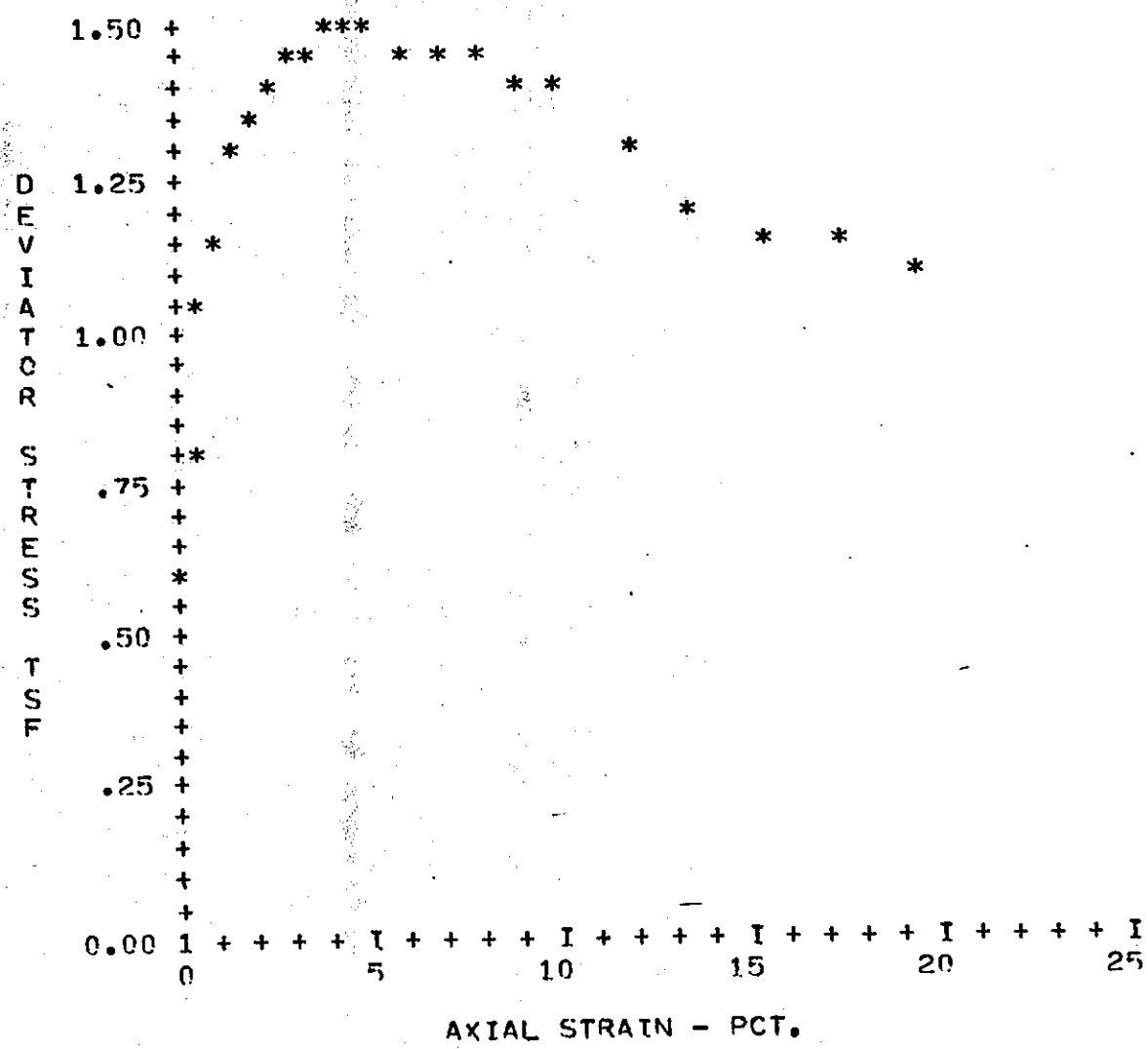

Fig. B-2

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TRIAXIAL SHEAR TEST
JACKURA
SAMPLE NO. R-13-7

JOB NO. 602-662564
SPECIMEN NO. 1

CONFINING PRESSURE: 3.5 TSF PROVING RING 9644

CORRECT STRAIN	DEVIATOR STRESS	MID. PORE PRES.	VOL. CHANGE PCT	TOTAL MAJ.PRIN. STRESS	EFFECTIVE MAJ.PRIN. STRESS	EFFECTIVE MIN.PRIN. STRESS	PRINCIPAL STRESS RATIO
.000	.00	.00	.0	3.50	3.50	3.50	1.00
.245	.08	.00	.0	3.58	3.58	3.50	1.02
.490	1.41	.00	.0	4.91	4.91	3.50	1.40
.735	1.87	.00	.0	5.37	5.37	3.50	1.54
.980	2.23	.00	.0	5.72	5.72	3.50	1.64
1.470	2.80	.00	.0	6.30	6.30	3.50	1.80
1.960	3.26	.00	.0	6.76	6.76	3.50	1.93
2.450	3.62	.00	.0	7.12	7.12	3.50	2.03
2.940	3.79	.00	.0	7.29	7.29	3.50	2.08
3.429	3.89	.00	.0	7.39	7.39	3.50	2.11
3.919	3.93	.00	.0	7.43	7.43	3.50	2.12
4.409	3.91	.00	.0	7.41	7.41	3.50	2.12
4.899	3.91	.00	.0	7.41	7.41	3.50	2.12
5.879	3.85	.00	.0	7.35	7.35	3.50	2.10
6.859	3.79	.00	.0	7.29	7.29	3.50	2.08
7.839	3.69	.00	.0	7.19	7.19	3.50	2.06
8.819	3.50	.00	.0	7.00	7.00	3.50	2.00
9.798	3.37	.00	.0	6.86	6.86	3.50	1.96
11.758	3.20	.00	.0	6.70	6.70	3.50	1.91
13.718	3.16	.00	.0	6.66	6.66	3.50	1.90
15.677	3.11	.00	.0	6.61	6.61	3.50	1.89
17.637	3.06	.00	.0	6.55	6.55	3.50	1.87
19.597	2.97	.00	.0	6.46	6.46	3.50	1.85

PCT. MOISTURE BEFORE TEST= 35.90
PCT. MOISTURE AFTER TEST= 35.90
WET U.W. BEFORE TEST= 116.01
DRY UNIT WEIGHT= 85.37

Fig. B-3

ClibPDF - www.fastio.com

TRIAXIAL SHEAR TEST
JACKURA
SAMPLE NO. R-13-7

JOB NO. 602-662564
SPECIMEN NO. 1

CONFINING PRESSURE: 3.5 TSF PROVING RING 9644

CONSOLIDATED-UNDRAINED
SOFT CLAY; DEPTH -120'
$\bar{\sigma}_3 \simeq \bar{\sigma}_v$

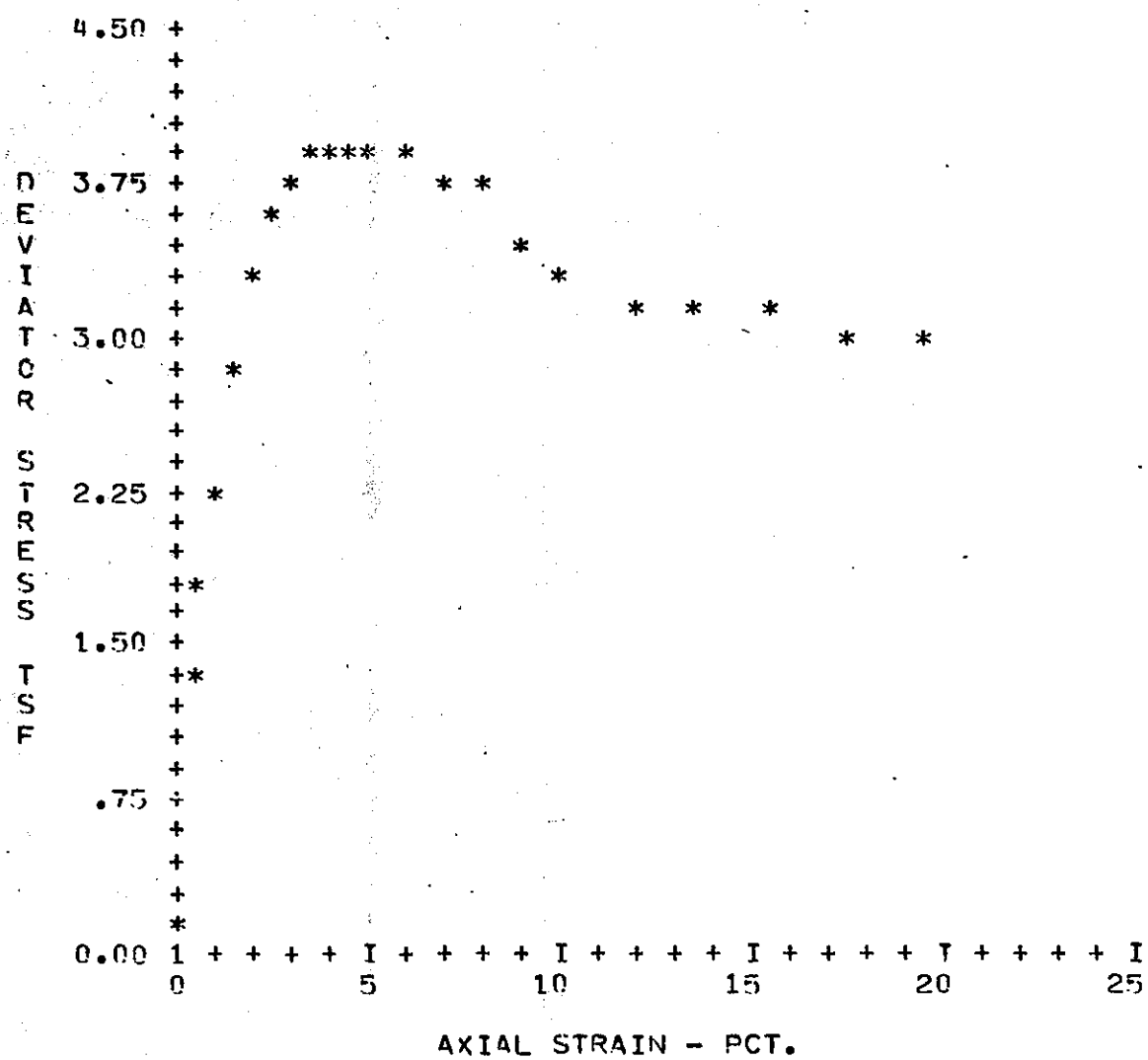

Fig.
B-4

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 10 PSI
$\bar{\sigma}_v \simeq 17$ PSI
PCT MOIST. BEFORE TEST 43.8
PCT MOIST. AFTER TEST 29.4
WET U.W. BEFORE TEST 110
DRY U.W. BEFORE TEST 76.5

DATE 26 JUNE 73
SAMPLE NO. R 1-6-1
AXIAL LOAD CELL -16.2 LBS

SPECIFIC GRAVITY 2.744
INITIAL VOID RATIO 1.2388
PCT SAT. BEFORE TEST 97

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	1.1228	.8747	484534	.4082
20	1.1228	1.2594	477787	.4331
30	1.1225	1.2270	476525	.4588
40	1.1225	1.3071	480575	1.2252
50	1.1221	1.4062	479403	.4579
60	1.1221	1.6369	471300	.4645
70	1.1221	2.0714	478052	.4620

Fig. B-5

ClibPDF - www.fastio.com

RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 20 PSI
$\bar{\sigma}_v \simeq 17$ psi

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -15.1 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	1.0290	.3400	873143	.4735
20	1.0290	.6852	865725	.4608
30	1.0288	.7110	859965	.4964
40	1.0288	.7659	859965	.4607
50	1.0286	.8319	863096	.4567
60	1.0286	.9982	861614	.4521
70	1.0283	1.2139	852896	.4760

Fig. B-6

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R E S O N A N T C O L U M N T E S T

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 30 PSI
$\bar{\sigma}_v \simeq 17$ PSI

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -14 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.8974	.4430	1263621	.3712
20	.8971	.6105	1246633	.3920
30	.8969	.6275	1238290	.3957
40	.8964	.6734	1242246	.3949
50	.8962	.7297	1233907	.3958
60	.8957	.8455	1254052	.3926
70	.8952	1.0864	1254574	.3907

Fig.
B-7

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 40 PSI
σ_v = 17 PSI

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -12.9 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.7640	.2883	1996638	.3818
20	.7640	.4502	1994110	.3797
30	.7633	.4653	1983038	.3816
40	.7624	.4987	1990157	.3774
50	.7619	.5371	1988666	.3788
60	.7607	.6439	1993792	.3778
70	.7607	.8052	2003934	.3768

Fig. B-8

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R E S O N A N T C O L U M N T E S T

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 50 PSI
$\bar{\sigma}_v \simeq 17$ PSI

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -11.8 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.7579	.2616	2259357	.3804
20	.7579	.4031	2246040	.3812
30	.7579	.4135	2246040	.3810
40	.7575	.4418	2256461	.3809
50	.7570	.4789	2244963	.3814
60	.7568	.5600	2254779	.3786
70	.7568	.7143	2241497	.3795

Fig. B-9

ClibPDF - www.fastio.com

RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 60 PSI
$\bar{\sigma}_v \simeq 17$ PSI

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -10.7 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.7008	1.0500	3272139	.3579
20	.7000	3.1913	3254748	.3555
30	.6998	5.0915	3229020	.3608
40	.6993	6.8465	3197685	.3681
50	.6984	8.3076	3148609	.3797
60	.6975	10.1425	3106450	.3957
70	.6970	11.8138	3056534	.4125

Fig.
B-10

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 70 PSI
$\bar{\sigma}_v$ = 17 PSI

DATE 26 JUNE 73
SAMPLE NO. R1-6-1
AXIAL LOAD CELL -9.6 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.6935	1.1011	3507927	.3588
20	.6925	2.8027	3525789	.3583
30	.6918	4.7519	3444830	.3650
40	.6914	6.2357	3391527	.3712
50	.6897	8.2053	3346692	.3806
60	.6888	9.3495	3306245	.3925
70	.6888	11.1286	3242487	.4057

Fig.
B-11

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 10 PSI
$\bar{\sigma}_v \simeq 97$ PSI
PCT MOIST. BEFORE TEST 38.3
PCT MOIST. AFTER TEST 35.9
WET U.W. BEFORE TEST 115.7
DRY U.W. BEFORE TEST 83.6

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -16.2 LBS

SPECIFIC GRAVITY 2.73
INITIAL VOID RATIO 1.0377
PCT SAT. BEFORE TEST 100.8

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.9820	3.5368	735463	.3594
20	.9815	10.0612	677756	.4031
30	.9811	14.2761	634879	.4819
40	.9804	18.2547	598830	.5398
50	.9804	22.8938	543209	.5854
60	.9800	27.0502	497010	.6545
70	.9800	31.4650	462252	.7140

Fig.
B-12

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 20 PSI
$\bar{\sigma}_v \simeq 17$ PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL --15.1 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.9833	3.2329	1052030	.2714
20	.9827	8.2095	1018730	.2995
30	.9822	11.9213	982739	.3651
40	.9822	15.4347	928840	.4055
50	.9822	18.5884	883822	.4515
60	.9818	22.1197	834260	.4798
70	.9815	25.0463	801032	.5204

Fig.
B-13

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 30 PSI
$\bar{\sigma}_v \simeq 47$ PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -14 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.9804	2.7735	1267745	.2574
20	.9800	7.4163	1238193	.2754
30	.9800	10.7013	1199560	.3208
40	.9793	14.0039	1153274	.3570
50	.9791	16.9466	1110042	.3732
60	.9787	19.4570	1073233	.4078
70	.9780	22.1118	1027131	.4466

Fig.
B-14

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 40 PSI
$\bar{\sigma}_v \simeq 97$ PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -12.9 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.9187	2.6723	1450837	.2622
20	.9160	6.3428	1421514	.2891
30	.9149	9.7004	1382395	.3176
40	.9142	11.9557	1354319	.3589
50	.9136	14.6317	1310629	.3928
60	.9129	18.2408	1265894	.4083
70	.9125	20.3603	1246174	.4278

Fig.
B-15

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 50 PSI
$\bar{\sigma}_v \simeq 47$ PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -11.8 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.8574	2.0886	1938088	.2376
20	.8569	5.8131	1910157	.2434
30	.8560	9.2500	1872360	.2584
40	.8552	11.3195	1832764	.2900
50	.8549	14.2010	1794449	.2976
60	.8540	16.5602	1736648	.3207
70	.8538	19.2294	1695170	.3434

Fig. B-16

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R E S O N A N T C O L U M N T E S T

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 60 PSI
$\bar{\sigma}_v \simeq 47$ PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -10.7 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.8580	1.9939	2074489	.2437
20	.8567	5.4584	2047493	.2488
30	.8558	8.6785	2010625	.2636
40	.8547	10.6108	1974561	.2944
50	.8540	13.1997	1922357	.2982
60	.8529	15.8018	1873937	.3243
70	.8520	17.9229	1834333	.3398

Fig.
B-17

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RESONANT COLUMN TEST

TEST NO. C
JOB NO. 662564
CONFINING PRESS. 70 PSI
$\bar{\sigma}_v$ ~47 PSI

DATE 8-6-72
SAMPLE NO. R1-13-1
AXIAL LOAD CELL -9.6 LBS

AMPL. SETTING	VOID RATIO	MAX% STRAIN X E-4	SHEAR MODULUS PSF	DAMP. RATIO
10	.8365	1.8701	2295448	.2481
20	.8358	5.0160	2274926	.2527
30	.8352	7.9655	2239876	.2624
40	.8347	9.7058	2226362	.2894
50	.8338	12.2563	2168280	.2906
60	.8329	14.3467	2125181	.3068
70	.8323	16.7707	2091400	.3255

Fig.
B-18

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APPENDIX C

GRAIN SIZE ANALYSES

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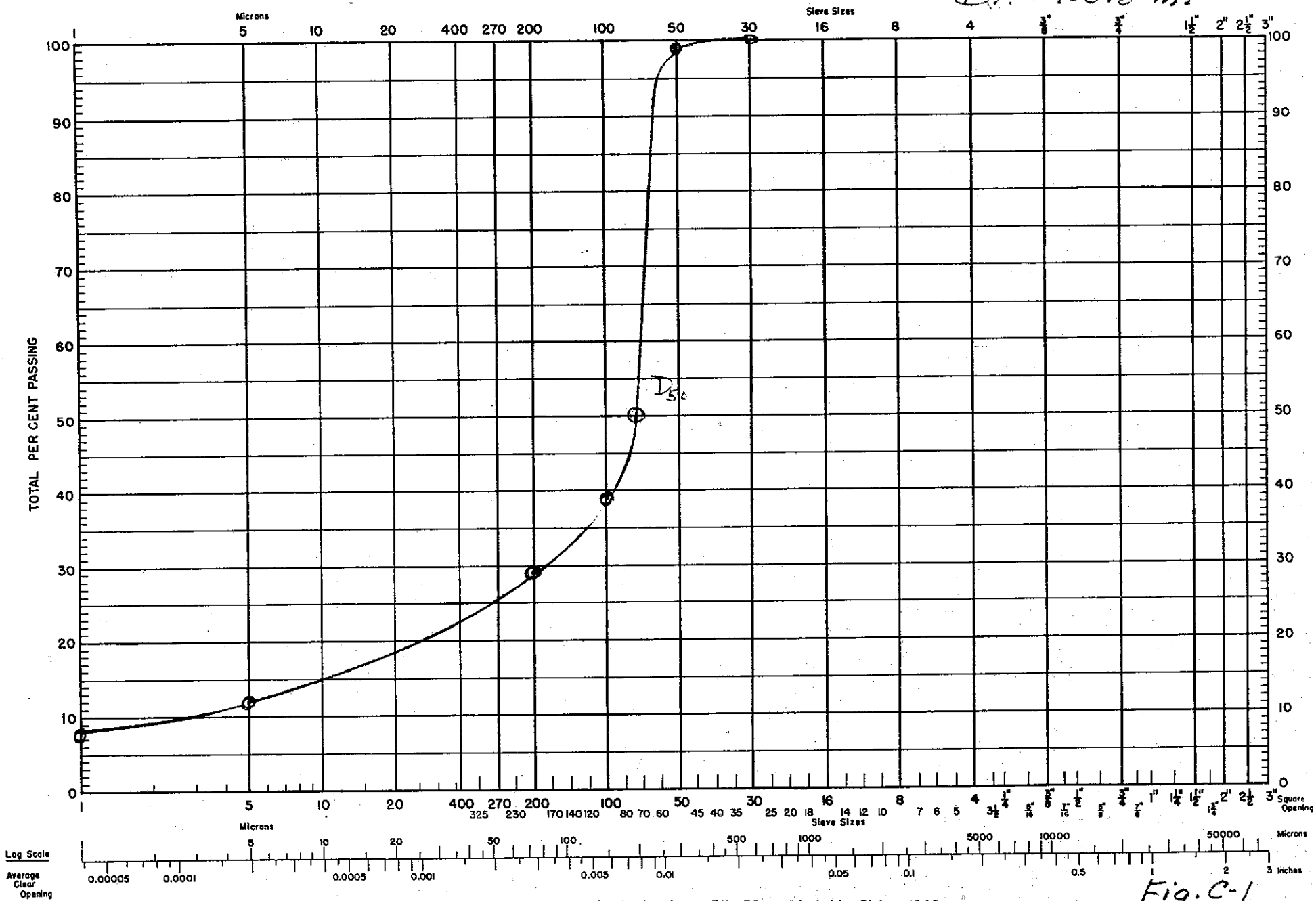
SAMPLE R1-3
DEPTH 17.0 to 17.5 ft.
D50 = .0078 IN.

GRADING ANALYSIS



A.A.S.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

Fig. C-1

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SOFT BAY MUD; DEPTH -37'

SAMPLE No. R1-5-6

GRADING ANALYSIS

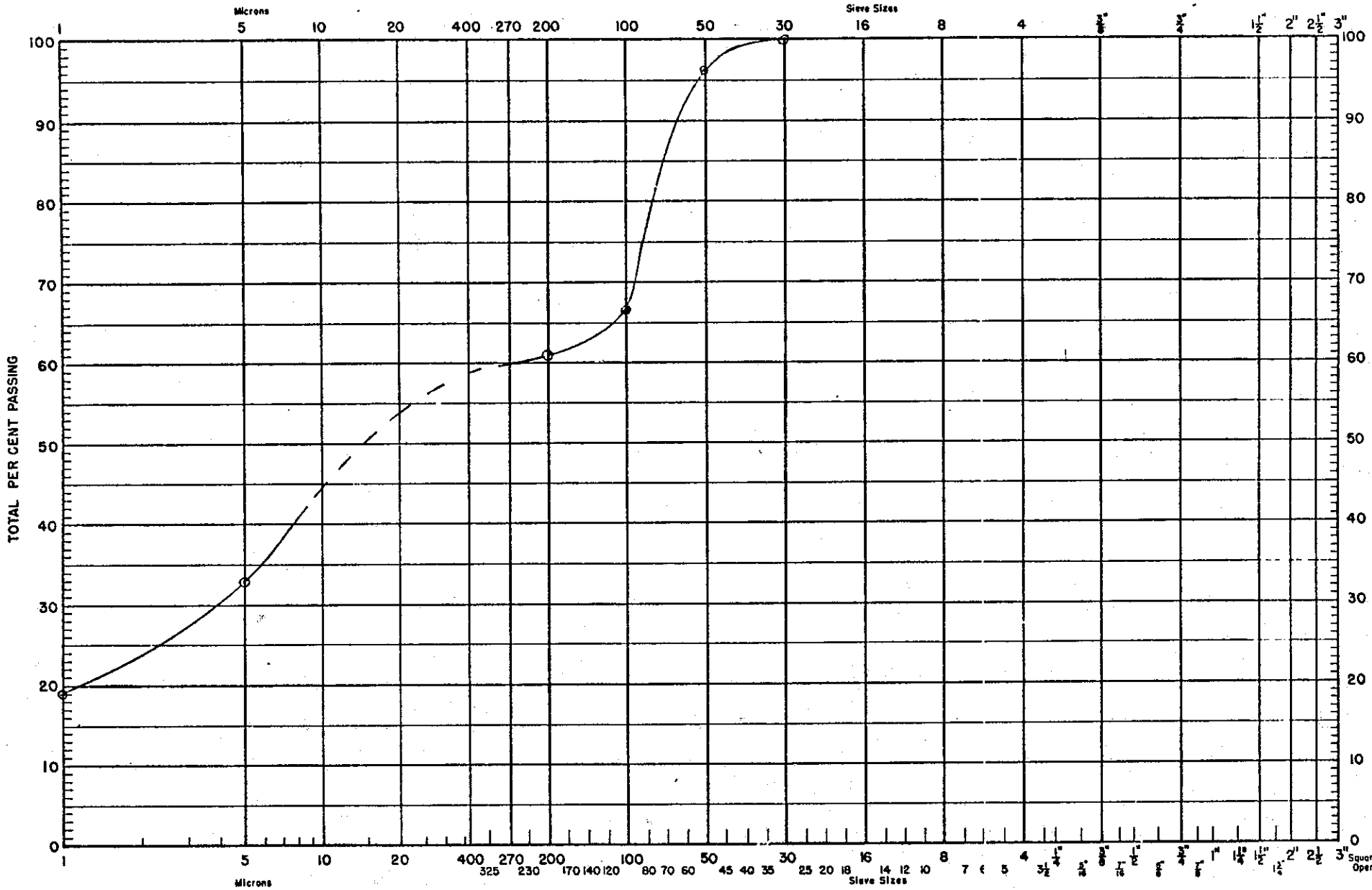

Fig. C-2

A.A.S.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

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GRADING ANALYSIS

SOFT BAY MUD; DEPTH -37'
SAMPLE NO. R1-6-1

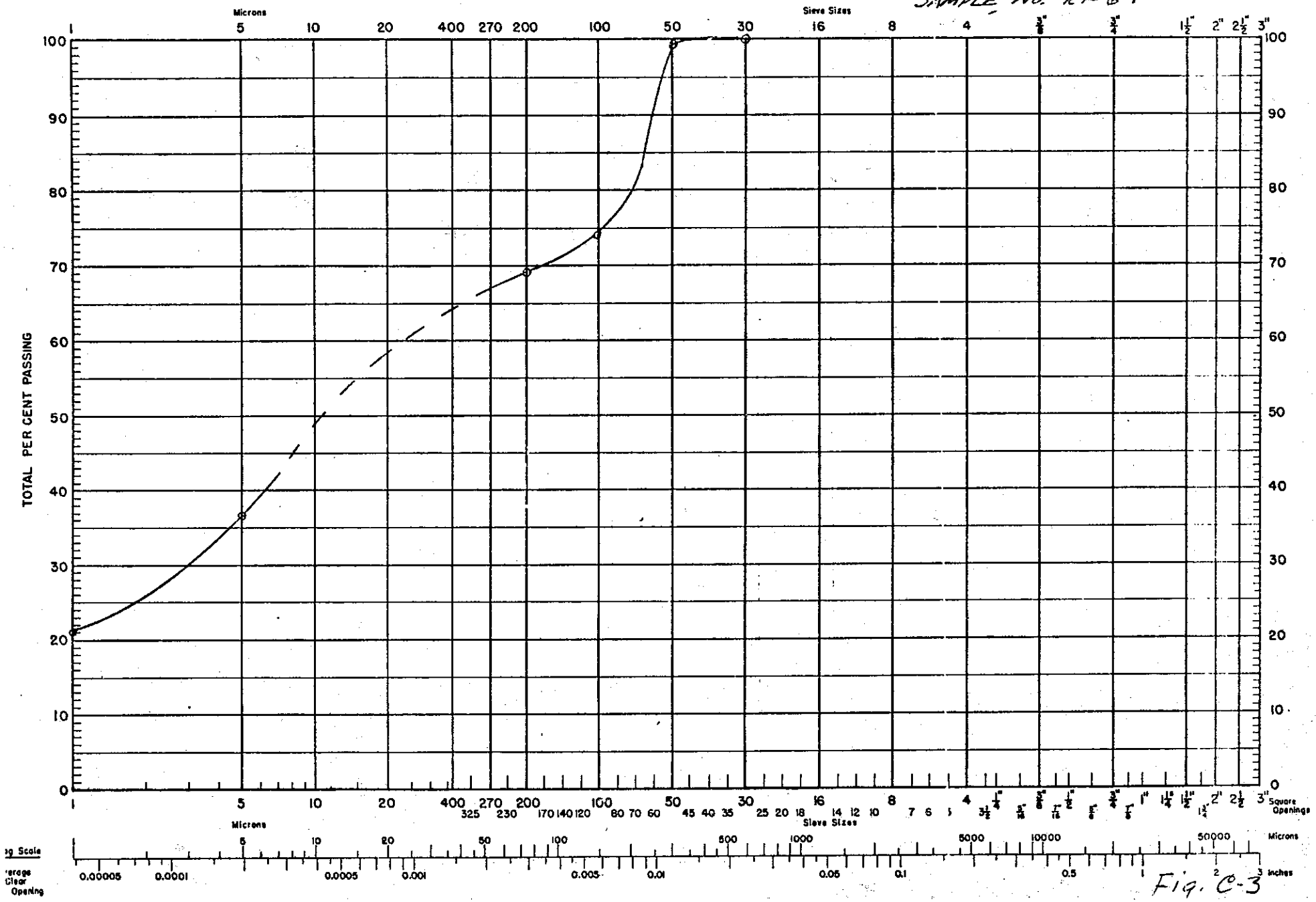

Fig. C-3

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GRADING ANALYSIS

Sample R1-7
DEPTH 45.0 TO 45.5 ft
D_{50} = .0105 in.

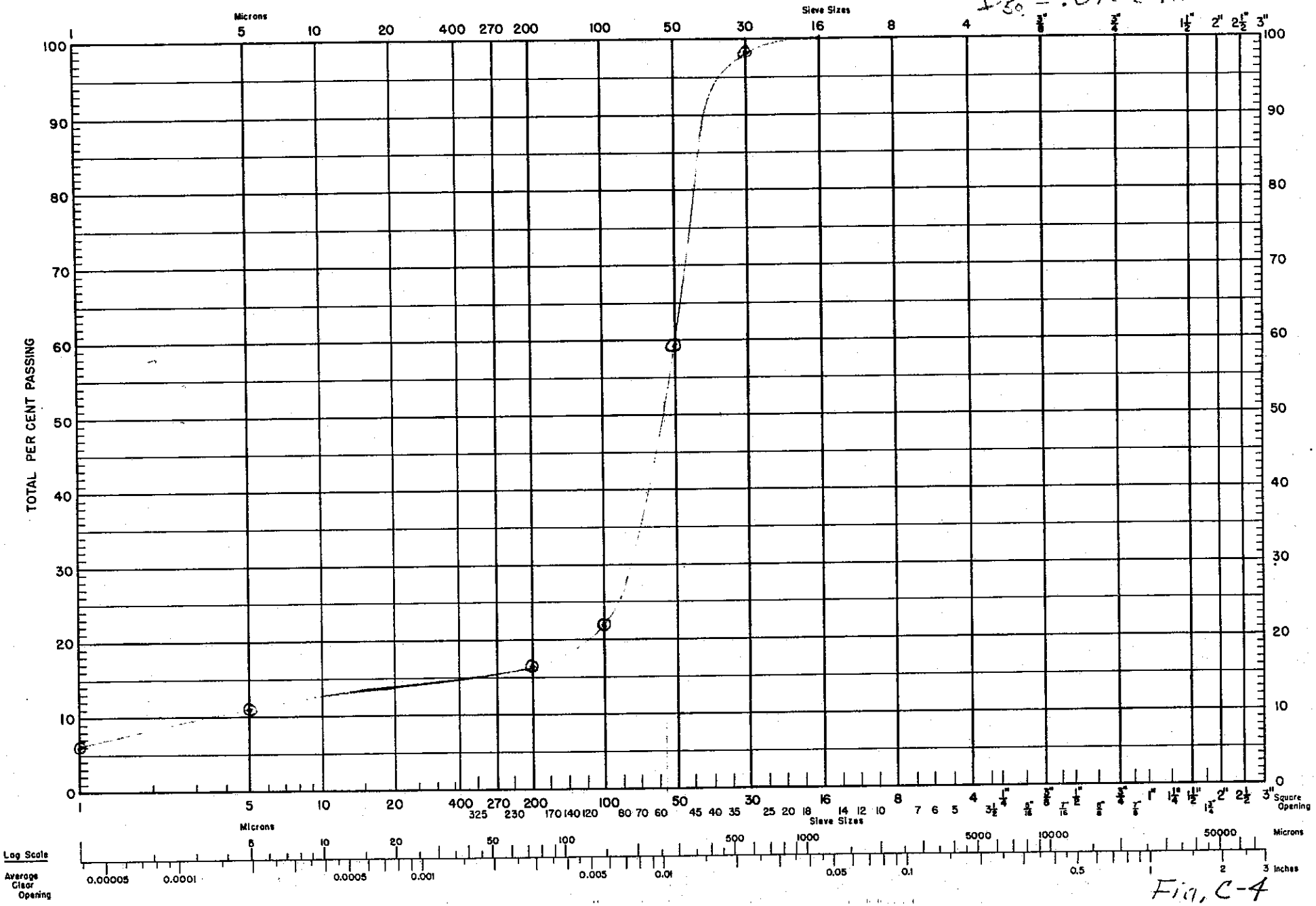

Fig. C-4

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SAMPLE R1-8
DEPTH 60.5 to 61.0 ft.
D_{50} = .0105 in

GRADING ANALYSIS

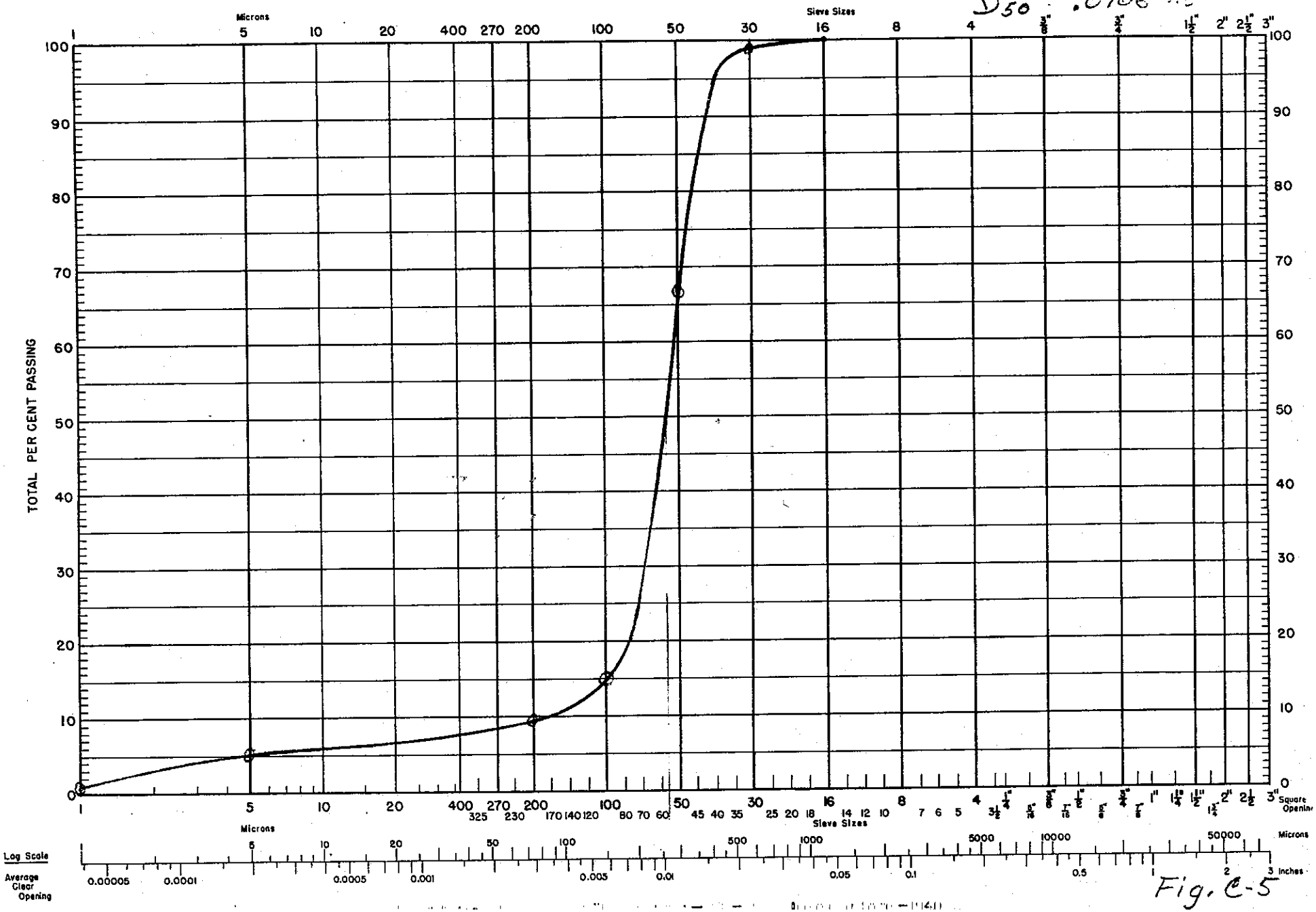

Fig. C-5

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SOFT CLAY; DEPTH -120'

SAMPLE NO. R1-13-1

GRADING ANALYSIS

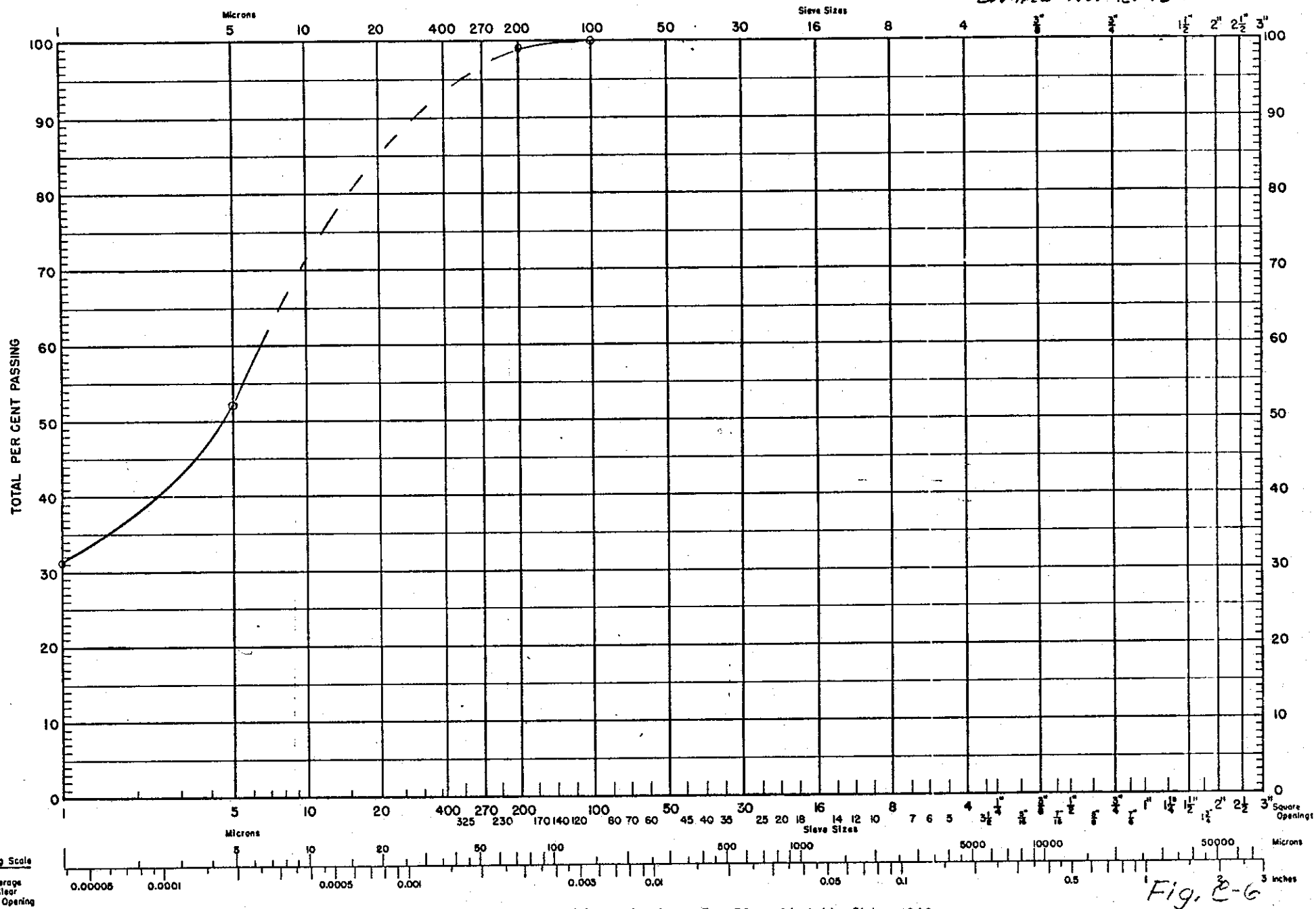

Fig. C-6

A.A.S.H.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

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SAMPLE R4-4
DEPTH 20' TO 20.5'.
D_{50} = .0082 in.

GRADING ANALYSIS

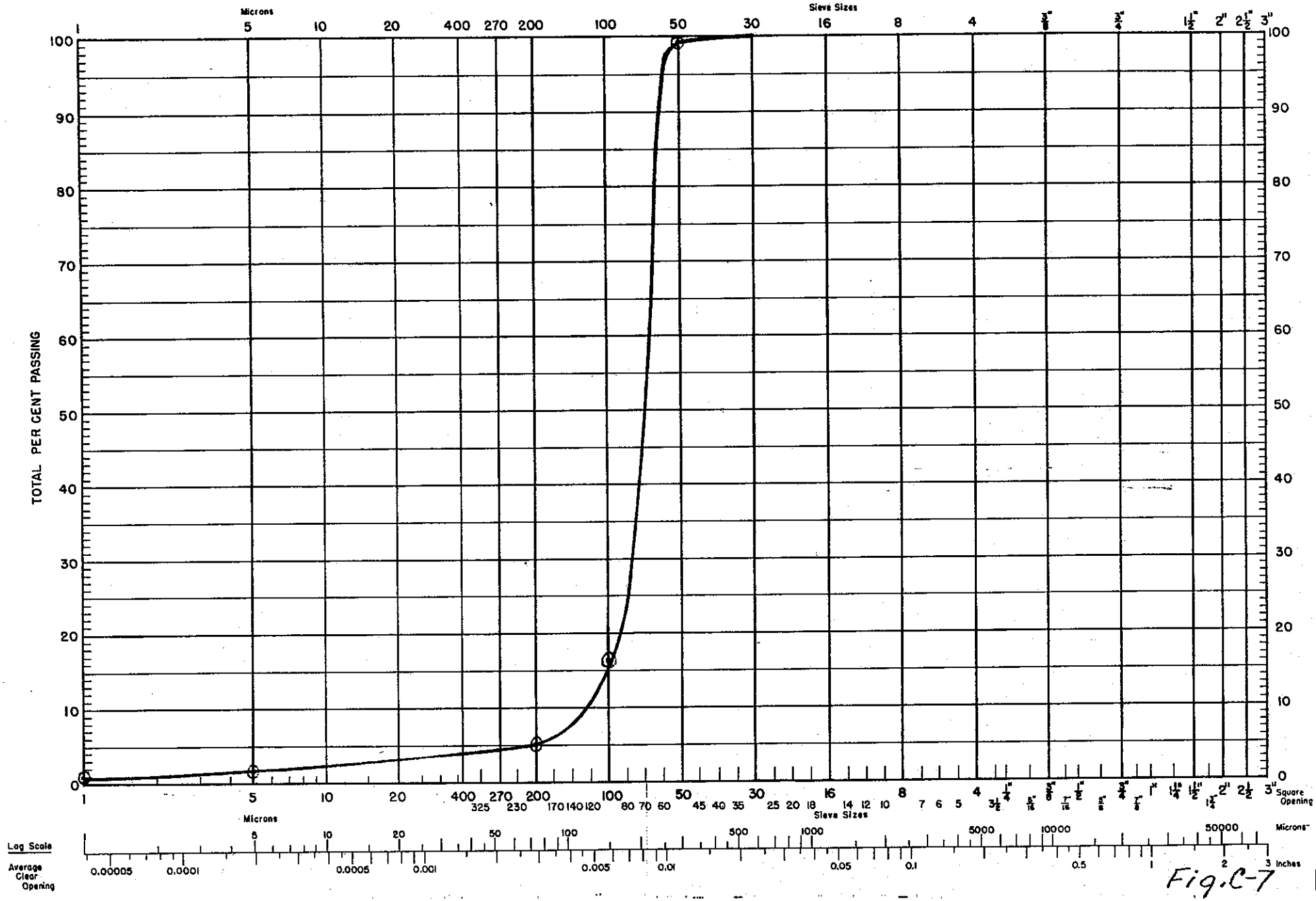

Fig. C-7

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SAMPLE R4-B

DEPTH: 60° to 60⁵ ft.

D_{50} = .0150 IN.

GRADING ANALYSIS

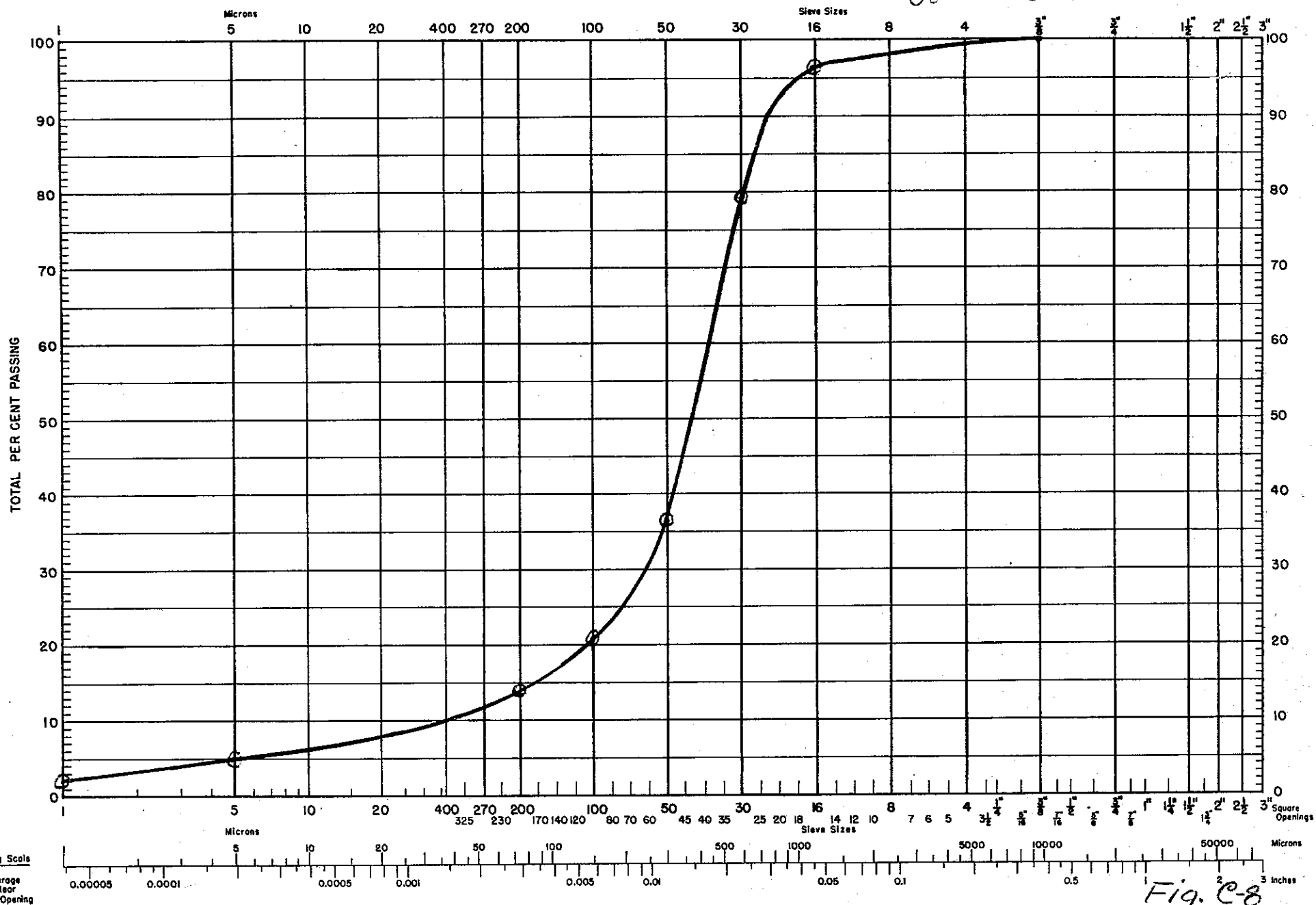

Fig. C-8

A.A.S.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

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SAMPLE R4-9
DEPTH 70° TO 70.5 FT.
D_{50} = .0085 IN

GRADING ANALYSIS

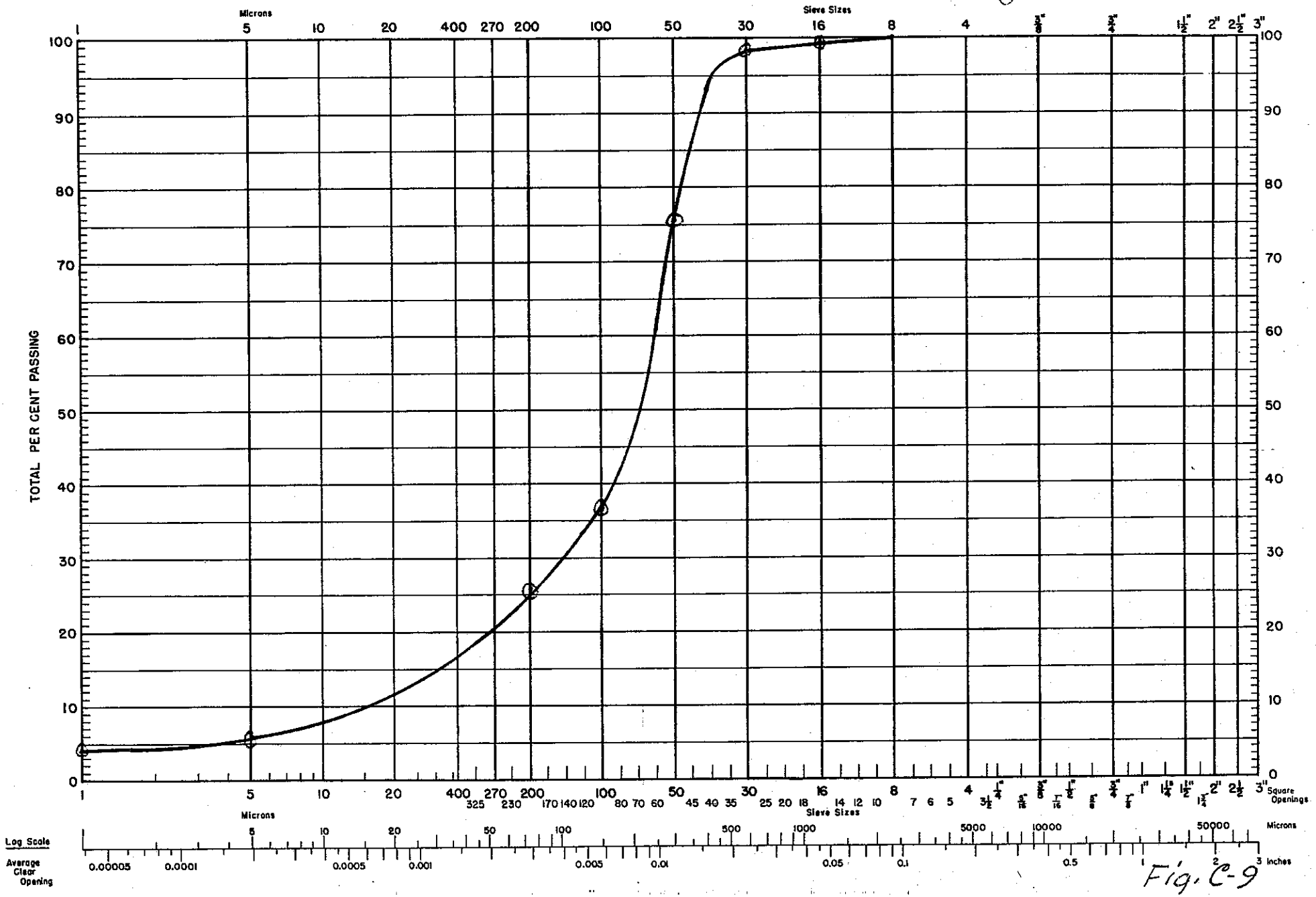

Fig. C-9

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GRADING ANALYSIS

SOFT CLAY; DEPTH -120'

SAMPLE NO. R-13-7

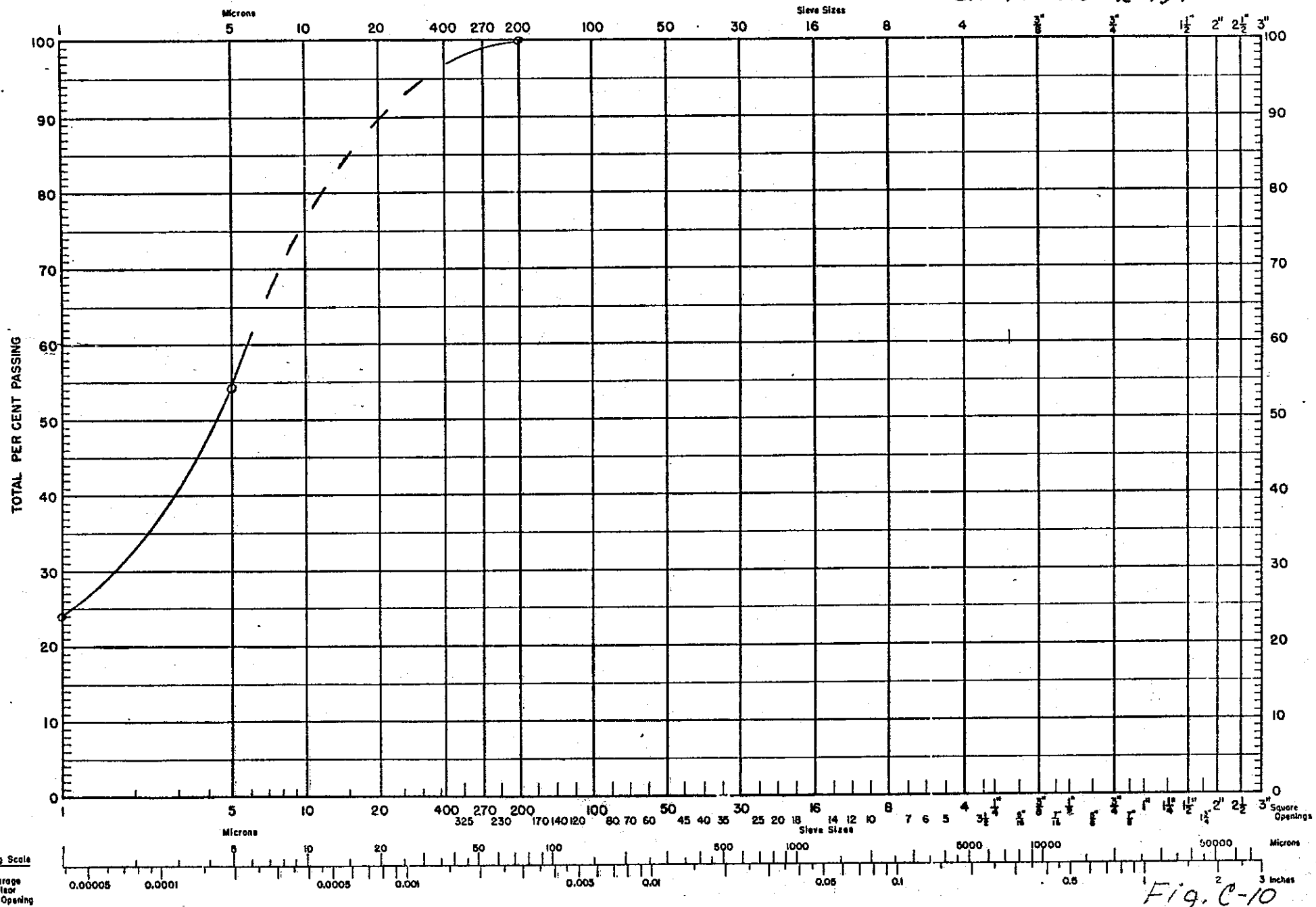

Fig. C-10

A.A.S.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

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