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Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

ON

GEOTECHNICAL SITE APPRAISAL
SOUTH EAST CITY
OPEN SPACE SYSTEM
OTTAWA-CARLETON · ONTARIO

TO

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ABSTRACT

The results of a geotechnical appraisal of some 4,000 acres of land recently acquired by the National Capital Commission (NCC), Ottawa, are presented. This land is adjacent to and forms a complementary open space system to the proposed Ontario Housing Corporation South East City development site in Ottawa-Carleton, Ontario.

The soil stratigraphy encountered across the NCC lands is similar to that encountered at the OHC site, i.e. an intermittent surficial sand cap overlies an extensive deposit of sensitive Leda clay. The upper portion of this deposit has been desiccated to a relatively stiff consistency to various depths across the site. The combined thickness of sand cap and desiccated crust varies from 4 to 12 feet in the south west sector and 4 to greater than 16 feet in the north east sector of the NCC lands. Within the south west sector the significant soil parameters (i.e. undrained shear strength and preconsolidation pressure, p_c) are similar to those measured in previous investigations on the OHC site. However, due mainly to a drop off in ground surface elevation north of Hwy. 417, the significant soil parameters in this area are higher than the equivalent values south of Hwy. 417. Although not directly proven, the depth to till or bedrock across the NCC lands to the north of Hwy. 417 is generally close to 100 feet while in the south west sector, the depth to bedrock from ground surface over an extensive portion of the sector is less than about 90 feet.

As well as describing the subsoil conditions across the NCC lands, this report summarizes the major pertinent geotechnical aspects of development within the general area and provides a general overview of the data obtained to date, including the results of the previous information from the OHC site.

With respect to foundation design, it is considered that, with reasonably judicious location of the various structures, all the buildings required by the development can be safely and, in the majority of cases, economically accommodated within the site. Various other aspects of site development such as the construction of roads and installation of services are considered and, from the geotechnical standpoint, there are no major geotechnical problems associated with these aspects of development. Based on available data, it is considered that development of the proposed site will not significantly affect the nature of the adjacent marshland Mer Bleue. Although no quantitative assessment can be made at this time, it is considered unlikely that there will be significant changes in regional and local groundwater levels.

In summary, while there are certain necessary geotechnical constraints as outlined in the report, there is no reason why development of the site can not proceed in a normal fashion.

1. INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd., Consulting Engineers to the National Capital Commission (NCC), Ottawa, to carry out a subsurface investigation within two areas in the Township of Gloucester in the Regional Municipality of Ottawa-Carleton, Ontario. The purpose of this investigation was to make an appraisal of the effect of development of the adjacent Ontario Housing Corporation (OHC) site on the NCC lands which are to form a complementary open space system. Recommendations regarding the development of the OHC site (i.e. the proposed South East City development site) have previously been presented in our report no. 73908, Vol. IV, dated June, 1974. In addition to reporting the results of the present investigation, this report also presents a general overview of the various geotechnical aspects of development of the complete South East City Growth area including the results of previous investigations related to the OHC site.

2. DESCRIPTION OF PROJECT

The site under consideration consists of about 4,000 acres of land in Gloucester Township, about 5 miles south east of the Ottawa City limits (see Figure 1 for Key Plan). The site forms parts of the boundary of the proposed "South East City Site" which is to be developed by the Ontario Housing Corporation (OHC) as a satellite city to the City of Ottawa. As an integral part of the planning study of the overall development of the area, it was considered necessary to include an assessment of the possible effects of development on the lands adjacent to the proposed South East City site.

Information on the soil conditions in the general area has been obtained during a number of investigations on the

OHC lands and in other studies carried out by the Division of Building Research of the National Research Council, Ottawa and the Ministry of Transportation and Communications. A list of the site investigations is given in Appendix I and this information has been used together with the results of the present investigation to define the subsurface conditions across the general area. To differentiate between the boreholes put down during the various investigations, the numbering system shown on the Site and Boring Plan (Figure 2) has been adopted.

3. SITE DESCRIPTION

The NCC lands are divided into two sections. The larger section, consisting of about 2,500 acres, is located immediately to the north of Hwy. 417, while the remaining land is located along the south west boundary of the OHC site, close to the Department of National Defence and Canadian Forces Station Gloucester (Figure 2). The latter sector, hereafter referred to as the "south west sector", is sparsely populated and is understood to be considered as marginal agricultural land. The terrain, which is partially covered by dense woodland, is flat-lying between about elevations 260 and 270. Drainage of the area is provided by a series of shallow natural and agricultural ditches which flow to the main watercourse in the area, i.e. Bear Brook.

The ground surface elevation in the larger sector of the site, hereafter referred to as the "north east sector", falls gently from about elevation 260 along Hwy. 417 to between about elevations 220 and 230 along a shallow depression running parallel to Hwy. 417 and which forms part of the Bear Brook valley. In the most northerly part of the north east sector the land rises in a local knoll which is adjacent to the extensive Mer Bleue marshland.

Within the north east sector, the land adjacent to Hwy. 417 is densely treed, changing to marginal agricultural land and, along the north east boundary of the sector, becoming relatively densely populated in the Town of Carlsbad Springs. The north east sector is traversed by Bear Brook which flows northerly across the site before abruptly changing direction immediately south of Carlsbad Springs and flows out of the site in an easterly direction. In its lower reaches within the site, the Bear Brook valley profile changes from being relatively steep-sided to a gently sloping flood plain.

4. GEOLOGY

Following the retreat of the ice sheet which occupied the Ottawa valley in the late Pleistocene period, the area was inundated by the marine waters of the Champlain Sea in which sensitive silty clay, known as Leda clay, was deposited. In some areas, the upper portions of the Leda clay deposits have been reworked and eroded by wave and current action during the last stages of the Champlain Sea and during subsequent estuarine and fluvial stages. Further changes in the character of the deposit were caused by the release into the area of large quantities of glacial meltwater and silt laden water from the contemporary Great Lakes region which resulted in brackish water conditions. As a result of the changes in sedimentary conditions and environment, the "Leda" clay deposit is the result of various deposition, erosion and redeposition cycles.

In time the flow of water into the Champlain Sea decreased as glacial meltwater was channeled into the immediate predecessor of the modern Great Lakes system. During the resulting estuarine and deltaic periods of the ancestral Ottawa River, widespread sand deposits were formed in the Ottawa valley. The deltaic sands are traversed by two wide flat bottomed channels. The present Ottawa River occupies the northern channel while the southern channel is drained in part by Bear Brook and the South Nation River. In two large undrained sections of the delta area, the Mer Bleue and Alfred peat bogs have been developed.

The complex of deposits described above are generally underlain by glacial deposits and Palaeozoic sandstones, shales, limestones or dolomites.

SECTION A

FACTUAL INFORMATION

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5. SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the borings, which were put down at the locations shown on Figure 2, is shown on the Record of Borehole sheets following the text of this report. Details of fieldwork and laboratory testing procedures are given in Appendix II. Following is a summarized account of the subsurface conditions at the site. A detailed description (including summaries of engineering properties) of the major overburden strata are presented separately in the succeeding section.

5.1 Generalized Subsoil Conditions

The stratigraphy encountered in the boreholes put down during the present investigation is basically similar to that encountered during previous investigations on the OHC site. In summary, a stratum of deltaic or alluvial sand overlies an extensive deposit of sensitive silty clay. The upper portion of this silty clay deposit has either been reworked (and combined with sand or silt layers) or desiccated to form a relatively stiff crust of varying thickness.*

Based on the results of the borings put down in the general area, Figure 3 has been compiled to show the approximate elevation of the surface of the silty clay deposit, including the weathered crustal zone. It is noted that post depositional erosion, probably by a distributary of the Ottawa delta, has formed a shallow depression running parallel to Hwy. 417 in the north east sector of the NCC lands. On either side, and particularly to the south of this depression within the silt boundary, there is a considerable build up of surficial sands (Figure 4). It is noteworthy that the general direction of

*Although these surficial cohesive deposits are, in the geological sense, probably different facies of the same geological unit, their general character and engineering properties are basically similar. Therefore, in this report, they have been treated as the same material and are referred to as "desiccated silty clay crust".

the sand "spits" is similar to the direction of flow in the Ottawa River; this is typical of deltaic formations. From the interpretation of the data presented in Figures 3 and 4, it appears that in the area south of Hwy. 417 the fall-off in elevation of the surface of the silty clay deposit towards Hwy. 417 is compensated by an increase in the thickness of sand cap. The result is that, within this area, the ground surface is approximately constant between about elevations 260 and 270.

However, from the engineering standpoint, the thickness of the surficial sand and/or desiccated silty clay crust across the site is of considerable importance in the preliminary planning of development within the area (Figure 5).

Within two local areas south of Hwy. 417 which overlap both the NCC and OHC lands, the combined sand cap/desiccated crust thickness is between about 8 and 10 feet. It should be noted that in previous interpretations of data available from borings on the OHC site only, the two areas noted above were conservatively considered to be underlain by less than 8 feet of sand cap and/or desiccated crust. The minor revisions shown on Figure 5 are based on more available information which gives credence to results of borings on the OHC site previously considered to be anomalous. The remainder of the area of the combined NCC and OHC lands to the south of Hwy. 417 is in the main part underlain by a combined sand cap/desiccated crust thickness of less than about 8 feet.

In a relatively narrow strip flanking Hwy. 417, the thickness of sand cap/desiccated crust increases to between about 10 and 16 feet. It is obvious that the highway route has been selected to take advantage of this relatively thick sand cap/desiccated crust ridge, since in the area north of the ridge, the sand cap/desiccated crust thickness decreases generally,lying between about 8 and 10 feet. In local areas, however, such as the Bear Brook flood plain south of Carlsbad Springs and the bluff overlooking Mer Bleue, the sand cap/desiccated crust thickness lies in the ranges of 4 to 8 feet and 12 to $>$16 feet, respectively.

The thickness of the Leda clay deposit varies considerably across the general area from a minimum of about 35 feet in the south east of the OHC site to a maximum of greater than 200 feet in the north west corner (Figure 6). Within the NCC lands, there appears to be a local rise in the till or bedrock surface in the south west sector. This observation, which is based on the indirect results of a geophysical (seismic) survey, confirms previous interpretation of available data in the adjacent area on the OHC site. Over a major portion of the north east sector of the NCC lands, the inferred surface of the till or bedrock is generally at a depth slightly in excess of 100 feet below ground surface. Within the combined OHC and NCC lands, there appears to be a general trend to decreasing till or bedrock surface elevation in a north westerly direction, combined with a "bowl-shaped" glacial topography.

Although none of the borings put down during the present investigation fully penetrated through the Leda clay deposit, it is known that the underlying glacial till varies in composition from sand and gravel to silty sandy clay. The till is generally dense to very dense or hard, of variable thickness and in the Carlsbad Springs area is known to contain pockets of gas. Shales of the Queenston and Carlsbad formations directly underlie the till and with the exception of a thin upper fractured zone, appear to be in fairly sound condition.

5.2 Groundwater Conditions

Readings taken in the piezometers and standpipes installed in the shallow borings during the present investigation indicate that the groundwater levels in the NCC lands range from about ground surface to about 7 feet below ground surface. Although there is widespread scatter in the results, there is some evidence to suggest that the depth to water level across the site decreases with fall-off in ground surface elevation.

Based on the readings taken in deep piezometers installed during previous investigations and studies in the area, it appears that only minor downward seepage is taking place through the Leda clay deposit (see Section 8.3).

6. ENGINEERING PROPERTIES OF THE MAJOR OVERBURDEN STRATA

6.1 Surficial Sands

Where present, the surficial sands generally consist of grey or red brown stratified sand to sandy silt. Gradation curves for representative samples fall within the range determined in previous investigations at the OHC site (Figure 7). The water content of the surficial sands lies in the range between about 9 and 48 percent, with an average value of about 25 percent. Based on the results of standard penetration tests, which gave 'N' values ranging between 3 and 36 blows per foot with an average value of about 14 blows per foot, the surficial sands are typically in a loose to compact state of packing.

6.2 Leda Clay

In general, the Leda clay deposit consists of a sensitive, lightly cemented, silty clay. However the upper zone of the deposit has been weathered or desiccated to a relatively stiff consistency during periods in the geological past when the groundwater level was below the surface of the deposit. It should be noted that the transition of material type from desiccated crust to unweathered parent material is gradual and therefore, changes in soil properties with depth through the desiccated zone are not abrupt. However, for the purposes of presentation of data and delineation of regions of varying sand cap/desiccated crust thickness for engineering analysis purposes, a demarcation line between the "weathered" and "unweathered" zones of the deposit has been shown on the Record of Borehole Sheets. The criteria on which the boundaries were decided, are based on the variations in relevant soil characteristics and properties (i.e. colour, water content, density and consistency). The engineering properties of the weathered and unweathered deposits, together with more descriptions of the respective constituent soils, are described below.

6.2.1 Desiccated Silty Clay Crust

The predominant soil type in the desiccated crustal zone is fissured red brown silty clay (see Figure 8 for representative grain size distribution curves). However, this material contains layers of clayey silt, silt and sand (see Figure 9). The variable nature of the desiccated crustal soils was described in detail in a previous investigation at the site (see Golder Associates report No. 73908, Vol. III, dated May, 1974). These earlier results have been broadly confirmed by the tests carried out during the present investigation.

The average water content of the cohesive soil layers in the crustal zone is about 42 percent. The results of Atterberg limit determinations indicate that the average liquid limit of these materials, is about 60 percent (range 27 to 90 percent), while the plastic limit ranges between about 15 and 26 percent with an average of about 21 percent. The Atterberg limit test results are summarized on Figure 12. Based on numerous bulk density determinations, the average in situ bulk density of the desiccated crustal soils appears to be about 105 pounds per cubic foot.

The undrained shear strength of the desiccated crustal soils obtained by field vane tests and unconsolidated undrained triaxial tests lies in the range between 480 to greater than 2,000 pounds per square foot, with an average value of about 1,000 pounds per square foot. These values agree well with values measured in the previous investigation on the OHC site (see Golder Associates report No. 73908, Vol. III, dated May, 1974). It has been assumed that, as found previously, the undrained shear strength of the crustal soils does not vary significantly with direction of shearing.

Consolidation tests were carried out on a sample of highly weathered silty clay (borehole 226) and on two other samples (boreholes 229 and 231) which are considered to be only slightly weathered. The results of these tests are given together with all consolidation results on Figures 13 to 26 and indicate that the desiccated soils are over-consolidated by between about 1,300 and 2,000 pounds per square foot. These values lie close to the lower limit of values measured in the detailed laboratory testing associated with the previous investigation on the OHC site (Golder Associates report No. 73908, Vol. III, dated May, 1974). Below the preconsolidation pressure, p_c, the coefficient of compressibility, C_{cr}, is small, (i.e. less than 0.05). However, at stresses in excess of p_c, there is an abrupt change in strain response to loading with values of the coefficient of compressibility in this stress range lying between 1.14 and 1.5.

6.2.2 Unweathered Silty Clay

The predominant soil type in the unweathered Leda clay deposit is a lightly cemented, highly sensitive, silty clay. At shallow depths in many locations, the deposit is highly variable and contains red grey clayey silt to silty clay, grey silt and grey silty sand layers up to 6 inches thick (see, for example, the test results shown on Record of Borehole Sheet 203). Typical gradation curves of the unweathered silty clay and the coarser layers within this deposit are shown on Figures 10 and 11, respectively. With increasing depth, the deposit becomes more uniform but throughout contains pockets or layers of black organic clay.

The unweathered silty clay is highly plastic with average liquid and plastic limits of about 70 and 24, respectively (Figure 12). The water content of the clayey layers is typically close to or in excess of the liquid limit, (i.e. liquidity index of about or greater than 1) which explains the "liquid-like" character of the soil when sufficiently disturbed or remoulded. Based on numerous bulk density measurements, the average in situ bulk density of the unweathered silty clay appears to be about 98 pounds per cubic foot.

The values of undrained shear strength within the NCC lands, as measured by field vane and unconsolidated undrained triaxial tests, ranges between 200 and 1,200 pounds per square foot. Within many individual boreholes, the undrained shear strength was found to increase with depth from a minimum value generally found immediately below the sand cap or desiccated crust. The trend is common in post-glacial, normally or lightly over-consolidated clayey deposits and can be described by the ratio of undrained shear strength to effective overburden pressure, S_u/p_o', which in the present investigation was found to average about 0.3.

However, the measured undrained strengths in the borings put down in the north east sector (i.e. north of Hwy. 417) are significantly higher than the values measured at corresponding depths below ground surface in the lands to the south of Hwy. 417. If the measured strength data is plotted against elevation (Figure 27), it becomes apparent that, on a regional basis, there is an overall increase in undrained shear strength with decreasing elevation. Therefore, where the ground surface elevation is relatively low, as is the case north of Hwy. 417, the measured undrained shear strength of the unweathered silty clay at shallow depths below ground surface is relatively high in comparision to shallow depth south of Hwy. 417.

It is emphasized that the above correlation denotes a general trend only. As noted previously the results of consolidation tests carried out during the present investigation (see Figures 13 to 26) and other geotechnical investigations at or close to the site suggest that the deposit may be banded (i.e. there are zones of nearly normally consolidated soil and over-consolidated soil at various depths within the deposit). The scatter in the overall plot of consolidation test results, which is given on Figure 28, tends to mask this banding effect. However, the data presented by Bozozuk and Leonards, 1972, obtained from research-quality laboratory tests carried out in conjunction with a test fill at CFS Gloucester close to the south west limit of the site, indicates that the upper surfaces of local over-consolidated zones may occur at about elevations 245 and 225 (see Figure 28).

It is noteworthy that in the area south of Hwy. 417 the surface elevation of the silty clay deposit is generally higher than elevation 250 (Figure 3). Consequently, the soil immediately underlying the sand cap/desiccated crust is probably in a near normally consolidated zone of the deposit. Hence, this material would be expected to exhibit relatively low strength and high compressibility characteristics. On the other hand, in much of the area north of Hwy. 417, the upper surface of silty clay deposit is at about elevations 225 or 245. It is considered that a combination of the two factors (i.e. general increase in strength with decreasing elevation and coincidence of ground surface elevations with local overconsolidated zones within the silty clay deposit) has resulted in higher undrained shear strengths and p_c values being measured in the area north of Hwy. 417; with the main factor being greater regional erosion in the geological past below previous Lake Champlain lake bottom level north of Hwy. 417.

The strength and compressibility data obtained in the study area during present and previous investigations has been compared with published results of other studies of the engineering characteristics of Leda clay in the Ottawa area (Figure 29). This comparison indicates that the data from the South East City area agrees reasonably well with the published data, and confirms the trend of test results obtained during the present investigation.

It should be noted that because of difficulty in gaining access to suitable areas for putting down test pits, no block samples were recovered during the present investigation. However, as reported previously (Golder Associates report No. 73908, Vol. III, dated May, 1974) the "disturbance" effect of sampling tubes can be considerable resulting in lower measured values of undrained shear strength and preconsolidation pressure. Therefore, although Osterberg piston sampling techniques have been used in the present study* it is likely that the measured values of soil parameters are lower than the corresponding field values. Analyses based on these results will therefore be conservative.

Finally, it should be noted that no attempt has been made to define the "cementation" or Coulomb-Mohr failure envelopes for this material. This work has already been carried out as part of our earlier investigation and is summarized in Golder Associates report No. 73908, Vol. III, dated May, 1974.

*Research has shown that in cemented Leda clay, less sample disturbance is associated with the Osterberg piston sampler than with any other form of tube sampling techniques, (Raymond, Townsend and Lojkasek, 1971).

SECTION B

ENGINEERING RECOMMENDATIONS

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7. FOUNDATIONS FOR STRUCTURES

7.1 Summary of Significant Subsurface Conditions

This section summarizes the various aspects related to the subsurface conditions which are significant to the design of foundations for structures associated with development of the general area. (A full description of the subsurface conditions across the combined sites is given in the preceding sections).

The area under consideration is generally flat-lying and is underlain by a deposit of highly sensitive silty clay, known as Leda clay. In some areas the upper portion of this deposit has been desiccated to a relatively stiff consistency. The Leda clay deposit is overlain by an intermittent alluvial sand layer. The combined thickness of silty sand cap and desiccated crust across the area varies from about 4 to greater than 16 feet, as shown on Figure 5. The unweathered Leda clay deposit ranges in thickness between about 35 feet to greater than 200 feet thick (Figure 6) and is underlain by a glacial till blanket which directly overlies shale bedrock of the Queenston and Carlsbad formations.

The groundwater level across most of the site is at or close to the ground surface although it is locally depressed in areas adjacent to the main watercourse in the area (i.e. Bear Brook), and its tributaries. Groundwater level fluctuations have occurred in the past, as evidenced by the desiccated crust.

7.2 Factors Affecting The Design of Shallow Foundations

In this section the factors affecting the design of shallow* foundations are discussed in relation to the present development. Values of the significant soil properties used in preliminary foundation design are also selected.

The major factors affecting the selection of the allowable bearing pressure for a conventional, isolated, shallow footing located in the relatively stiff crust of a "soft" clay deposit can be summarized as follows:

i) Soil Properties:
   - undrained shear strength of the crustal soils
   - undrained shear strength of the unweathered clay immediately below the crust
   - preconsolidation pressure immediately below crust (i.e. the pressure above which further stressing of the soil will, in time, cause excessive settlement)

ii) Groundwater:
   - changes in groundwater table (due to seasonal variation and effect of development)

iii) Footing Geometry:
   - size and shape of footing
   - founding elevation and crust thickness

iv) Structure type:

   - nature and extent of regrading or excavation close to the footing

*In soil mechanics terminology shallow foundations are defined as those footings constructed at a depth less than the width of the footing. However, for frost protection purposes, footings in the proposed development will have to be at a depth of at least 5 feet below final grade and therefore, depending on their width, may not be "shallow". Nevertheless, for preliminary design purposes all spread and strip footings have been analysed using "shallow" footing criteria. It is considered that the error thus involved is small and conservative.

The effect of each of the above factors is shown diagrammatically on Figure 32 and is discussed separately below.

i) In the previous investigation at the OHC site, the parameters selected for preliminary foundation design were as follows:

- undrained shear strength of crustal soil* = 800 lb/sq. ft.
- undrained shear strength of unweathered clay in zone immediately below crust = 300 lb/sq. ft.
- p_c-p_o' of unweathered clay in zone immediately below crust - narrow footings = 500 lb/sq. ft.
  - wide footings = 400 lb/sq. ft.

The results of tests carried out during the present investigation on samples recovered from the south west sector of the NCC lands fall within the ranges of values of soil properties measured in the OHC investigation (see Figures 30 and 31). Therefore, the soil parameters given above are applicable to the design of foundations in the south west sector of the NCC lands as well as in the OHC site (i.e. the entire area of the combined sites south of Hwy. 417).

Because of the banded nature of the Leda clay deposit and the variation in ground surface elevation in this area, the strength and compressibility characteristics of the uppermost unweathered soils underlying the area north of Hwy. 417 differ considerably from those measured in the same zone across the remainder of the site, (see Figures 30 and 31). In general, the soil in the critical zone under the crust north of Hwy. 417 is stronger and can tolerate higher stresses without excessive strain than the equivalent soil south of Hwy. 417. The following are the soil properties

*To simplify analyses, shallow (i.e. 8 ft. thick) deposits of the surficial sands were conservatively assumed to be as competent in terms of foundation design as the desiccated clay crustal soils.

selected for preliminary foundation design purposes in the area north of Hwy. 417:

- undrained shear strength of crustal soil = 800 lb/sq. ft.
- undrained shear strength of unweathered clay in zone immediately below crust = 400 lb/sq. ft.
- p_c-p_o' (all footings) = 600 lb/sq. ft.

It should be noted that as shown on Figures 30 and 31, the values of soil parameters chosen for design are somewhat conservative (i.e. they fall close to the minimum measured value of the relevant parameter as opposed to arithmetic average). It is considered that conservative selection of soil parameters for design is necessary for a preliminary investigation of this nature because of the limited data available. Also, it is considered that as well as providing design guidelines for preliminary planning, a supplementary but no less important purpose of the present series of investigations is to establish the feasibility of the proposed development from the geotechnical standpoint. The decision as to the feasibility of the project must be based on an interpretation of the facts which is beyond reasonable doubt. Therefore, as in other aspects of the present work, the selection of soil parameters is purposely conservative.

ii) If local groundwater levels fall due to the effect of development (i.e. installation of services and the like), the effective overburden pressure, p_o', on the foundation soil is increased. Since the preconsolidation pressure is effectively constant, the magnitude of the parameter p_c-p_o' is reduced and therefore the allowable bearing capacity of the foundation subsoil (to limit consolidation settlement to tolerable limits) is also decreased (see Figure 32). For the purpose of analyses carried out during this investigation, the groundwater level has been assumed to be above founding elevation.

iii) In shallow foundation design in homogeneous soils, an increase in footing size generally results in an increase in allowable load/unit length of footing. However, in the case of a footing located in a stiff upper crust of a "soft" clay deposit, this may not be the case. As shown on Figure 32, the wider a foundation at any given elevation the deeper is the zone of influence of the increased stress on the unweathered soil. Hence this factor which controls settlement must be taken into account in determination of allowable bearing pressures for shallow footings.

Similarly, the closer the footing is to the underside of the crust, the larger is the applied stress on the soil in the critical zone and the deeper the "soft" clay is stressed (see Figure 32). Again, this factor must be taken into account during design.

iv) The actual footing configuration will depend on the type of structure to be supported. For example, the structure may include a basement which has the beneficial effect of "off-loading" the foundation soil. On the other hand, fill placed along the outside wall of a structure supported on shallow strip footings (for frost protection purposes), will cause additional stress on the foundation subsoil. These factors are shown on Figure 32 and have been taken into account in our analyses.

From the above fundamental discussion, it is clear that there is a wide range of variables which affects the allowable bearing pressure of shallow foundations. For clarity of presentation, and because more detailed analyses are not warranted at this stage, only three footing configurations have been chosen for detailed analyses (see Figures 33 to 35). The number of combinations is further reduced since within each of the regions north and south of Hwy. 417, the soil parameters are considered constant. Therefore the results of analyses have been presented in terms of allowable bearing loads/lineal foot of footing for various combinations of crust thickness and footing width. The loading characteristics for the various types of structures can then be compared with these charts to give an indication of the suitability of a given area for different types of development. It is noted that because of the necessary generalizations which have been made with respect to soil parameters, thickness of crust and the like, the results of analyses should be used for preliminary design purposes only. Individual commercial or industrial developments should be planned in detail only after a detailed geotechnical investigation of the proposed site is carried out.

7.3 Residential Housing

Numerous single family detached houses exist on the site at present and range from older masonry structures to more modern bungalows. A preliminary visual inspection of some of these buildings did not reveal any cases of significant structural distress due to settlement. It is noted that a common feature of a large majority of houses at the site is that the lots are regraded to permit adequate drainage to take place away from the buildings. With the final grade around the buildings being at a higher elevation than the surrounding land, the footing elevations are higher with respect to the underside of the crustal zone, a factor which minimizes settlements due to consolidation.

In determining the allowable bearing pressures for shallow foundations for detached houses, the buildings were assumed to incorporate a basement and to have a founding elevation 2-1/2 feet below original grade. Although the actual loading will vary with the type of building construction, it is understood that the loads on the footings will be of the order of 1.2 tons per lineal foot of footing. The results of analyses shown on Figure 33 indicate that except in marshy areas these loads can be supported by strip footings everywhere on the site. A maximum footing width of about 4 feet will be required in areas south of Hwy. 417 where the crust thickness is about 4 feet. In areas north of Hwy. 417, the maximum required footing width is about 3 feet.

Semi-detached or row townhouses, which are restricted to two storeys in height, have an approximate loading of between about 1.3 and 1.5 tons per lineal foot of footing. As shown on Figure 33, these structures can be successfully constructed in most areas of the site, although in areas south of Hwy. 417 where the crust thickness is minimal (about 4 feet) relatively wide (i.e. 5 feet) footings may be required. It is considered that footing widths of this magnitude reflect the conservative nature of the design soil parameters.

The column loads associated with single, semi-detached and two storey townhouses are generally less than 10 tons. This load can be supported on spread foundations over most of the site.

In some areas it may be desirable to construct "low rise" (i.e. 3-4 storeys) apartment buildings. Provided these structures incorporate basements, they can be supported on conventional strip footings about 4 feet wide in areas where the crust thickness is greater than 10 feet, south of Hwy. 417. North of Hwy. 417 where the crust thickness is greater than 8 feet the required footing width is about 5 feet. Without a basement, these structures cannot be supported on conventional strip footings south of Hwy. 417 except in areas where the crust thickness is at least 13 feet (see Figure 34). North of Hwy. 417, this minimum value decreases to about 10 feet.

However, "low rise" apartment buildings can be constructed everywhere on the site if the basement walls and floor slabs are designed and constructed as a structural unit. The founding level is then chosen so that the weight of the structure is at least partially compensated by removal of the soil above the founding level (i.e. floating raft principle). The following table is included as a guide to the required depth of excavation and should be used for preliminary design purposes only:

Number of Storeys	Approximate Founding Level Ft. (below original grade)	
	South of Hwy. 417	North of Hwy. 417
3	3	1
4	5	3
5	7	5

In residential areas it will be necessary to carry out some regrading both within individual buildings lots and within larger townhouse developments. The regrading of individual building lots for detached or semi-detached houses is necessary for drainage purposes. However, as stated earlier, the actual height of fill adjacent to the outside of single and multi-family residential buildings has a significant effect on the allowable bearing pressure for strip footings. For illustration purposes, the effect of varying fill height on allowable bearing pressure is shown on Figure 36 for a given footing geometry and a specific set of soil conditions. It should be noted that the optimun height of fill to achieve maximum allowable bearing pressure is about 2 to 3 feet for normal footing widths (i.e. below 4 feet). The design value chosen, as illustrated on Figure 36, is 2.5 feet.

For aesthetic reasons, it is often the case that substantial regrading of the sites of townhouse developments is carried out and footings are located at varying elevations in an effort to introduce changes in elevations of groups of buildings. It is noted that for reasons described above, earthfill landscaping immediately adjacent to structures should be limited to a height consistent with ensuring the optimum bearing capacity for strip footings. In some areas, embankments less than about 3 to 4 feet high along the front and back outside walls of row townhouses may cause differential settlement across transverse strip footings(i.e. between the front and back of the building). Depending on the subsoil conditions at the individual sites, it may be necessary to incorporate "flexible" construction joints in these footings to prevent cracking or structural failure of the transverse footings. Further, variations in footing elevations may not be possible in some areas of the site where the crust is relatively thin.

7.4 Light Industrial, Commercial and Institutional Buildings

The type of buildings being considered in this section are single storey light industrial or commercial structures, schools and community centres with loadings on external strip footings of about 1.5 tons per lineal foot. The column loads for light industrial structures are expected to be about 30 tons for single storey structures with a column spacing of about 20 feet. In commercial structures it is anticipated that, for the normal type of construction, column loads will be of the order of 25 tons for a column spacing at 30 foot centres. The loadings associated with community facilities such as schools, churches and community centres are not easily categorized because of the considerable variation in design of the individual structures. However, it is anticipated that strip footing loadings will be of the same order of magnitude as in the case of light industrial or commercial structures.

In normal use, these structures do not require a basement and as a result the floor slab is usually constructed at or above original ground surface elevation. Consequently, only minor regrading beneath and around the walls of the structure will be necessary and as a result conventional exterior strip footings will be at a depth of about 4 feet below original grade.

South of Hwy. 417, these structures can be supported by conventional strip footings when the crust thickness is greater than about 8 to 10 feet (see Figure 34). Spread footings for column loads will be of the order of about 7 feet square. It should be noted that because of minor revisions to the "crust thickness" plan (Figure 5) there are two areas close to the south west boundary of the site which have been redesignated as being overlain by a crust of thickness ranging between 8 and 12 feet. Therefore, it appears that these two areas can be developed as light industrial or commercial areas together with the strip of land adjacent to and immediately south of Hwy. 417.

North of Hwy. 417 where the soil parameters which determine the allowable bearing capacity of the soil are significantly higher, light industrial and commercial structures can be supported by shallow foundations in areas where the crust thickness is greater than about 6 feet. The maximum width of strip footings would be of the order of 5 feet while the maximum size of spread footing is of the order of 7 to 8 feet square.

In preliminary planning, it will probably be acceptable that major commercial and industrial centers are concentrated in areas of relatively thick crust (i.e. adjacent to Hwy. 417) However, it will be necessary to provide schools within residential areas and therefore in some cases these buildings will be located in areas having a relatively thin crust. One or two storey schools will probably impose a maximum loading of about 1.5 tons per lineal foot of footing. These loads can be accommodated on strip footings in most areas of the site. It should, however, be noted that in areas south of Hwy. 417 where the crust is 4 to 6 feet thick, these footings may have to be of the order of 8 feet wide. The column loads associated with gymnasiums for schools will depend on the type of construction and the size of the structure. Obviously, larger high school gymnasiums will impose higher column loads (probably of the order of about 30 tons), than equivalent loads in the junior schools. Column loads in the order of 30 tons can be accommodated in most areas north of Hwy. 417 and in areas south of Hwy. 417 where the crust thickness is greater than 8 to 10 feet. Since only a limited number of these high school facilities will be required south of Hwy. 417 they should be located in areas underlain by sufficiently thick crustal deposits to take advantage of conventional foundation techniques.

Nevertheless, numerous primary schools will be required and it is inevitable that some of these will be located in areas where the crust thickness is between 4 and 6 feet. From Figure 35 it appears that north of Hwy. 417, there should be no major problems in supporting 20 to 30 ton column loads in areas where the crust thickness is as low as between 4 and 6 feet. In this area the location of schools is not a constraint to planning. However, south of Hwy. 417, the maximum allowable column loads in areas where the crust thickness is between 4 and 6 feet appear to be restricted to about 15 to 20 tons. This therefore will have to be taken into account during final design of these structures.

Where this restriction cannot be accommodated, it may be necessary to preload sites to loads in excess of those resulting from final structural loadings. This technique has been discussed previously (see Golder Associates report no. 73908, Vol. IV, dated June, 1974) and the restriction regarding the proximity of structures to preloads are applicable to the present investigation. One of the major practical considerations with the use of the preload technique is the time requirement for surcharging (of the order of one to two years depending on the flexibility of the structures).

However, it should also be noted that within any given residential area, which requires provision of a junior school, there will be local areas in which the crust thickness is greater than "average for the area". Thus, it is definitely possible that within an area at present designated as being underlain by a crust 4 to 6 feet thick, for example, there are local areas in which the crust thickness could be 8 feet or greater. On the basis of available data, it is not possible to define these areas. However, during subsequent detailed investigation within specific proposed subdivisions, a more detailed investigation of the nature of soil conditions within that area will be carried out and structures such as schools and other community facilities should be strategically located to make best use of the range of crust thickness within the area under consideration.

7.5 Heavy Institutional Buildings and High Rise Structures

The types of structures being considered in this section are those greater than about 3 to 4 storeys in height such as high rise apartment buildings or institutional buildings which impose relatively high loadings. It is clear that, except possibly for some local areas close to Hwy. 417 and on the bluff adjacent to Mer Bleue, these structures cannot be supported on shallow foundations at the sites under consideration. They can, however, be economically supported on end bearing piles in areas of the site where bedrock or dense glacial till is at a relatively shallow depth below ground surface. For example, in the south east corner of the site, close to the Town of Edwardson, the till or bedrock surface is as close to the ground surface as about 30 feet. Also in a relatively widespread area close to the south west boundary of the site, the till or bedrock surface is at a depth ranging between about 60 and 90 feet below the ground surface. In the area north of Hwy. 417, the till or bedrock surface is generally about 100 to 125 feet below ground surface. For the purpose of the present investigation this latter range of required pile length has been assumed to be somewhat excessive and as a consequence, this area has thereforce been regarded as not ideal for the widespread use of piled foundations.

In general, the comments related to piled foundations contained in a previous report (Golder Associates report no. 73908, Vol. IV, dated June, 1974), are applicable to the present investigation. However, it is re-emphasized that the use of friction piles to support structural loads is not generally recommended. This is due to the fact that the deposit is banded(i.e. there are zones within the deposit which are nearly normally consolidated) and therefore susceptible to consolidation settlements under small additional loads. The action of a group of friction piles is such that a large part of the total imposed load is transmitted through the base of the pile group. Depending on the relative location of the near normally consolidated zone, considerable settlement of the pile group could occur under the action of loads applied through the pile group.

The alternative to piled foundations for heavy structures is to use wholly or partially "compensated" foundations. Again, recommendations made regarding this foundation treatment have been given in our previous report (number 73908, Vol. IV, dated, June, 1974).

7.6 Summary

Within the framework of design criteria, conservative soil parameters and generalized structural loading characteristics selected as a basis for the analyses carried out during the present investigation, it is considered that all of the structures required for the proposed development can be successfully constructed within the general area defined by the OHC and NCC sites using conventional foundation methods. Since the foundation soil in some areas can accommodate heavier loads than in other areas, there is obvious economic advantage in taking this geotechnical fact into account during preliminary planning.

The "geotechnical constraints" regarding the development of the site for building use have been taken into account in the compilation of the summary in Table I and Figure 37. It should be noted from Figure 37 that because of the more favourable soil conditions north of Hwy. 417, the only type of structure which cannot be supported on conventional shallow foundations in the majority of the north eastern sector are relatively heavy institutional buildings and high rise structures. However, these can be accommodated in the south western half of the site in the two areas where the till or bedrock surface is relatively close to ground surface and where piled foundations are therefore economical. Except for an area parallel to and immediately south of Hwy. 417 and other localized areas, much of the south western half of the area is suitable for use as residential developments only. Although it is considered that junior schools can be accommodated within these residential areas, it is suggested that sub-community facilities such as churches, small commercial centres and high schools are located in the two local areas of higher crust thickness.

It is again noted that while the structural loading characteristics used in analyses are considered realistic, actual loadings of specific structures such as schools and the like will probably vary and may be lower than those used in this report. Further, as stated earlier, the soil parameters used in analyses are considered to be conservative and therefore depending on actual structural loading details and following detailed investigations at individual sites, the use of higher values of allowable bearing pressures than have been given in this report may be possible.

Finally, it should be noted that the recommendations regarding allowable bearing pressures given in the preceding sections are based on conventional foundation methods. However, a significant increase in allowable bearing pressure may be gained by using more innovative methods such as styrofoam insulation for frost protection. This method allows the footing to be placed at a relatively high elevation in the crust, without the disadvantage of having to observe the "5 foot frost protection requirement". For example, analyses indicate that a footing suitably insulated using styrofoam sheeting, located at 2 feet below original grade in an area of 5 foot thick crust, can carry 1.4 tons per lineal foot of footing on a 3 foot wide footing. This allowable bearing pressure compares favourably with a conventional footing of the same width (see Figure 33). It should be noted that such foundation practices have been successfully applied in various projects in Northern Ontario where the climate is considerably more severe that in Ottawa. As there will be variations in the actual costs of the alternative foundation methods which will depend on soil conditions at specific sites and on relative costs of various construction materials and labour at the time of construction, the use of insulated foundations should not be disregarded at this time for economic reasons.

8. SITE DEVELOPMENT

There are a number of aspects related to site development, other than foundation design, which must be considered. These factors include the construction of roads and installation of services. Both of these activities involve earth excavation and utilization of the native soils. Also, it will be necessary to provide adequate site drainage which in turn requires that consideration be given to the possibility of significant regional and local groundwater lowering and possible increased run-off into Bear Brook.

The factors related to site development have been discussed in our previous report (number 73908, Vol. IV, dated June, 1974). The following sections of this report reiterate the more salient of these earlier recommendations and outline regional differences in the factors affecting development within the combined OHC and NCC sites. Where relevant, suggestions are made as to the methods of obtaining the data necessary for final planning and design of roads, services and the like (see also Appendix III of this report).

Before discussing the various individual aspects of site development it should be noted that the basic geotechnical behaviour of the native soils affect a number of the different aspects of development. For discussion purposes, there are three major soil types at the site, namely, the surficial sands, the desiccated clayey crust and the softer unweathered clay. It should be noted that the surficial sands are suitable for general landscaping and should present no major problem during the course of shallow earthworks. Although the desiccated crust has a relatively high natural water content, it is considered that it is also suitable for general regrading purposes. Where it is necessary to use the desiccated crust or surficial sands as backfill in areas where only small settlements can be tolerated, the selection and compaction of this material should be carefully controlled. The unweathered clay at depth is unsuitable for general use as fill and will be difficult to handle during excavation because of its high natural water content and sensitive nature. This material can only be used as fill where settlement is not a critical governing criterion. Because of the lack of "quality" granular material on the site, emphasis should be placed on keeping the extent of earthworks requiring backfill to a minimum.

However, because of the flat nature of the site some regrading will be carried out during the course of development. For reasons discussed later in this report, regrading and earthworks will be primarily to a shallow depth within the desiccated crust or sand cap. Due to the soil conditions at the site, deep excavations associated with cuts or deep basement foundations, or earthworks associated with large fills should be kept to a minimum.

8.1 Roads

Because of the flat nature of the site, it will be necessary to carry out some regrading to accommodate the necessary minimum road gradients. The combination of decreasing supporting capacity and poorer trafficability with increasing depth of excavation for roadway cuts requires that all earthworks for roads be limited to the soil within the zone at least 2 feet above the underside of the crust. This restriction will necessitate "above grade" roadway construction in some areas of the site, particularly where the crust is thin.

The surficial sands and silty sand layers which are evident in the desiccated crust are highly susceptible to frost action (i.e. heaving). Since the groundwater level across most of the site is close to or at the ground surface it will therefore be necessary to ensure that good underdrainage of road pavements is provided. Clearly this drainage process will be considerably simpler to achieve if the roads are constructed above existing grade.

The predominant geotechnical factor in the design of road pavements, whether rigid or flexible, is related to the strength and variability of the subgrade material. Assuming that the backfill in service trenches located under roads is at least as capable of supporting traffic loads as the native soils at the site*, preliminary design of flexible pavements should be based on the criteria of equivalent pavement thickness of 30 inches and 24 inches of granular base and subbase for arterial and subdivision roads, respectively. It is suggested that a Benkleman Beam survey of the performance

*The use of various types of backfill materials in service trenches and the location of trenches is discussed fully in Section 8.2 of this report.

of the existing roads at the site is carried out and compared with the relevant road structure. This survey, which would in fact, constitute a large scale field test under operating conditions, should provide the necessary data on which the economical design of flexible pavement structures can be based. Local factors such as the presence of varying thickness of sand cap and the like could be studied and could lead to significant savings in the requirement for imported granular material.

Other methods for reducing the amount of granular required for road pavement construction have been discussed in our earlier report and include stabilization of the subgrade soils by treatment with lime and the use of concrete or deep strength asphalt pavements. The former of these techniques, (i.e. subgrade stabilization) is to be applied in the construction of new arterial roads in the Ottawa area. It is suggested that the performance of these pavement structures be monitored to provide guidelines regarding the use of this technique in the proposed development. It should be noted that the use of concrete or deep strength asphalt pavements will result in a lower total pavement thickness. Therefore, if used in areas of thin crust, these techniques will allow more flexibility in accommodating minimum road gradient requirements. A further saving on imported granular, the cost of which may escalate significantly in the period prior to development, is to install synthetic filter membranes in road bases instead of conventional granular filters. Recent experience with this type of material has been successful in overcoming similar problems in other areas of Southern Ontario.

As stated previously the underside of pavement structures should not be lower than about 2 feet above the underside of the crust. To accommodate this restriction, grade separations for roads should be elevated. However, embankments for overpass structures should not exceed 10 to 15 feet high and should be constructed as far in advance of actual construction as possible. In this respect, it is noteworthy that preload techniques have proven useful in minimizing settlements under an approach embankment for a bridge structure located close to the site of the proposed development and in an area in which the thickness of the Leda clay was of the order of 150 feet (Devata and Darch, 1972). Embankments were initially constructed to a height of about 14 feet and subsequently reduced to the design height of about 9 feet after one year. During this surcharge period the settlement under the centre of the embankment was about 8 inches. In the two-year period following surcharge removal, the settlement under the centre of the embankment was about 2 inches.

In some areas of the site, roads will be constructed on saturated sands. Although this type of material can be subject to compaction by vibration, it is unlikely that, given the traffic loads and probable distance of most structures from roads, any distress to structures will be caused by traffic vibrations during the normal course of development.

8.2 Services

There are two major geotechnical factors which affect the installation of municipal services for the proposed development:

i) excavation and construction methods;
ii) backfill materials.

The actual construction method will depend on cost comparison and is basically a question of the depth at which tunnelling becomes economically more viable than "cut and cover" methods. However, because of the relative soft consistency of the silty clay at the trench invert level, there is a limitation as to the maximum depth of open cut without risk of basal heave/instability. As shown on Figure 38, the analyses of basal stability of trenches which was confirmed by test pits and a test trench in areas south of Hwy. 417 indicate a critical depth of between 15 and 20 feet. However, because of the overall higher strength of the silty clay at trench invert level north of Hwy. 417, the maximum depth of trench excavation is between about 20 and 25 feet (see Figure 38). Based on recent experience in soft ground tunnelling, which is relatively common in the Ottawa area, it is considered likely that tunnelling for major services will become economically viable at invert levels in excess of about 25 to 30 feet. Therefore, within the area north of Hwy. 417, the transition from cut and cover to tunnel sections appears to be economically suitable.

In the area south of Hwy. 417, excavation in the depth zone from about 20 to 25 feet may prove more expensive than normal, in that trenches may require greater bracing support and sheeting capacity below invert level. However, there is a major aspect which should be taken into account when assessing the impact of this factor in preliminary design in that the soil shear strength of 300 pounds per square foot which was used in analyses (Figure 30) is conservative. Therefore, the results of further detailed site investigation for a given section of sewer may indicate that the use of a higher design value is applicable and hence that the cut can be made safely to depths greater than 15 to 20 feet in the areas south of Hwy. 417.

However, it is anticipated that a large percentage of the total length of sewer trench excavation will be at relatively shallow depth (i.e. 15 feet). In this case the trench could be excavated using sloping sides no steeper than 1 horizontal to 1 vertical. The contractor may elect to sheet and brace the excavation (as opposed to open cut with sloping sides), since easy driving conditions will allow continued re-use of sheet piling. Further, if interlocking sheet piling is used in areas of relatively thick sand cap, pumping of inflowing groundwater from sumps located within the floor of the trench may be sufficient and would probably represent a considerable saving on other forms of groundwater control required for "open cut" excavation.

It is noteworthy that a test sewer pipe was installed on a site within the OHC lands in January, 1975 (see Golder Associates report No. 741285, dated February, 1975). Although the excavation was made predominantly in the silty clay deposit, to depths ranging between 9 and 13 feet, numerous silty sand layers up to 1-1/2 feet thick were intercepted below the water table. Observations made during construction indicated that water seepage from these layers was minor and there was no detrimental effect on the stability of the trenches. The frequent occurrence and thickness of the coarse layers within the silty clay deposit underlying the test pipe site has not been observed in the majority of shallow borings put down during the course of the present and previous investigations in the general area. Therefore it is considered that except where excavations are predominantly in the surficial sands, and below the water table, groundwater should be readily handled by pumping from sumps located in the excavation floors. However, sand deposits below the water table along the route of the proposed sewers will probably require dewatering by a wellpoint system (see Figure 39) if sheeting is not employed.

The use of soft ground tunnelling techniques is common practice in the Ottawa area. It is considered that there should be no major geotechnical difficulties associated with tunnelling at the proposed site although the use of compressed air may be necessary in some areas of the site. Unfortunately documented case records of large diameter sewer tunnel construction in Leda clay in the Ottawa area are relatively limited. However Eden and Bozozuk (1969) describe the geotechnical aspects of tunnelling at a depth of about 65 feet in a deposit of highly sensitive Leda clay leading to the Green Creek Sewage Treatment Plant, which like the site under examination, is located in the easterly limits of the city. During tunnelling operations, which were carried out using a rotary tunnelling machine, surface settlements were minimal and the clay behaved as a stiff, brittle material.

Further useful information related to tunnelling operations in Leda clay may be gained from construction of the South Ottawa Collector Sewer which will encounter mixed face and soft ground conditions along its eastern section.

One of the major constraints on the maximum depth of storm sewer installation is the relatively high outlet level fixed by Bear Brook. This restriction will require the provision of a number of pumping stations across the site. As with trenches, pumping station excavations will be restricted in depth because of possible basal instability.

Based on the design strength parameters given earlier, it is considered that for open cut with sloping sides or conventional braced and sheeted excavations, the maximum depth of excavation in the areas south and north of Hwy. 417 are about 20 and 25 feet, respectively. It should be noted, however, that excavation in narrow strips or the provision of "toed-in" sheet piling, will allow the above values to be increased to some extent. Again, the actual design values may vary from those given above depending on the strength values determined during the detailed geotechnical investigation stage for the structure. It should be further noted that advantage may be gained by locating a pumping station(s) in the south east corner of the OHC site where the depth to competent till is known to be relatively shallow. Within this area, the appropriate method of construction would be to excavate from within braced sheeting which has been driven into the till. In this case, excavation can proceed to considerably greater depths than the maximum values given above.

The various methods of backfilling service trenches are discussed in detail in our previous report. As no major sources of quality granular material were exposed during the present investigations at the site, these earlier recommendations apply and have been summarized as follows:

i) The only suitable native backfill materials in the area are carefully selected portions of the sand cap and desiccated crust zones.

ii) If service trenches are to be located under roadways as is normal practice in Ontario, then the trenches must be backfilled with adequately compacted suitable native material or imported granular.

iii) If service trenches are relocated at the back of housing lots where subsequent settlement is not critical, the native excavated soils can be used as backfill.

Since making the above recommendations, the information gained during the present investigation has allowed more complete mapping of the surficial sands across the site (Figure 4). It is noted that since there will be restrictions as to the proximity of structures to Hwy. 417, it may be possible to strip suitable backfill material from undeveloped barren lands in this zone. The results of a Standard Proctor compaction test carried out on 'select' native backfill used in the recent test sewer pipe construction on the OHC site, are shown on Figure 40. It is clear that although the maximum dry unit weight of 106 pounds per cubic foot is relatively low, the material can be compacted and is suitable for use as a backfill material. Also shown on Figure 40 are the results of a standard Proctor test carried out on a sample of granular conforming to Gloucester Township specifications.

8.3 Site Drainage

There are two major aspects of site drainage which affect development:

i) changes in regional and local groundwater levels as they affect the performance of structures.

ii) changes in the groundwater recharge system by which the groundwater level in Mer Bleue is maintained.

In an area which is underlain by a near normally consolidated clay deposit (as is the case in question) significant groundwater lowering may cause regional settlements. Settlements of this nature are not necessarily detrimental to the performance of individual structures but should be taken into account during preliminary design. On the other hand, depending on variation in subsoil conditions and amount of drawdown experienced, settlements may be more localized and may affect the performance of sewer pipes and the like over relatively short distances.

The installation of granular backfilled services and the practice of pumping from sumps located in basements of homes would tend to lower "local" groundwater levels. On the other hand there will be some "artificial" recharge of the groundwater level due to such activities as lawn watering and the like. As part of our preliminary study into the geotechnical aspects of the development, (Golder Associates report 73908, Vol. IV, dated June, 1974) a literature review was carried out in an attempt to quantify the effect of development on the local and regional groundwater levels. However, no documented evidence on this "problem" is available from the usual sources except for cases where pumping was carried out from aquifers underlying developed or proposed development sites. This latter information is not applicable in the present study as it is understood that no deep pumping will be carried out during the proposed development.

Because of the lack of detailed information and reported case histories it is not possible to precisely assess quantitatively the effect of development on local and regional groundwater levels. On a qualitative basis, it is considered that the installation of services in even the most swampy sites causes drainage of the upper portions of the site subsoil. However, except where services are backfilled with granular materials and are located in areas underlain by predominantly cohesionless soils, the lowering of groundwater tables is probably modest. Therefore it is considered that development of the South East City site will not cause significant lowering of groundwater levels and that subsequent settlements of properly designed structures should not be a major problem. Clearly, a modest drawdown of the groundwater table particularly at some low-lying locations across the site, during peak run off periods when the groundwater table is at or close to the ground surface, is not only desirable but mandatory from the viewpoint of trafficability and to encourage the growth of vegetation in these areas.

Although it is recognized that a very detailed study of this aspect of development is required in order to properly assess the effect of the various hydrological parameters on local groundwater conditions, it is hoped that some valuable information can be gained from the results of the sewer monitoring programme which has been instigated at the site.

The second aspect of the effect of development on the groundwater regime at the site is related to the maintenance of the present groundwater in, and therefore the conservation of, Mer Bleue. In this respect it is important to establish the interrelationship between run off from the development site and its contribution to the recharge of the groundwater level in Mer Bleue. Such a recharge mechanism could be effected either by surface flow or by percolation of surface water into the granular till blanket which underlies the post-glacial overburden deposits at the site. On the basis of a preliminary examination of surface topography at the site, there does not appear to be a significant catchment area contributing to surface recharge of Mer Bleue. It should be noted that both the major watercourses draining the area (i.e. Bear Brook and Ramsay Creek) flow north towards the edge of Mer Bleue before abruptly changing direction and flowing east and west respectively along the southern edge of the marsh. In fact, tributaries to both Bear Brook and Ramsay Creek partially drain the fringes of the marshland.

Therefore, if surface water from the development site does not provide a surficial recharge source for the Mer Bleue, the only alternate route is through the till blanket underlying the Leda clay deposit. Based on the water level readings taken in piezometers installed at various elevations within the overburden deposits, it appears that some downward seepage of groundwater is taking place. However, given the impermeable nature of the silty clay and the thickness of the deposit, it is considered that only a minor amount of water is entering the till blanket through the clay.

However, it is known that at least to the south and west of the study area, there are outcrops of bedrock and therefore possibly till. Therefore it is possible that water entering the till in these "catchment" areas flows through this deposit into the Mer Bleue area where the till would have to be in direct communication with the upper portions of the marsh subsoils. It should be noted that the bedrock elevation contours shown on Figure 6 indicate that the glacial topography underlying the site is "bowl shaped" (i.e. there is a bedrock or till ridge immediately to the south and west of the site and some evidence to suggest that there may be a bedrock or till ridge parallel to the north boundary of the site). This configuration would be conducive to the "channelling" of groundwater through the till blanket towards the Mer Bleue area.

Factual data supporting the above postulation is given by deep piezometric data. These results indicate that along the line of deep boreholes put down at the site (see Figure 2) the piezometric pressure in the till blanket is equivalent to a groundwater level at or above elevation 260. However, the deep piezometers installed during investigations for Hwy. 417 which lies closer to the marshland indicate that along this line the equivalent piezometric head is between about elevation 230 and 250. It is noteworthy that the lowest piezometric head was obtained at the Hwy. 417/Anderson Road crossing which in plan is close to a trough in the bedrock or till surface (see Figure 6) and which may represent the eventual underground seepage channel towards Mer Bleue.

On the basis of the above, it appears that the groundwater in the Mer Bleue is not recharged from a source within the proposed development site. Therefore, subject to confirmation as indicated in Appendix III, it is tentatively concluded that the proposed development of the site will not have a detrimental effect on the conservation of the Mer Bleue marshland.

8.4 Stability of the Side Slopes of Bear Brook

Provided that reasonable precautions are taken, it is considered that major flow slides within the valley walls of Bear Brook will not be caused by development of the site. This conclusion is based on work currently being carried out on the stability of the existing slopes at five locations within the Bear Brook valley (Golder Associates report no. 741231, dated March, 1975) and the results of previous analyses (Golder Associates report no. 73908, Vol. IV, dated June,1974). The precautions basically require that the flow (and hence erosional capability) within the river should not be significantly increased as the result of development. Further, any existing areas along the river inside and downstream of the site of marginal stability must be defined and protected against further erosion.

Until more detailed information becomes available it is recommended that for preliminary planning purposes, no structure should be located within 250 feet and 150 feet of the crest of the river bank where the bank height is respectively greater than and less than 15 feet. In no case should fill be placed within these limits. Subexcavation of the valley should only be carried out after a detailed site investigation within the proposed excavation area is carried out.

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TABLE I - SHALLOW FOUNDATIONS RELATED TO BUILDING TYPE AND CRUST THICKNESS

Type of Structure	Conventional Foundation Type	CRUST THICKNESS (t_c)-ft. Area North of Hwy. 417				Area South of Hwy. 417				NOTES
		4-6	6-8	8-12	>12	4-6	6-8	8-12	>12	
A-1 Single family houses (Fig. 33)	Strip Ftgs. (with basement)	✓	✓	✓	✓	✓	✓	✓	✓	
A-2 Semi-detached or row houses (1-2 storey only) (Fig. 33)	Strip Ftgs. (with basement)	✓	✓	✓	✓	? (1)	✓	✓	✓	1. Mat fdns. possible where $t_c \approx 6$ ft.
A-3 Three storey row houses or apt. bldgs. (Fig. 33)	Strip Ftgs. (with basement)	? (1)	? (1)	✓	✓	NO	? (1)	(2)	✓	2. Best located where $t_c > 10$ ft.
B Light industrial, commercial and institutional structures (One to one and half storeys in height) (Figs. 34 & 35)	Strip Ftgs. for external walls sp. ftgs. for isolated column loads (no basement)	?	(3)	✓	✓	NO	NO (4)	(5)	✓	3. Loads on strip ftgs. $\ngtr 2\frac{1}{2}$ tons/lin. ft. 4. Preload possible where $t_c > 6'$ but $< 12'$ 5. Conventional fdns. most appropriate where $t_c > 10$ ft.
C Heavier institutional bldgs., high rise structures	Strip ftgs. for external walls, isolated column loads on sp.ftgs.	NO ←	NO (6)	NO →	? (7)	NO ←	NO (6)	NO →	? (7)	6. Does not take unload due to basement into account. 7. Allowable fdn.press. generally $\ngtr 2$ tons per square foot.

GENERAL NOTE: The information given in the above table is for preliminary planning purposes only. Detailed site investigations will be required for each development or individual structure (types B and C) to define allowable bearing pressures at specific locations.

APPENDIX I

LIST OF PREVIOUS SITE INVESTIGATIONS IN THE SOUTH EAST CITY DEVELOPMENT AREA

March, 1975 741230

The following list of site investigations carried out previously in the general South East City area has been included for reference purposes. Factual data from these reports has been used in the preparation of this report.

Ministry of Transportation and Communications, Ontario; Site Investigations for Hwy. 417 Bridge Structure - Personal Communication.

K.H. King Associates Limited Report to Ontario Housing Corporation, "Preliminary Investigation, Alternative Sites, Land Assembly, Ottawa-Carleton Region". Ref. No. 209-5.15 (A to D); undated, - probably 1972.

K.H. King Associates Limited Report to Ontario Housing Corporation, "Preliminary Investigation of Subsurface Conditions, South East City Growth Area, Ottawa Region, Ontario". Ref. No. 209-5.15; undated, - probably 1972.

K.H. King Associates Limited Report to Ontario Housing Corporation, "Supplementary Report on Subsurface Condition, Gloucester Township, Carleton County". Ref. No. 209-5.15; undated, - probably 1972.

K.H. King Associates Limited Report to DeLeuw Cather, Canada Ltd., "Report Volume 1 of Geotechnical Investigation, South East City, Regional Municipality of Ottawa-Carleton". Ref. No. 312-5.2; undated, - probably 1974.

K.H. King Associates Limited, Report to DeLeuw Cather, Canada Ltd., "Report Volume 2 of Geotechnical Investigation, South East City, Regional Municipality of Ottawa-Carleton". Ref. No. 312-5.2; undated - probably 1974.

H.Q. Golder & Associates Ltd., Report No. 73908, to DeLeuw Cather, Canada Ltd., Volume 3. "Subsurface Investigations, Proposed South East City, Township of Gloucester, Regional Municipality of Ottawa-Carleton", dated May, 1974.

H.Q. Golder & Associates Ltd., Report No. 73908-1, to DeLeuw Cather, Canada Ltd., Volume 4. "Engineering Recommendations, South East City, Regional Municipality of Ottawa-Carleton, Ottawa, Ontario", dated June, 1974.

APPENDIX II

FIELDWORK AND LABORATORY PROCEDURES

March, 1975 741230

FIELDWORK

Boring Programme

The shallow boring programme, which consisted of a total of 32 borings put down at the locations shown on Figure 2, was carried out between December 2, 1974 and January 3, 1975. The boreholes were numbered 201 to 232, inclusive, to differentiate between borings put down in previous investigations at the OHC site. (A full description of the borehole numbering system is given on Figure 2.) About half of the borings were put down to a depth of some 32 feet with a trailer-mounted diamond drillrig using wash boring techniques. The remaining holes, which were located in areas not easily accessible by truck, were put down to a depth of about 34 feet using a bombardier-mounted power auger. Both drillrigs were supplied and operated by a local drilling contractor.

Samples of the overburden were obtained at depth intervals ranging between 2-1/2 and 5 feet using conventional split spoon sampling equipment. In the more cohesive overburden soils, the split spoon samples were augmented by relatively undisturbed 2 and 3 inch diameter thin-walled Osterberg and fixed piston samples. The Osterberg sampler was used to prevent disturbance to the soil due to overdriving. Standard penetration tests were carried out during split spoon sampling operations, and in situ vane shear tests were carried out in the cohesive overburden soils between samples. At periodic intervals during the course of the fieldwork, samples were transported to our laboratory for detailed examination and testing.

The fieldwork was supervised throughout by members of our engineering staff who directed the drilling, sampling and field testing operations, logged the borings and cared for the samples obtained. The ground surface elevations and locations of the borings were given to us by DeLeuw Cather, Canada Ltd. It is understood that the elevations are referred to Geodetic datum.

Geophysical Survey

A geophysical survey was carried out using the seismic method to determine the approximate depth to the till or bedrock surface across the site. The survey was carried out during the period December 30, 1974 and January 13, 1975, by specialist personnel. A single channel seismograph was used to record the arrival time of seismic waves from an energy source provided by an instrumented sledge hammer striking a steel plate on the ground surface. The seismic survey stations were located at or close to the locations of the shallow borings put down during this investigation to obviate the necessity for additional surveying to be carried out and to ensure that there was adequate coverage of the site.

It is noted that as with all geophysical survey methods, the results are necessarily approximate. The accuracy of the results have been compared with the results of two dymanic cone penetration tests carried out during the present investigation (see Record of Penetration Test sheets 202 and 221). At borehole 202 the results of the deep cone penetration test, which gave the depth to the till or bedrock surface as 97 feet, was in good agreement with the seismic result of 94 feet at this location. At borehole 221 in the north east sector of the NCC lands, the dynamic cone penetration test was terminated at refusal (550 blows per foot) at a depth of about 79 feet. However, the seismic results at this point indicated a depth to the till or bedrock of about 120 feet. It is unlikely that the dynamic cone penetration was terminated on a boulder in the glacial till layer since published well drilling records close to this location indicate that the depth to bedrock is between 85 and 96 feet below ground surface. Despite this discrepancy and based on previous experience of the accuracy of the seismic method in predicting the depth to the till or bedrock surface at the site, it is considered that in general the results given by the seismic methods are accurate to about ± 10 percent. In the relatively small number of locations in which interpretation of the seismic results was uncertain, this was taken into account in inferring the till or bedrock surface elevation contours across the site.

LABORATORY TESTING PROCEDURES

The results of numerous studies, which have been carried out in recent years, into the engineering behaviour of highly sensitive clays have shown that the engineering properties of these soils as measured in laboratory tests are strongly influenced by variations in the test techniques. A number of these studies were carried out on the cemented Leda clays found in Eastern Canada, (Jarrett, 1967; Bozozuk, 1970) and are therefore of particular interest in the present investigation. In view of the results of this work, it was considered important to report the test techniques used in the present study to determine the soil properties.

Initially, the laboratory testing programme was designed to provide information on the general nature of the overburden deposits. To this end, water content and Atterberg limit determinations, together with grain size analyses were carried out on representative samples of the overburden soils. Based on these results, consolidation and triaxial tests were performed on selected samples to provide information on the strength and compressibility characteristics of the cohesive overburden soils.

As the engineering properties of the clay immediately underlying the surficial sand cap or desiccated clayey crust have an important influence on the design of shallow foundations, a major portion of the laboratory testing programme was concentrated on samples recovered from within this zone. In this respect, it is noted that, with one exception, the consolidation tests were carried out on soil specimens trimmed from Osterberg samples. (Previous studies in the Ottawa area have shown that in sensitive cemented marine clays less disturbance to the soil structure is associated with Osterberg sampling techniques than with any other form of tube-sampling techniques).

The consolidation tests were carried out on 2 inch diameter samples using a fixed ring consolidation cell. Except for samples from boreholes 226 and 229, load increments were applied at 24 hour intervals to allow sufficient time for consolidation to occur under individual load increments. A load increment ratio typically less than 0.5 was used at stresses in excess of existing effective overburden pressure. In some tests, the samples were off-loaded to below the existing effective overburden pressure when the applied stress reached a value close to this preconsolidation pressure. The subsequent reloading curve has previously been found to give a more accurate indication of the compression index of cemented marine clays in the stress range below the preconsolidation pressure. In the two samples noted above, a faster rate of loading was used (see Figures 23 and 24). Although this technique is known to overestimate the preconsolidation pressure of some soil types (Jarrett, 1967), the results obtained during these tests agree reasonably well with the results from the remainder of the consolidation tests.

Unconsolidated undrained triaxial compression tests were carried out on 2 inch diameter and 4 inch high samples to determine the undrained shear strength of the soil. The tests were carried out at a constant rate of strain of 2 percent per minute and using lubricated end platens. The samples were trimmed using a wire saw and soil lathe to minimize sample disturbance during preparation. Nevertheless, examination of the test results indicated that some samples had been disturbed during sampling operations as evidenced by relatively high axial strains at maximum deviator stress (i.e. 3 percent). However, except for samples which had obviously suffered excessive disturbance, the results of these tests have been included in the overall appraisal of the strength characteristics of the soil. (It is important to note that disturbance of sensitive soils results in lower measured shear strengths and higher compressibility characteristics than would exist in the field and consequently design based on these results, will be conservative.)

APPENDIX III

RECOMMENDATIONS FOR FURTHER
DETAILED GEOTECHNICAL STUDIES
(GOLDER ASSOCIATES LETTER TO DELEUW CATHER,
CANADA LTD., DATED JANUARY 31, 1975)

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

January 31, 1975

De Leuw Cather, Canada Ltd.
133 Wynford Drive
DON MILLS, Ontario
M3C 1K1

ATTENTION: Mr. E.M. Jefferson, P. Eng.

RE: ONGOING DETAILED STUDIES
SOUTH EAST CITY DEVELOPMENT
TOWNSHIP OF GLOUCESTER
OTTAWA-CARLETON, ONTARIO

Dear Sirs:

In your letter dated January 8, 1975, we were requested to make recommendations regarding future detailed studies which should be instigated in the next planning stage of the above development project. This letter outlines our recommendations for further studies, including our comments on recommendations made by the Division of Building Research, National Research Council, (DBR/NRC) in their research proposal dated November 7, 1973.

1. GROUNDWATER LEVELS

In their research proposal for the South East City Development area, DBR/NRC recommended that changes in the groundwater level be monitored to determine the effect with respect to the deformation of the underlying clay deposits. To this end, it was recommended that four sites be selected for installation of piezometers and ground movement gauges.

In general, we agree with this recommendation and consider that a programme should be initiated in the early stages of development. However, it is considered that since the effect of development on the groundwater level is twofold, any further study should concentrate on the following points:

- changes in local groundwater levels as they affect the performance of structures,
- the effect of development on the regional groundwater regime, particularly with respect to the effect of these changes on the conservation of Mer Bleu.

The above points have been discussed in our letter to you dated December 20, 1974 and the following recommendations for future studies are based on those discussions.

i) Local Groundwater Level Monitoring Programme

The purpose of this study would be to define the amount of lowering of local groundwater level, if any, due to activities related to municipal development. Clearly there are a multitude of factors affecting the groundwater level at the development site. A partial list of these includes installation of services, increased run-off due to extensive paving, pumping from sumps located in basements of buildings, recharge due to artificial means (lawn watering and the like). It is considered that, although the effect of some of these factors can be quantified in separate studies (such as the Test Sewer Pipe programme presently underway), the overall effect can only be reasonably determined by studying other municipal developments in sites where the subsoil conditions are similar to those existing at the site presently under consideration. To this end, we suggest that groundwater levels at other development sites, particularly in the Ottawa area, are monitored and compared with the measurements taken prior to development. The latter information should be available from site investigations for the relevant developments from existing well records.

Initially, this work would involve the selection of appropriate sites and a summary of the "pre-development" information at each site. Depending on the existence or otherwise of the standpipes/piezometers installed during pre-development site investigations, the second stage could require the installation of standpipes or piezometers at various selected locations within the developed areas. The information thus obtained would preclude possible accumulative errors which are inevitable in combining the results of individual studies of the effect of the various individual factors related to groundwater level changes.

At this time it is considered that a study of this nature would likely require a low capital input and could therefore be economically terminated at any stage if the continuing review of available data indicated that the study was not producing the required information.

ii) Regional Groundwater Level Monitoring Programme

Although a knowledge of the probable regional settlements due to development of the site would be extremely useful in the preliminary and subsequent planning stages, this programme would primarily serve the purpose of defining the hydrological inter-relationship between the proposed development site and the adjacent Mer Bleu conservation area. It is considered that the "recharge" mechanism whereby the groundwater level in Mer Bleu is maintained can be effected either by surface flow or by percolation of surface water into the relatively permeable glacial till deposit which is thought to underly the general area under consideration.

An examination of existing topographical information in the general area, should provide the necessary information to appraise the significance of the first of these, i.e. surface recharge. However, the "deep" recharge mechanism can only be established by monitoring the piezometric pressures in the overburden deposits at various locations in the development and conservation areas. Depending on the availability of this data from other sources, such as the Ministry of Environment Studies, it may be necessary to install piezometers as part of this study. Further, a knowledge of the topography of the till and bedrock surfaces across the general area may indicate the source of recharge water and the possible existence of preferential flow channels through the till blanket. Although some information is available regarding the topography of the till/bedrock surface across the development site, a further study should incorporate available data from the surrounding area. For example, rock outcrops may act as cachment areas for collection of surface water which eventually percolates through into the till blanket. If this is indeed the case, then it could readily be accepted that the development, per se, would not affect the recharge of the groundwater within Mer Bleu. Finally it is recommended that, because of the possible serious effect of development on the conservation of Mer Bleu, this study should be initiated prior to the final planning stage.

2. SOIL PROPERTIES

In the second part of the DBR/NRC proposal, it was noted that the physical properties and engineering behaviour of the weathered clay crust at the site would have an important influence on the design of shallow foundations. It was therefore recommended that these factors be the subject of a detailed study which would include a review of the existing data, use of special sampling and field testing techniques and the construction and monitoring of a test fill or structure.

We agree that such a study would be of considerable benefit in advancing the state of knowledge within the geotechnical engineering field. However, it is considered that to derive more immediate benefit, the study programme should be designed to relate to the significant geotechnical aspects of the present development. To this end, we suggest that the programme be subdivided as follows:

(a) The results of the proposed review of existing data and the study into the engineering behaviour of clayey crustal soils and the underlying unweathered deposits should be compared with the results of routine sampling and testing techniques which would be an integral part of site investigations, carried out within the area as development proceeds. In this respect, it should be noted that routine sampling, together with conventional commercial field and laboratory techniques, are known to disturb the soil structure and hence the results of these tests generally indicate a lower strength and higher compressibility of the Leda clay deposits than actually exists in the field. Therefore, in order to avoid conservative and hence possibly uneconomical foundation design, it would be prudent to have a working knowledge of the differences which can be expected between the results obtained in routine testing and the actual in situ conditions. To this end, we suggest that "good quality" tests be carried out on "undisturbed: samples obtained from either shallow test pits OR deeper borings when "sophisticated" sampling gear is utilized.

(b) With respect to (a) above, it is considered that the construction and monitoring of one test fill or loading structure at a selected location in the general area would serve the purpose of checking the validity of conventional methods of foundation design (with respect to bearing capacity) and prove the validity of the "soil parameter" correlation resulting from (a) above. However, it is noted that a short-term full scale loading test of this nature will not provide much information on the long-term consolidation settlement characteristics of the Leda clay deposit. Information on this aspect can however be gained from a review of the existing data obtained in monitoring two other test fills in the area, i.e. the Gloucester Test Fill (NRC) and the embankment on Anderson Road at Highway 417 (MTC).

3. SEASONAL WATER CONTENT VARIATION

In order to provide realistic and economical design standards for constuction of roads in the development, it will be necessary to define the seasonal variation of water content in the upper 4 to 5 feet of the desiccated crustal soils. To this end we suggest a study of the seasonal variations in this parameter along the routes of proposed roads within the development. It is envisaged that such a programme would be of a long-term nature and should, therefore, be initiated as soon as possible prior to the final planning stage of the development. The results of this study would also be of considerable use in defining the effect of growth of various tree types on the subsoils at the site (i.e. swelling and shrinkage); a factor which is of considerable significance with respect to deformations of structures associated with the proposed development.

4. FROST PENETRATION

It is generally accepted that, within the Ottawa area, exterior foundations for heated structures should be constructed at a minimum depth of 5 feet below final grade for frost protection purposes. This design depth is based on reliable past experience and is not in dispute. However, it is noted that, to accommodate this restriction, the foundations for structures in some areas of the site will be close to or below the stronger surficial sand and clayey crustal deposits. Therefore the allowable bearing pressures available for foundation support are considerably reduced. The alternative method of locating the foundation about 2 to 3 feet below existing ground surface and gaining the required 5 feet frost protection by regrading the ground surface around the structure requires a considerable amount of earthworks, often involving difficult soils.

Therefore to obviate the necessity of excessive regrading/ earthworks, it is suggested that frost protection for foundations could be provided by a styrofoam insulation barrier. This technique has proven useful in smaller development areas in North Canada where the climatic conditions are considerably worse that in the Ottawa area. To our knowledge, this technique is not prohibitively expensive in terms of capital or installation costs. Therefore we suggest that a long-term monitoring programme be instigated to determine the effectiveness of this technique and to define the necessary design parameters which should be used in the presently proposed development. To this end we suggest that various forms of insulated "simulated" foundations are constructed at a selected location of the site and their performance monitored over a long term.

5. ROAD PAVEMENT DESIGN

There are a number of aspects of the design of road pavements which deserve special attention:

(a) The engineering characteristics of the surficial clayey subsoils which underlie considerable areas of the proposed development site are such that a considerable thickness of road base structure may be required to provide satisfactory road pavement performance. In this respect, it is noted that the improvement of the subgrade using stabilizing techniques such as lime in stabilization will reduce the amount of granular road base required and hence may lead to considerable overall savings in the cost of importing granular. It is suggested that a review of the performance of other roads and highways in the Ottawa area which have been constructed on stabilized subgrades may yield valuable information on this aspect of development. Further, a modest laboratory programme could be carried out to determine the optimum quantity of lime which is necessary to increase the CBR values of the subgrade material at the site presently under investigation.

(b) In our experience, one of the major contributing factors to pavement distress is deformation within clayey material used to backfill service trenches which are installed down the centre of the roadways. Obviously, there is considerable economic advantage in using selected native clayey backfill in service trenches installed as part of the proposed development. However if these are to be located below the roads, tests should be carried out to ascertain the likely settlements within the backfill and the period over which these settlements will take place. To this end, Golder Associates have developed apparatus to simulate the behaviour of subgrade materials under operative conditions and we consider that a series of tests run on selected samples of the native clayey crust may prove useful in providing realistic data on which this aspect of road pavement can be based.

(c) Although it is unlikely that all of the roads presently in use at the site have been carefully designed and constructed to the rigid standards required for future development, they nevertheless provide an opportunity for evaluation of pavement performance on the existing subgrade soils at the site. Therefore we recommend that a survey be carried out of the actual pavement structure of these roads and this data related to the measured performance of the roads. The performance can be qualitatively assessed by visual inspection but should also be confirmed by carrying out a Benkleman Beam survey of selected roads in the area.

6. DETAILED GENERAL GEOTECHNICAL INVESTIGATIONS

As stated by the DBR/NRC in their proposal, the development area "is one of the few for which detailed soil conditions will be taken into account in the initial planning and....". To date a preliminary geotechnical investigation of the proposed development site has been carried out (see Golder Associates report No. 73908, Volume IV, dated June, 1974) and a second investigation is currently underway on other lands adjacent to the site (see Golder Associates preliminary report No. 741230, dated January, 1975). Further, supplementary studies are currently underway, related to the control of run-off from the development site into Bear Brook (Golder Associates preliminary report NO. 741231, dated January, 1975) and to the monitoring of the performance of a length of test sewer pipe (Golder Associates report No. 741232, dated December, 1974).

The first of the above mentioned investigations has served to define the general character of the subsoils at the site together with variations in crust thickness and depth to bedrock across the site. Based on the findings of this investigation, we have defined optimum locations for various types of structures with respect to foundation use for overall planning purposes. However, when the preliminary planning stage is complete and the development plans within the site are finalized, it will be necessary to carry out detailed investigations in heavily developed areas to confirm that the soil conditions in the areas are at least as good as inferred from the results of the earlier preliminary investigations. Further, it is essential that detailed investigations are carried out at the sites of large structures, to ensure that the foundation design is both safe and economical.

Finally it is understood that, of necessity, the proposed development will take place over a considerable time period. As a good appreciation of and respect for the soil conditions at the site is essential to the overall success of the project, the data which has been obtained to date and which will be obtained in future investigations should be well documented and assimilated periodically. It is possible that, based on an ongoing review of the available data, changes in both detailed design procedures and in various aspects of the overall planning of the development may be advantageously made.

In outlining our recommendations for future studies which we feel would be of considerable benefit in ensuring the success of the project, we have tried to be practical. Therefore most of the suggestions relate primarily to geotechnical aspects of the development which are of immediate interest in the planning stages of the project. Further, in the majority of the studies recommended above, we consider that the work can and should be carried out in stages. It is considered important that not only should the problem be defined, and related to the present development, but at an early stage (i.e. prior to the commitment of capital outlay), the applicability of actual study itself should be assessed. However, it should be noted that many of the studies outlined in this letter require little capital outlay.

We trust that the suggestions and recommendations given in this letter are sufficient for your present requirements. If you have any questions regarding this or any other aspect of the proposed development related to geotechnical engineering, please do not hesitate to call us.

Yours truly,

H.Q. GOLDER & ASSOCIATES LTD.

J.H.A. Crooks, P. Eng.

J.L. Seychuk, P. Eng.

JHAC:JLS:def
741269

APPENDIX IV

REFERENCES

March, 1975 741230

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, N: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$
e = base of natural logarithms 2.7183
$\log_e a$ or $\ln a$, natural logarithm of a
$\log_{10} a$ or $\log a$, logarithm of a to base 10

t time
g acceleration due to gravity
V volume
W weight
M moment
F factor of safety

II. STRESS AND STRAIN

u pore pressure
σ normal stress
σ' normal effective stress ($\bar{\sigma}$ is also used)
τ shear stress
ϵ linear strain
ϵ_{xy} shear strain
ν Poisson's ratio (μ is also used)
E modulus of linear deformation (Young's modulus)
G modulus of shear deformation
K modulus of compressibility
η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ unit weight of soil (bulk density)
γ_s unit weight of solid particles
γ_w unit weight of water
γ_d unit dry weight of soil (dry density)
γ' unit weight of submerged soil
G_s specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e void ratio
n porosity
w water content
S_r degree of saturation

(b) *Consistency*

w_L liquid limit
w_P plastic limit
I_P plasticity index
w_S shrinkage limit
I_L liquidity index $= (w - w_P)/I_P$
I_C consistency index $= (w_L - w)/I_P$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
D_r relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

h hydraulic head or potential
q rate of discharge
v velocity of flow
i hydraulic gradient
k coefficient of permeability
j seepage force per unit volume

(d) *Consolidation (one-dimensional)*

m_v coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$
C_c compression index $= -\Delta e/\Delta \log_{10} \sigma'$
c_v coefficient of consolidation
T_v time factor $= c_v t/d^2$ (d, drainage path)
U degree of consolidation

(e) *Shear strength*

τ_f shear strength
c' effective cohesion intercept
ϕ' effective angle of shearing resistance, or friction
— in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$

c_u apparent cohesion*
ϕ_u apparent angle of shearing resistance, or friction
— in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$

μ coefficient of friction
S_t sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

Form G.A.-D-1.

Project No. 74[redacted]30[redacted]

RECORD OF BOREHOLE 201

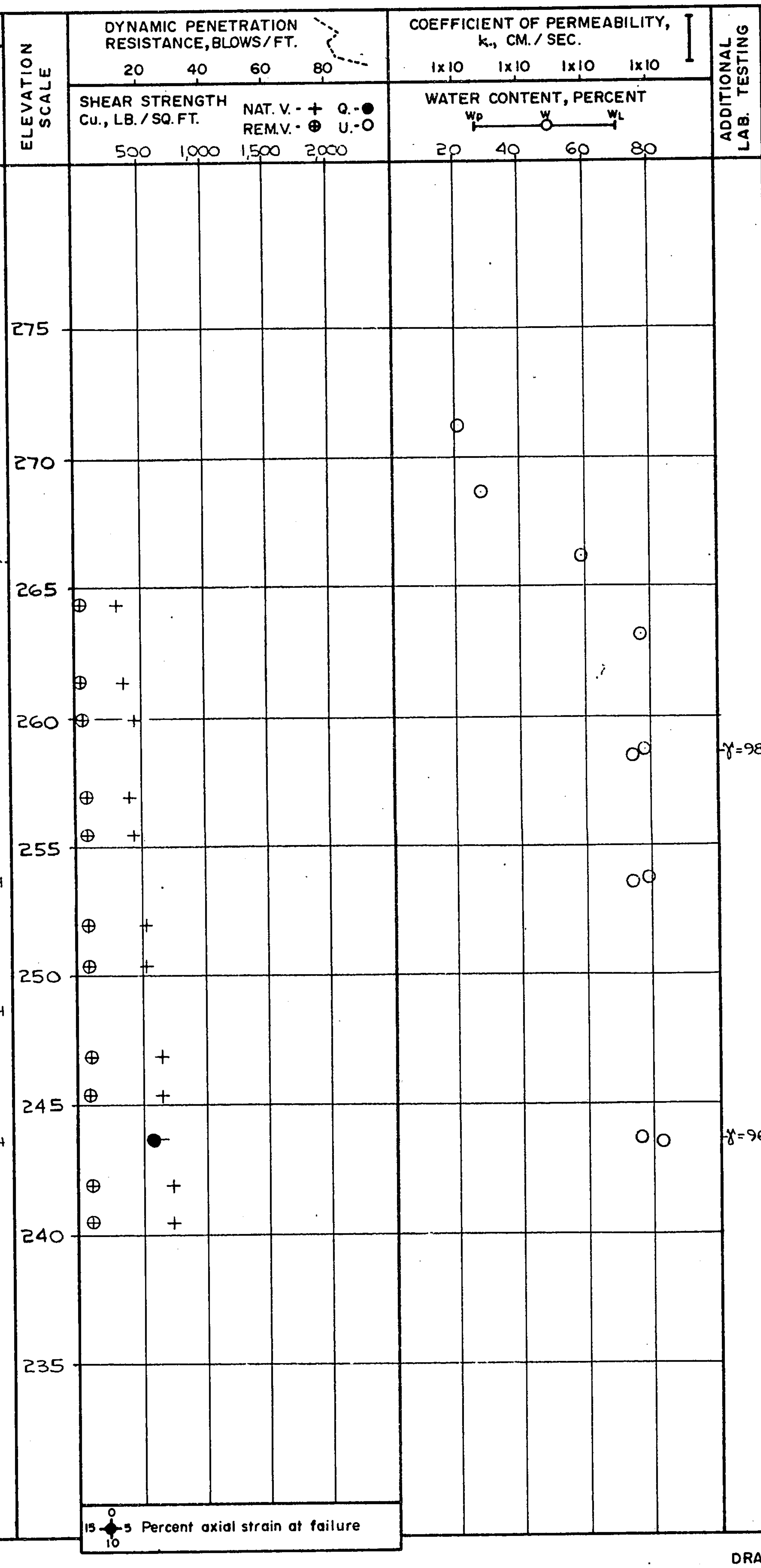
LOCATION See Figure 2 BORING DATE DEC. 13, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION
	274.4	GROUND SURFACE
POWER AUGER, 7" DIA. HOLLOW STEM	273.9 / 0.5	SANDY TOPSOIL
	267.4 / 7.0	COMPACT BROWN SILTY SAND TO SAND
	239.9 / 34.5	SOFT BECOMING FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY OCCASIONAL SILT LAYERS AND POCKETS OF BLACK ORGANIC CLAY
		END OF HOLE

SAMPLES

NUMBER	TYPE	BLOWS/FT.
1	2" D.O.	20
2	"	13
3	"	W.R.
4	3" T.P.	PH
5	2" T.P.	PH
6	3" T.P.	PH
7	2" T.P.	PH
8	3" T.P.	PH



PIEZOMETER OR STANDPIPE INSTALLATION

GROUND SURFACE

CLAY BACKFILL

3/8" DIA. PLASTIC TUBING

GRAVEL FILL

STANDPIPE

CLAY BACKFILL

W.L. IN STANDPIPE AT EL. 268.9 ON JAN. 10, 1975.

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED RG

Form G.A.-D-1.

Project No. 741230

RECORD OF BOREHOLE 202

LOCATION See Figure 2 BORING DATE JAN. 7, 1975 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	SAMPLE NUMBER	TYPE	BLOWS/FT.
	262.9	GROUND SURFACE				
WASH BORING – HX CASING	0.0 / 259.4	COMPACT BROWN SILTY SAND TO SAND		1	2" D.O.	11
				2	"	5
	3.5 / 255.4	STIFF FISSURED MOTTLED BROWN SILTY CLAY WITH SAND LAYERS (DESICCATED CRUST)		3	2" T.O.	P.M.
	7.5	SOFT TO FIRM GREY AND RED-GREY SENSITIVE SILTY CLAY WITH LAYERS OF CLAYEY SILT AND SILTY SAND		4	2" D.O.	P.M.
				5	3" O.S.	W.P.
WASH BORING – UNCASED				6	2" D.O.	P.M.
				7	3" O.S.	W.P.
				8	2" D.O.	P.M.
	231.4			9	3" T.P.	P.L.
	31.5	END OF SAMPLED HOLE. DYNAMIC CONE PENETRATION TEST TO REFUSAL AT EL. 164.9 (SEE RECORD OF PENETRATION TEST SHEET)				

ELEVATION SCALE: 265, 260, 255, 250, 245, 240, 235, 230

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.: 20 40 60 80

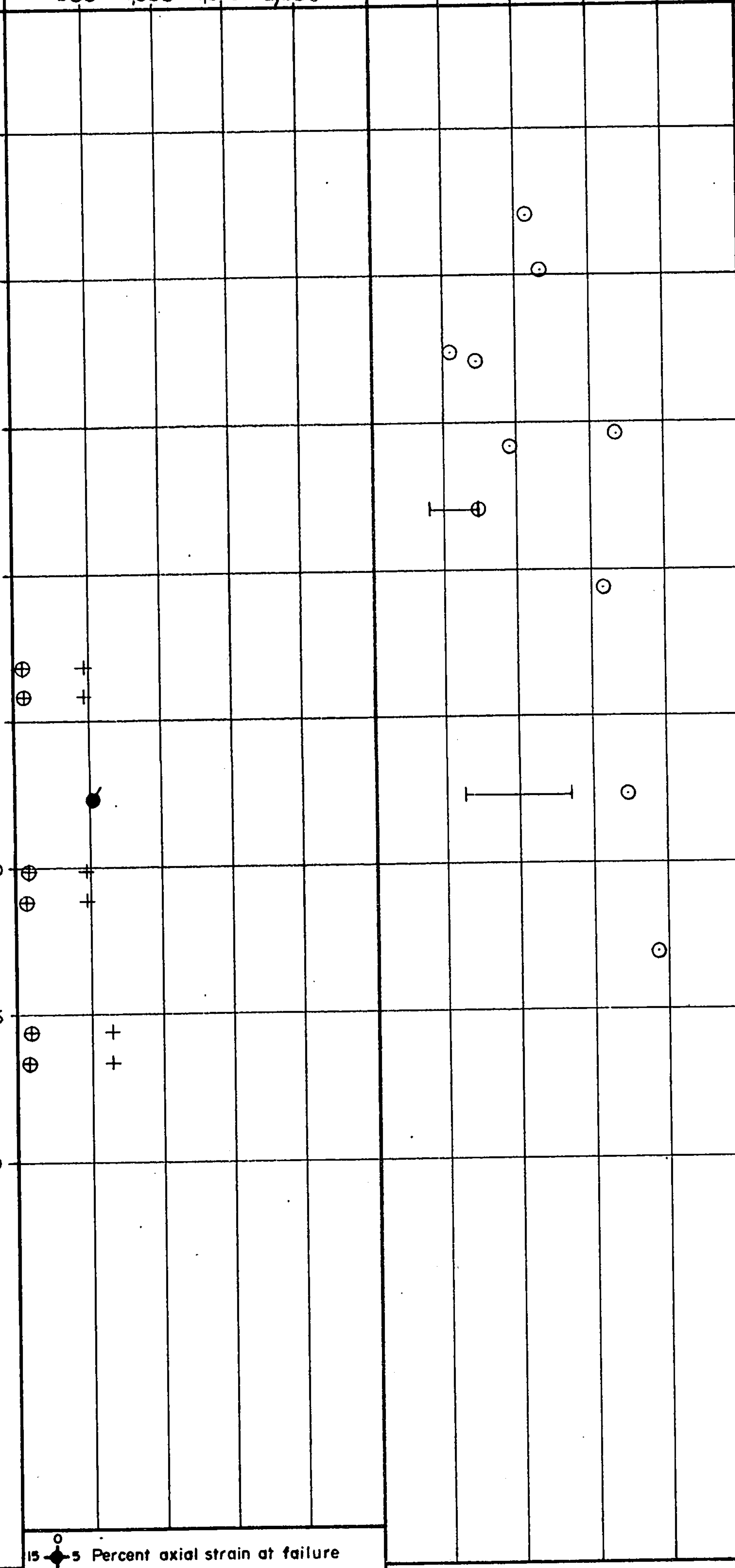
SHEAR STRENGTH Cu., LB./SQ.FT.: NAT. V. - + Q. - ● REM.V. - ⊕ U. - O — 500 1,000 1,500 2,000

COEFFICIENT OF PERMEABILITY, k, CM./SEC.: 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT: Wp W Wl — 20 40 60 80

ADDITIONAL LAB. TESTING: γ=123 C; γ=100

PIEZOMETER OR STANDPIPE INSTALLATION



15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED RG

[redacted] Form [redacted] [redacted] PROJECT No. [redacted]

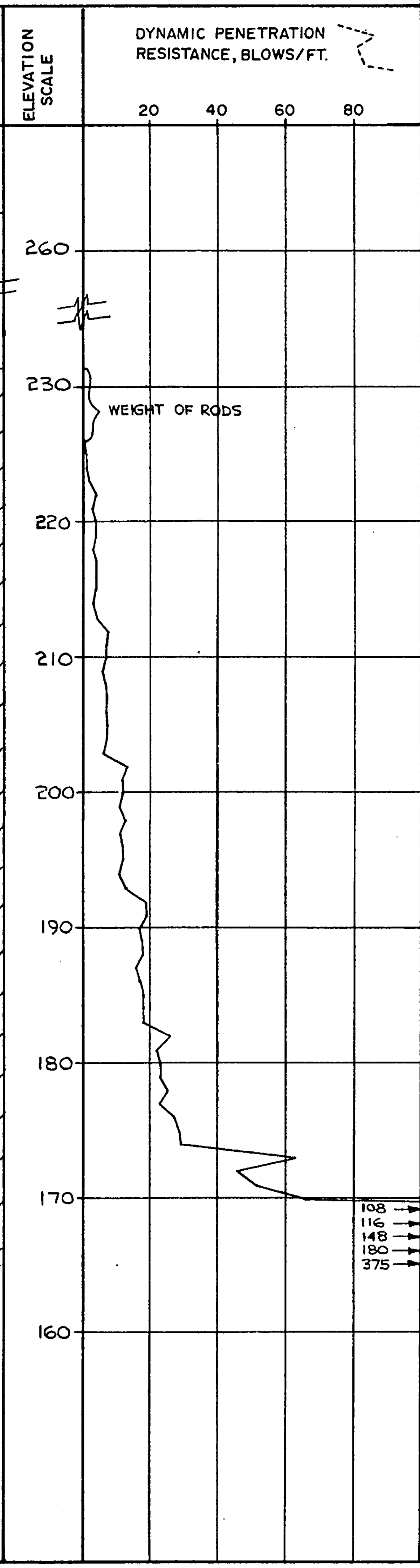
RECORD OF PENETRATION TEST 202

LOCATION See Figure 2 DRIVING DATE JAN. 8, 1975 DATUM GEODETIC

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST No. 202

ELEV'N. / DEPTH	INFERRED SOIL PROFILE
262.9 / 0.0	GROUND SURFACE
	FOR STRATIGRAPHY SEE RECORD OF BOREHOLE SHEET 202
231.4 / 31.5	END OF BOREHOLE 202
	PROBABLY FIRM TO STIFF SENSITIVE SILTY CLAY
164.9 / 98.0	END OF PEN. TEST



PENETRATION TEST No.

ELEV'N. / DEPTH	INFERRED SOIL PROFILE	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN J.A.
CHECKED RG

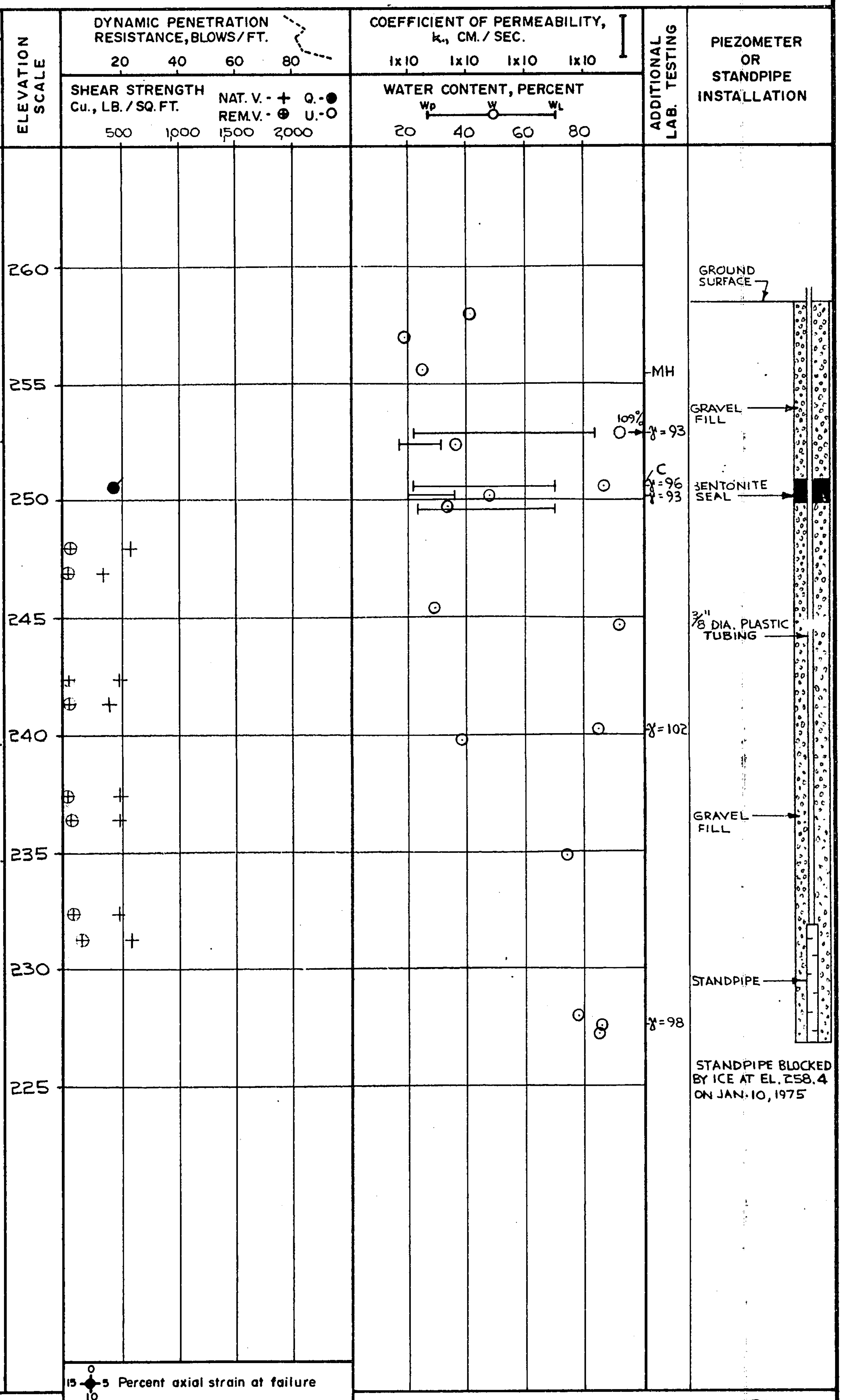
Form G.A.-D-1. Project No. 74[illegible]0

RECORD OF BOREHOLE 203

LOCATION See Figure 2 BORING DATE DEC. 3, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE: ELEV'N. DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FT.
	258.4	GROUND SURFACE			
WASH BORING, HX CASING	257.9	TOPSOIL			
	0.5	LOOSE STRATIFIED RED-BROWN SILTY SAND TO SANDY SILT	1	2" D.O.	6
			2	"	9
WASH BORING, HX CASING / UNCASED	254.4 4.0	SOFT BECOMING FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, SOME GREY CLAYEY SILT AND SANDY SILT LAYERS, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY WITH DEPTH	3	2" T.O.	P.M.
			4	3" O.S.	W.P.
			5	2" D.O.	1
			6	3" T.O.	P.M.
			7	2" D.O.	P.M.
			8	3" O.S.	W.P.
	226.9 31.5	END OF HOLE			



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED RG

Form G.A.-D-I. Project No. 74[illegible]

RECORD OF BOREHOLE 204

LOCATION See Figure 2. BORING DATE DEC. 5 & 6, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: POWER AUGER, 7" DIA. HOLLOW STEM

ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
271.8	GROUND SURFACE			
271.3 / 0.5	BROWN SANDY TOPSOIL			
	COMPACT TO DENSE BROWN SAND	1	2" D.O.	12
		2	"	19
		3	"	35
261.8 / 10.0	SOFT BECOMING FIRM, SENSITIVE, GREY AND RED-GREY SILTY CLAY, SOME GREY SILT TO CLAYEY SILT LAYERS	4	"	WH.
		5	3" T.P.	P.H.
		6	"	"
		7	2" D.O.	2
		8	3" T.P.	PH.
		8a	2" D.O.	WH.
		9	3" T.P.	PH.
		9a	2" T.P.	"
237.8 / 340	END OF HOLE			

ELEVATION SCALE: 275, 270, 265, 260, 255, 250, 245, 240, 235

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.: 20, 40, 60, 80

SHEAR STRENGTH Cu., LB./SQ.FT.: 500, 1,000, 1500, 2,000 — NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

COEFFICIENT OF PERMEABILITY, k., CM./SEC.: 1x10, 1x10, 1x10, 1x10

WATER CONTENT, PERCENT: W_P — W — W_L: 20, 40, 60, 80

ADDITIONAL LAB. TESTING: γ = 97; γ = 107

15 / 0 / 5 / 10 Percent axial strain at failure

PIEZOMETER OR STANDPIPE INSTALLATION:

GROUND SURFACE

CLAY SEAL

3/8" DIA. PLASTIC TUBING

CLEAN GRAVEL

STANDPIPE

CAVE-IN MATERIAL

WATER LEVEL IN STANDPIPE AT ELEV. 268.8 ON JAN. 9, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED BG

Form G.A.-D-1. Project No. 74[redacted]0

RECORD OF BOREHOLE 205

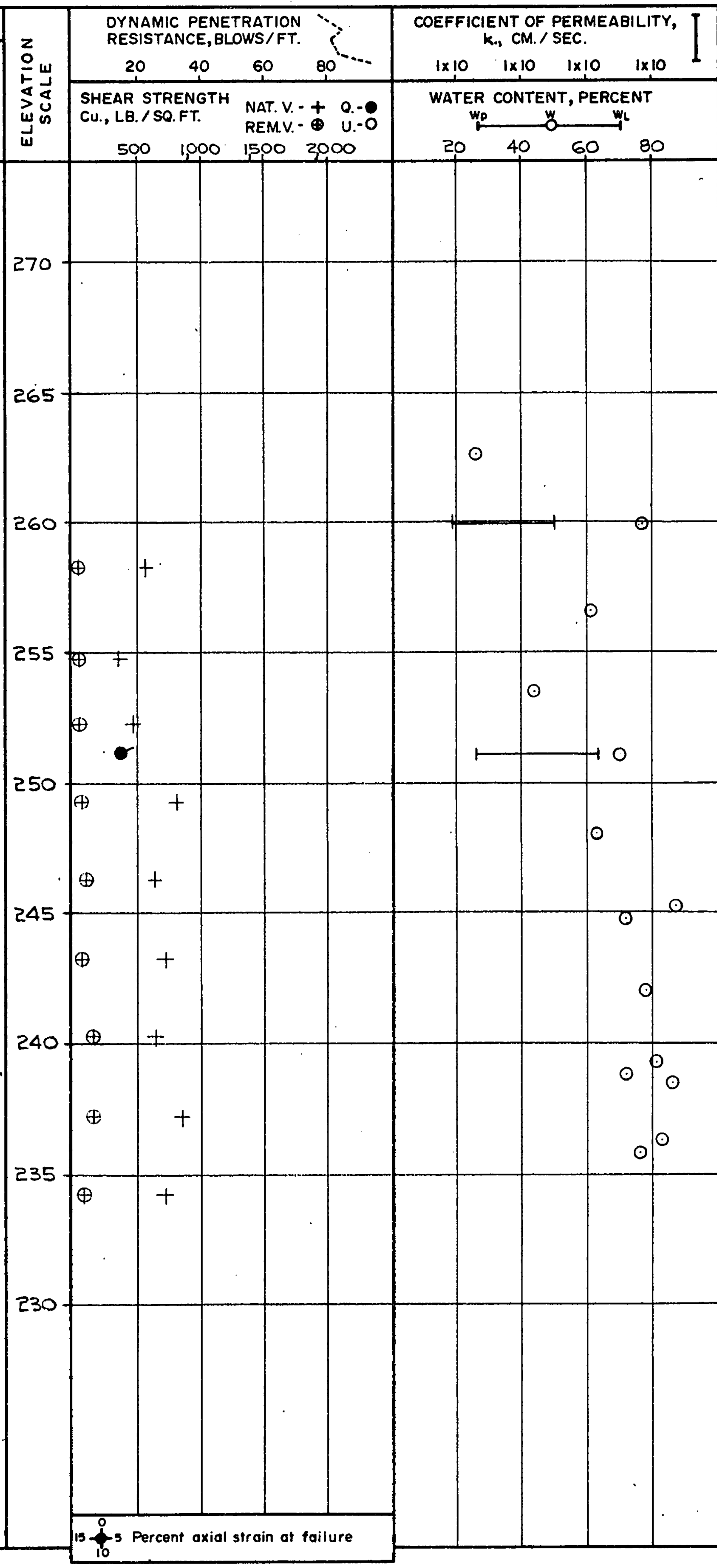
LOCATION See Figure 2 BORING DATE DEC. 5 & 6, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: POWER AUGER, 7" DIA. HOLLOW STEM

ELEV'N. / DEPTH	SOIL PROFILE DESCRIPTION
265.8	GROUND SURFACE
265.3 / 0.5	BROWN SANDY TOPSOIL
261.3 / 4.5	LOOSE GREY-BROWN TO BROWN SANDY SILT
233.8 / 32.0	SOFT BECOMING FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY
	END OF HOLE

SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
1	2" D.O	10	M
2	"	2	
3	3" T.P.	PH.	γ=106
4	2" D.O.	3	
5	3" T.P.	PH.	C, γ=102, γ=98
6	2" D.O.	1	
7	3" T.P.	PH.	γ=100
8	2" D.O.	2	
9	3" T.P.	PH.	γ=100
10	"	"	γ=98



PIEZOMETER OR STANDPIPE INSTALLATION

GROUND SURFACE

CLAY FILL

BENTONITE SEAL

3/8" DIA. PLASTIC TUBING

CLEAN GRAVEL

STANDPIPE

CAVE-IN MATERIAL

STANDPIPE INSTALLATION DESTROYED BEFORE READING COULD BE TAKEN

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

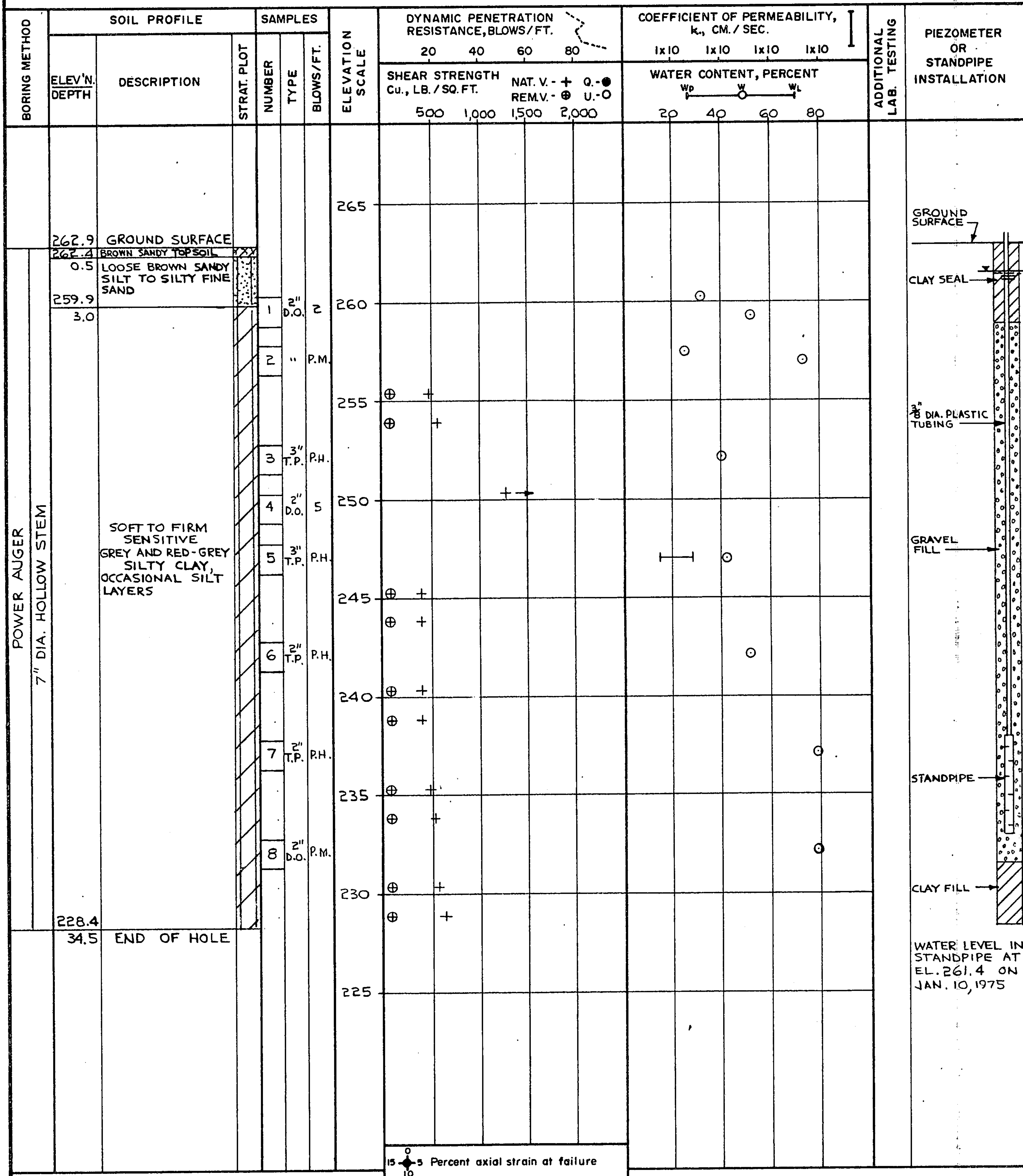
CHECKED RG

Form G.A.–D–I. Project No. 74[redacted]0[redacted]

RECORD OF BOREHOLE 206

LOCATION See Figure 2 BORING DATE JAN. 2, 1975 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

Form G.A.-D-1. Project No. 74[redacted]0

RECORD OF BOREHOLE 207

LOCATION See Figure 2 BORING DATE DEC. 9, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE ELEV'N. / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
						265		GROUND SURFACE
POWER AUGER 7" DIA. HOLLOW STEM	263.1	GROUND SURFACE						
	262.6 / 0.5	BROWN SANDY TOPSOIL						CLAY FILL
	260.6 / 2.5	BROWN SILTY FINE SAND						
		FIRM TO STIFF FISSURED, MOTTLED BROWN SILTY CLAY, SOME SAND LAYERS (DESICCATED CRUST)	1	2" D.O.	5	260		BENTONITE SEAL
	255.1 / 8.0		2	3" T.P.	PH.		γ=105	
	253.6 / 9.5	COMPACT GREY SILTY SAND	3	2" D.O.	15	255	MH	3/8" DIA. PLASTIC TUBING
		SOFT BECOMING FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILT LAYERS AND POCKETS OF BLACK ORGANIC CLAY	4	"	2			
			5	3" T.P.	PH.	250	γ=102	
			6	"	"	245	γ=99	CLEAN GRAVEL
			7	2" D.O.	2	240		STANDPIPE
			8	2" T.P.	PH.	235	γ=97	CAVE-IN MATERIAL
	230.6 / 32.5	END OF HOLE				230		WATER LEVEL IN STANDPIPE AT ELEV. 259.4 ON JAN. 10, 1975

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O 500 1000 1500 2000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT W_P W W_L 20 40 60 80

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED RG

Form G.A.-D-1.

Project No. 74[illegible]30

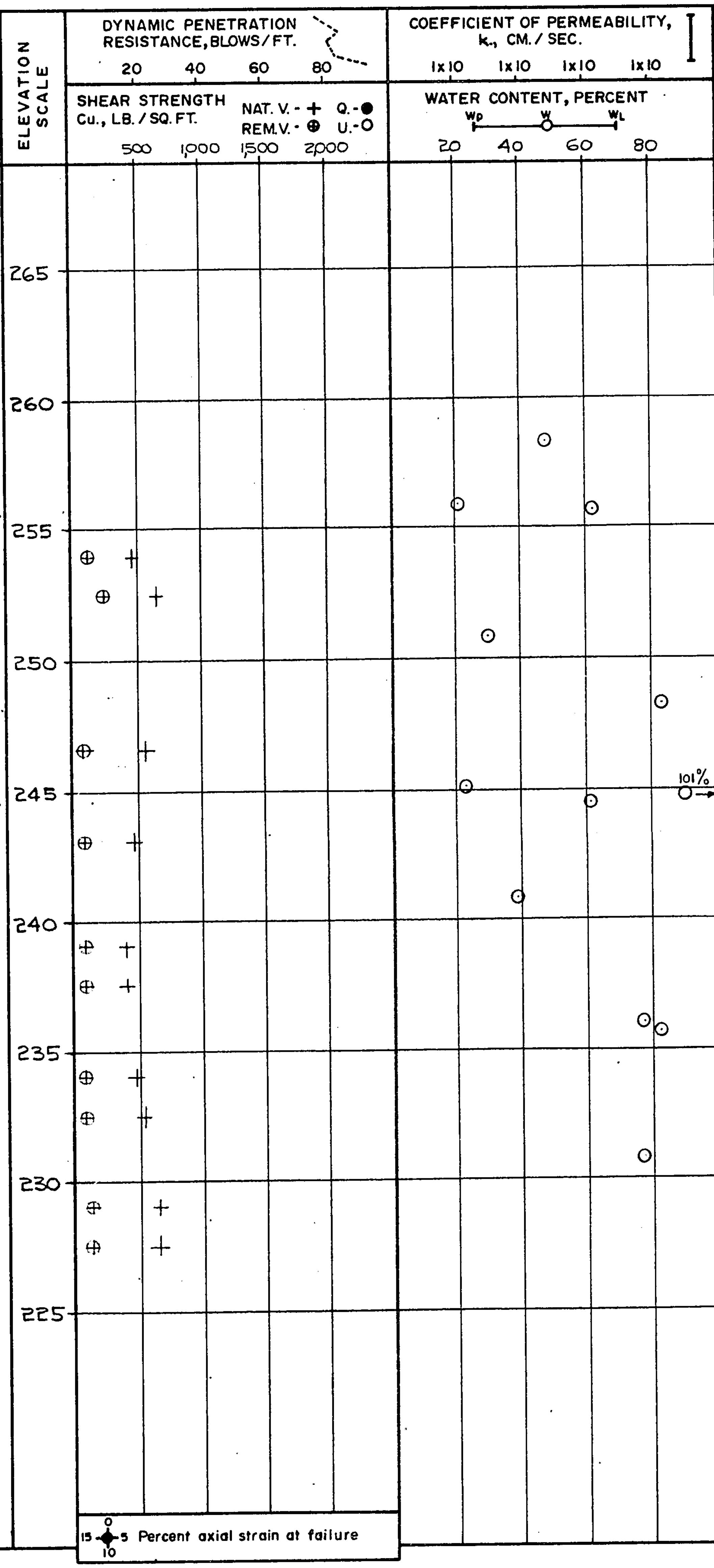
RECORD OF BOREHOLE 208

LOCATION See Figure 2 | BORING DATE DEC. 9, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: POWER AUGER, 7" DIA. HOLLOW STEM

ELEV'N. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
261.5	GROUND SURFACE				
261.0	BROWN SANDY TOPSOIL				
0.5	BROWN SILTY SAND				
259.0 / 2.5	FIRM, MOTTLED RED-BROWN SILTY CLAY, SOME SILT AND SAND LAYERS (DESICCATED CRUST)	1	2" D.O.	5	
		2	3" T.P.	PH.	γ=109
251.5 / 10.0	COMPACT GREY SILTY FINE SAND	3	2" D.O.	20	MH
249.5 / 12.0	SOFT, BECOMING FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, SOME SILT LAYERS AND POCKETS OF BLACK ORGANIC CLAY.	4	"	W.R.	
		5	2" T.P.	PH.	
		6	"	"	
		7	"	"	γ=96
		8	2" D.O.	2	
227.0 / 34.5	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

GROUND SURFACE

CLAY FILL

BENTONITE SEAL

3/8" DIA. PLASTIC TUBING

GRAVEL BACKFILL

STANDPIPE

CAVE-IN MATERIAL

WATER LEVEL IN STANDPIPE AT ELEV. 259.5 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED

Form G.A.-D-I.

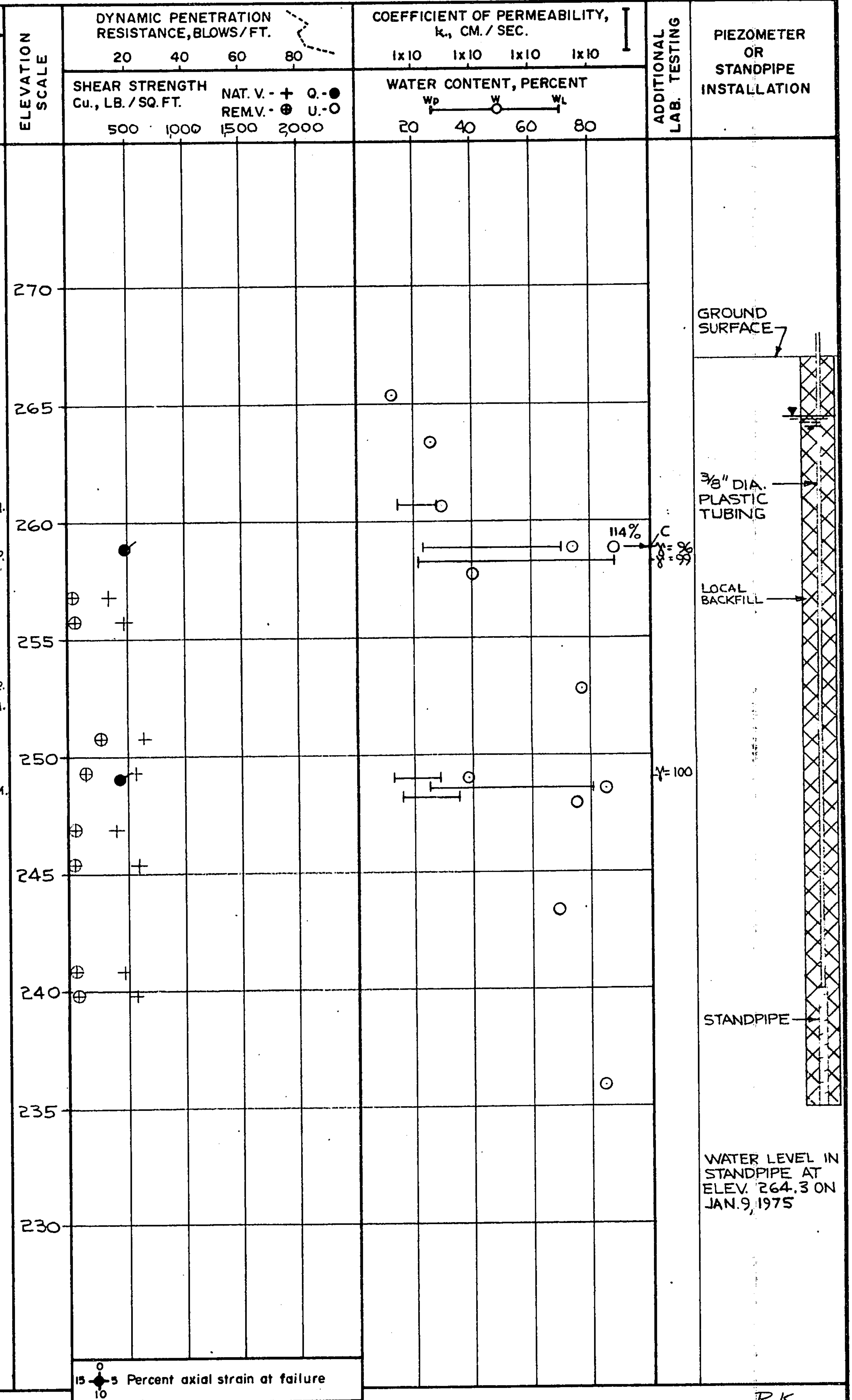
Project No. 74[redacted]0

RECORD OF BOREHOLE 209

LOCATION See Figure 2 BORING DATE DEC. 2 & 3, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
	266.8	GROUND SURFACE			
WASH BORING – HX CASING	266.3	BROWN SANDY TOPSOIL			
	0.5	LOOSE BROWN SAND TO GREY SILTY SAND	1	2" D.O.	10
			2	"	7
	262.8 / 4.0	MOTTLED BROWN SILTY CLAY, SOME SAND SEAMS (DESICCATED CRUST)			
	260.8 / 6.0	SOFT TO FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, SOME CLAYEY SILT AND SILT LAYERS	3	2" T.O.	PM.
			4	3" O.S.	W.P.
WASH BORING – UNCASED			5	2" D.O.	WR. & PM.
			6	3" T.O.	PM.
			7	2" D.O.	"
			8	2" D.O.	"
	234.8 / 32.0	END OF HOLE			



VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN RK
CHECKED BG

Form G.A.–D–I.

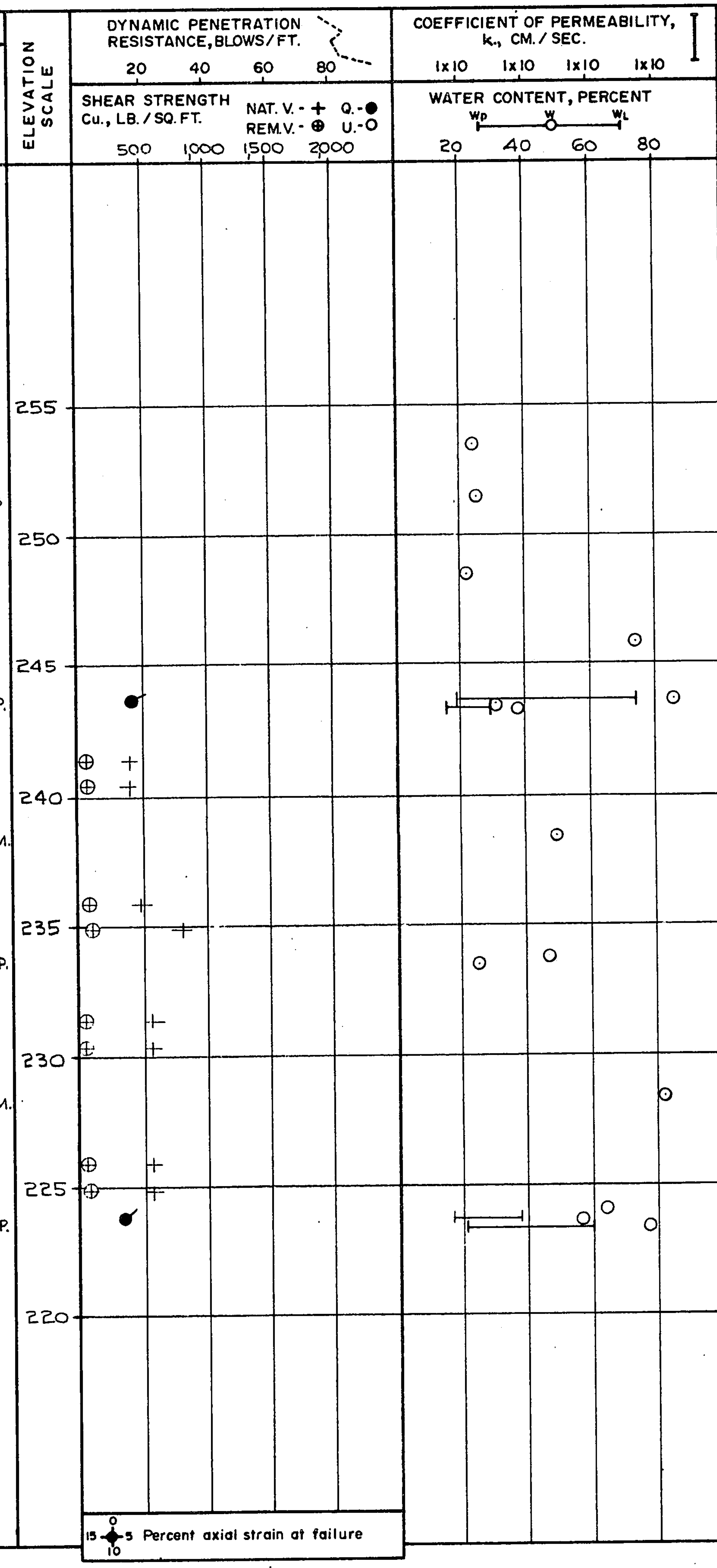
Project No. 741[redacted]

RECORD OF BOREHOLE 210

LOCATION See Figure 2 BORING DATE DEC 4, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	254.4	GROUND SURFACE					GROUND SURFACE
WASH BORING – HX CASING	253.9	TOPSOIL					
	0.5	LOOSE TO COMPACT BROWN TO GREY SILTY SAND TO SAND	1	2" D.O.	8		
			2	"	13		BENTONITE SEAL
			3	"	6		
WASH BORING – UNCASED	247.4 / 7.0	SOFT BECOMING FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY NUMEROUS CLAYEY SILT TO SILT LAYERS (UP TO 6" THICK) OCCASIONAL POCKETS OF BLACK ORGANIC CLAY	4	2" D.O.	1		3/8" DIA. PLASTIC TUBING
			5	3" O.S.	W.P.	MH γ=105; C γ=116	
			6	2" D.O.	P.M.		GRAVEL BACKFILL
			7	3" O.S.	W.P.	γ=123	
			8	2" D.O.	P.M.		STANDPIPE
			9	3" O.S.	W.P.	γ=104	
	222.9 / 31.5	END OF HOLE					STANDPIPE BLOCKED BY ICE AT EL. 255.4 ON JAN. 9, 1975



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED BG

Form G.A.-D-1.

Project No. 74[redacted]0

RECORD OF BOREHOLE 211

LOCATION See Figure 2 | BORING DATE DEC. 5, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	254.2	GROUND SURFACE				
WASH BORING – HX CASING	0.0	LOOSE TO COMPACT BROWN TO GREY SILTY SAND	1	2" D.O.	7	
			2	"	23	
			3	"	17	
			4	"	(76)	
	243.7 / 10.5	SOFT TO FIRM SENSITIVE RED-GREY AND GREY SILTY CLAY, OCCASIONAL THIN SILT AND SAND LAYERS	5	"	P.M.	
			6	3" O.S.	W.P.	γ = 97
WASH BORING – UNCASED			7	2" D.O.	P.M.	
			8	3" O.S.	W.P.	γ = 99
			9	"	"	γ = 102
	222.7 / 31.5	END OF HOLE				

ELEVATION SCALE: 255, 250, 245, 240, 235, 230, 225, 220'

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.: 20, 40, 60, 80

SHEAR STRENGTH Cu., LB./SQ.FT.: 500, 1000, 1500, 2000 — NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

COEFFICIENT OF PERMEABILITY, k., CM./SEC.: 1x10, 1x10, 1x10, 1x10

WATER CONTENT, PERCENT: Wp W WL — 20, 40, 60, 80

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND SURFACE; GRAVEL BACKFILL; BENTONITE SEAL; 3/8" DIA. PLASTIC TUBING; GRAVEL BACKFILL; STANDPIPE

WATER LEVEL IN STANDPIPE AT ELEV. 251.7 ON JAN. 9, 1975

15 – 0 – 5 – 10 Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED RQ

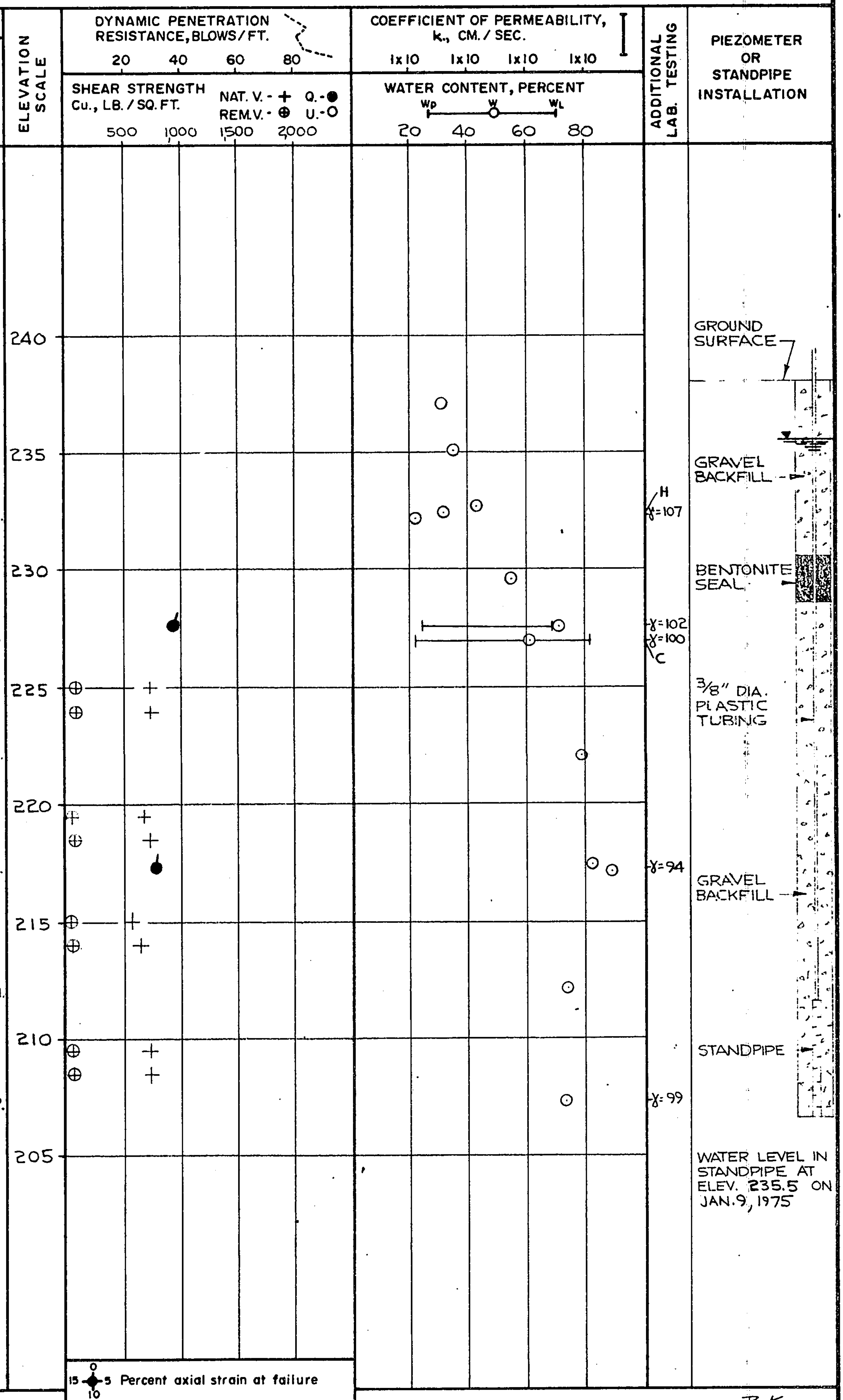
Form G.A.-D-1. Project No. 741[redacted]

RECORD OF BOREHOLE 212

LOCATION See Figure 2 BORING DATE DEC. 5 & 6, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	238.0	GROUND SURFACE				
WASH BORING – HX CASING	0.0	STIFF TO SOFT WEATHERED AND FISSURED RED-BROWN SILTY CLAY, SOME SILT LENSES (DESICCATED CRUST)	1	2" D.O.	6	
			2	"	15	
			3	2" T.O.	PM.	H, γ=107
	229.0 / 9.0		4	2" D.O.	3	
WASH BORING – UNCASED		FIRM SENSITIVE RED-GREY TO GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY	5	3" O.S.	W.P.	γ=102, γ=100, C
			6	2" D.O.	PM.	
			7	3" O.S.	W.P.	γ=94
			8	2" D.O.	PM.	
	206.5		9	3" O.S.	W.P.	γ=99
	31.5	END OF HOLE				



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED BG

Form G.A.-D-1.

Project No. 74[redacted]30

RECORD OF BOREHOLE 213

LOCATION See Figure 2

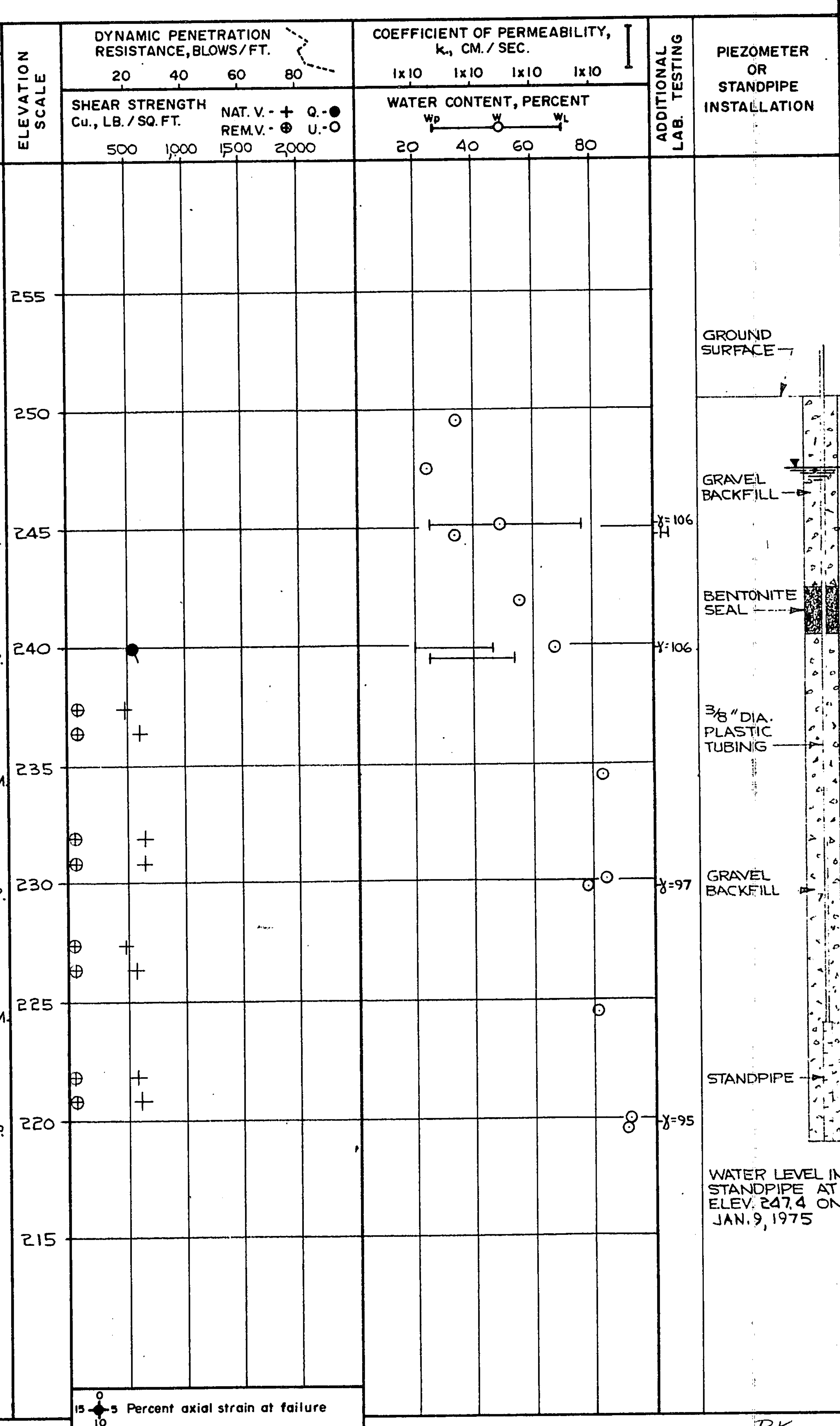
BORING DATE DEC. 6, 1974

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
	250.4	GROUND SURFACE			
WASH BORING – HX CASING	0.0	LOOSE TO COMPACT BROWN SILTY FINE SAND	1	2" D.O.	11
			2	"	7
	246.4 / 4.0	STIFF WEATHERED AND FISSURED RED-BROWN SILTY CLAY, NUMEROUS SILT AND SAND LAYERS (DESICCATED CRUST)	3	2" T.O.	PL.
			4	2" D.O.	2
WASH BORING – UNCASED	240.4 / 10.0	SOFT TO FIRM SENSITIVE GREY AND GREY-RED SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY	5	3" O.S.	W.P.
			6	2" D.O.	PM.
			7	3" O.S.	W.P.
			8	2" D.O.	PM.
			9	3" O.S.	W.P.
	218.9 / 31.5	END OF HOLE			



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED BG

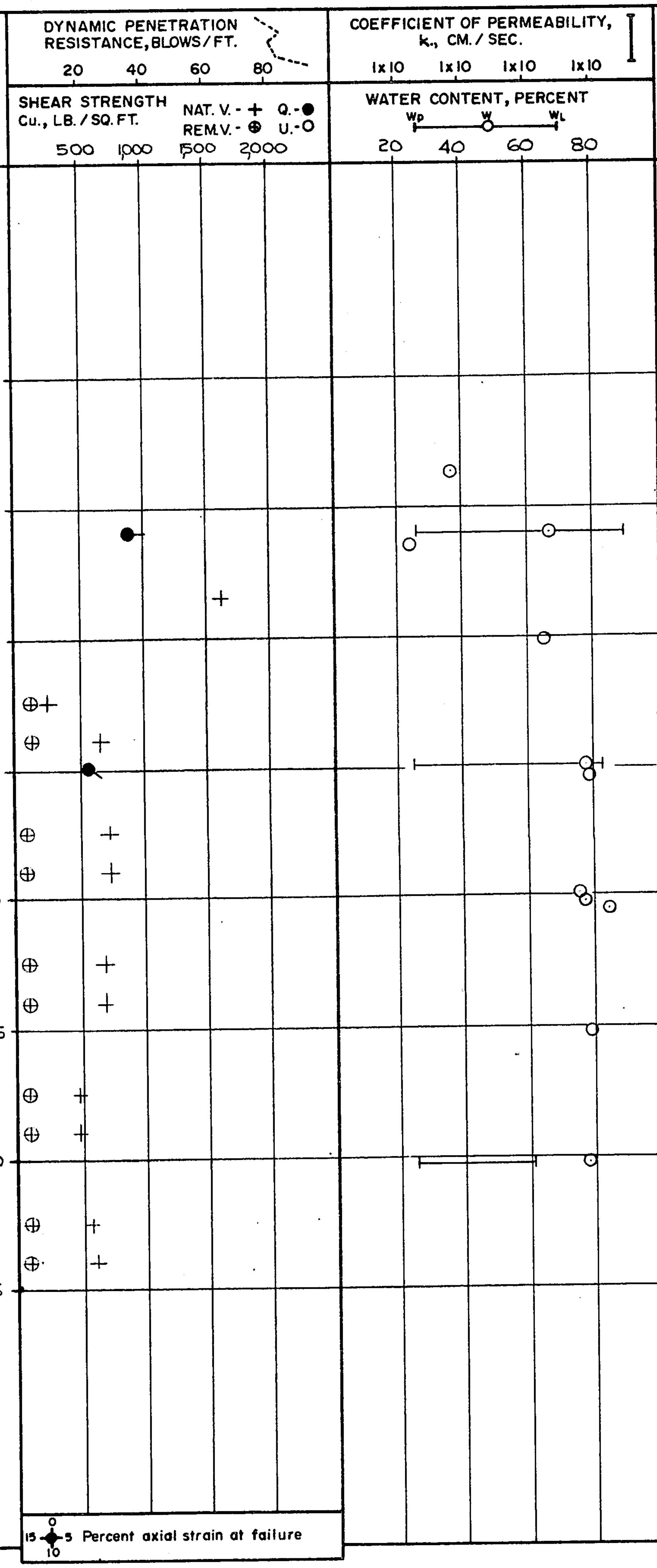
Form G.A.-D-I. Project No. 741[redacted]0

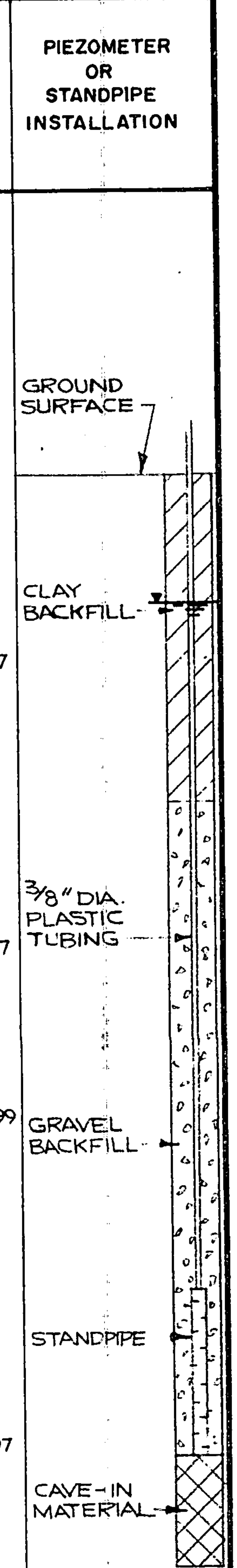
RECORD OF BOREHOLE 214

LOCATION See Figure 2 BORING DATE DEC. 10, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
	249.5	GROUND SURFACE				250	
POWER AUGER 7" DIA. HOLLOW STEM	249.0	BROWN SANDY TOPSOIL					
	0.5 / 247.0	BROWN SILTY SAND					
	2.5	FIRM TO STIFF WEATHERED AND FISSURED RED-BROWN SILTY CLAY, OCCASIONAL SILT LAYERS (DESICCATED CRUST)	1	2" D.O.	14	245	
			2	3" T.P.	PH.		γ=97
	240.5 / 9.0	SOFT TO FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY.	3	2" D.O.	4	240	
			4	3" T.P.	PH.	235	H γ=97
			5	"	"	230	γ=99
			6	2" D.O.	2	225	
			7	3" T.P.	PH.	220	γ=97
	216.0 / 33.5	END OF HOLE				215	





WATER LEVEL IN STANDPIPE AT ELEV. 245.5 ON JAN. 9, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

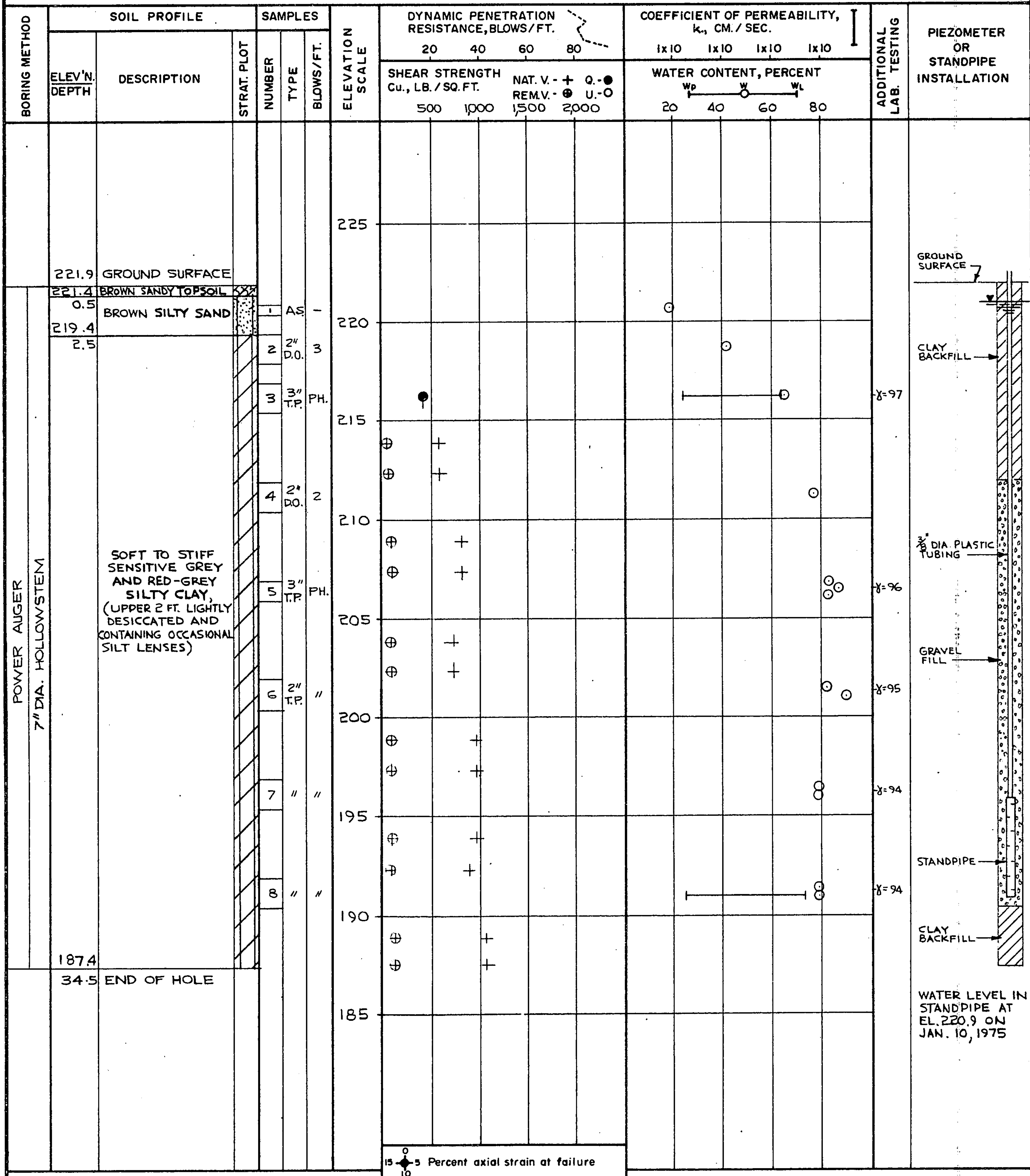
DRAWN RK
CHECKED RG

Form G.A.-D-I. Project No. 74[redacted]

RECORD OF BOREHOLE 215

LOCATION See Figure 2 BORING DATE DEC. 10, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN RK

CHECKED BG

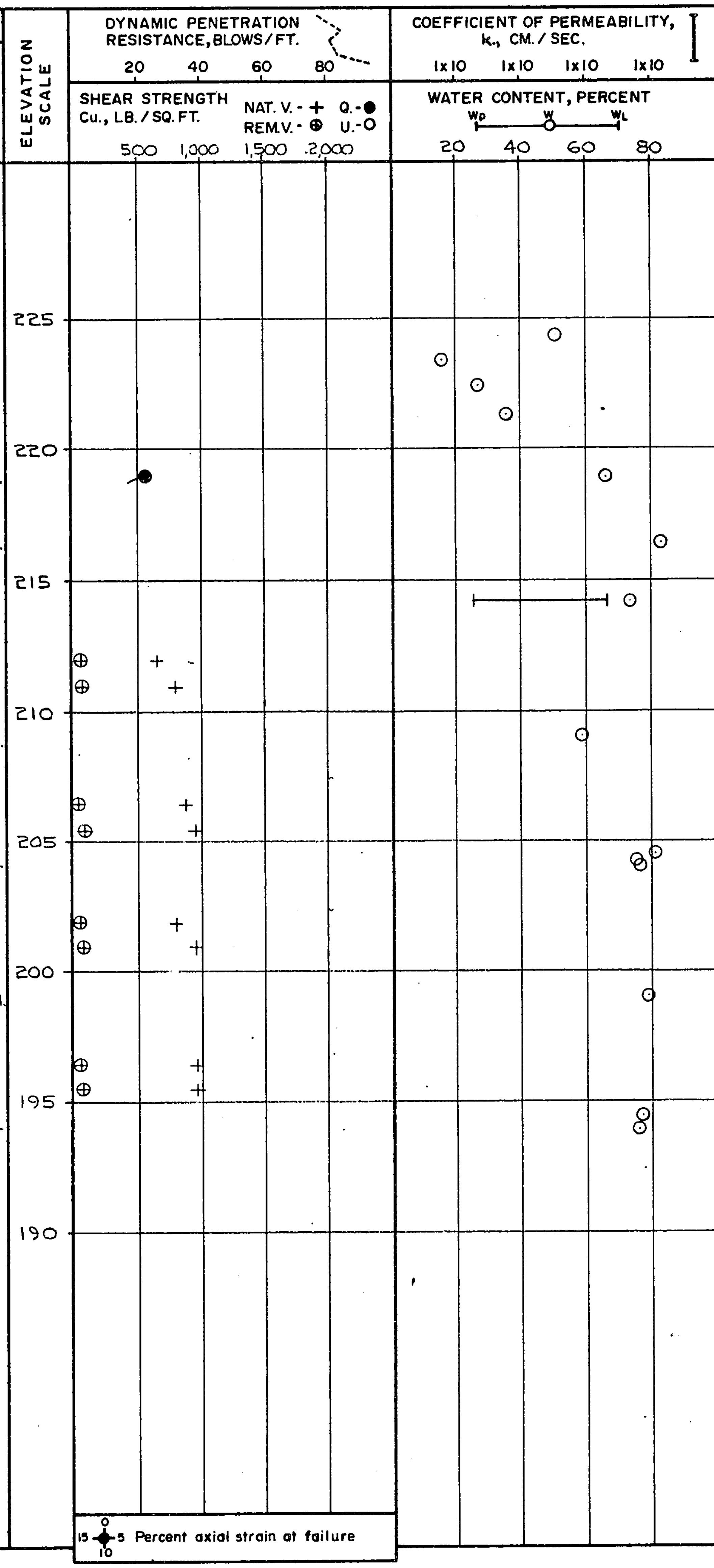
Form G.A.-D-1. [redacted] Project No. 74[redacted]30

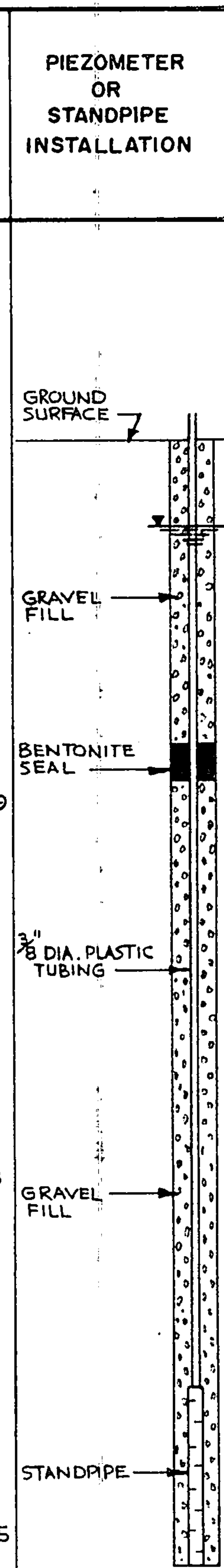
RECORD OF BOREHOLE 216

LOCATION See Figure 2 | BORING DATE DEC. 11, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	224.9	GROUND SURFACE				
WASH BORING – HX CASING	224.4 / 0.5	TOPSOIL				
		LOOSE BROWN SILTY SAND	1	2" D.O.	8	
	221.9 / 3.0		2	"	4	
		FIRM, WEATHERED AND FISSURED MOTTLED GREY AND BROWN SILTY CLAY (DESICCATED CRUST)	3	2" T.O.	P.M.	γ=95
WASH BORING – UNCASED	217.9 / 7.0		4	2" D.O.	P.M.	
		FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY	5	3" O.S.	W.P.	γ=99 C
			6	2" D.O.	P.M.	
			7	3" O.S.	W.P.	γ=98
			8	2" D.O.	P.M.	
	193.4		9	3" O.S.	W.P.	γ=95
	31.5	END OF HOLE				





WATER LEVEL IN STANDPIPE AT EL. 222.4 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED B.G.

Form G.A.-D-1.

Project No. 74[redacted]30

RECORD OF BOREHOLE 217

LOCATION See Figure 2 BORING DATE JAN. 2, 1975 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE: ELEV'N. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT.
	261.6	GROUND SURFACE				
POWER AUGER, 7" DIA. HOLLOW STEM	0.0 / 259.1	SAND AND GRAVEL FILL (ROAD BASE)				
	258.1 / 3.5	COMPACT BLACK ORGANIC SAND		1	2" D.O.	21
		COMPACT TO DENSE BROWN, SLIGHTLY STRATIFIED SILTY FINE SAND		2	"	21
				3	"	21
				4	"	35
	242.6			5	A.S.	–
	19.0	SOFT TO FIRM GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILT LAYERS		6	2" T.P.	P.H.
				7	"	P.H.
	227.1			8	"	P.H.
	34.5	END OF HOLE				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

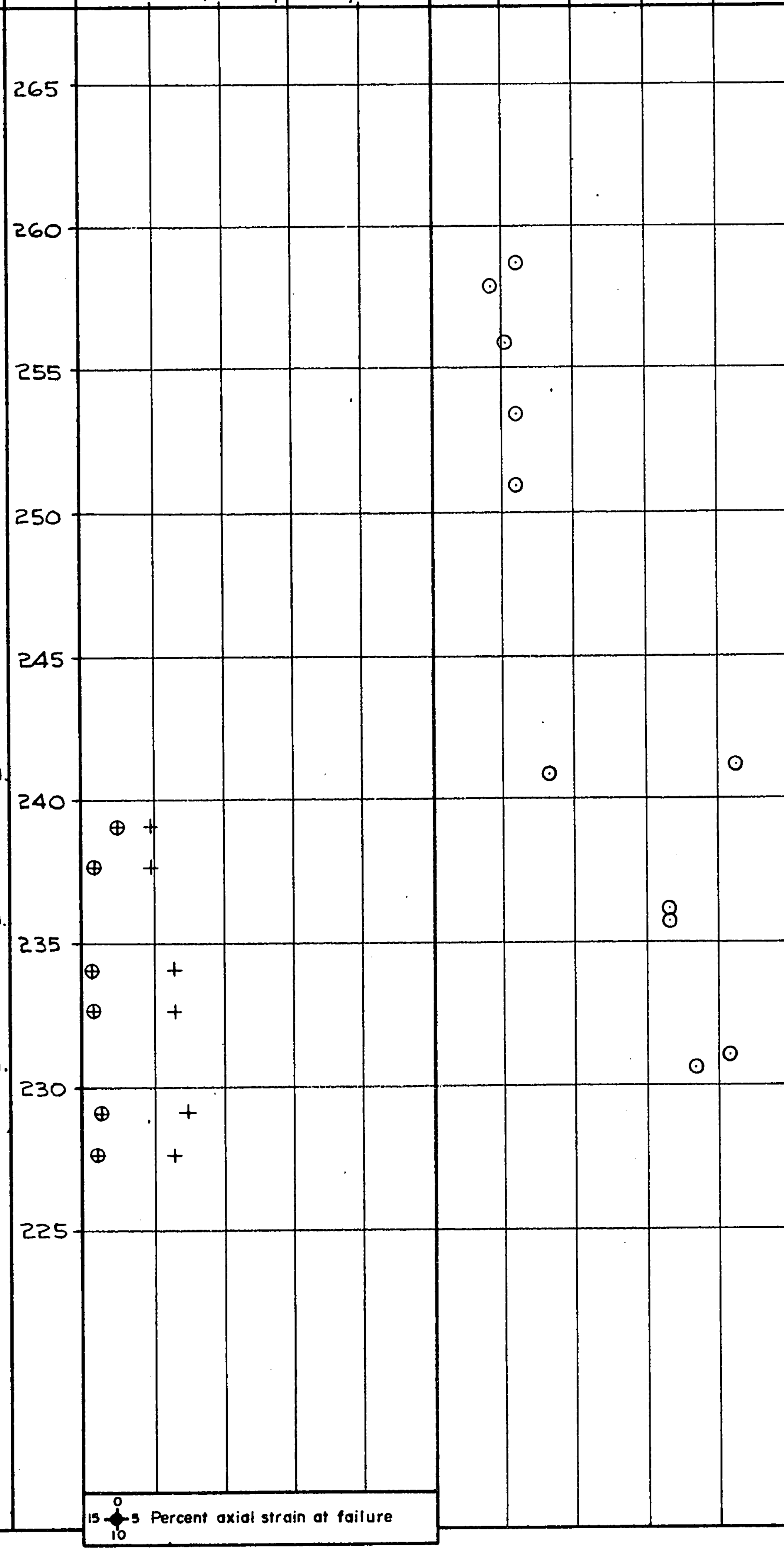
500 1,000 1,500 2,000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

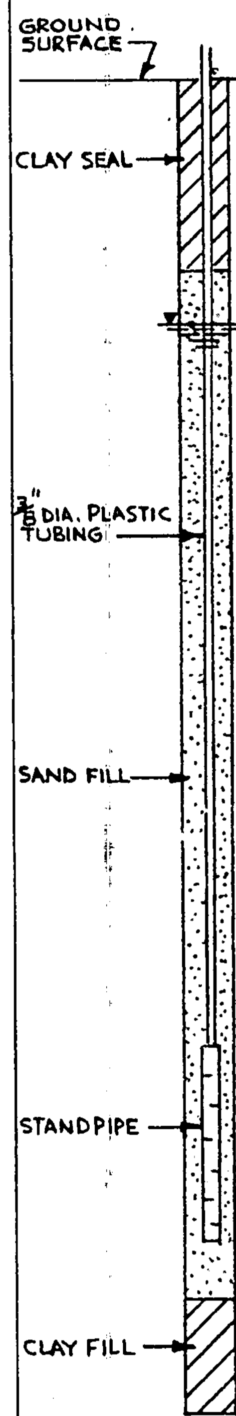
WATER CONTENT, PERCENT Wp W WL

20 40 60 80

ADDITIONAL LAB. TESTING



PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT EL. 255.1 ON JAN. 9, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

Form G.A.-D-1.

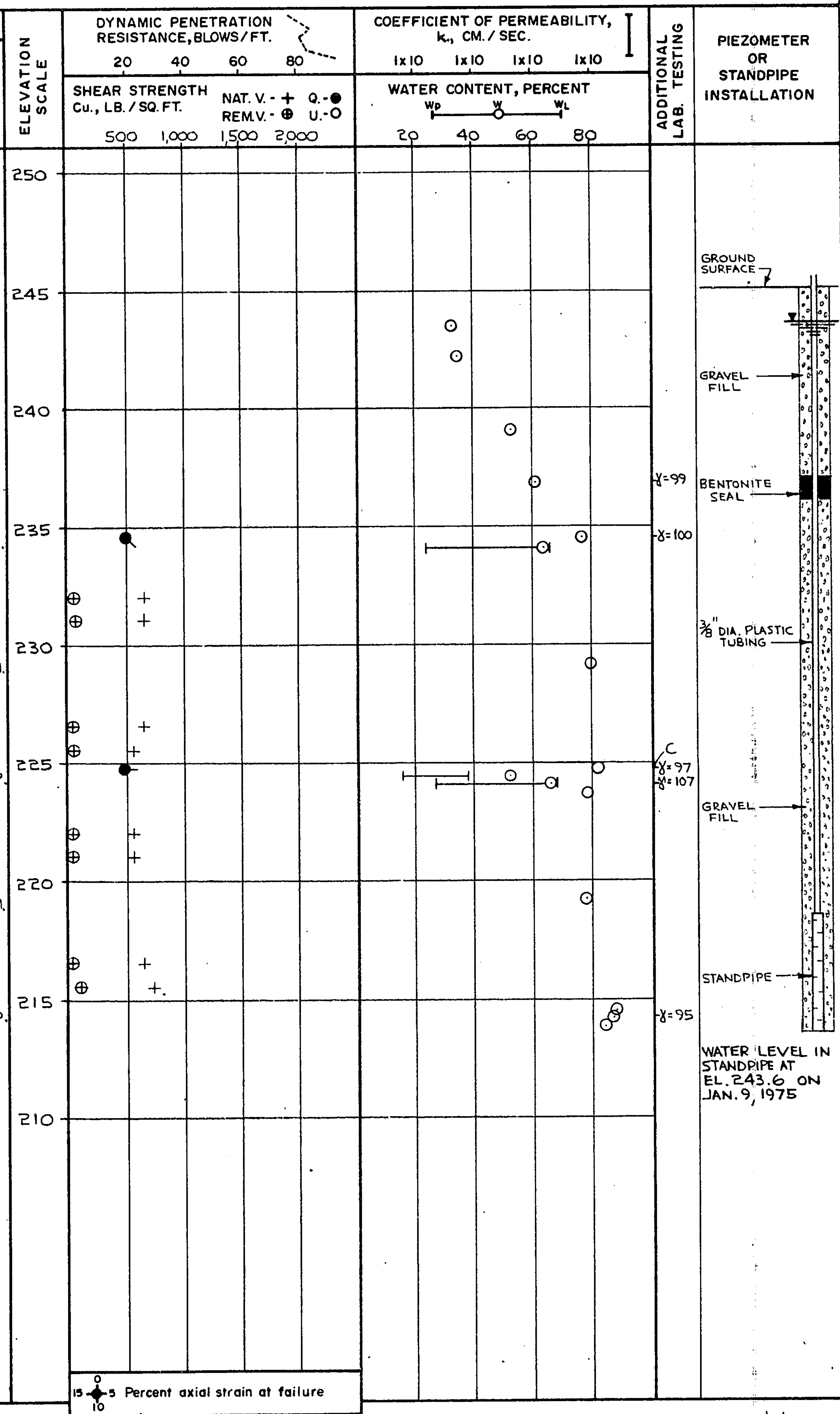
Project No. 7[redacted]30

RECORD OF BOREHOLE 218

LOCATION See Figure 2 | BORING DATE DEC. 9, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.
	245.1	GROUND SURFACE			
WASH BORING – HX CASING	244.6 / 0.5	TOPSOIL	1	2" D.O.	5
		FIRM TO STIFF WEATHERED AND FISSURED RED-BROWN SILTY CLAY, OCCASIONAL SILT AND SAND LAYERS (DESICCATED CRUST)	2	"	17
			3	"	6
			4	2" T.O.	P.L.
WASH BORING – UNCASED	235.1 / 10.0	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY	5	3" O.S.	W.P.
			6	2" D.O.	P.M.
			7	3" O.S.	W.P.
			8	2" D.O.	P.M.
			9	3" O.S.	W.P.
	213.6 / 31.5	END OF HOLE			



WATER LEVEL IN STANDPIPE AT EL. 243.6 ON JAN. 9, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

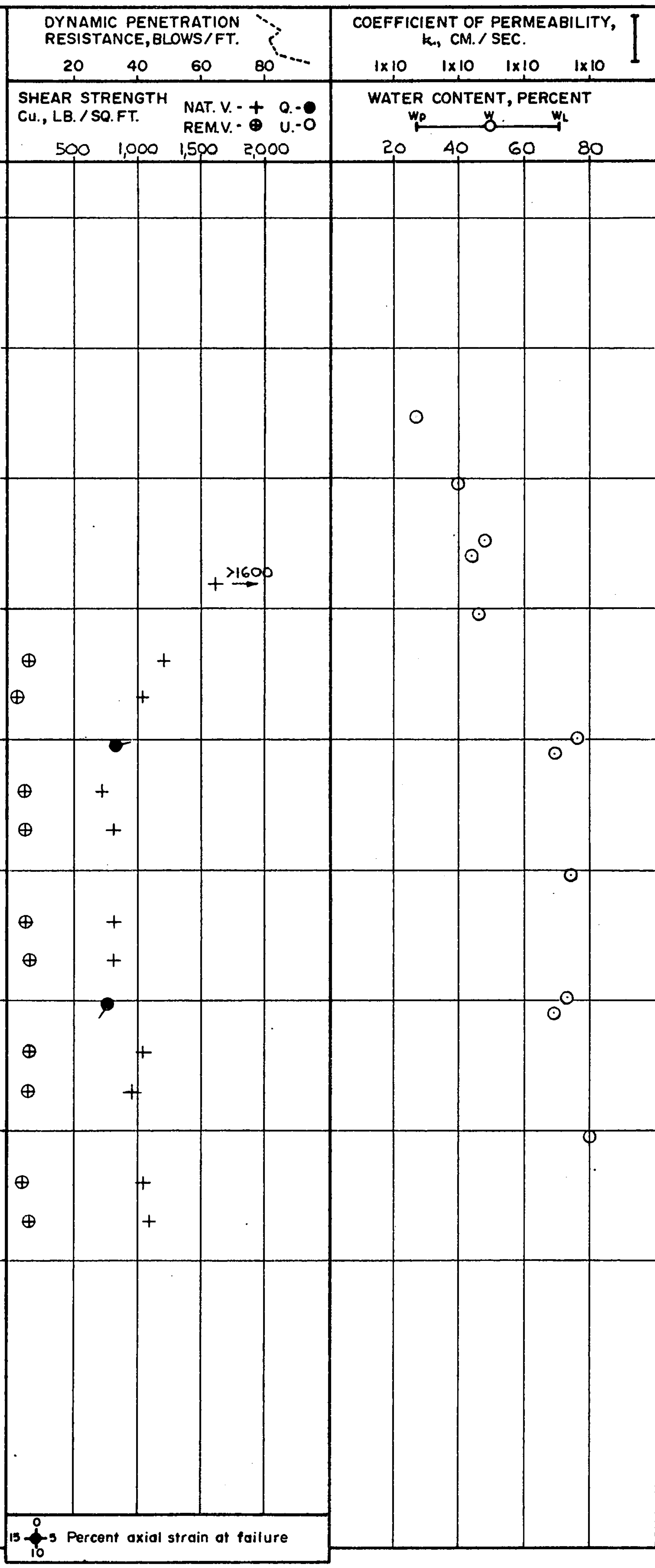
Form G.A.–D–I. Project No. 74[redacted]

RECORD OF BOREHOLE 219

LOCATION See Figure 2 BORING DATE DEC. 11, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
						240	
	235.5	GROUND SURFACE					
POWER AUGER, 7" DIA. HOLLOW STEM	235.0 / 0.5	BROWN SANDY TOPSOIL				235	
		VERY STIFF TO FIRM WEATHERED AND FISSURED GREY-BROWN TO BROWN SILTY CLAY, OCCASIONAL SILT AND SAND LAYERS (DESICCATED CRUST)	1	2" D.O.	18		
			2	"	15	230	
			3	2" T.P.	P.H.		γ=107
			4	2" D.O.	4	225	
	221.5 / 14.0	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY					
			5	2" T.P.	P.H.	220	γ=98
			6	2" D.O.	P.H.	215	
			7	2" T.P.	P.H.	210	γ=98
			8	2" D.O.	3	205	
	201.0 / 34.5	END OF HOLE				200	



PIEZOMETER OR STANDPIPE INSTALLATION

GROUND SURFACE

CLAY BACKFILL

3/8" DIA. PLASTIC TUBING

GRAVEL FILL

STANDPIPE

CLAY BACKFILL

WATER LEVEL IN STANDPIPE AT EL. 229.5 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED RG

Form G.A.-D-I.

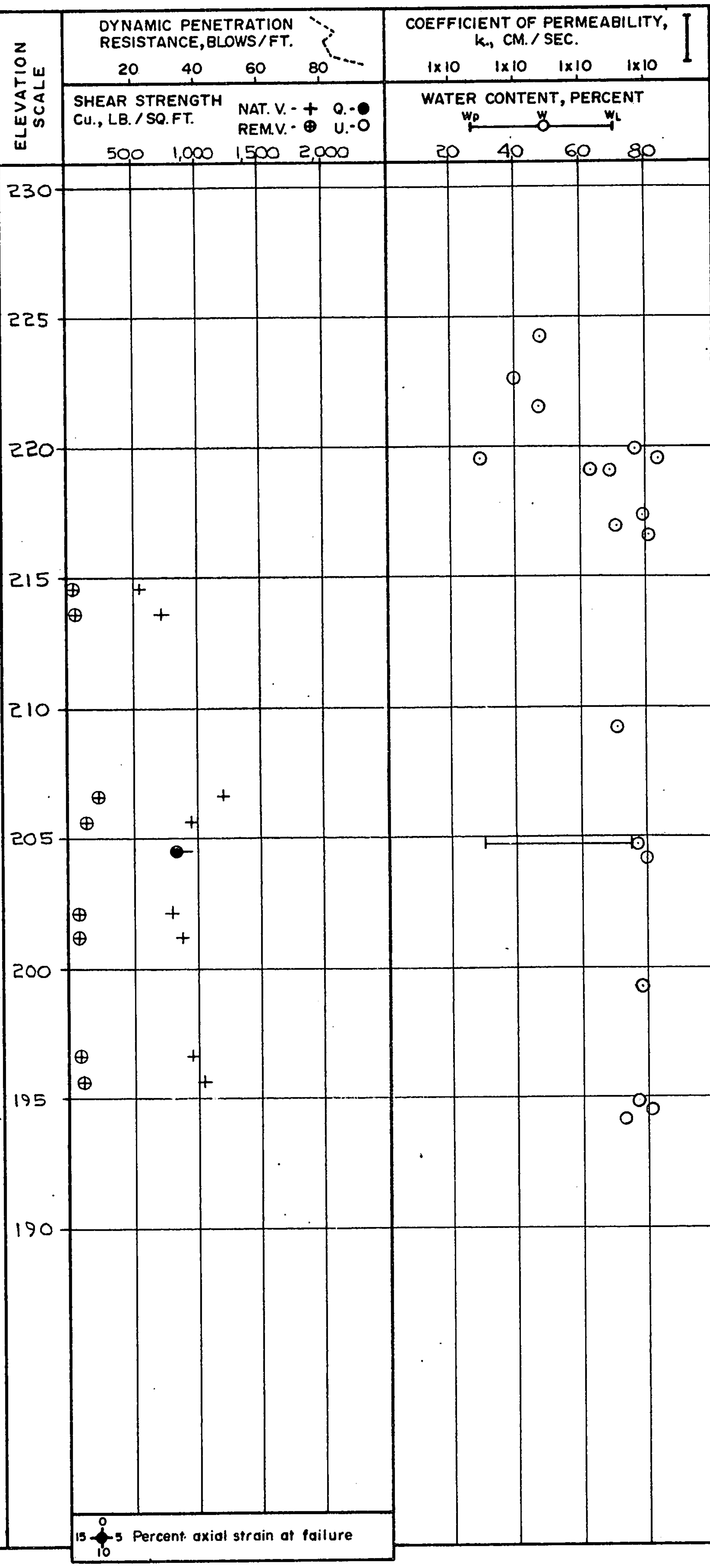
Project No. 74[redacted]80

RECORD OF BOREHOLE 220

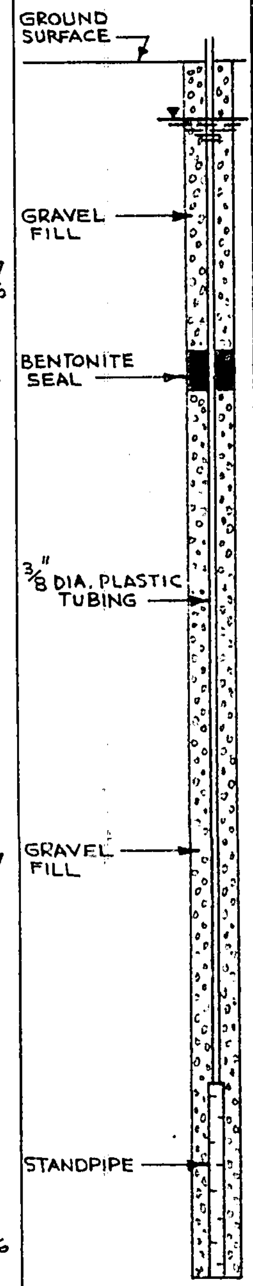
LOCATION See Figure 2 | BORING DATE DEC. 10, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	225.2	GROUND SURFACE				
WASH BORING – HX CASING	0.0	LOOSE TO VERY LOOSE BROWN SILTY SAND	1	2" D.O.	5	
			2	"	3	
	221.7 / 3.5	GREY-BROWN SILTY CLAY (DESICCATED CRUST)				
	219.7 / 5.5	FIRM TO STIFF SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILT AND SAND LAYERS, POCKETS OF BLACK ORGANIC CLAY	3	2" T.O.	P.M.	γ=97, γ=96
			4	3" O.S.	W.P.	γ=98
WASH BORING – UNCASED			5	2" D.O.	P.M.	
			6	3" O.S.	W.P.	H, γ=97
			7	2" D.O.	P.M.	
	193.7		8	3" O.S.	W.P.	γ=96
	31.5	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT EL. 223.7 ON JAN. 9, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED B.G.

Form G.A.-D-1. Project No. 741230

RECORD OF BOREHOLE 221

LOCATION See Figure 2 BORING DATE DEC. 11, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	226.4	GROUND SURFACE				
WASH BORING – HX CASING	225.9 / 0.5	BROWN SANDY TOPSOIL				
		STIFF TO VERY STIFF WEATHERED AND FISSURED RED-BROWN SILTY CLAY (DESICCATED CRUST)	1	2" D.O.	14	
			2	"	20	
			3	2" T.O.	PL	γ=100, H
WASH BORING – UNCASED	218.9 / 7.5	SOFT TO STIFF SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILT SEAMS AND POCKETS OF BLACK ORGANIC CLAY.	4	2" D.O.	PM	
			5	3" O.S.	W.P.	C, γ=99, γ=95
			6	2" D.O.	PM	
			7	3" O.S.	W.P.	γ=97, H
			8	2" D.O.	PM	
			9	3" O.S.	W.P.	γ=96
	194.9 / 31.5	END OF HOLE				

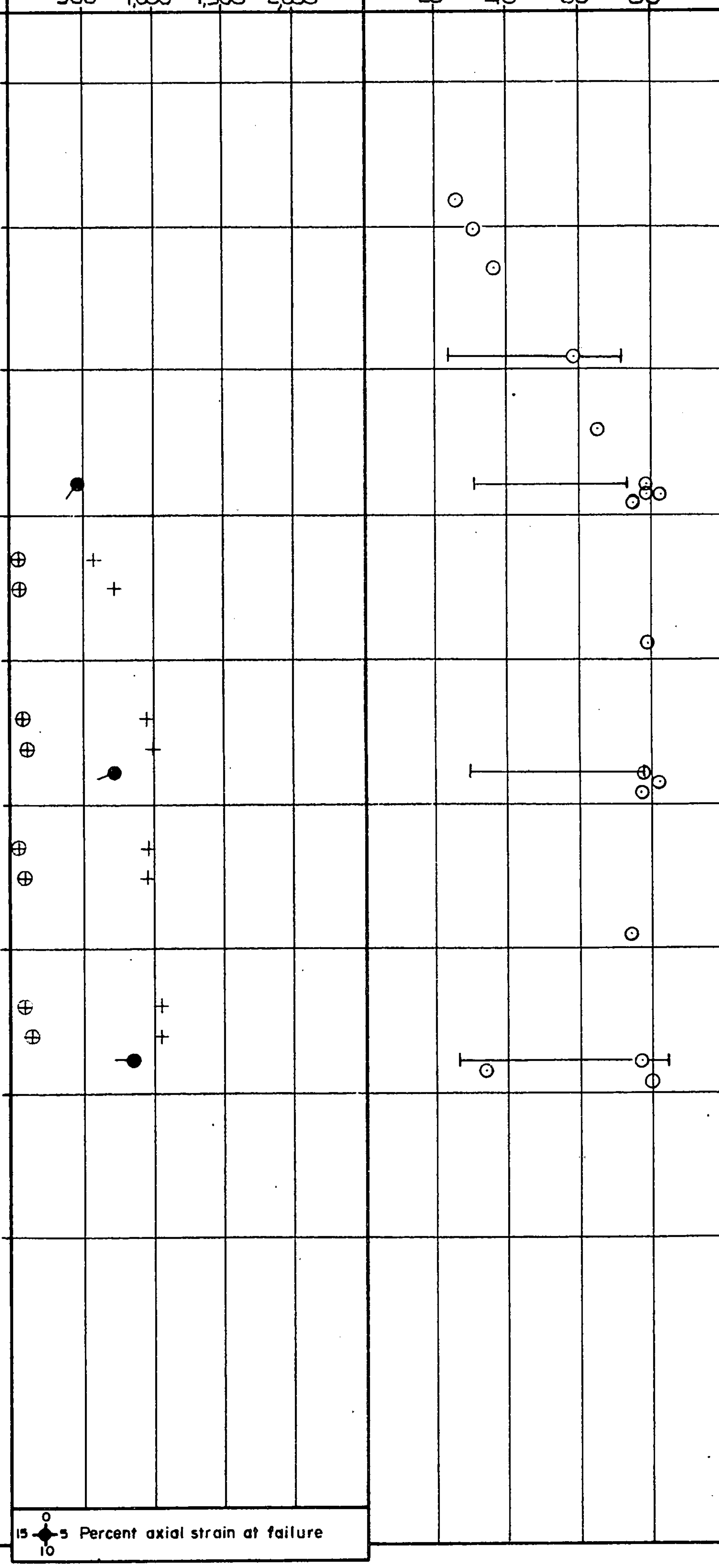
ELEVATION SCALE: 230, 225, 220, 215, 210, 205, 200, 195, 190

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

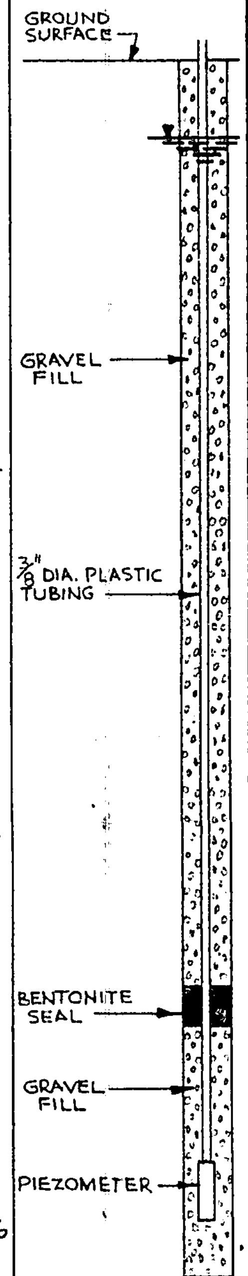
SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O 500 1,000 1,500 2,000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT W_P W W_L 20 40 60 80



PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN PIEZOMETER AT EL. 224.4 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

RECORD OF PENETRATION TEST 221

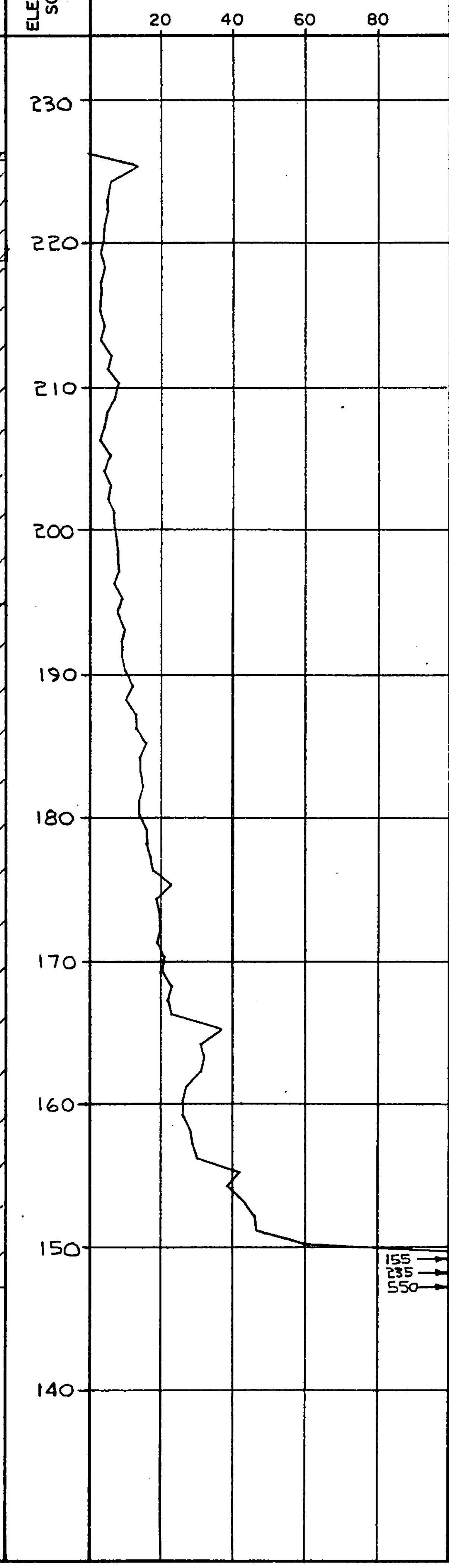
LOCATION See Figure 2 DRIVING DATE JAN. 6, 1975 DATUM GEODETIC

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST No. 221

ELEVN. / DEPTH	INFERRED SOIL PROFILE
	PROBABLY TOPSOIL
226.4	GROUND SURFACE
0.5	PROBABLY STIFF TO VERY STIFF SILTY CLAY (DESICCATED CRUST)
218.9 / 7.5	PROBABLY SOFT TO STIFF SENSITIVE SILTY CLAY
194.9 / 31.5	END OF BOREHOLE 221
	PROBABLY FIRM TO STIFF SENSITIVE SILTY CLAY
147.4 / 79.0	END OF PEN. TEST

ELEVATION SCALE — DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.



PENETRATION TEST No.

ELEVN. / DEPTH	INFERRED SOIL PROFILE	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN J.A.
CHECKED BG

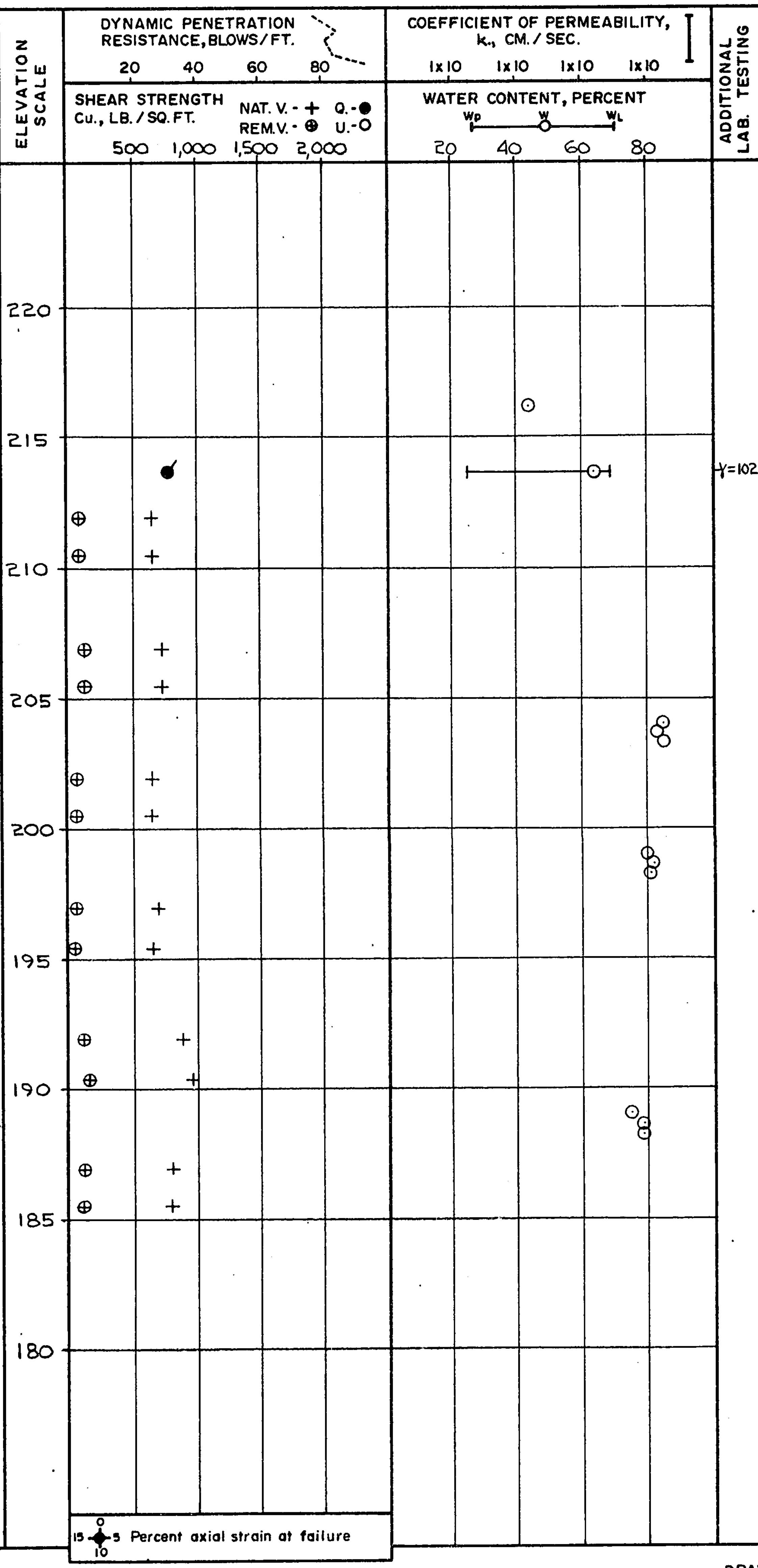
Form G.A.-D-1.   Project No. 74[redacted]30

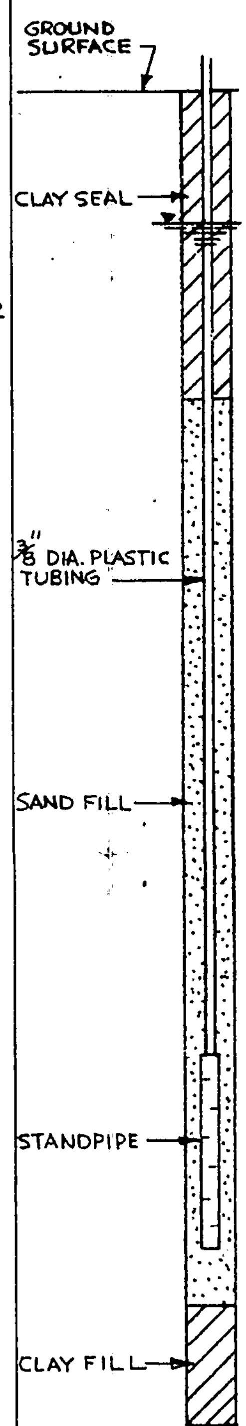
RECORD OF BOREHOLE 222

LOCATION See Figure 2   BORING DATE JAN. 2-3, 1975   DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.   PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
	219.4	GROUND SURFACE			
POWER AUGER, 7" DIA. HOLLOW STEM	0.0	BLACK ORGANIC SILTY CLAY			
	216.4 / 3.0	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY.	1	2" D.O.	6
			2	3" T.P.	P.H.
			3	2" D.O.	P.M.
			4	3" T.P.	P.H.
			5	2" T.P.	P.H.
			6	"	P.H.
			7	"	P.H.
	184.9 / 34.5	END OF HOLE			





WATER LEVEL IN STANDPIPE AT EL. 215.9 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.
CHECKED BG

Form G.A.-D-1. Project No. 74[redacted]30

RECORD OF BOREHOLE 223

LOCATION See Figure 2 BORING DATE DEC. 13, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	232.1	GROUND SURFACE				
WASH BORING — HX CASING	0.0 / 230.1	LOOSE BROWN SILTY SAND	1	2" D.O.	7	
WASH BORING — HX CASING	2.0	STIFF WEATHERED AND FISSURED GREY-BROWN SILTY CLAY, NUMEROUS SILT AND SAND LAYERS (DESICCATED CRUST)	2	"	16	
			3	2" T.O.	P.L.	γ=104
			4	2" D.O.	P.L.	
WASH BORING — UNCASED	222.1 / 10.0	SOFT TO FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY.	5	3" O.S.	W.P.	γ=97, C, γ=93
			6	2" D.O.	P.M.	
			7	3" O.S.	W.P.	γ=116
			8	2" D.O.	P.M.	
	200.6		9	3" O.S.	W.P.	γ=95
	31.5	END OF HOLE				

ELEVATION SCALE: 235, 230, 225, 220, 215, 210, 205, 200

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

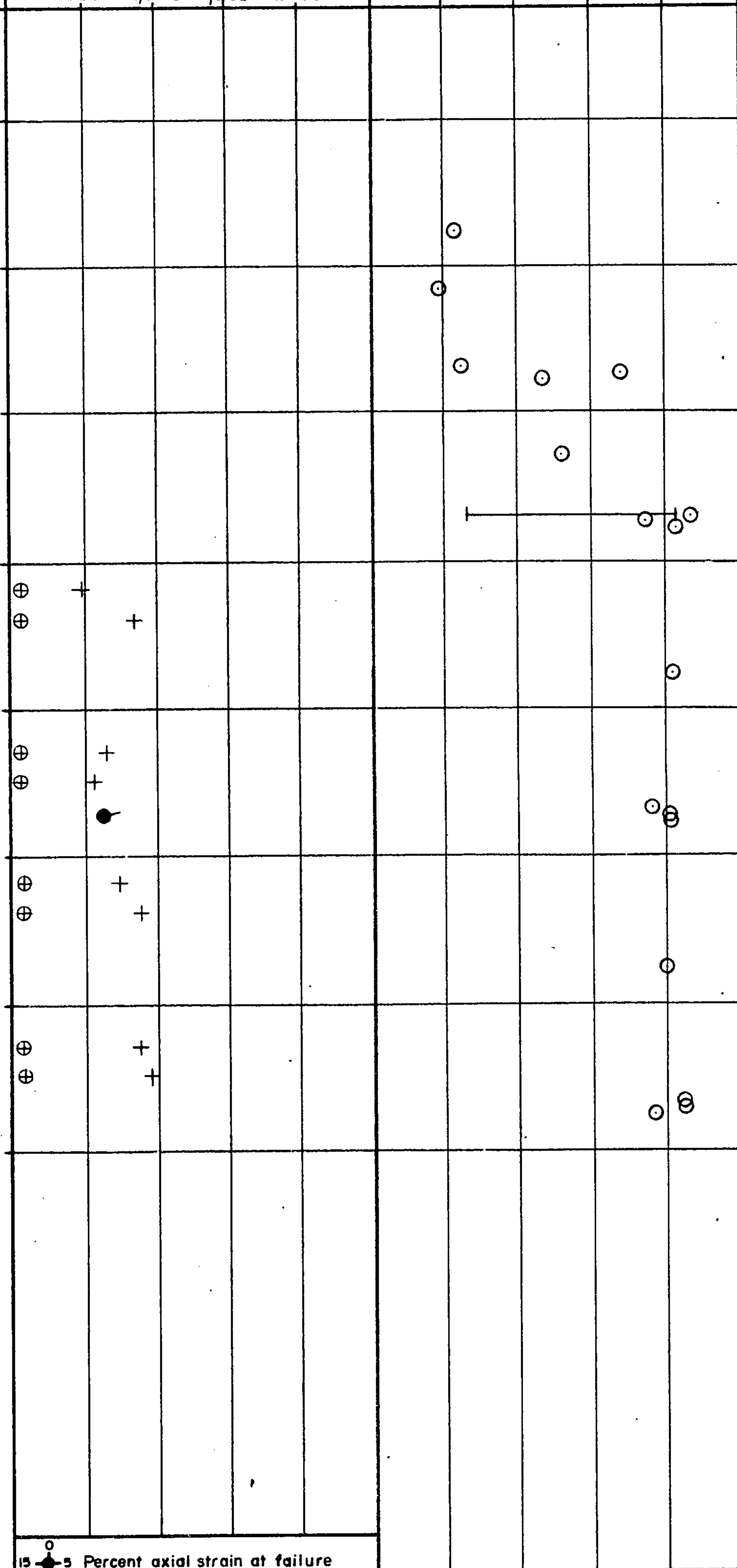
SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

500 1,000 1,500 2000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

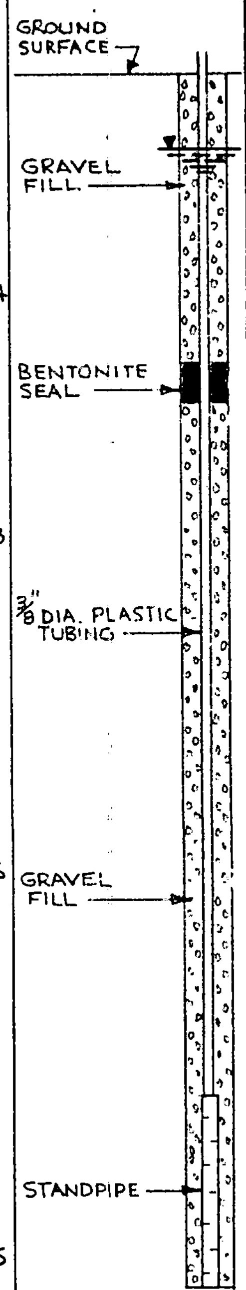
WATER CONTENT, PERCENT Wp W WL

20 40 60 80



15 10 0 5 Percent axial strain at failure

PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT EL. 230.1 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

Form G.A.-D-1. Project No. 74[redacted]30

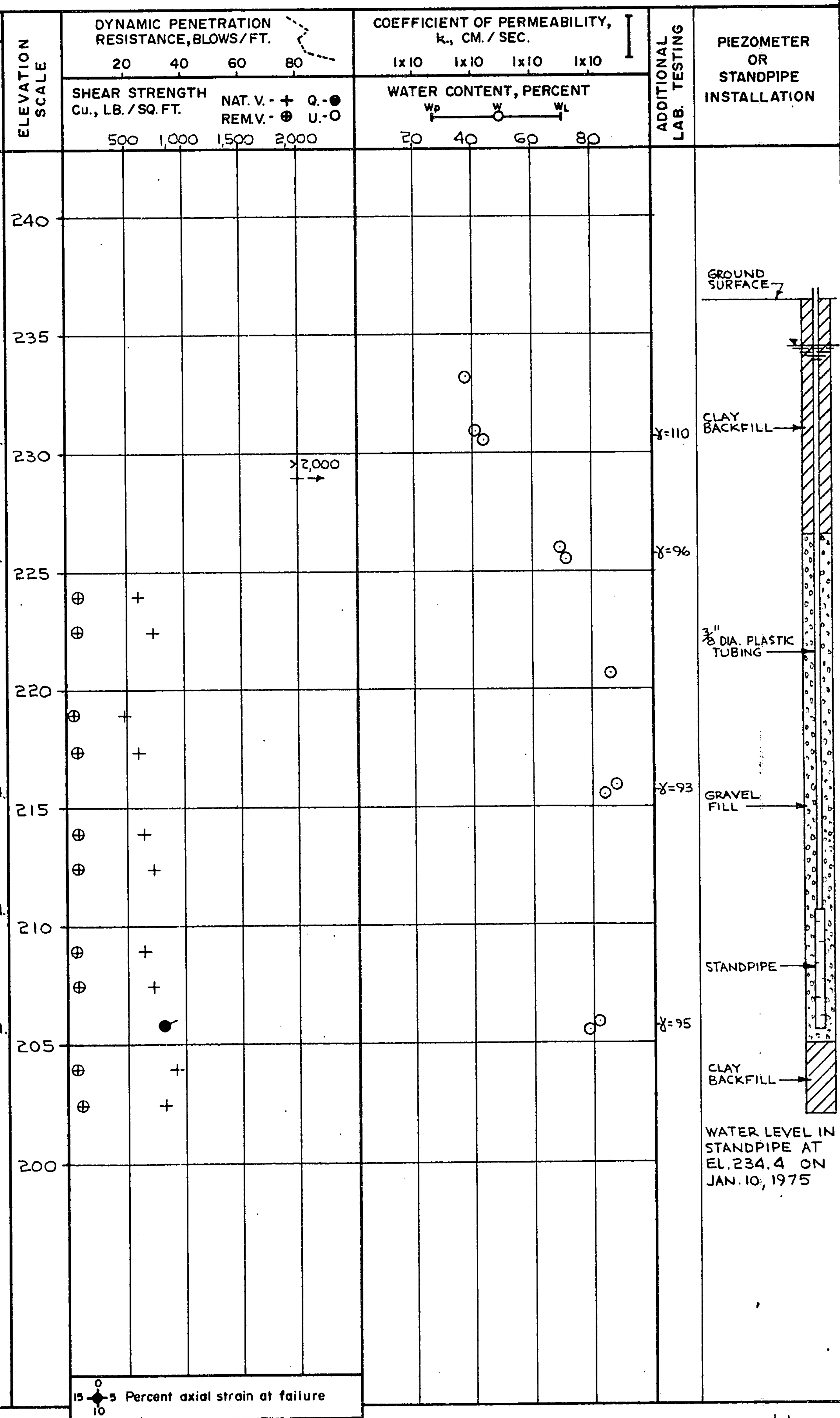
RECORD OF BOREHOLE 224

LOCATION See Figure 2 BORING DATE DEC. 12, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION (SOIL PROFILE)
	236.4	GROUND SURFACE
POWER AUGER, 7" DIA. HOLLOW STEM	235.9 / 0.5	BROWN CLAYEY TOPSOIL
		STIFF TO VERY STIFF WEATHERED AND FISSURED GREY-BROWN SILTY CLAY, SOME THIN SILT LAYERS (DESICCATED CRUST)
	227.4 / 9.0	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL THIN SILT LAYERS
	201.9 / 34.5	END OF HOLE

SAMPLES NUMBER	TYPE	BLOWS/FT.
1	2" D.O.	12
2	2" T.P.	P.H.
3	2" T.P.	P.H.
4	2" D.O.	2
5	2" T.P.	P.H.
6	2" D.O.	P.M.
7	2" T.P.	P.H.



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.
CHECKED BG

Form G.A.-D-1.

Project No. 74[illegible]30

RECORD OF BOREHOLE 225

LOCATION See Figure 2   BORING DATE DEC. 12, 1974   DATUM GEODETIC

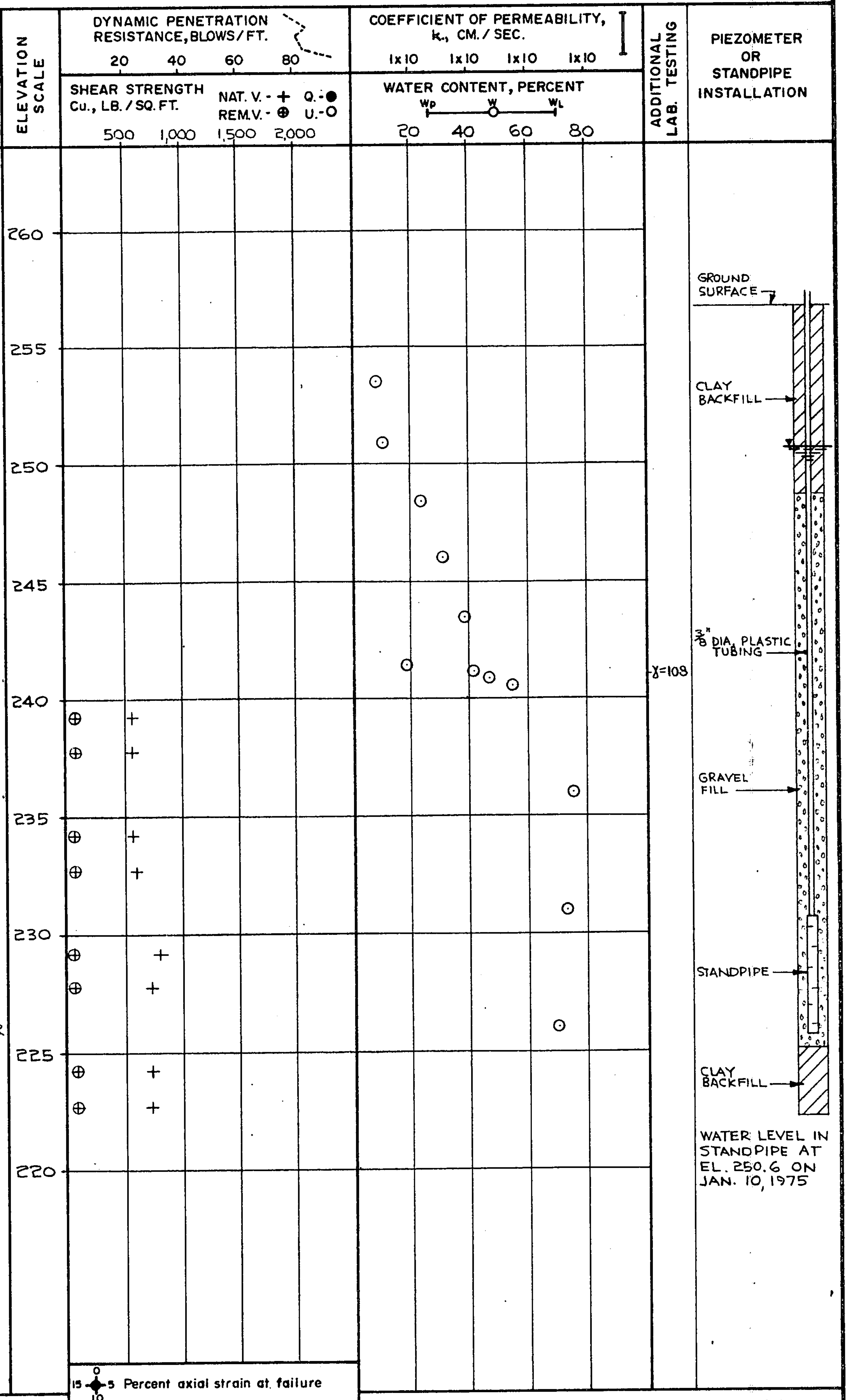
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.   PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: POWER AUGER, 7" DIA. HOLLOW STEM

ELEV'N. / DEPTH	SOIL PROFILE DESCRIPTION
256.6	GROUND SURFACE
256.1 / 0.5	BROWN SANDY TOPSOIL
244.1 / 12.5	COMPACT TO DENSE BROWN STRATIFIED SAND TO SILTY FINE SAND
222.1 / 34.5	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, TRACE GRAVEL, OCCASIONAL SILT AND SAND LAYERS
	END OF HOLE

SAMPLES NUMBER	TYPE	BLOWS/FT.
1	2" D.O.	9
2	"	30
3	"	22
4	"	12
5	"	3
6	3" T.P.	P.H.
7	2" T.P.	P.H.
8	2" D.O.	2
9	"	W.R.

ADDITIONAL LAB. TESTING: γ=108



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.

CHECKED BG

Form G.A.-D-1.

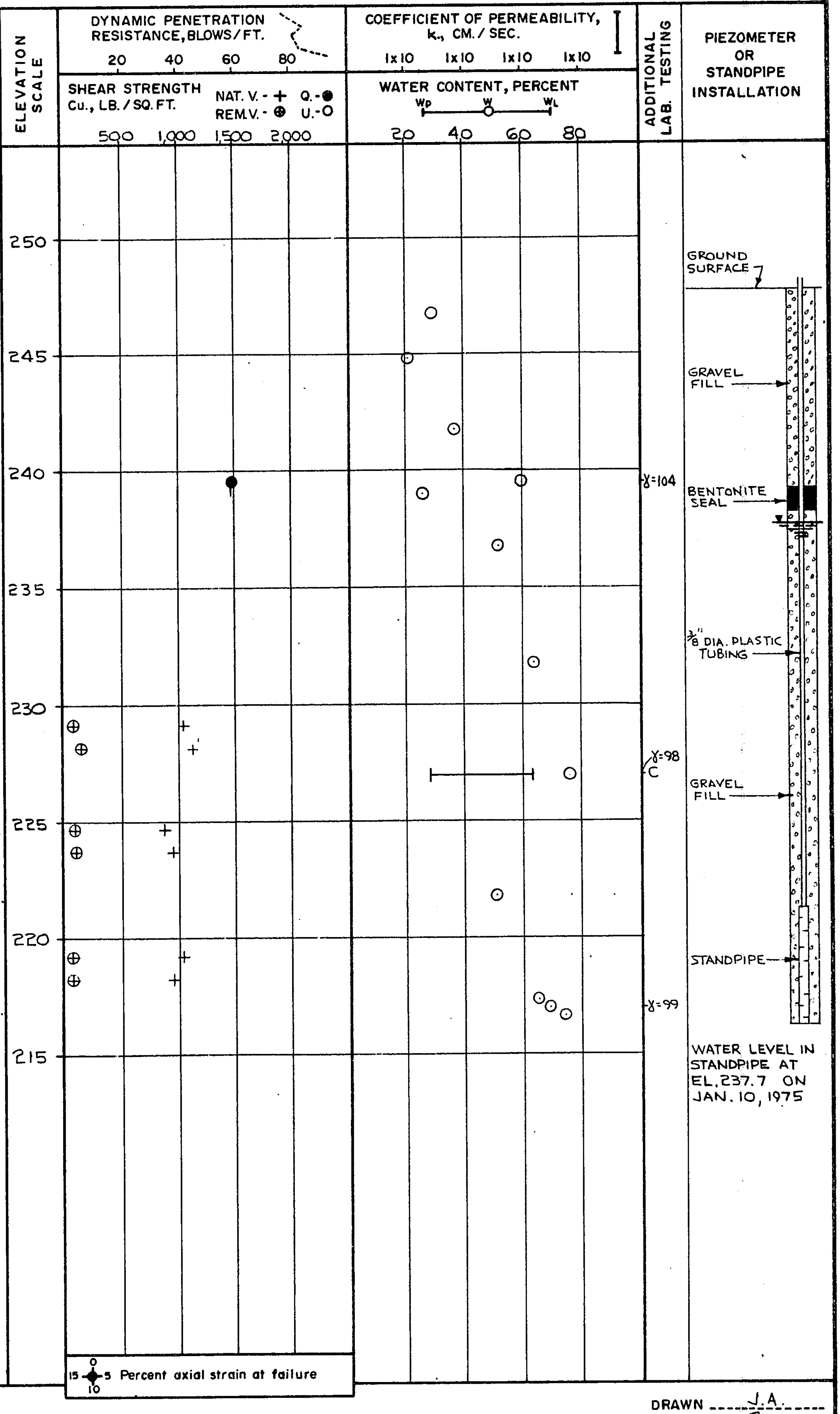
Project No. 741530

RECORD OF BOREHOLE 226

LOCATION See Figure 2 BORING DATE DEC. 15, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.
	247.7	GROUND SURFACE			
WASH BORING – HX CASING		LOOSE TO COMPACT BROWN SILTY SAND	1	2" D.O.	8
			2	"	16
	242.7 5.0	STIFF RED-BROWN SILTY CLAY BECOMING STIFF TO FIRM SENSITIVE GREY SILTY CLAY, OCCASIONAL THIN SILT LAYERS WITH DEPTH	3	"	7
			4	2" T.O.	P.L.
WASH BORING – UNCASED			5	2" D.O.	6
			6	"	4
			7	3" O.S.	W.P.
			8	2" D.O.	P.M.
			9	3" T.O.	P.H.
	216.2 31.5	END OF HOLE			



VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN J.A.
CHECKED BG

Form G.A.–D–I. Project No. 7[redacted]3[redacted]

RECORD OF BOREHOLE 227

LOCATION See Figure 2 BORING DATE DEC. 14, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	230.2	GROUND SURFACE				
WASH BORING, HX CASING	0.0	COMPACT TO VERY LOOSE BROWN SILTY SAND TO SAND	1	2" D.O.	14	
			2	"	10	
			3	"	7	
	221.7 / 8.5		4	"	3	
WASH BORING, UNCASED		FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY.	5	3" O.S.	W.P.	γ=94
			6	2" D.O.	P.M.	
			7	3" O.S.	W.P.	γ=95
			8	2" D.O.	P.M.	
	198.7		9	3" O.S.	W.P.	γ=94
	31.5	END OF HOLE				

ELEVATION SCALE: 235, 230, 225, 220, 215, 210, 205, 200, 195

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu.; LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O 500 1,000 1,500 2,000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT W_P W W_L 20 40 60 80

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND SURFACE; GRAVEL FILL; BENTONITE SEAL; 3/8" DIA PLASTIC TUBING; GRAVEL FILL; STANDPIPE

WATER LEVEL IN STANDPIPE AT EL. 225.2 ON JAN. 10, 1975

0 / 15 - 5 / 10 Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

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Form G.A.-D-1.

Project No. 74[redacted]30

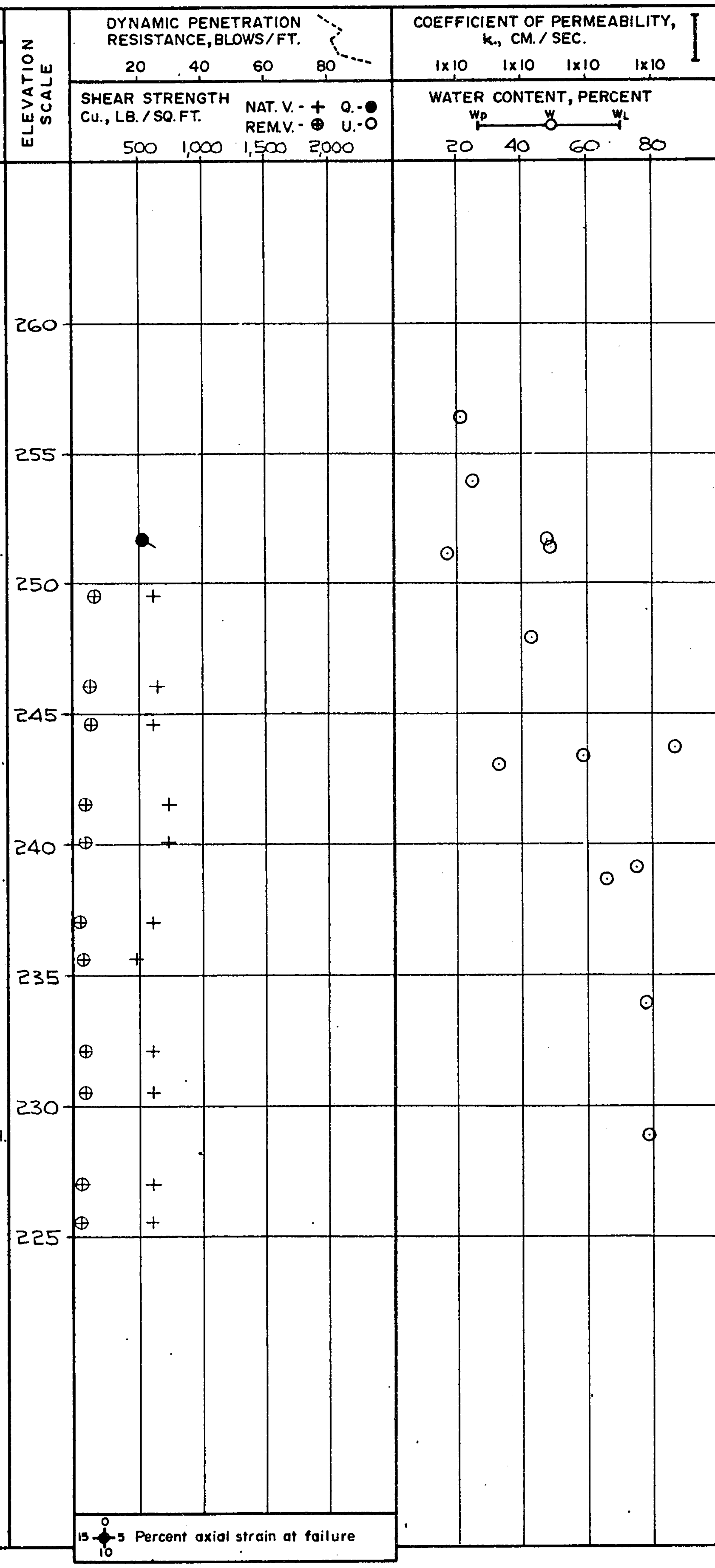
RECORD OF BOREHOLE 228

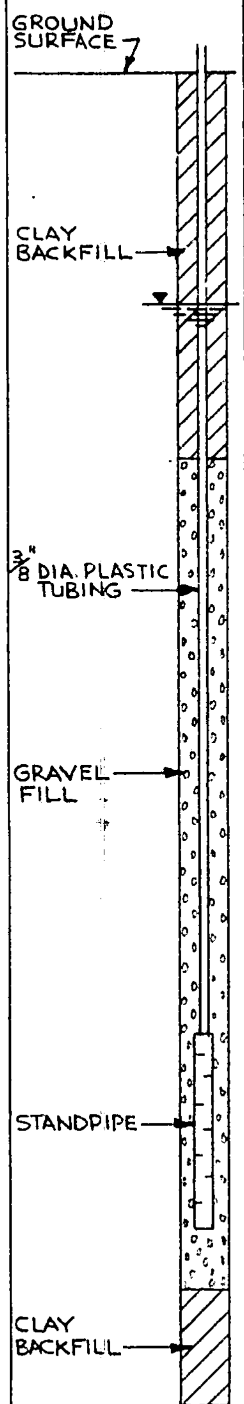
LOCATION See Figure 2 BORING DATE DEC. 11, 1974 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD: POWER AUGER, 7" DIA. HOLLOW STEM

ELEV'N. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
259.6	GROUND SURFACE				
259.1 / 0.5	BROWN SANDY TOPSOIL				
	COMPACT BROWN TO GREY SAND, TRACE SILT.	1	2" D.O.	20	
		2	"	14	
251.1		3	3" T.P.	P.H.	γ=110, γ=114
250.1 / 9.5	RED-BROWN SILTY CLAY (DESICCATED CRUST)				
	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILT LAYERS AND POCKETS OF BLACK ORGANIC CLAY	4	2" D.O.	2	
		5	2" T.P.	P.H.	γ=103
		6	"	P.H.	γ=101
		7	2" D.O.	3	
		8	2" T.P.	P.H.	γ=98
225.1 / 34.5	END OF HOLE				





WATER LEVEL IN STANDPIPE AT EL. 253.6 ON JAN. 10, 1975.

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

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CHECKED B.G.

Form G.A.-D-1. [redacted] Project No. 74[redacted]0 [redacted]

RECORD OF BOREHOLE 229

LOCATION See Figure 2 — BORING DATE DEC. 12 & 13, 1974 — DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. — PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
	260.9	GROUND SURFACE					
WASH BORING – HX CASING		LOOSE TO DENSE RED-BROWN TO GREY SILTY SAND TO SAND	1	2" D.O.	8	260	MH
			2	"	26		
			3	"	36	255	
			4	"	13		
	250.4 / 10.5		5	3" O.S.	W.P.	250	γ=96, C
WASH BORING – UNCASED		SOFT TO FIRM SENSITIVE GREY AND RED-GREY SILTY CLAY	6	2" D.O.	P.M.	245	
			7	3" O.S.	W.P.	240	γ=95
			8	2" D.O.	P.M.	235	
	229.4		9	3" O.S.	W.P.	230	γ=91
	31.5	END OF HOLE					

Elevation scale: 265, 260, 255, 250, 245, 240, 235, 230, 225

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.: 20, 40, 60, 80

SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O: 500, 1,000, 1,500, 2,000

COEFFICIENT OF PERMEABILITY, k., CM./SEC.: 1x10, 1x10, 1x10, 1x10

WATER CONTENT, PERCENT (W_p, W, W_L): 20, 40, 60, 80

15 – 0 – 5 – 10: Percent axial strain at failure

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND SURFACE; GRAVEL FILL; 3/8" DIA. PLASTIC TUBING; BENTONITE SEAL; GRAVEL FILL; STANDPIPE

WATER LEVEL IN STANDPIPE AT EL. 258.9 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

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Form G.A.-D-1.

Project No. 74[illegible]30

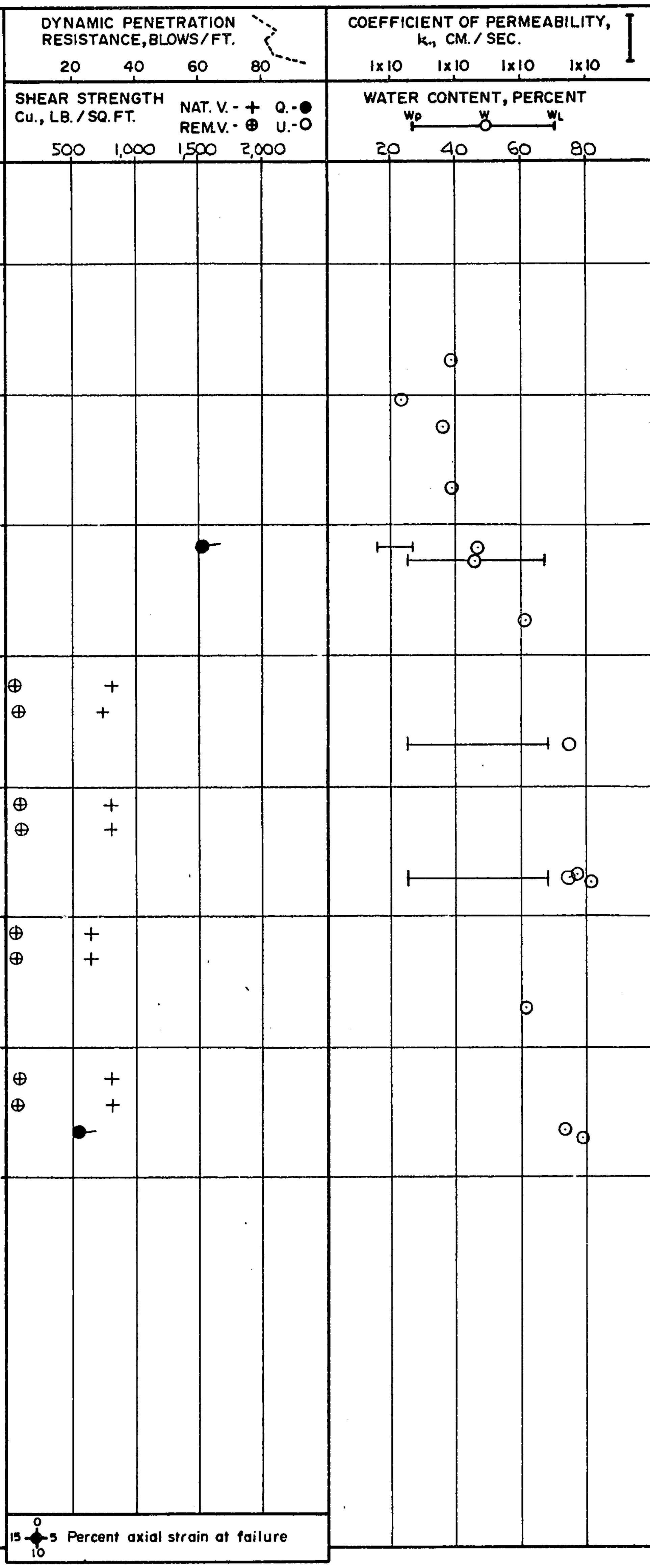
RECORD OF BOREHOLE 230

LOCATION See Figure 2 BORING DATE DEC. 16, 1974 DATUM GEODETIC

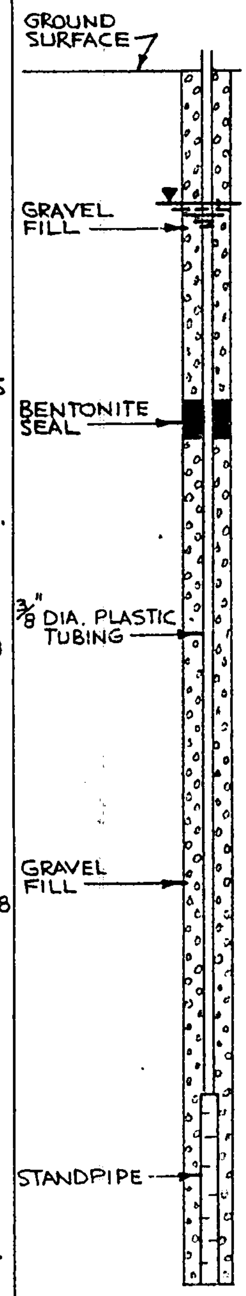
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	247.3	GROUND SURFACE				
HX CASING	246.8	TOPSOIL				
HX CASING	0.5	LOOSE RED-BROWN SILTY SAND	1	2" D.O.	6	
HX CASING	244.3 3.0	STIFF, WEATHERED AND FISSURED RED-BROWN SILTY CLAY, OCCASIONAL GREY CLAYEY SILT TO SILT LAYERS (DESICCATED CRUST)	2	"	12	
HX CASING			3	"	4	
WASH BORING UNCASED			4	2" T.O.	P.L.	H, γ=105
WASH BORING UNCASED	236.3 11.0	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY.	5	2" D.O.	P.M.	
WASH BORING UNCASED			6	3" O.S.	W.P.	γ=98, C
WASH BORING UNCASED			7	"	W.P.	γ=96, γ=98
WASH BORING UNCASED			8	2" D.O.	P.M.	
WASH BORING UNCASED	215.8		9	3" O.S.	W.P.	γ=96
	31.5	END OF HOLE				

ELEVATION SCALE: 250, 245, 240, 235, 230, 225, 220, 215



PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT EL. 243.8 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

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Form G.A.-D-1.   Project No. 74[redacted]30

RECORD OF BOREHOLE 231

LOCATION See Figure 2 | BORING DATE DEC. 17, 1974 | DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. | PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	252.3	GROUND SURFACE				
WASH BORING – HX CASING	0.0	COMPACT BROWN SILTY FINE SAND, NUMEROUS ROOTS	1	2" D.O.	20	
	250.3 / 2.0	STIFF TO FIRM WEATHERED AND FISSURED RED-BROWN SILTY CLAY (DESICCATED CRUST)	2	"	19	
			3	"	5	
			4	2" T.O.	P.L.	γ=108
WASH BORING – UNCASED	242.8 / 9.5	FIRM, SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL POCKETS OF BLACK ORGANIC CLAY	5	3" O.S.	W.P.	C γ=99
			6	2" D.O.	P.M.	
			7	3" O.S.	W.P.	γ=95
			8	2" D.O.	P.M.	
	220.8		9	3" O.S.	W.P.	γ=97
	31.5	END OF HOLE				

ELEVATION SCALE: 255, 250, 245, 240, 235, 230, 225, 220

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

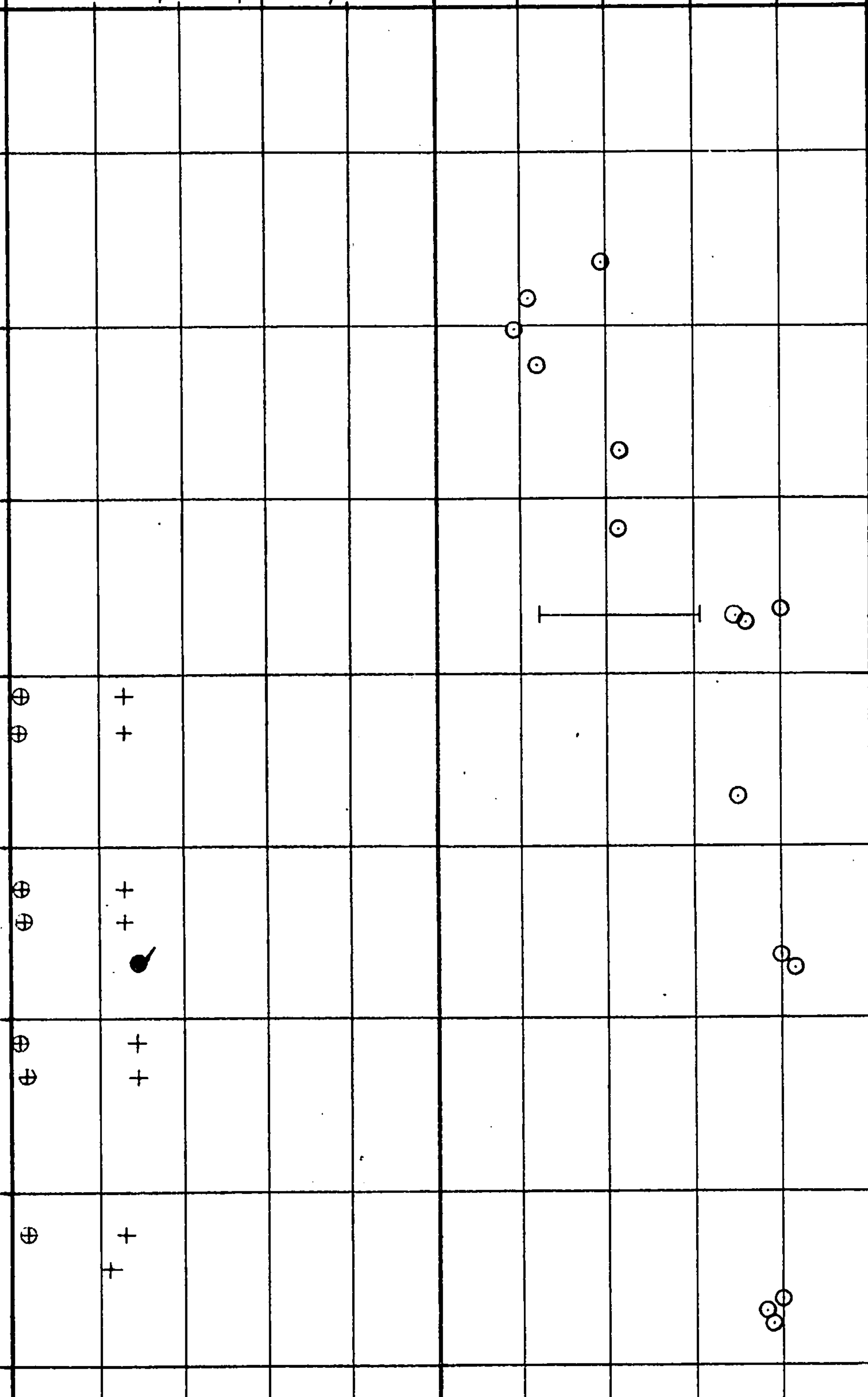
SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

500 1,000 1,500 2,000

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

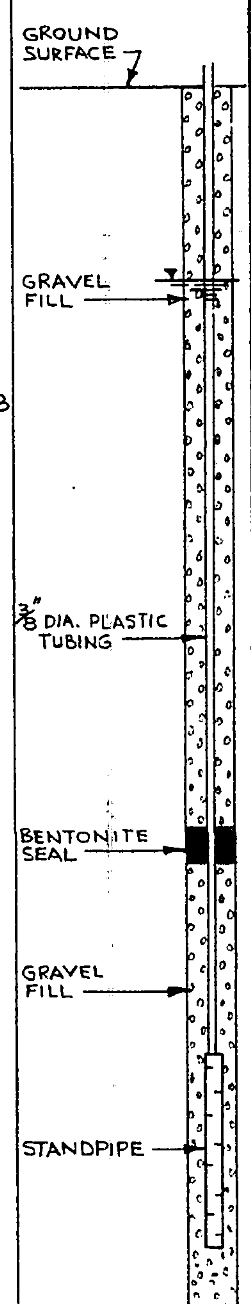
WATER CONTENT, PERCENT Wp W WL

20 40 60 80



15 0 5 10 Percent axial strain at failure

PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN STANDPIPE AT EL. 247.3 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

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[redacted]
Form G.A.-D-1.

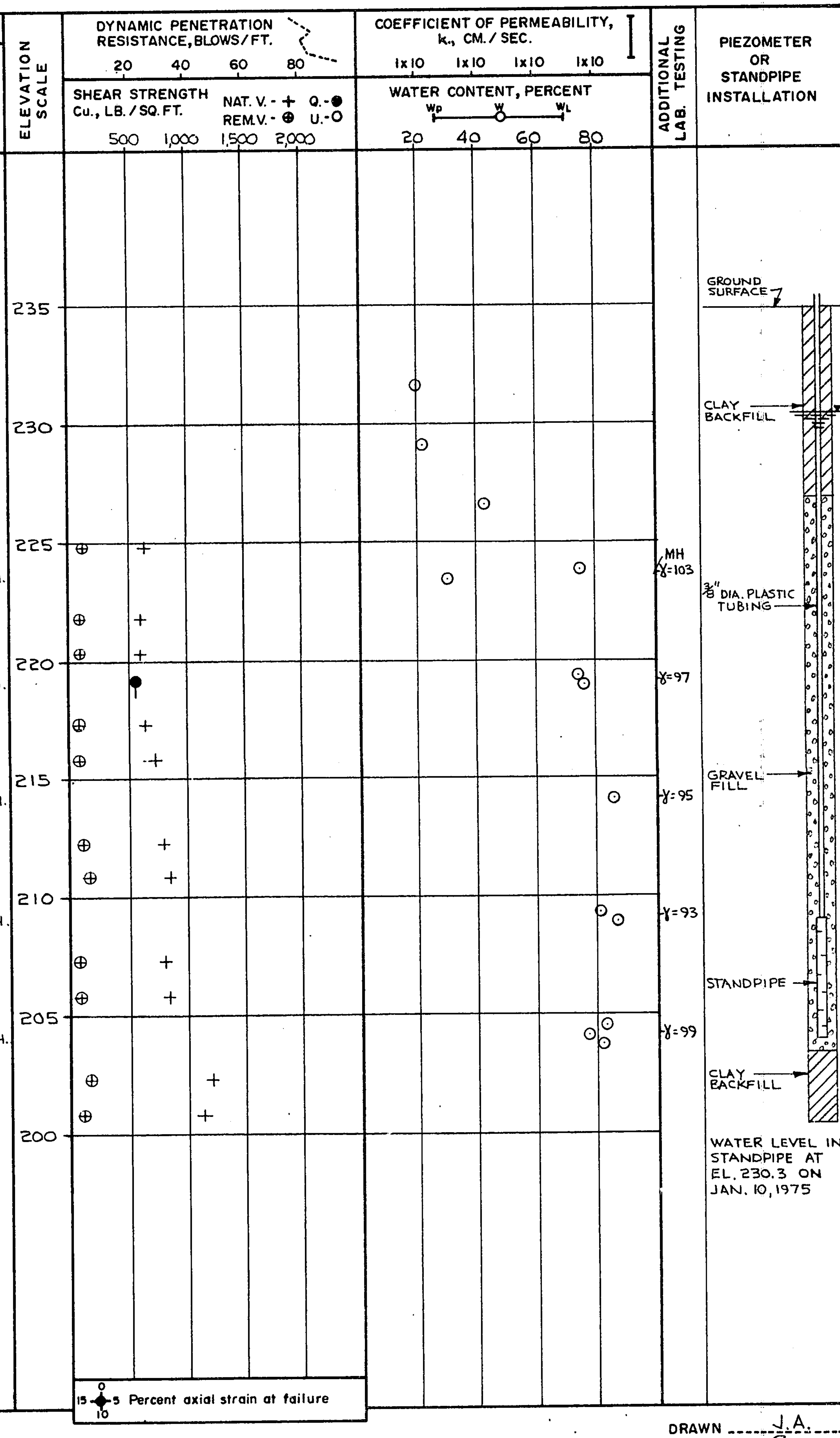
Project No. 74[redacted]30

RECORD OF BOREHOLE 232

LOCATION See Figure 2 — BORING DATE DEC. 13, 1974 — DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. — PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
	234.8	GROUND SURFACE				
POWER AUGER, 7" DIA. HOLLOW STEM	234.3 / 0.5	BROWN SANDY TOPSOIL				
		COMPACT BROWN STRATIFIED SAND, TRACE SILT	1	2" D.O.	19	
			2	"	19	
	227.8 / 7.0	FIRM TO STIFF SENSITIVE GREY AND RED-GREY SILTY CLAY, OCCASIONAL SILTY SAND LAYERS AND POCKETS OF BLACK ORGANIC CLAY (LIGHTLY DESICCATED IN UPPER 2 FT.) VERTICAL CROSS-SHAPED SAND DRAIN BETWEEN EL. 223 AND EL. 224.	3	"	2	
			4	3" T.P.	P.H.	MH, γ=103
			5	2" T.P.	P.H.	γ=97
			6	3" T.P.	P.H.	γ=95
			7	2" T.P.	P.H.	γ=93
			8	3" T.P.	P.H.	γ=99
	200.3 / 34.5	END OF HOLE				



WATER LEVEL IN STANDPIPE AT EL. 230.3 ON JAN. 10, 1975

VERTICAL SCALE 1 IN. TO 5 FT.

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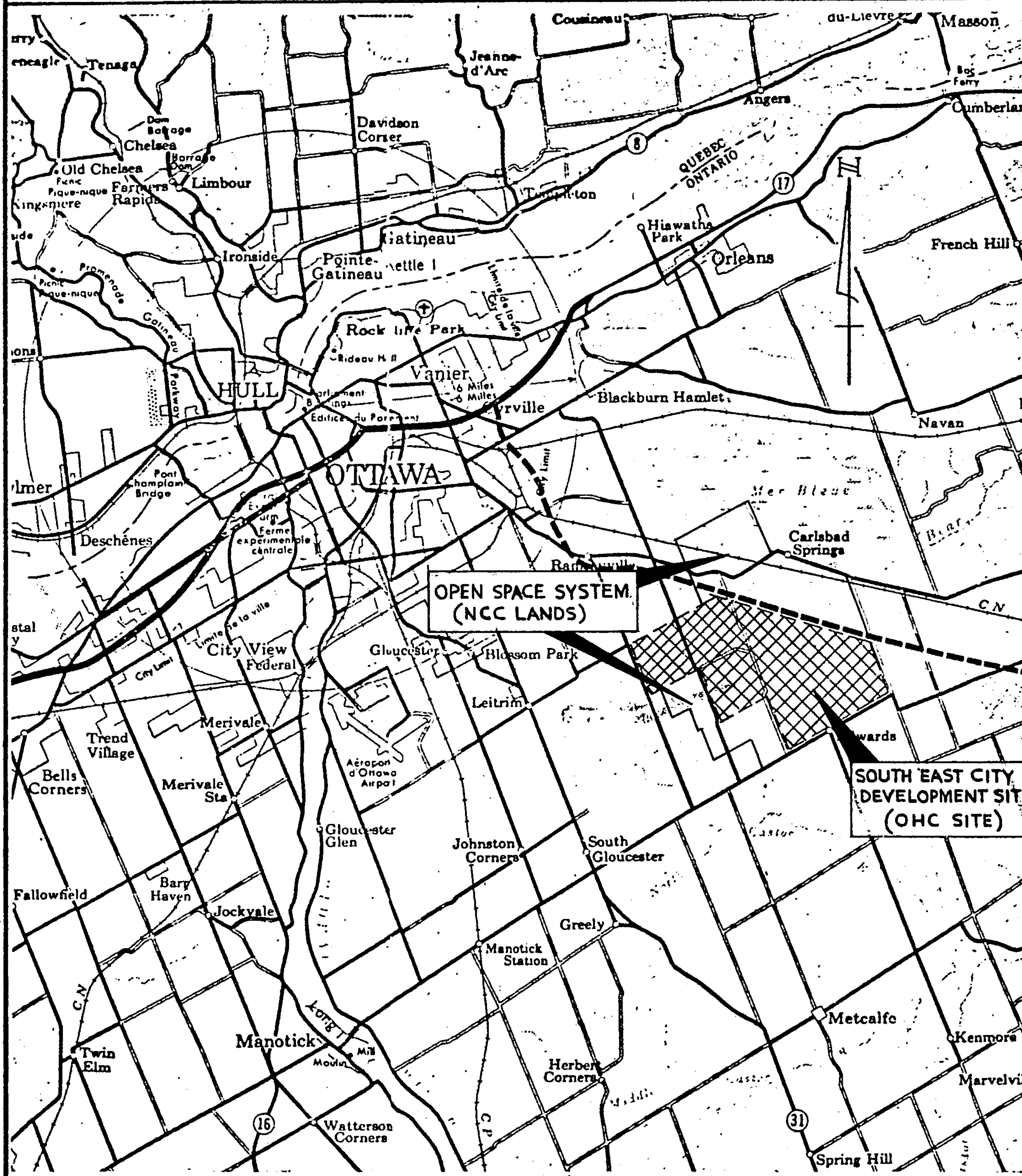
CHECKED BG

KEY PLAN

FIGURE 1



SCALE: 1 INCH TO 3 MILES (APPROX.)

Date MARCH 20, 1975

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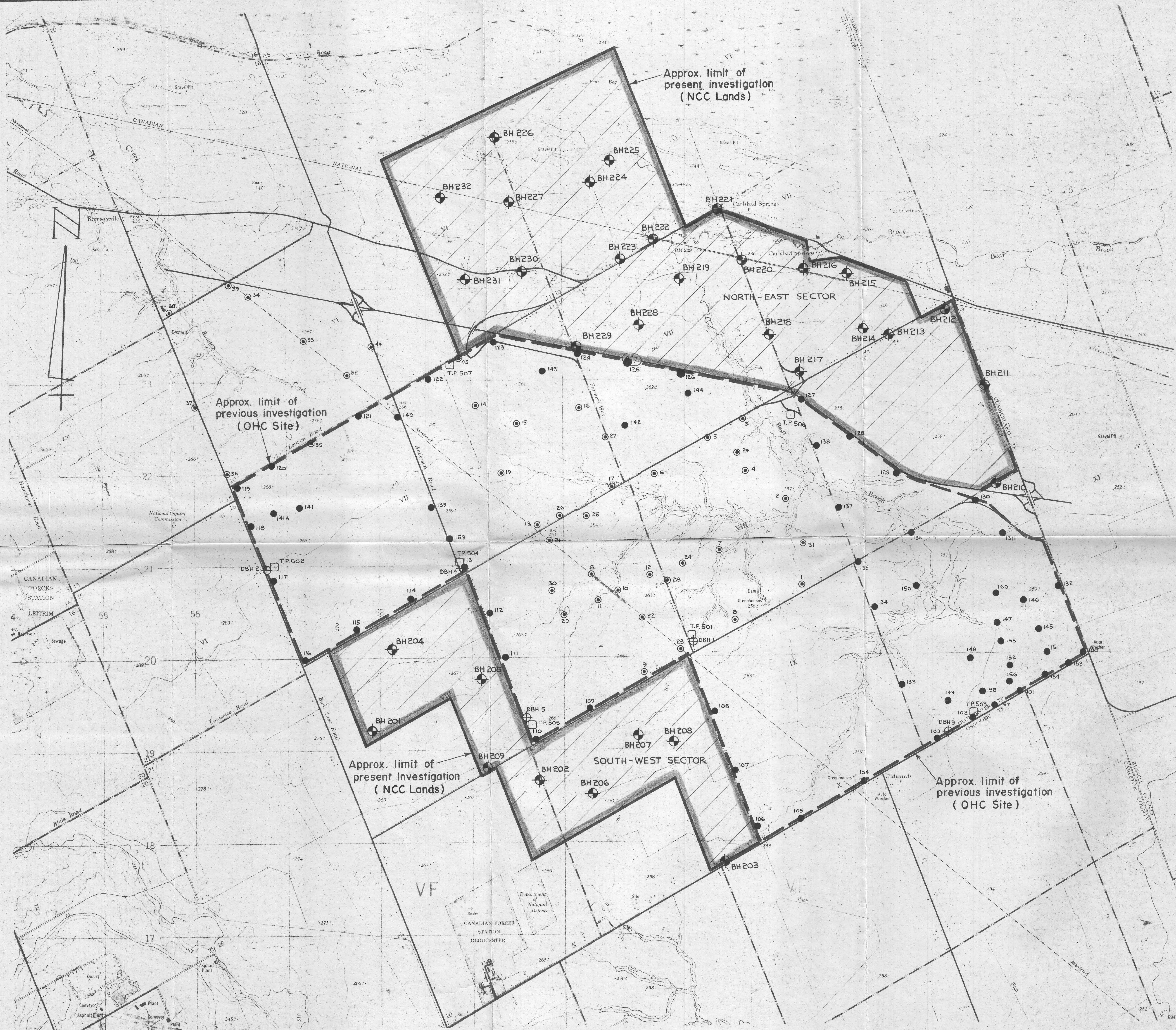
PROJECT No.

Form G.A.-D-4

741230

SITE AND BORING PLAN	FIGURE 2



Scale 1:25,000
MILE 1 0 1 MILE

LEGEND

- BOREHOLE IN PLAN, (200 SERIES) - PRESENT INVESTIGATION (NCC LANDS)
- BOREHOLE IN PLAN, (100 SERIES) - PREVIOUS INVESTIGATION BY K. H. KING ASSOCIATES LIMITED, JAN.-MARCH, 1974, REPORT No. 312-S.2, VOL. I & II. (OHC SITE)
- BOREHOLE IN PLAN, (DBH SERIES) - PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES, REPORT No 73908, VOL. III & IV, DATED MAY & JUNE, 1974. (OHC SITE)
- BOREHOLE IN PLAN, (O SERIES) - PREVIOUS INVESTIGATION BY K. H. KING ASSOCIATES LIMITED, SEPT. & OCT., 1973, REPORT No. 209-S.15. (OHC SITE)
- TEST PIT IN PLAN, (500 SERIES) - PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES, REPORT No. 73908, VOL. III & IV, DATED MAY & JUNE, 1974. (OHC SITE)

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d, NAVAN 31G/6e, ed. 2 SOUTH GLOUCESTER 31G/5a, BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

Date: JAN. 14, 1975

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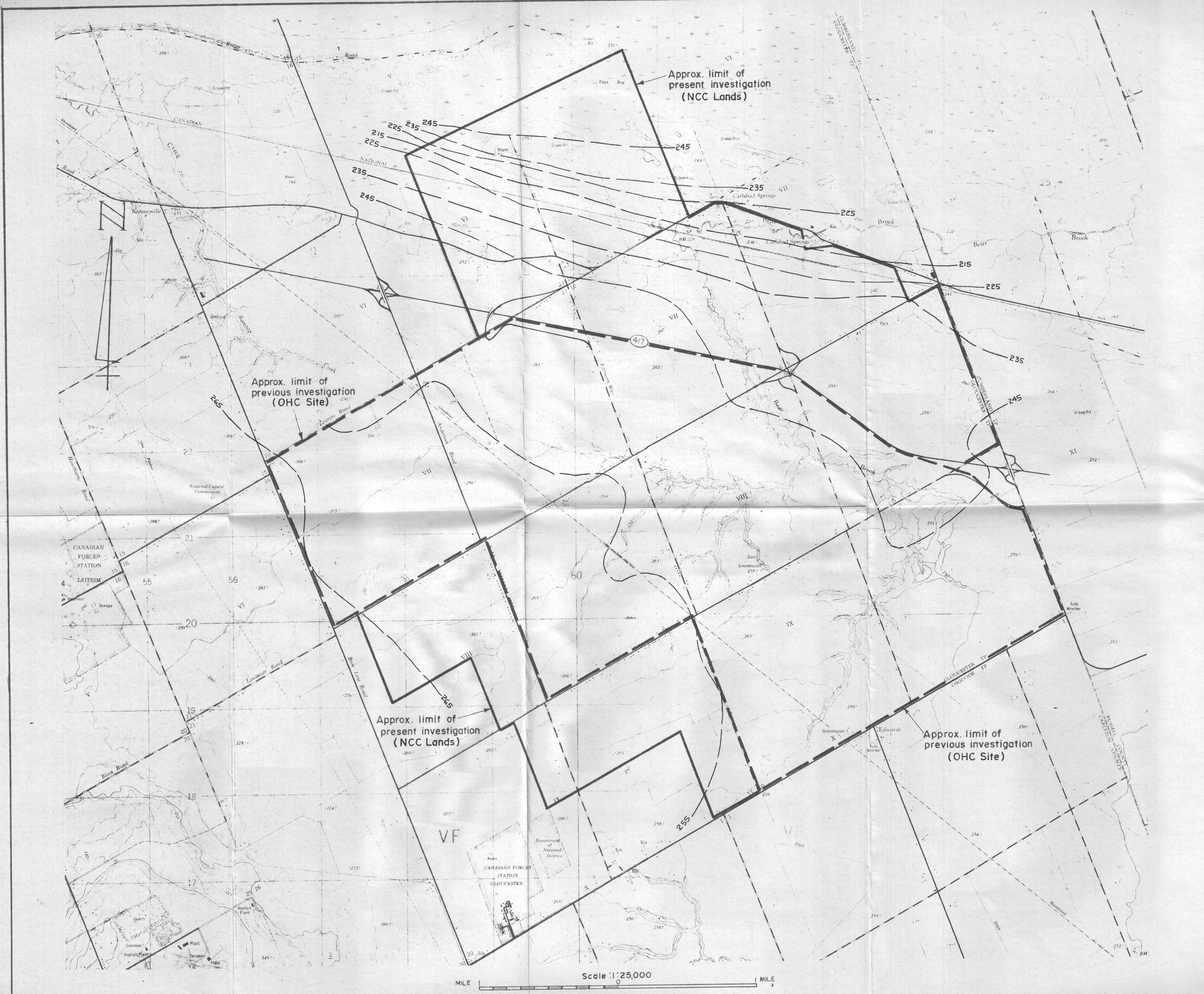
Chkd. [illegible]
Appd. [illegible]

ELEVATION CONTOURS OF SURFACE OF LEDA CLAY DEPOSIT

FIGURE 3



LEGEND

— — — 235 APPROXIMATE ELEVATION CONTOUR OF SURFACE OF LEDA CLAY DEPOSIT – INCLUDING DESICCATED CRUSTAL ZONE.

NOTES

1) ELEVATIONS ARE REFERRED TO GEODETIC DATUM

2) CONTOURS ARE BASED ON INFORMATION OBTAINED AT WIDELY SPACED LOCATIONS AND SHOULD THEREFORE BE CONSIDERED AS APPROXIMATE ONLY.

3) DATA OBTAINED FROM :-

a) K.H. KING ASSOCIATES LIMITED REPORT No. 209-S.15 (UNDATED)

b) K.H. KING ASSOCIATES LIMITED REPORT No. 312-S.2, VOL. I & II (UNDATED)

c) GOLDER ASSOCIATES REPORT No. 73908, VOL. III & IV, DATED MAY & JUNE, 1974

d) GOLDER ASSOCIATES – PRESENT INVESTIGATION

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d , NAVAN 31G/6e , ed. 2 SOUTH GLOUCESTER 31G/5a , BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

Date: MARCH 14, 1975

Golder Associates

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Chkd. B.G.
Appd. J.H.K.

741230

THICKNESS OF SURFICIAL SAND DEPOSITS

FIGURE 4

LEGEND

RANGE OF THICKNESS OF SURFICIAL SANDS

(blank)	0 FT. - 4 FT.
(vertical lines)	4 FT. - 8 FT.
(diagonal lines)	8 FT. - 16 FT.
(grid)	>16 FT.

NOTES

1) BOUNDARIES BETWEEN "SAND THICKNESS" REGIONS ARE INFERRED FROM WIDELY SPACED BOREHOLE INFORMATION AND SHOULD THEREFORE BE CONSIDERED AS APPROXIMATE ONLY.

2) DATA OBTAINED FROM :-

a) K.H. KING ASSOCIATES LIMITED REPORT No. 209-S.15 (UNDATED)
b) K.H. KING ASSOCIATES LIMITED REPORT No. 312-S.2, VOL. I & II (UNDATED)
c) GOLDER ASSOCIATES REPORT No. 73908, VOL. III & IV, DATED MAY & JUNE, 1974
d) GOLDER ASSOCIATES - PRESENT INVESTIGATION.

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d, NAVAN 31G/6e, ed. 2 SOUTH GLOUCESTER 31G/5a, BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

Date: MARCH 13, 1975

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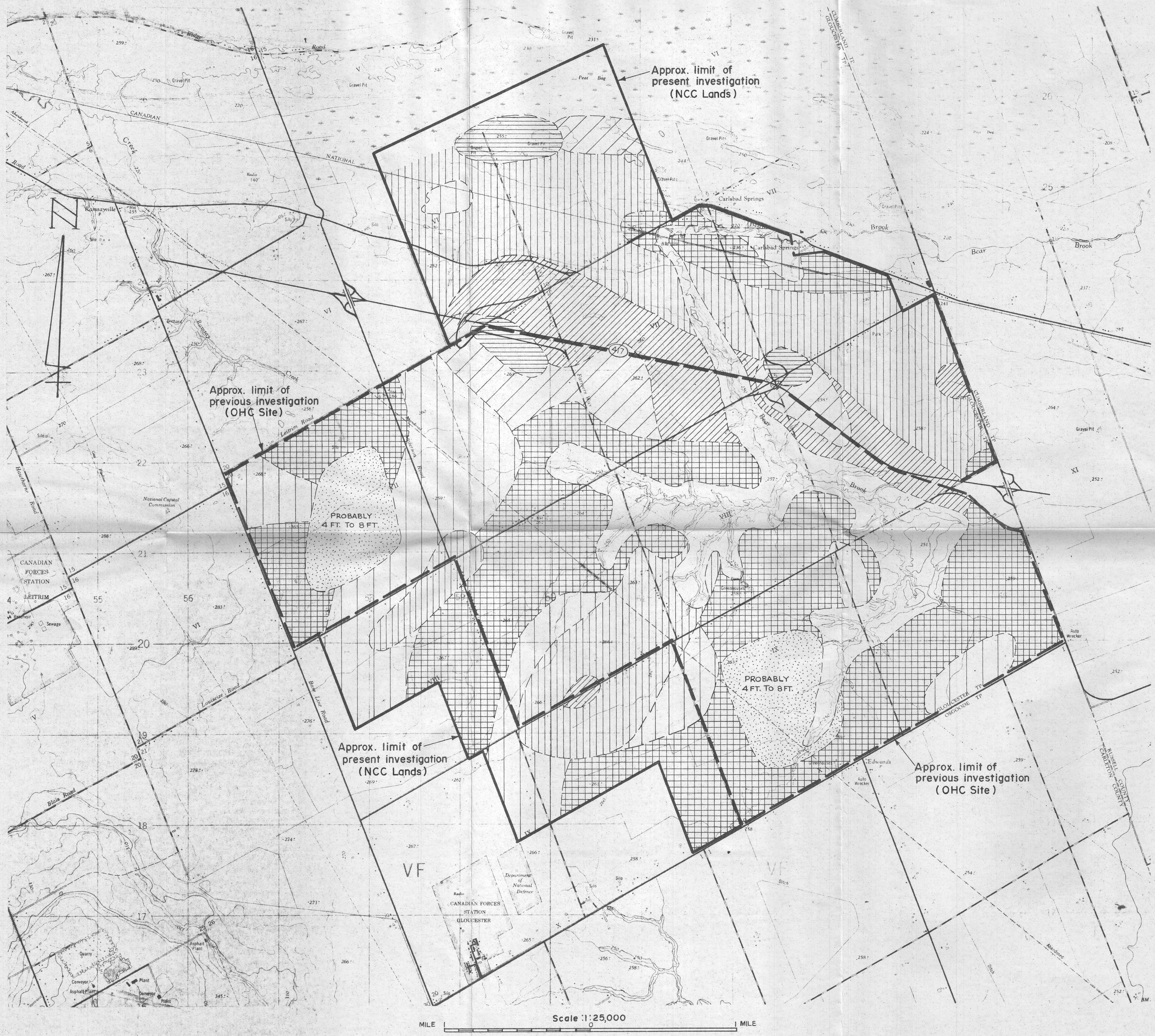
Chkd. B.G.
Appd. JHH

741230

THICKNESS OF SAND CAP/ DESICCATED CRUST

FIGURE 5



LEGEND

RANGE OF DESICCATED CRUST/ SAND CAP THICKNESS

- 4 FT. TO 6 FT.
- 6 FT. TO 8 FT.
- 8 FT. TO 10 FT.
- 10 FT. TO 12 FT.
- 12 FT TO 16 FT.
- >16 FT.
- INSUFFICIENT INFORMATION
- BEAR BROOK AND TRIBUTARY AREA

NOTES

1) BOUNDARIES BETWEEN "CRUST THICKNESS" REGIONS ARE BASED ON WIDELY SPACED BOREHOLE INFORMATION AND SHOULD THEREFORE BE CONSIDERED AS APPROXIMATE ONLY.

2) DATA OBTAINED FROM:-
   a) K.H. KING ASSOCIATES LIMITED REPORT No. 209-S.15 (UNDATED)
   b) K.H. KING ASSOCIATES LIMITED REPORT No. 312-S.2, VOL. I & II (UNDATED)
   c) GOLDER ASSOCIATES REPORT No. 73908, VOL. III & IV, DATED MAY & JUNE, 1974
   d) GOLDER ASSOCIATES - PRESENT INVESTIGATION

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d , NAVAN 31G/6e , ed. 2 SOUTH GLOUCESTER 31G/5a , BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

Date: MARCH 18, 1975

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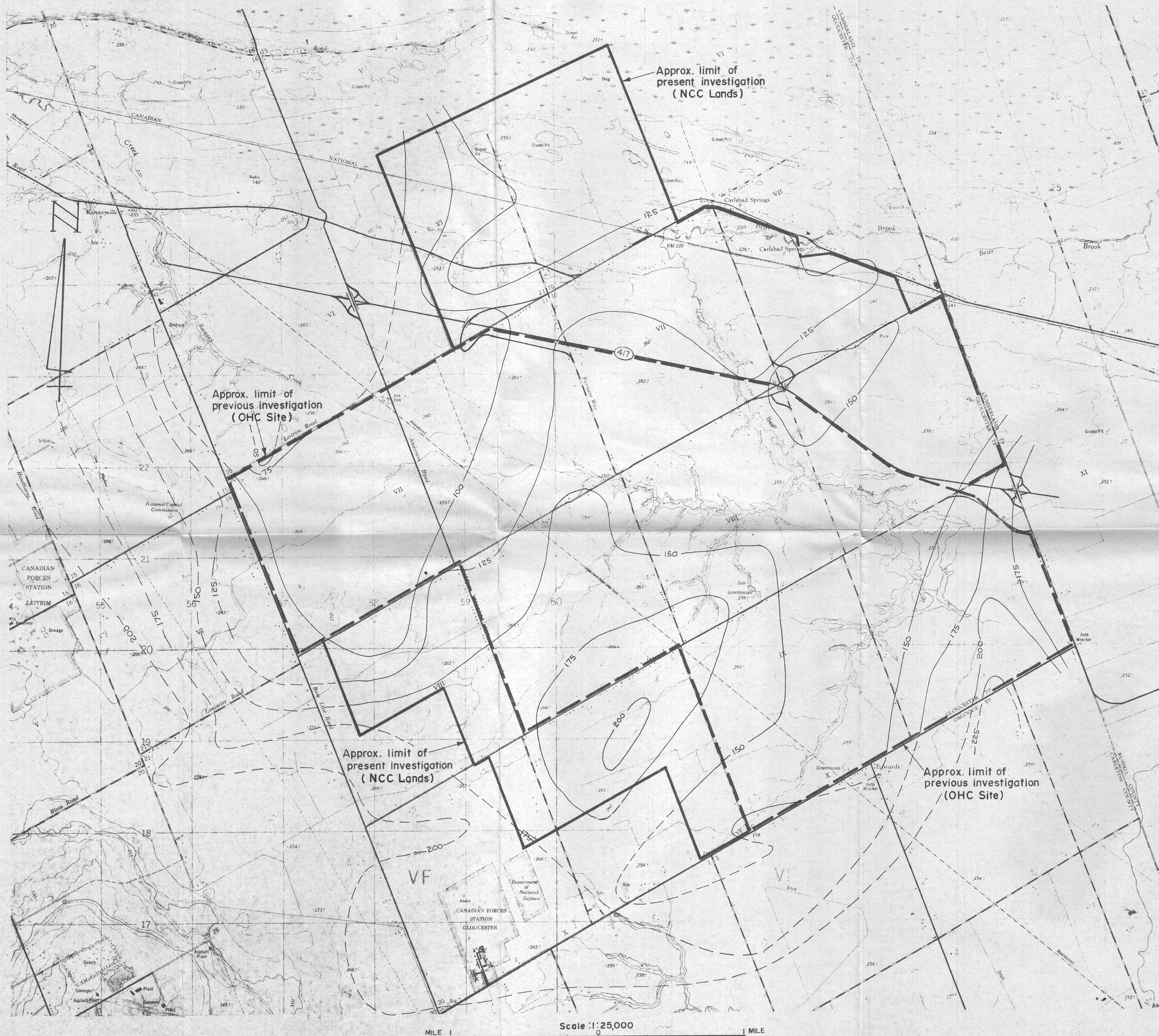
Chkd.
Appd.

741230

INFERRED TILL OR BEDROCK SURFACE ELEVATION CONTOURS

FIGURE 6



LEGEND

—175— INFERRED TILL OR BEDROCK SURFACE AT ELEVATION 175 WITHIN SITE LIMITS — BASED ON RESULTS OF DEEP BORINGS, DYNAMIC CONE PENETRATION TESTS, GEOPHYSICAL SURVEY AND PUBLISHED WELL DRILLING RECORDS.

— —150— — INFERRED TILL OR BEDROCK SURFACE AT ELEVATION 150 OUTSIDE SITE LIMITS — BASED ON PUBLISHED WELL DRILLING RECORDS ONLY.

NOTES

1) ELEVATIONS ARE REFERRED TO GEODETIC DATUM.

2) CONTOURS ARE BASED ON INFORMATION OBTAINED AT WIDELY SPACED LOCATIONS AND SHOULD THEREFORE BE CONSIDERED AS APPROXIMATE ONLY.

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d, NAVAN 31G/6e, ed. 2 SOUTH GLOUCESTER 31G/5a, BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

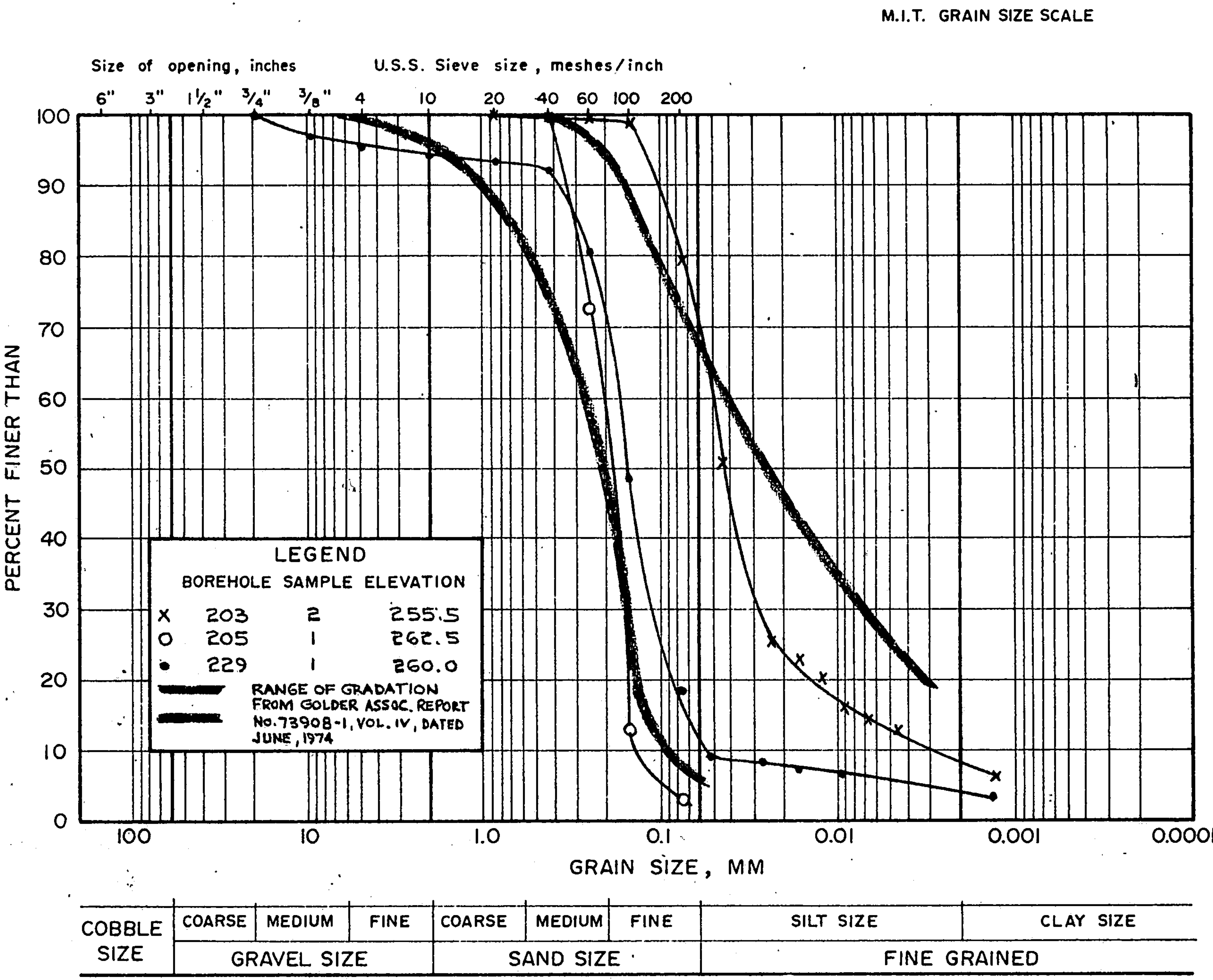
Date: MAR. 20, 1975

Golder Associates

Drawn
Chkd.
Appd.

Form G.A.-D.-6

PROJECT No. 741230



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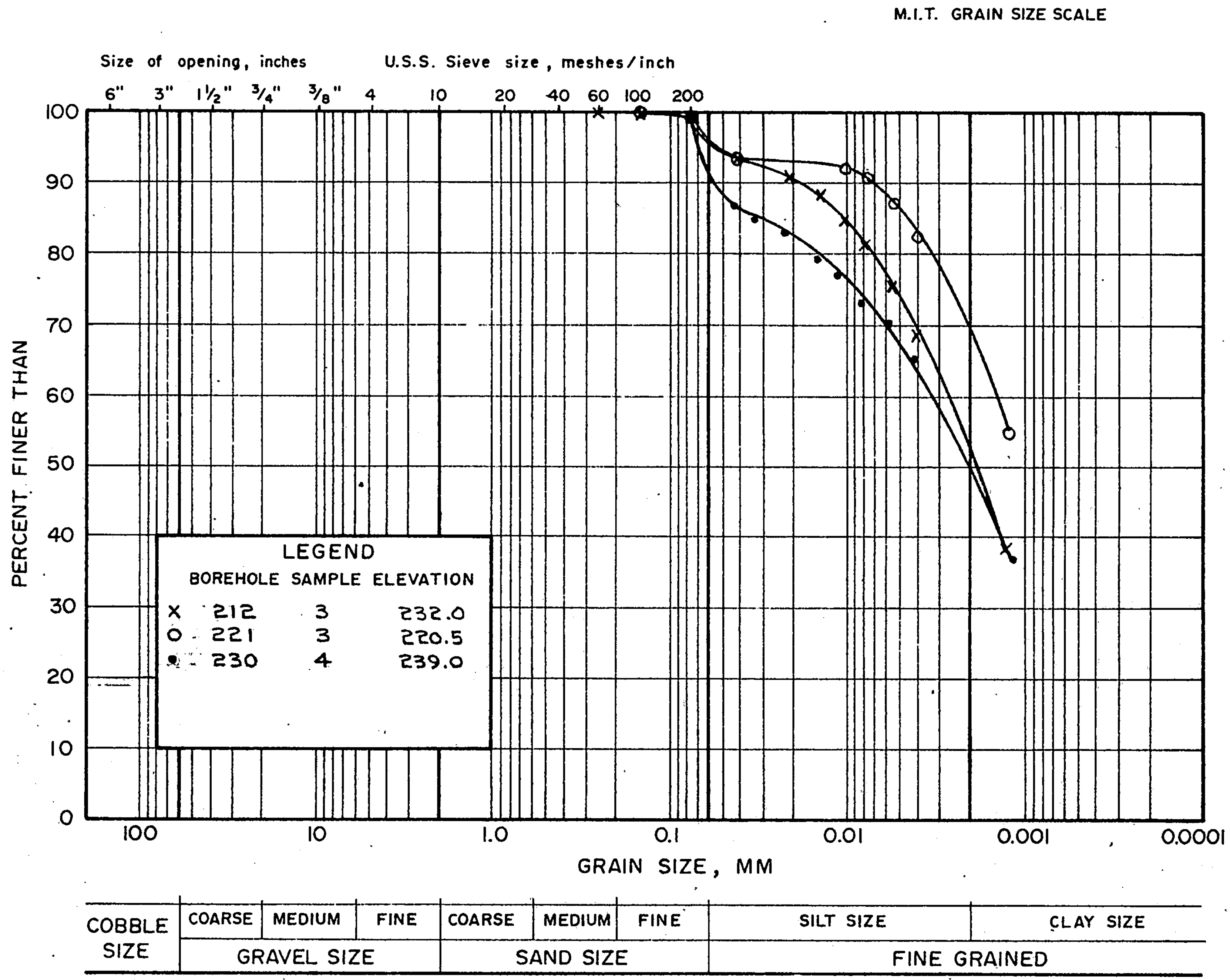
GRAIN SIZE DISTRIBUTION

SURFICIAL SANDS

FIGURE 7

Form G.A.-D.-6

PROJECT No. 741230



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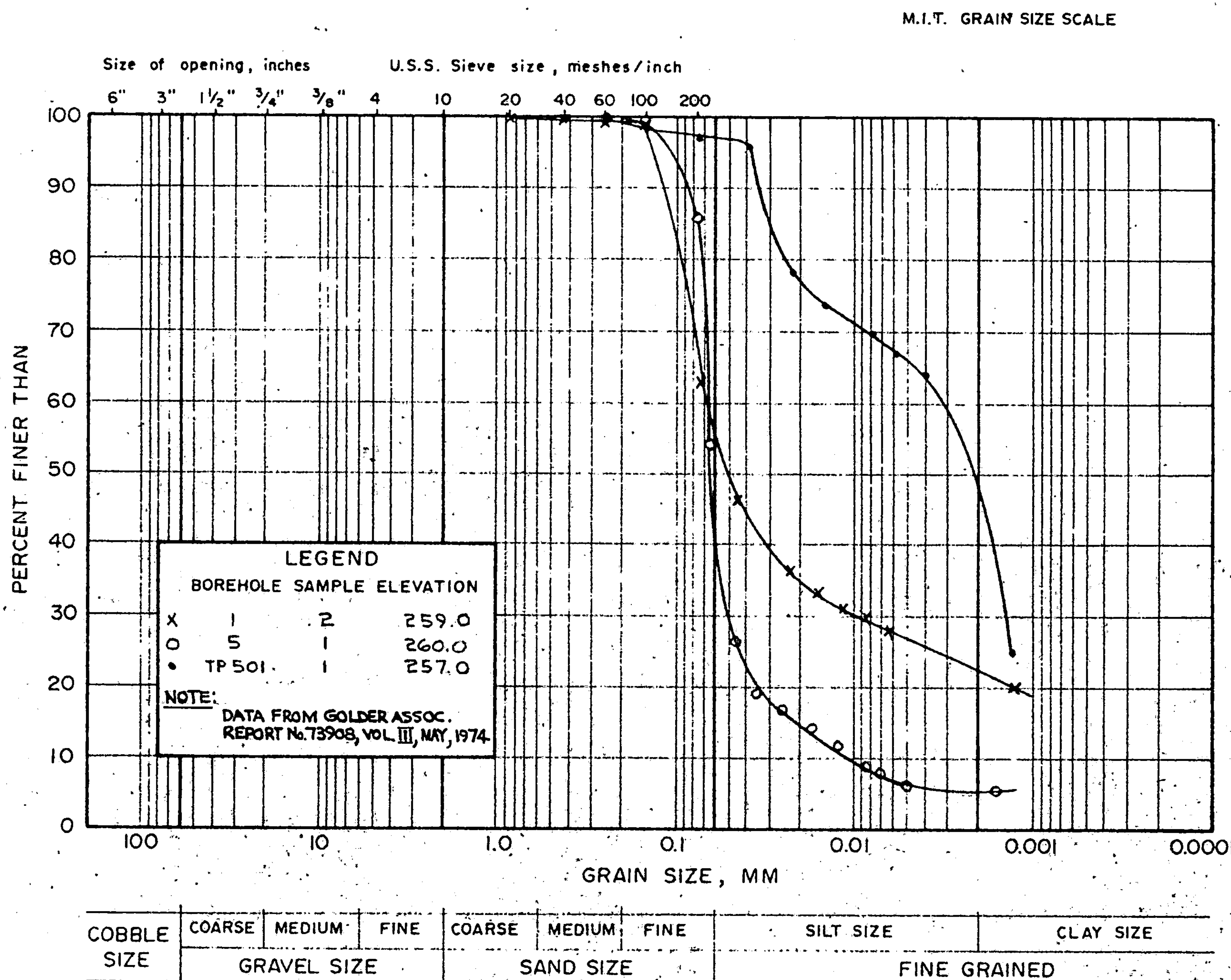
GRAIN SIZE DISTRIBUTION

DESICCATED SILTY CLAY CRUST

FIGURE 8

Form G.A.-D.-6

PROJECT No. 741230



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GRAIN SIZE DISTRIBUTION

COARSE LAYERS IN DESICCATED SILTY CLAY CRUST

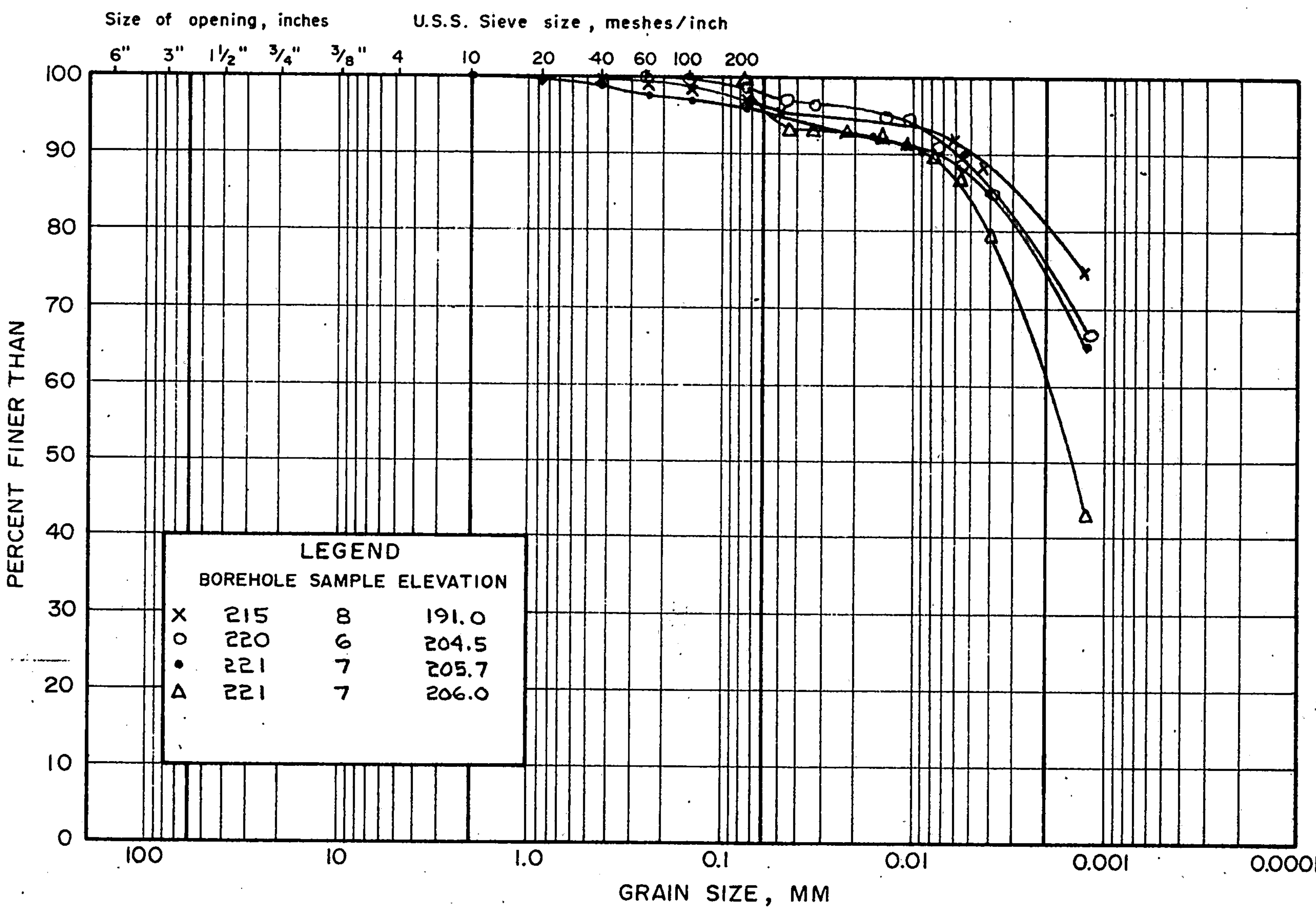
FIGURE 9

Form G.A.-D.-6

PROJECT No. 741236

GRAIN SIZE DISTRIBUTION

UNWEATHERED SILTY CLAY



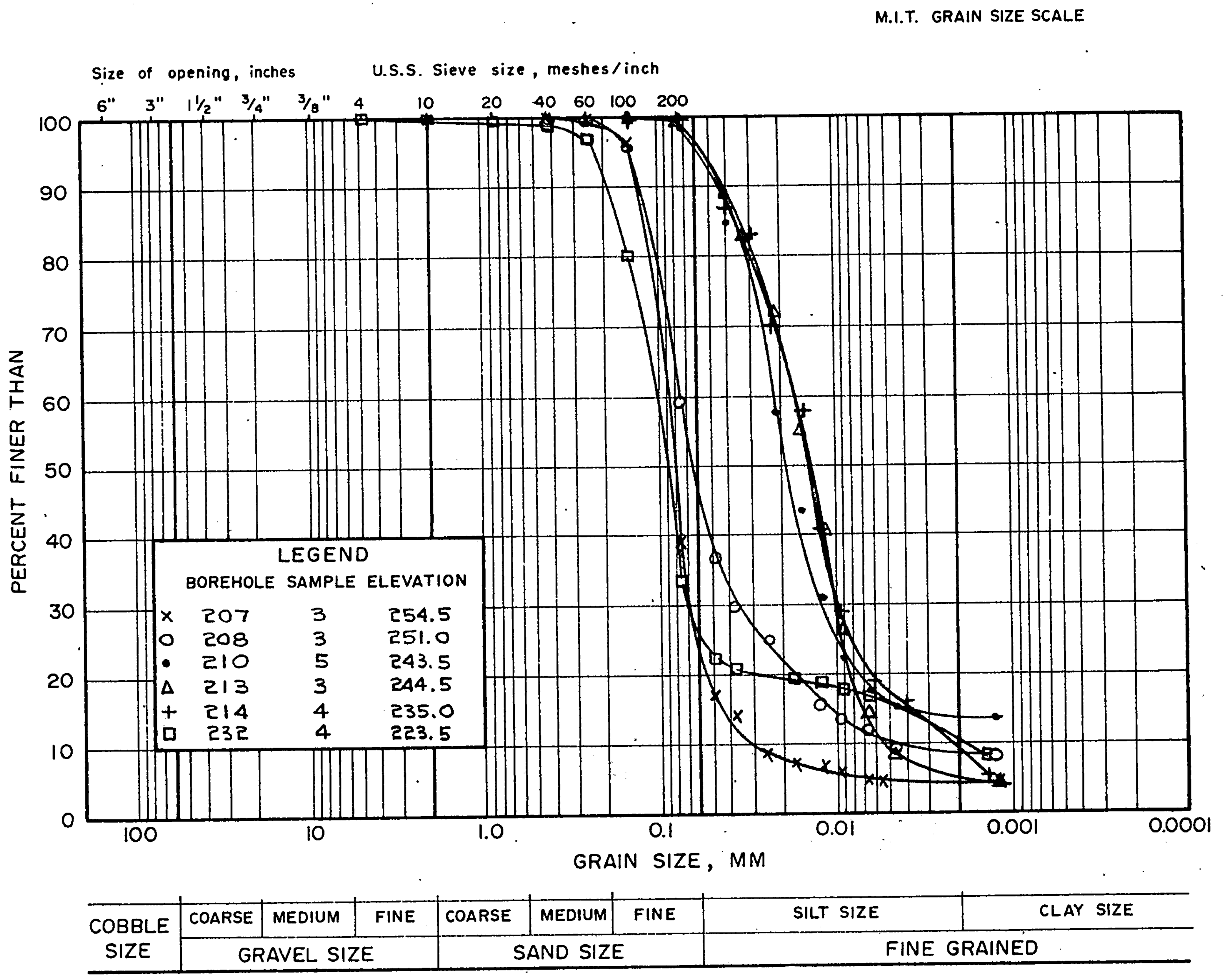
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

Golder Associates

FIGURE 10

Form G.A.-D.-6

PROJECT No. 741230



Golder Associates

GRAIN SIZE DISTRIBUTION

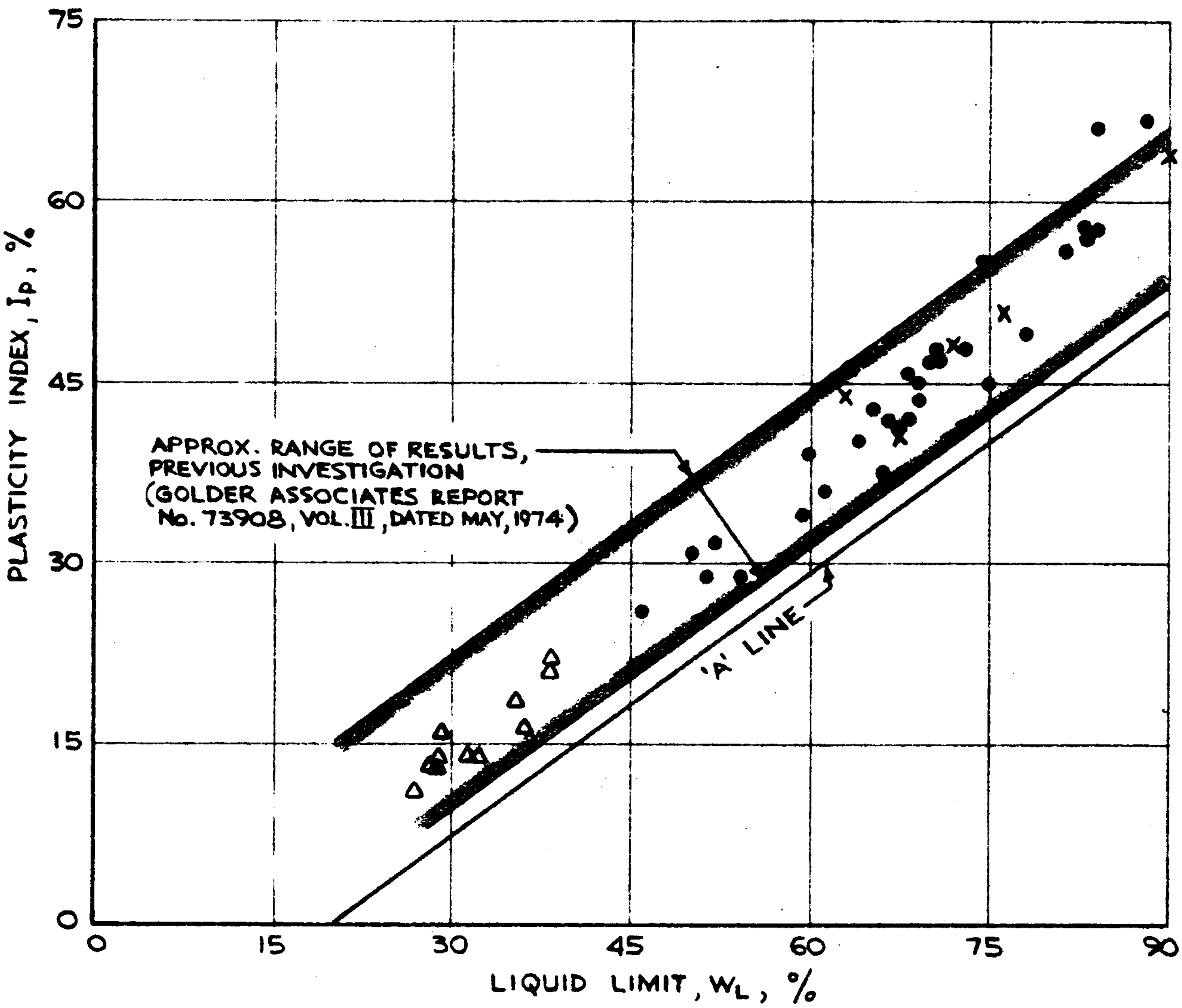
COARSE LAYERS IN UNWEATHERED SILTY CLAY DEPOSIT

FIGURE 11

PROJECT No. 741230

PLASTICITY CHART

FIGURE 12



LEGEND

X DESICCATED SILTY CLAY CRUST

O UNWEATHERED SILTY CLAY DEPOSIT

Δ COARSER LAYERS WITHIN SILTY CLAY DEPOSIT

Date FEB. 10, 1975

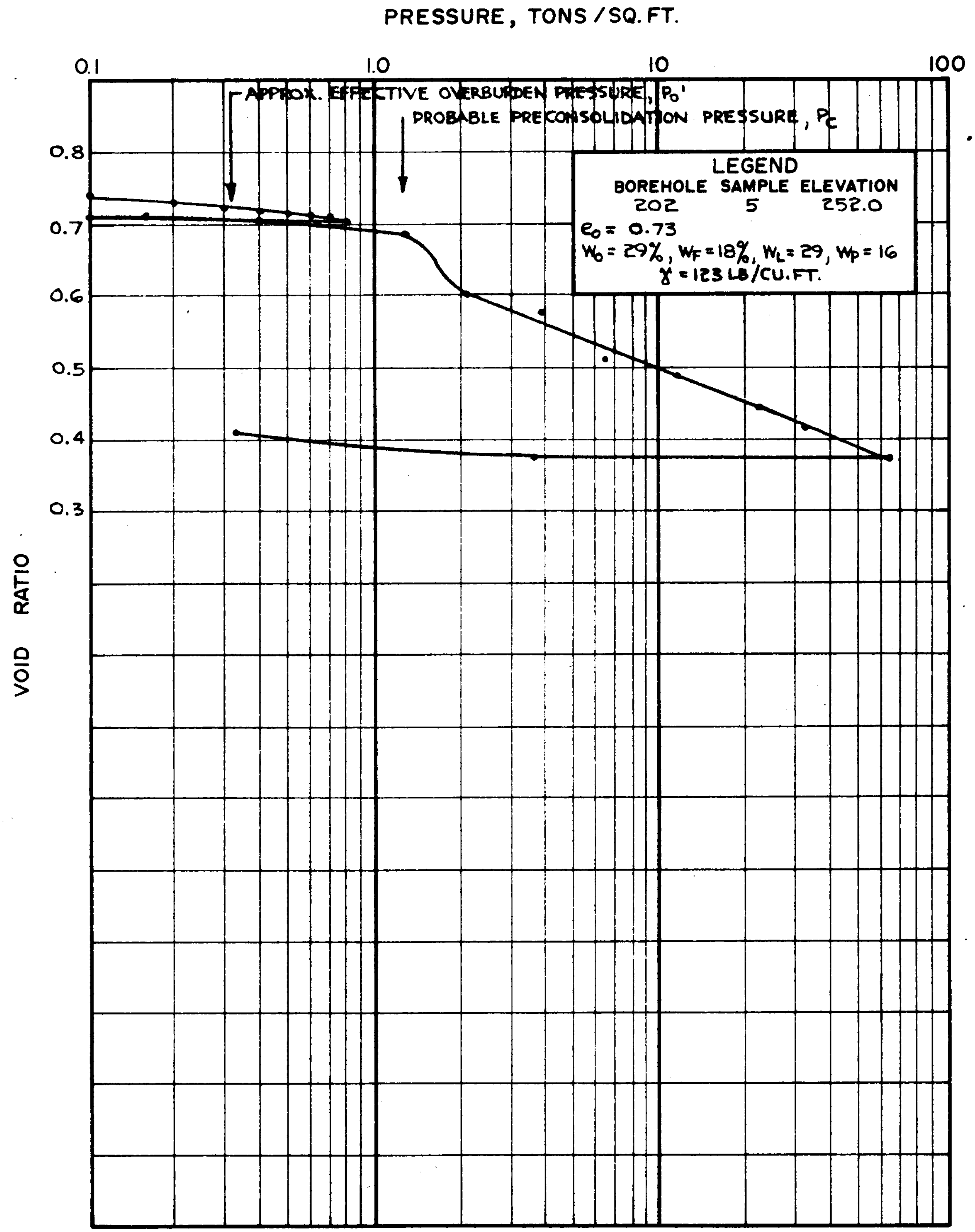
Golder Associates

Drawn J.A.
Chkd. BG
Appd. JWHC

Form G.A - D - 4

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 13



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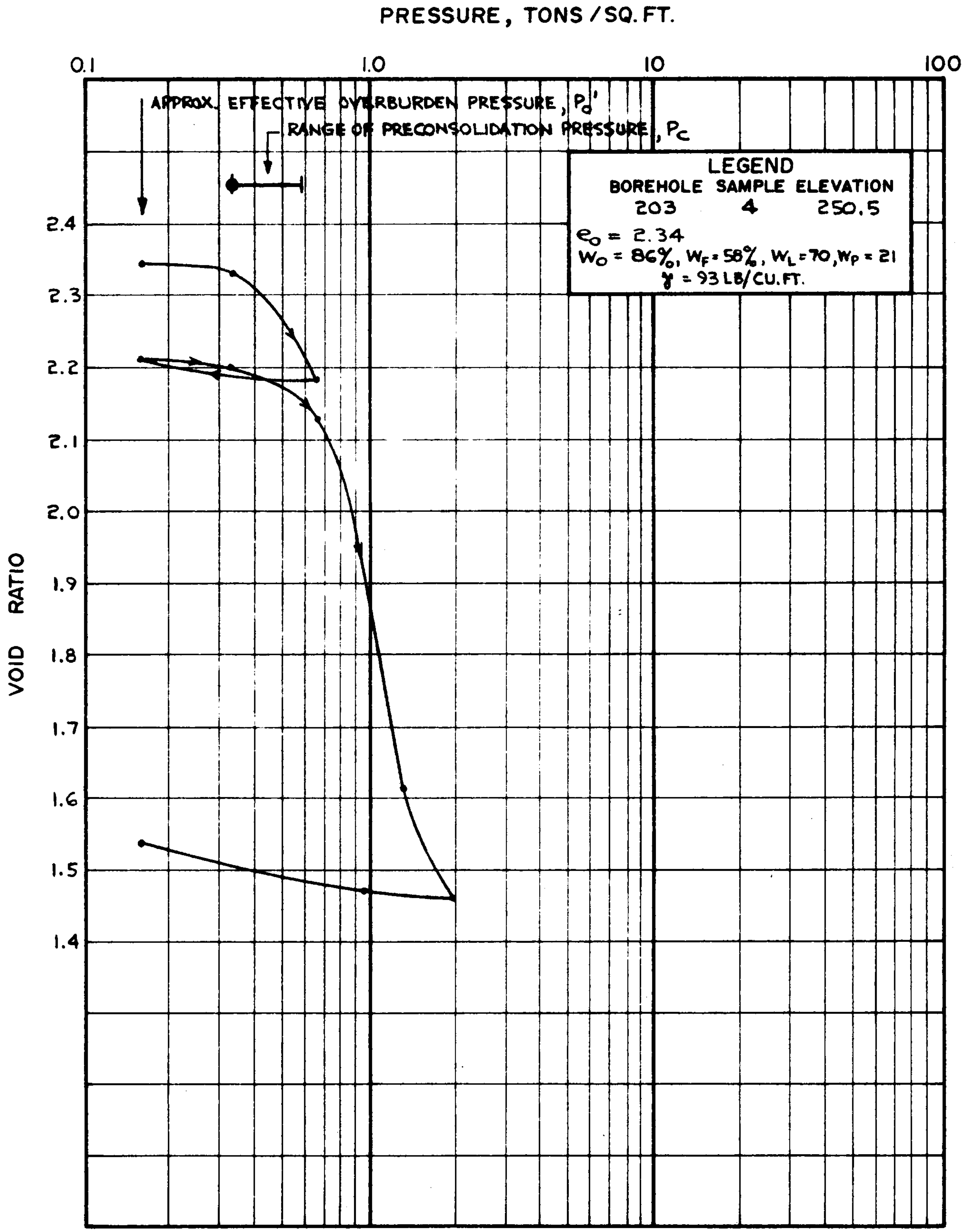
PROJECT No. 741230

Form G.A. D 10

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST	FIGURE 14



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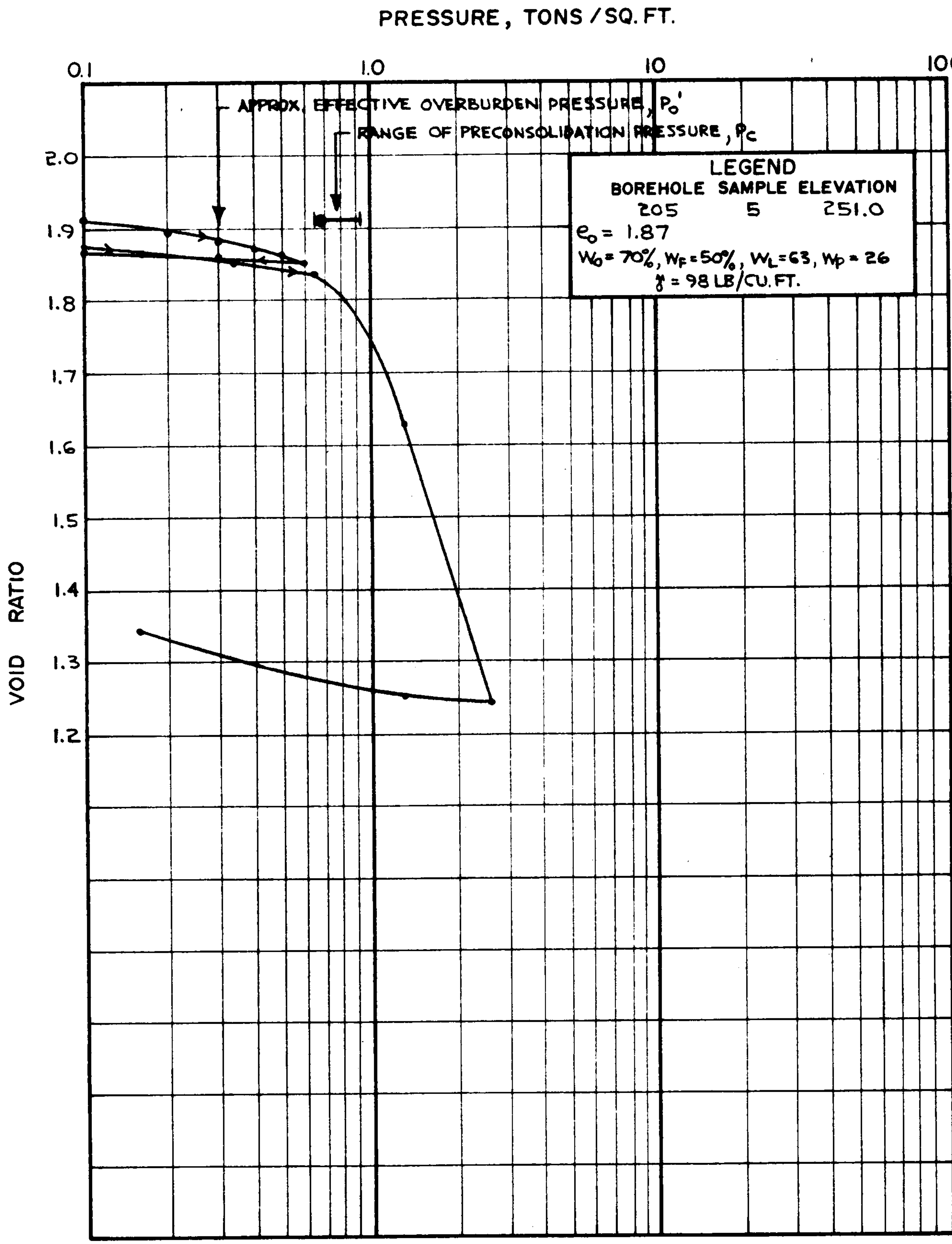
PROJECT No. 741230

Form GA-D-10

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 15



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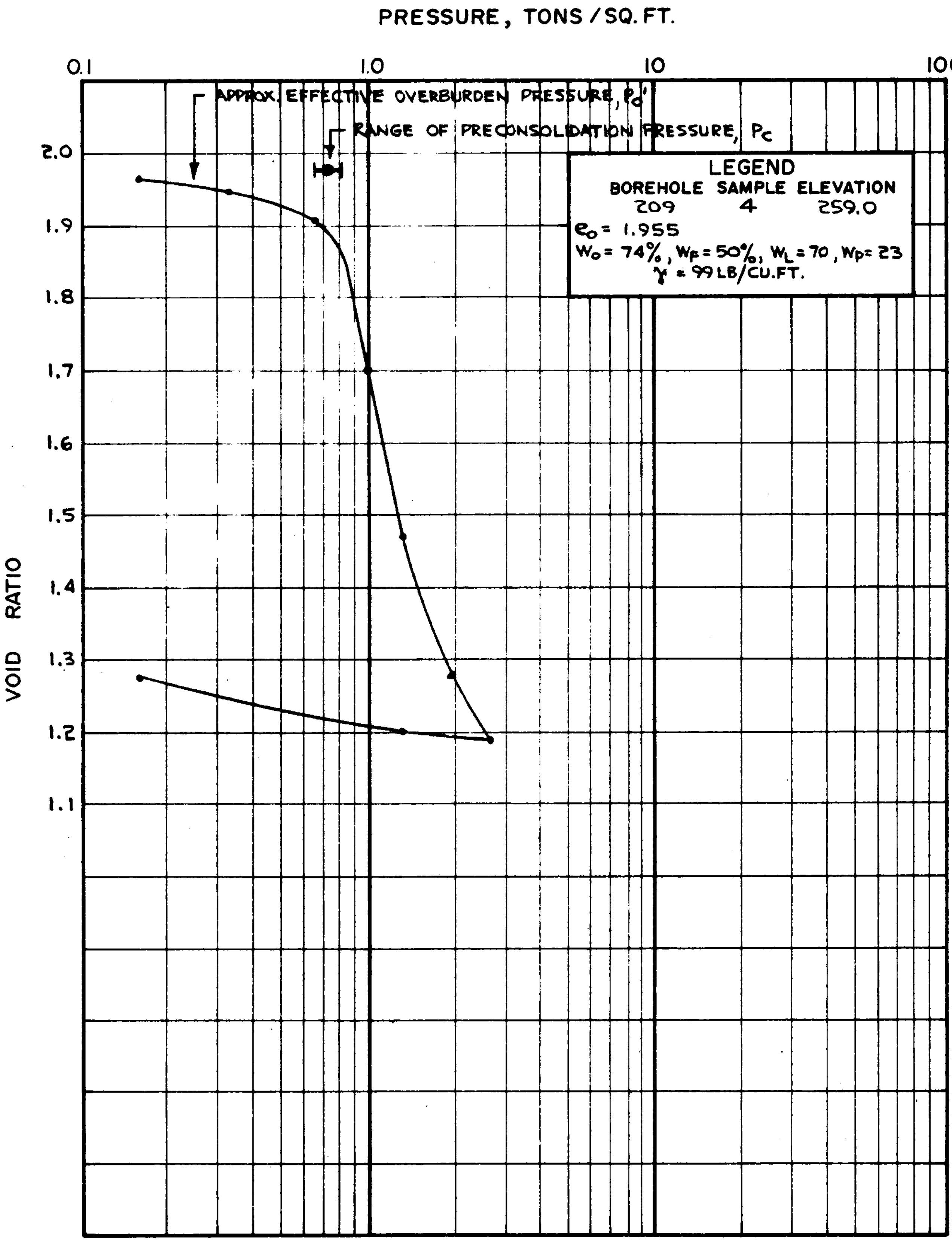
PROJECT No. 741230

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 16



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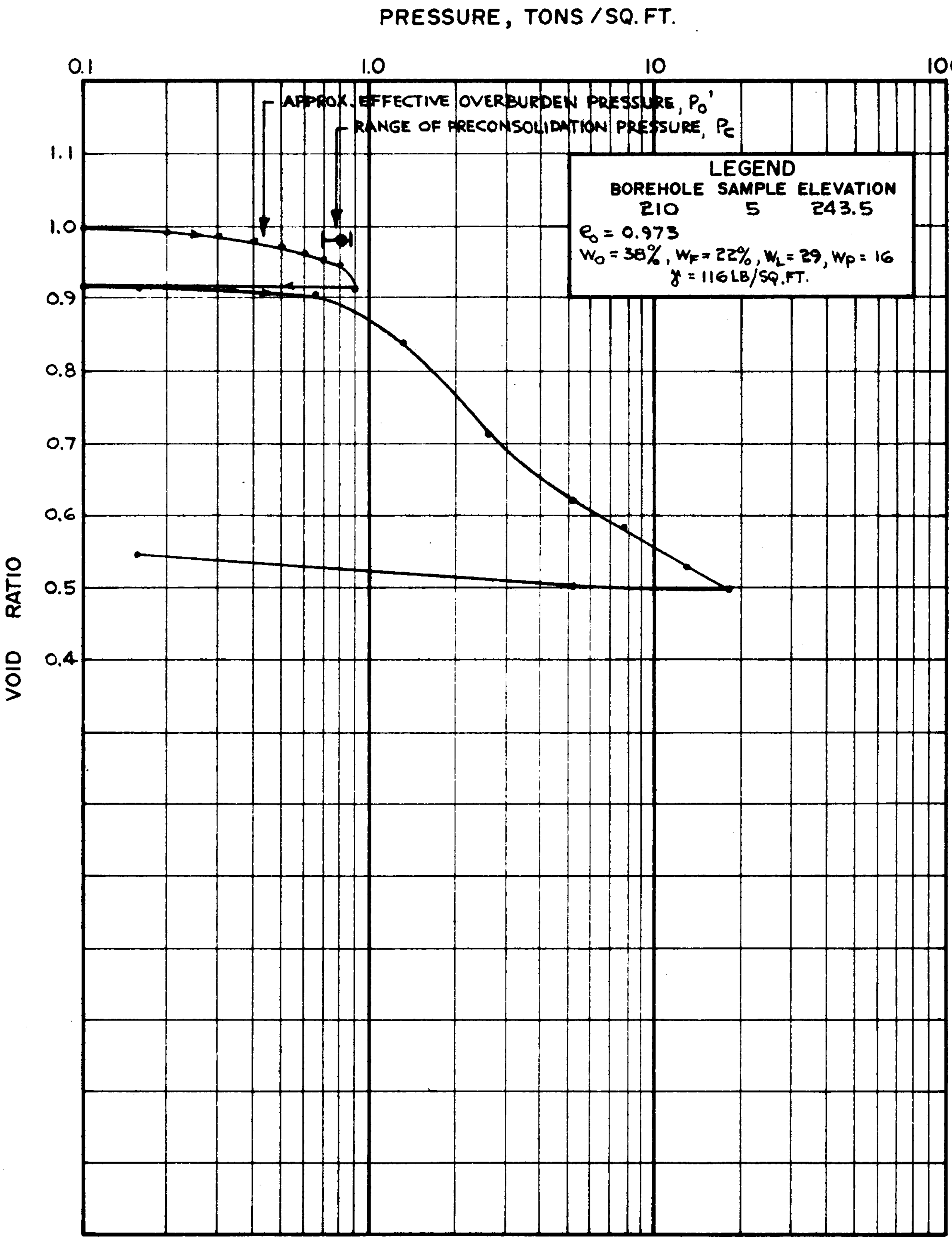
Form G.A.-D-10

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 17

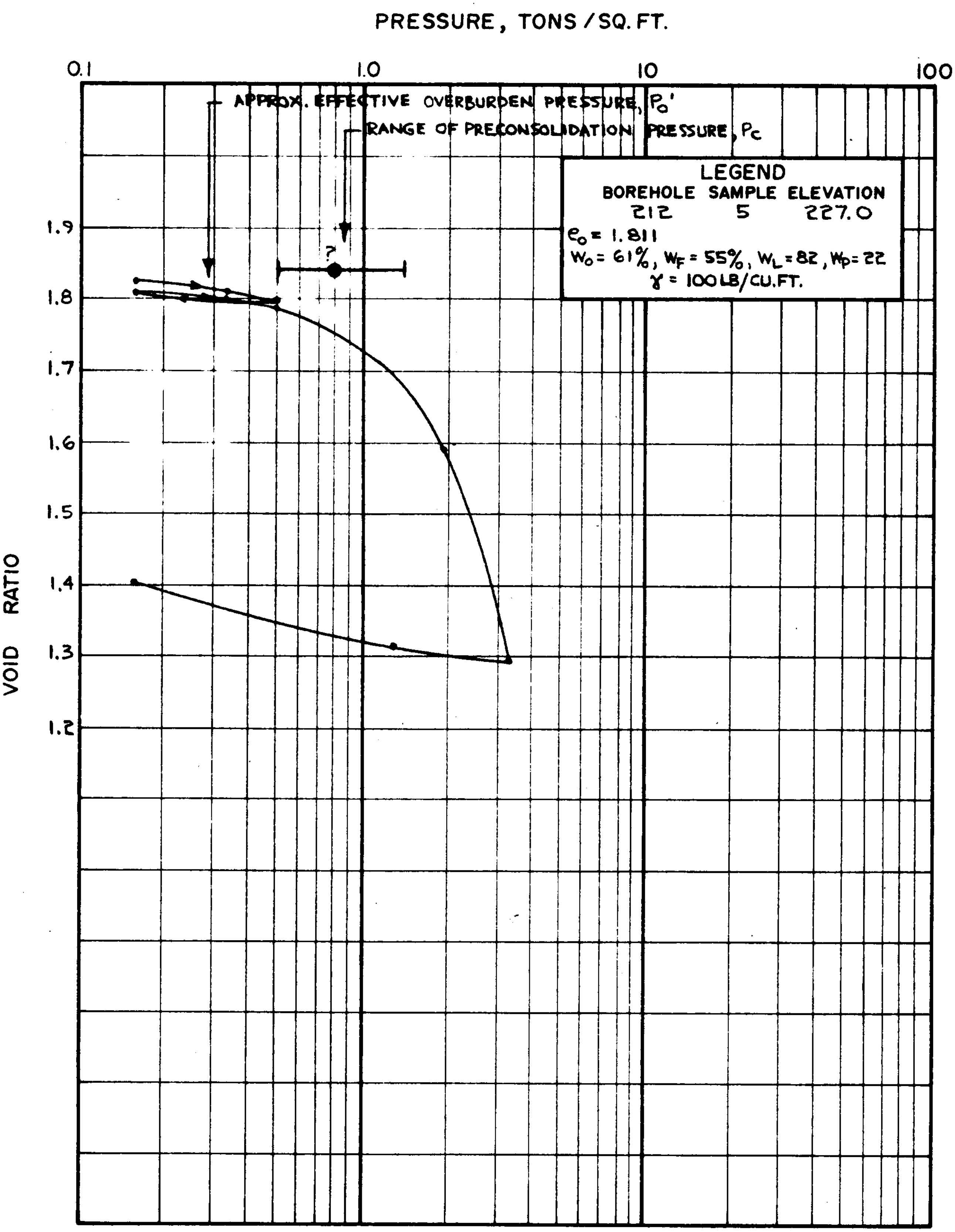


Golder Associates

PROJECT No. 741230

Form GA-D-10

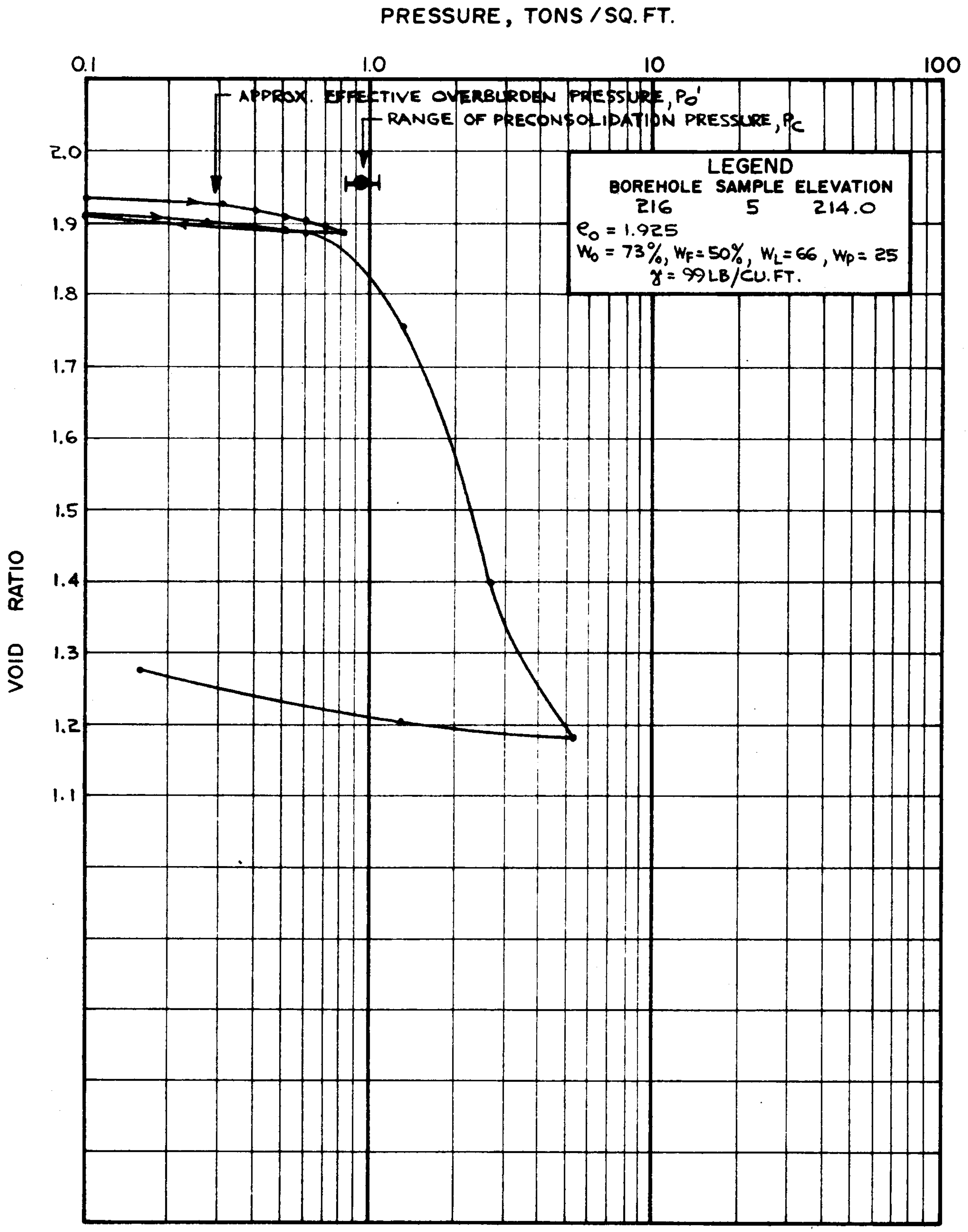
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST	FIGURE 18



Golder Associates

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 19

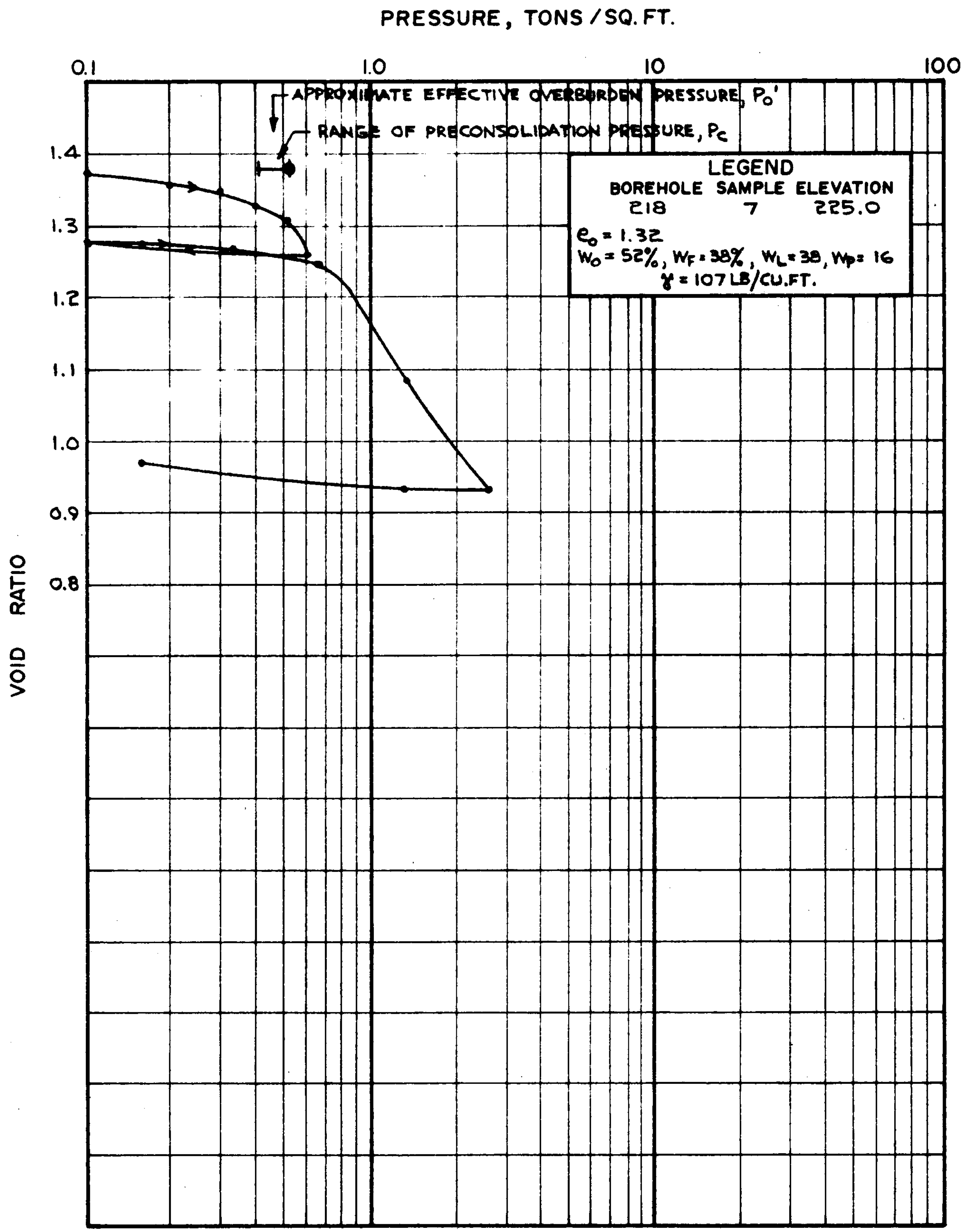


Golder Associates

PROJECT No. 741230

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 20



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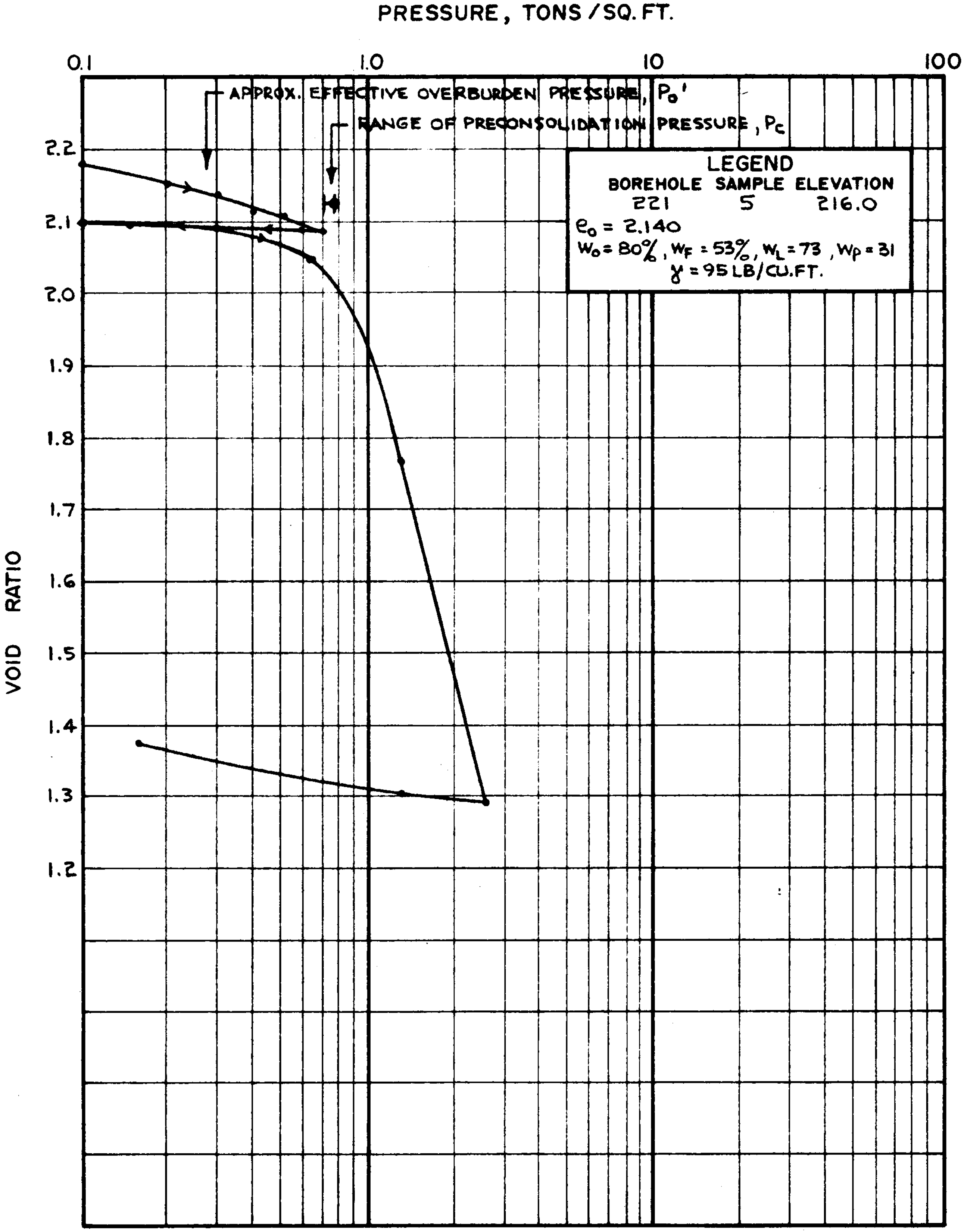
Form GA-D-10

PROJECT No. 741230

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 21



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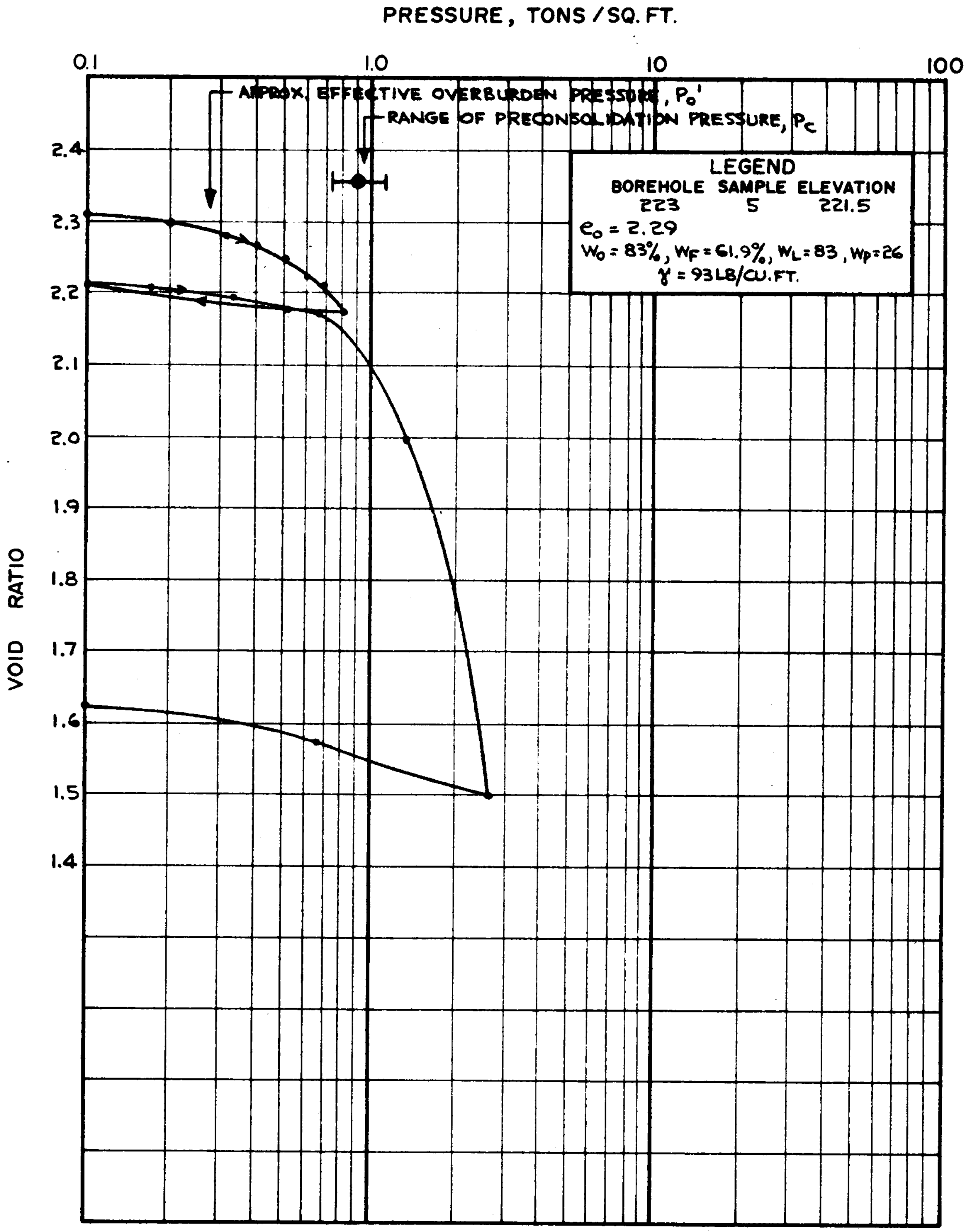
Form G.A.-D.-10

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 22



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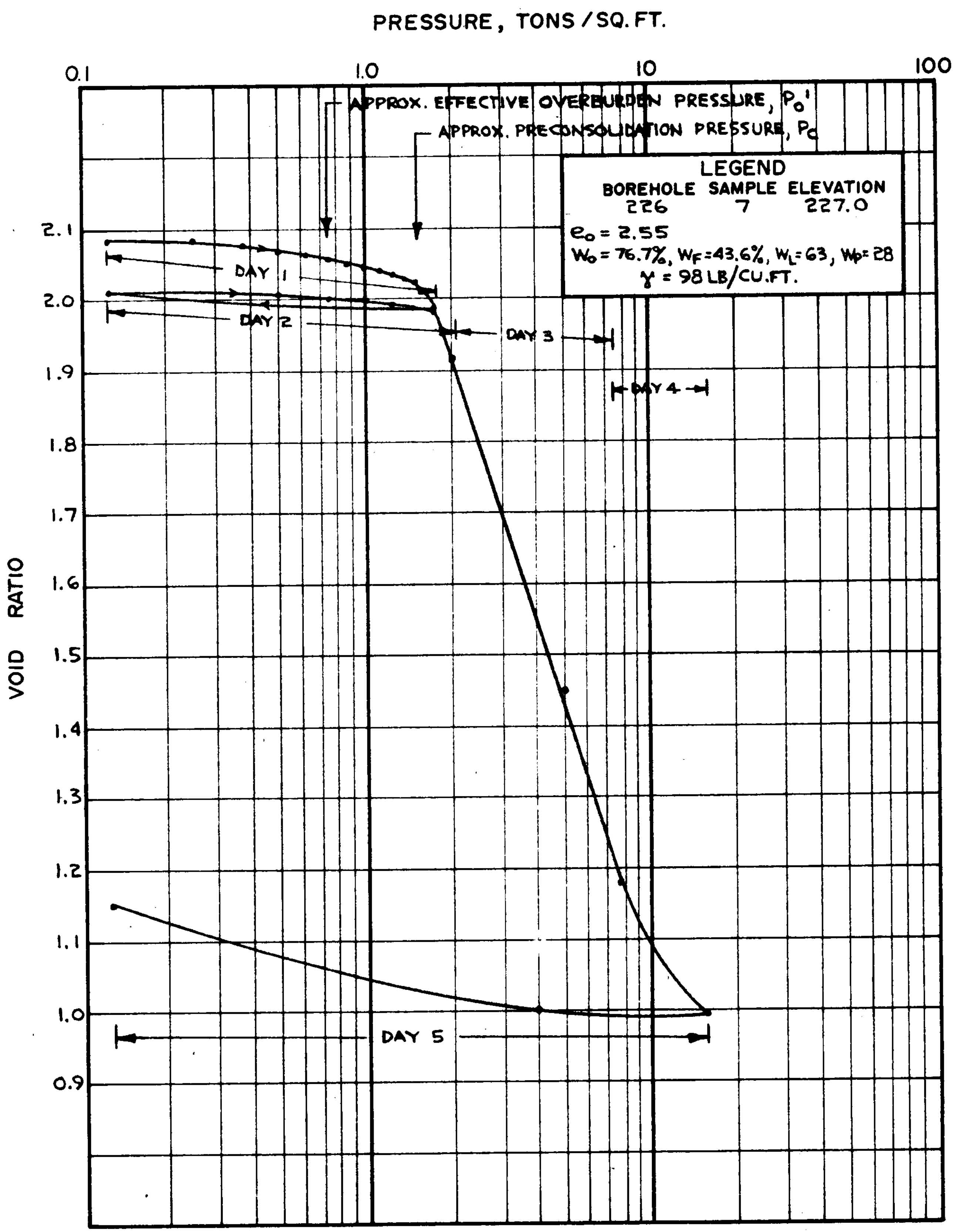
Form G.A.-D.-10

PROJECT No. 741230

PROJECT No. 791-E-30

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 23



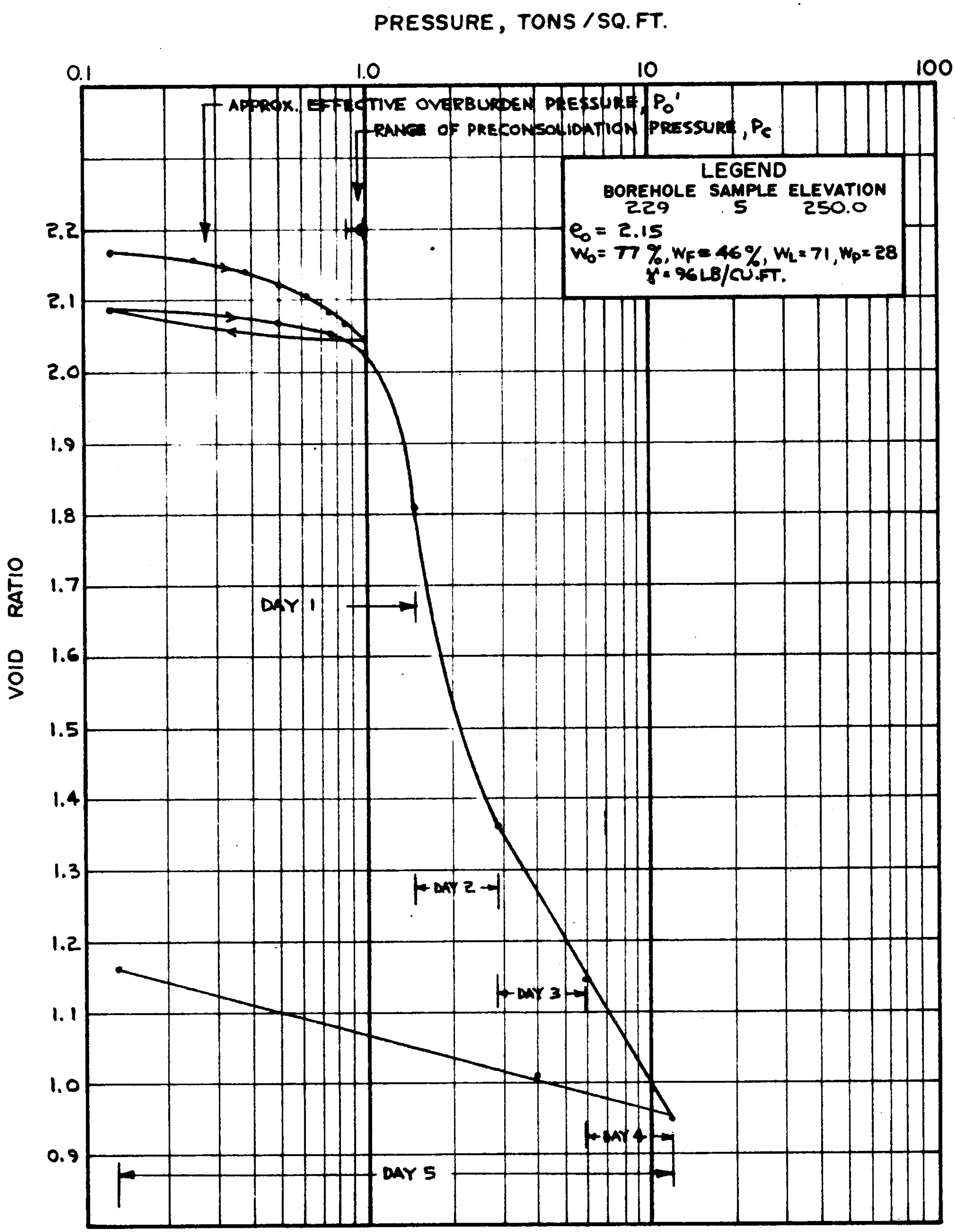
Golder Associates

Form G.A.-D.-10

PROJECT No. 741230

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 24

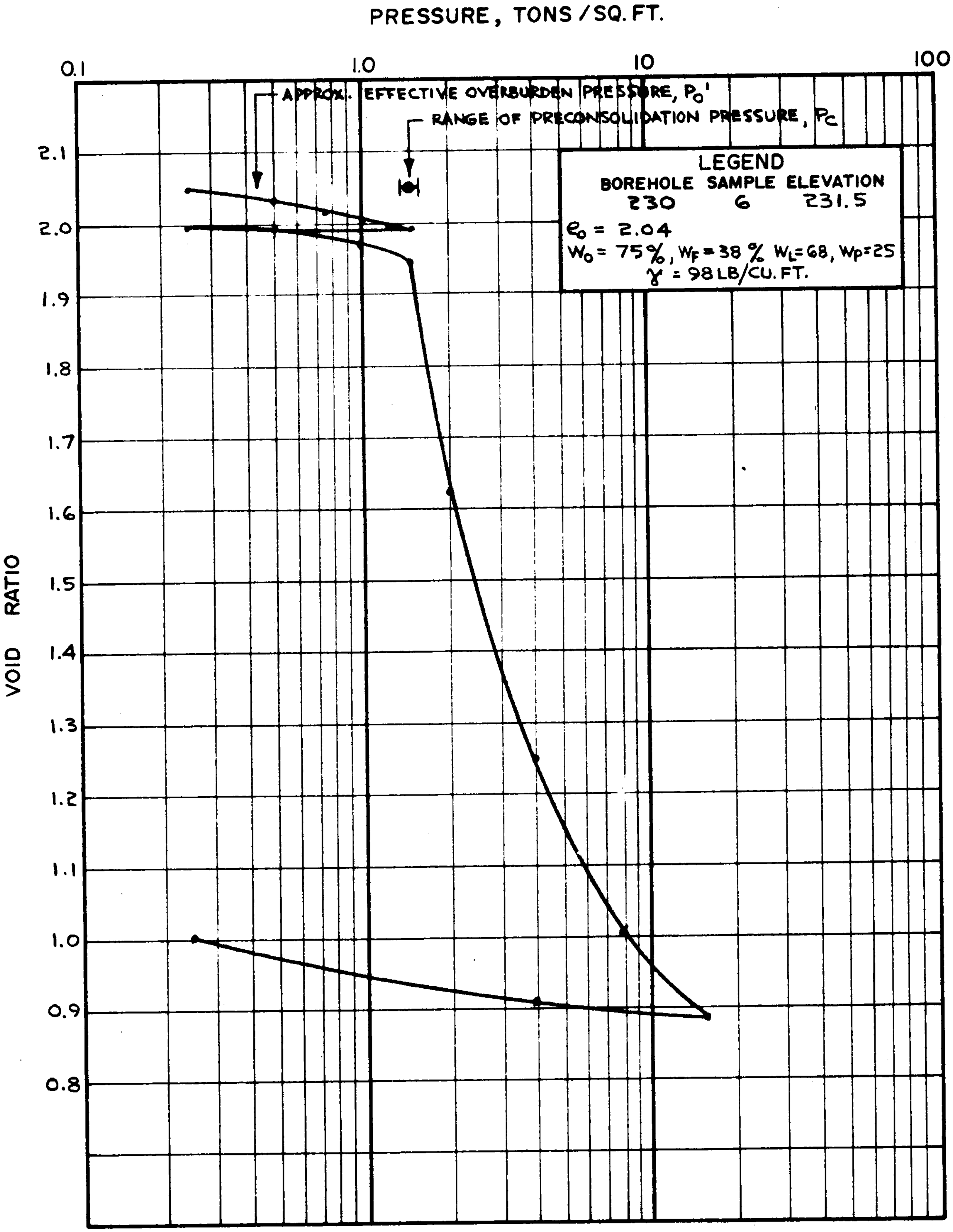


Golder Associates

Form G.A.-D-10

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 25



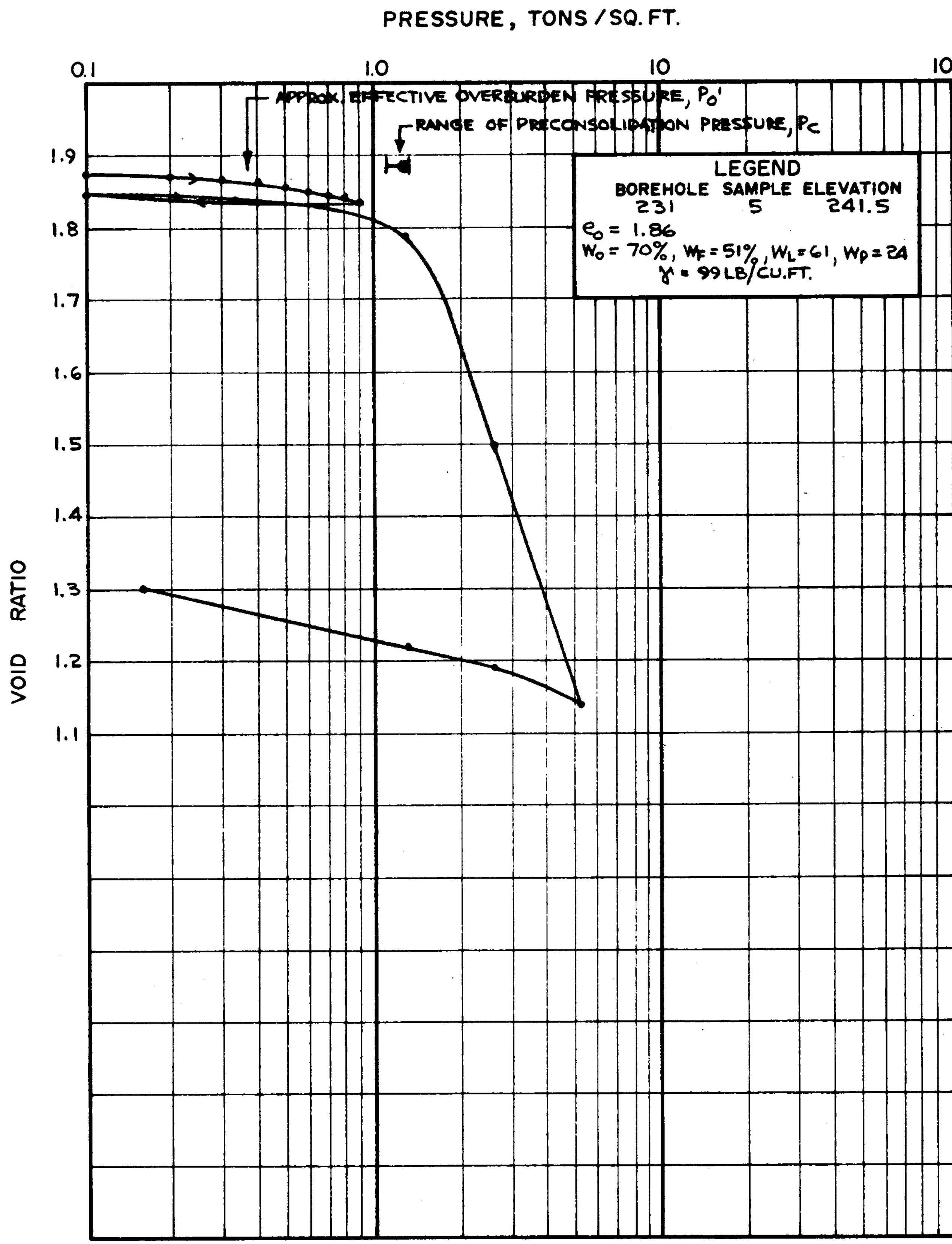
Golder Associates

PROJECT No. 741230

m G-10

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 26

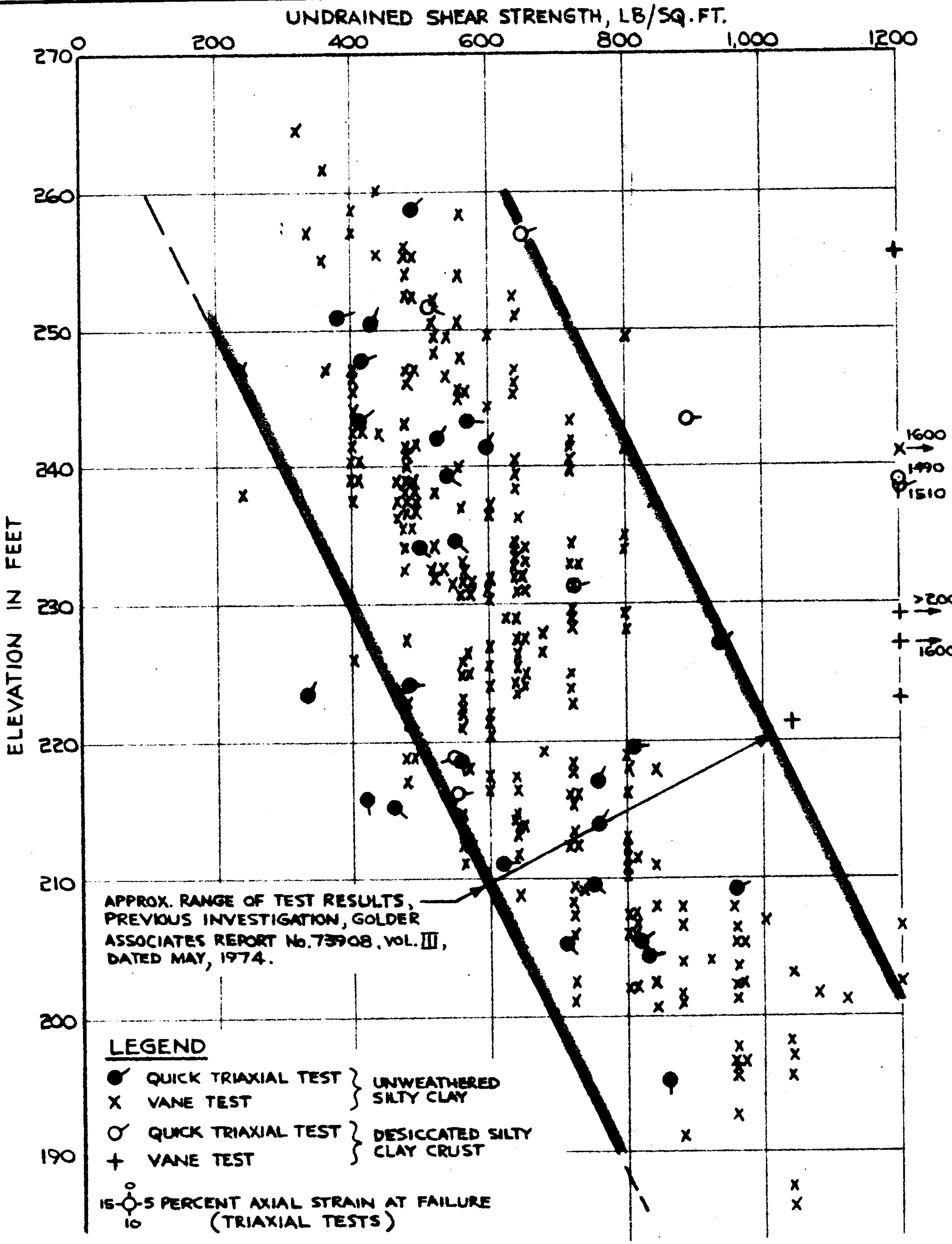


Golder Associates

PROJECT No. 741530

GENERAL PATTERN OF UNDRAINED SHEAR STRENGTH VS ELEVATION

FIGURE 27



Date FEB. 12, 1975

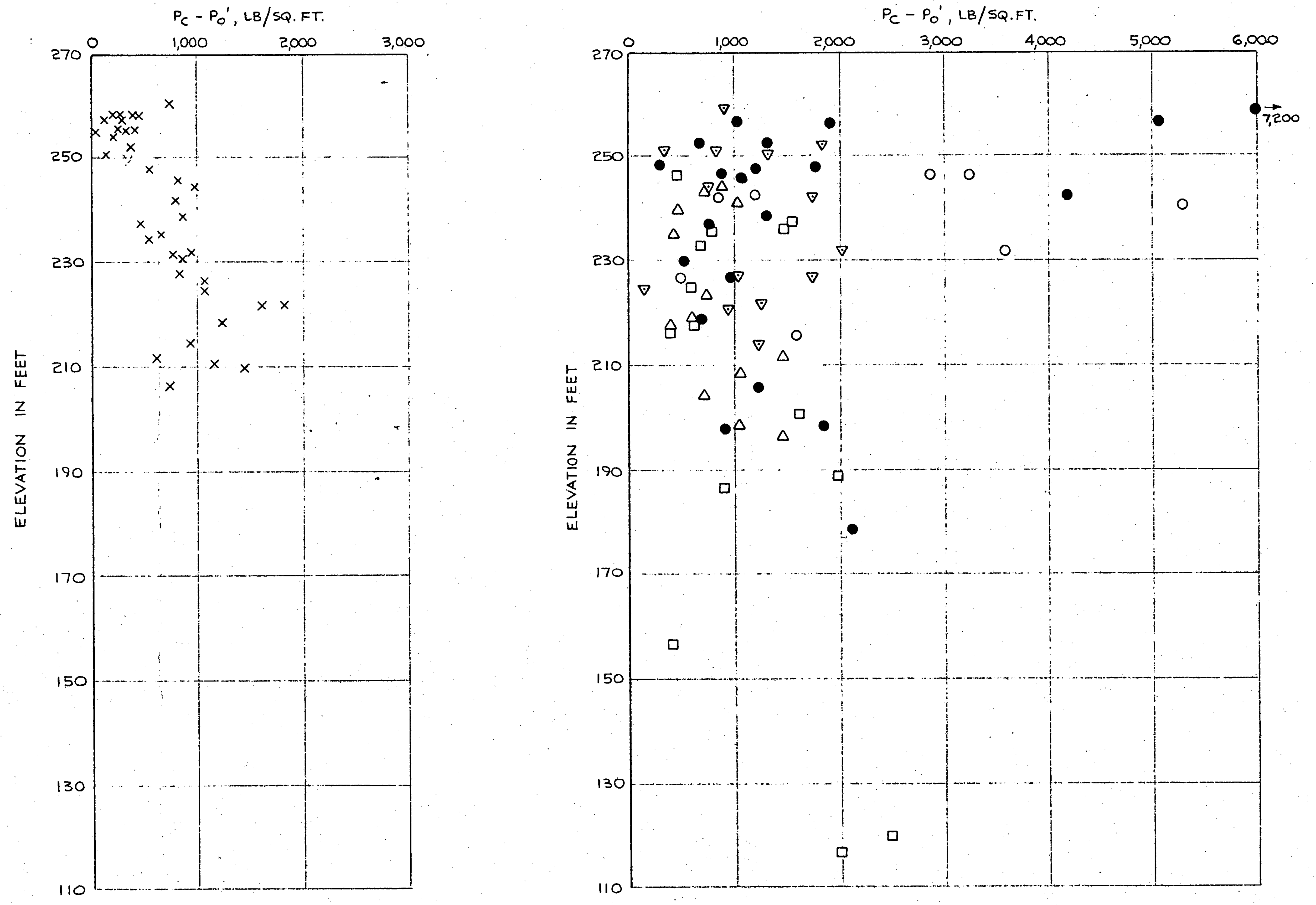
Golder Associates

Drawn J.A.
Chkd. BG
Appd. JHMC

Form G.A. - D. -4

$P_c - P_o'$ VS ELEVATION

FIGURE 28



LEGEND

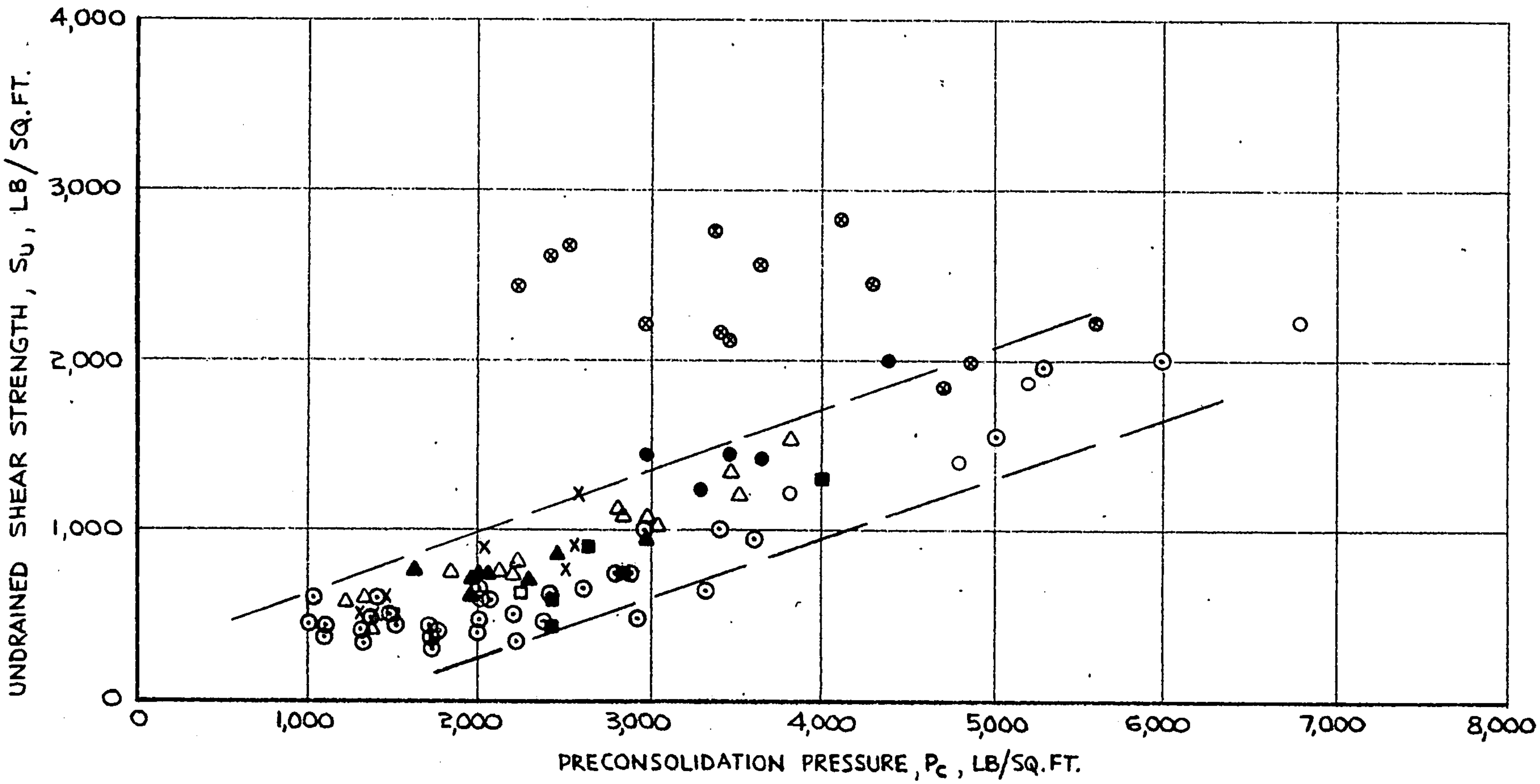
- ▽ GOLDER ASSOCIATES - PRESENT INVESTIGATION
- ● GOLDER ASSOCIATES, PREVIOUS INVESTIGATION (REPORT No. 73908, 1974)
- ○ K.H. KING ASSOCIATES, PREVIOUS INVESTIGATION (REPORT No. 312-S.2, 1974)
- △ HIGHWAY No. 417 AND BOUNDARY ROAD (N.R.C. PRIVATE COMMUNICATION)
- □ HIGHWAY No. 417 SITE INVESTIGATION (M.T.C. PRIVATE COMMUNICATION)
- × GLOUCESTER TEST FILL (BOZOZUK AND LEONARDS, 1972)

Date MARCH 20, 1975

Golder Associates

Drawn J.A.
Chkd. B.G.
Appd. [illegible]

RELATIONSHIP BETWEEN PRECONSOLIDATION PRESSURE AND UNDRAINED SHEAR STRENGTH OF LEDA CLAY



LEGEND

- ● N.R.C., OTTAWA
- ○ GREEN CREEK
- ▲ BEAUHARNOIS, P.Q.
- △ H.M.C.S. GLOUCESTER
- ■ NICOLET, P.Q
- □ RICHARDS LANDING, N.Y.
- ⊗ NATIONAL MUSEUM
- X HAWKESBURY, ONT.

} AFTER W. J. EDEN "DISCUSSION ON BANDED SEDIMENTS", ASTM SPECIAL TECHNICAL PUBLICATION No. 239, 1959.

⊙ GOLDER ASSOCIATES PRESENT INVESTIGATION / GOLDER ASSOCIATES REPORT No. 73908, 1974 / K.H. KING ASSOCIATES REPORT No. 312-5, 2, 1974

NOTE: S_u OBTAINED BY FIELD VANE

FIGURE 29

Golder Associates

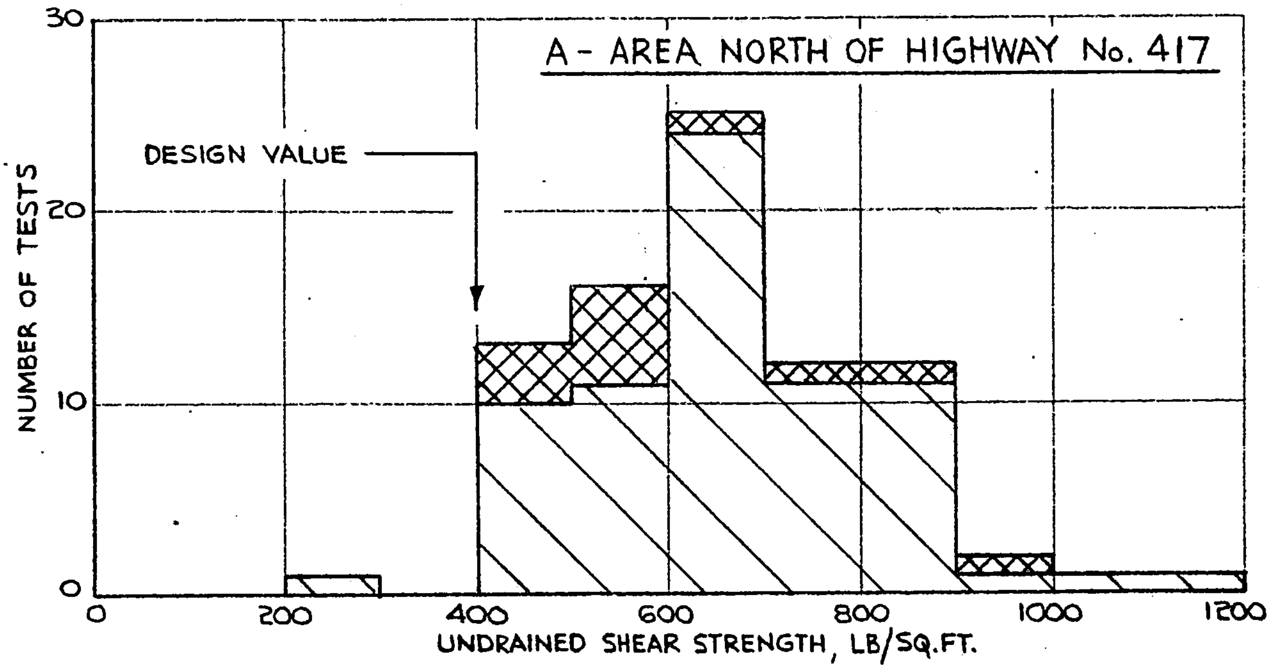
Date FEB. 13, 1975

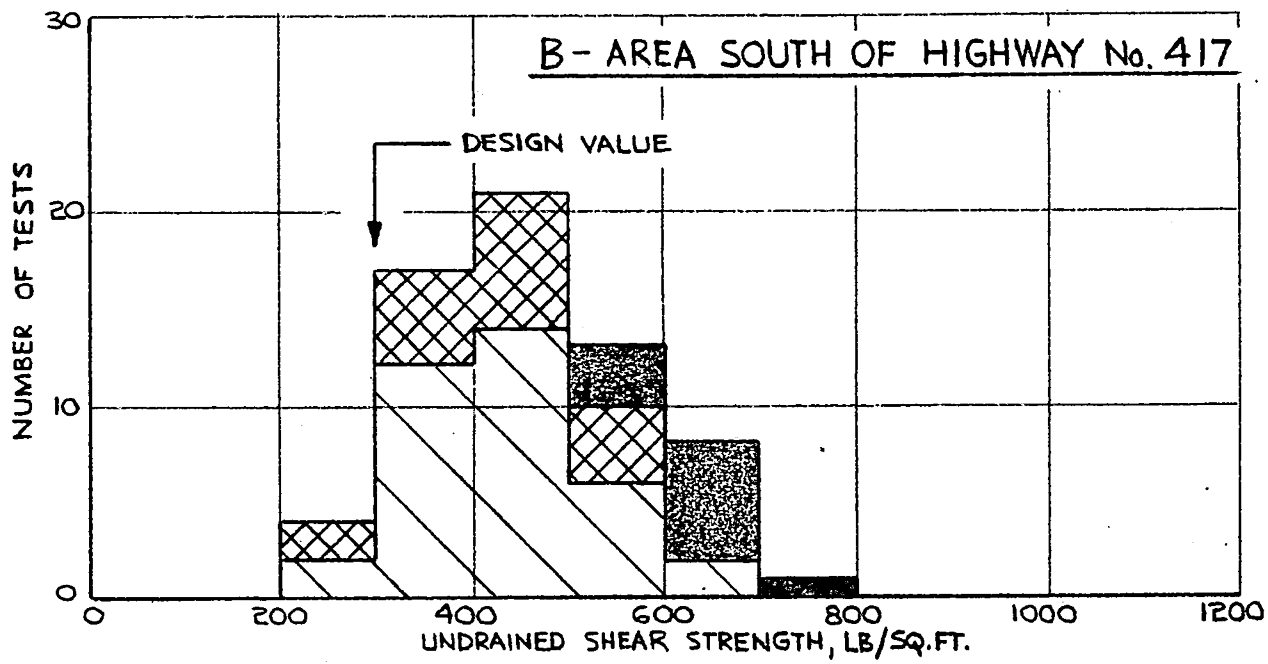
Drawn J.A.
Chkd. RJ
Appd. [illegible]

PROJECT No. 741230

DISTRIBUTION OF UNDRAINED SHEAR STRENGTH IN UPPER 10' OF UNWEATHERED CLAY

FIGURE 30





NOTES

1) DATA IN (A) OBTAINED FROM PRESENT INVESTIGATION

2) DATA IN (B) OBTAINED FROM PRESENT INVESTIGATION AND PREVIOUS INVESTIGATION, GOLDER ASSOCIATES REPORT No. 73908, VOL. IV DATED JUNE, 1974.

LEGEND

FIELD VANE TESTS

UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS ON TUBE SAMPLES

UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS ON BLOCK SAMPLES

Date FEB. 13, 1975

Golder Associates

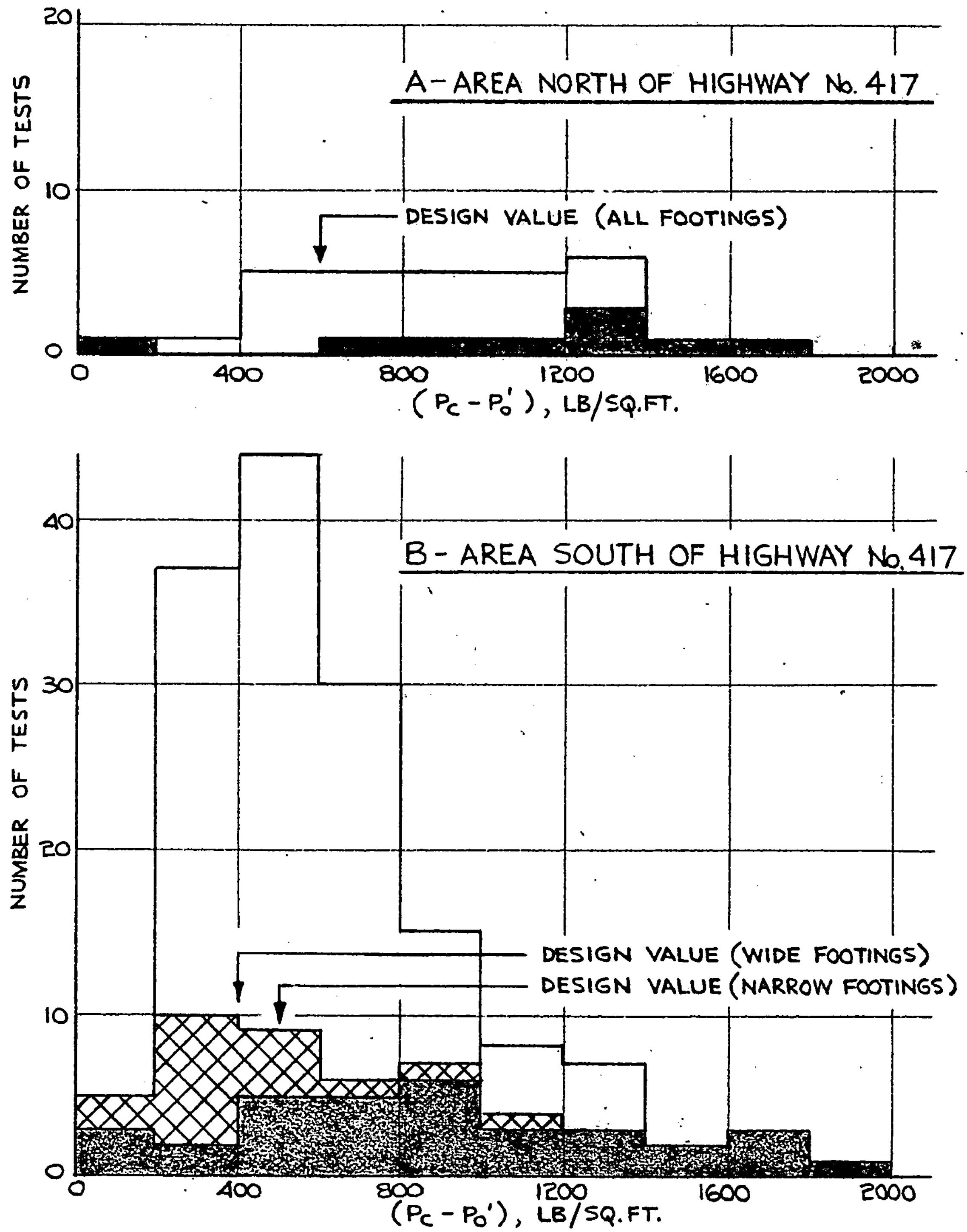
Drawn J.A.
Chkd. BG
Appd. [illegible]

Form G.A. - D. -4.

PROJECT No. 741230

DISTRIBUTION OF $P_c - P_o'$ IN UPPER 20' OF UNWEATHERED CLAY

FIGURE 31



LEGEND

A - CONSOLIDATION TESTS, PRESENT INVESTIGATION
B - CONSOLIDATION TESTS, DATA FROM GOLDER ASSOCIATES REPORT No. 73908 VOLS. III & IV, DATED MAY/JUNE, 1974.

CONSOLIDATION TESTS, GLOUCESTER TEST FILL

INFERRED FROM S_u (VANE) RESULTS (IN EACH BOREHOLE MINIMUM VALUE TAKEN)

Date FEB. 12, 1975

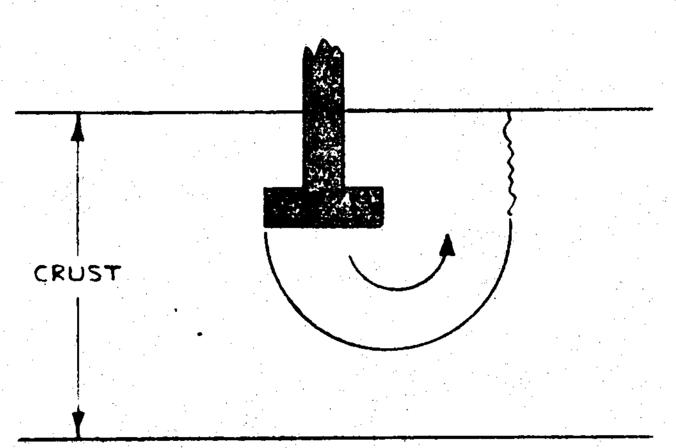
Golder Associates

Drawn J.A.
Chkd. BG
Appd. [illegible]

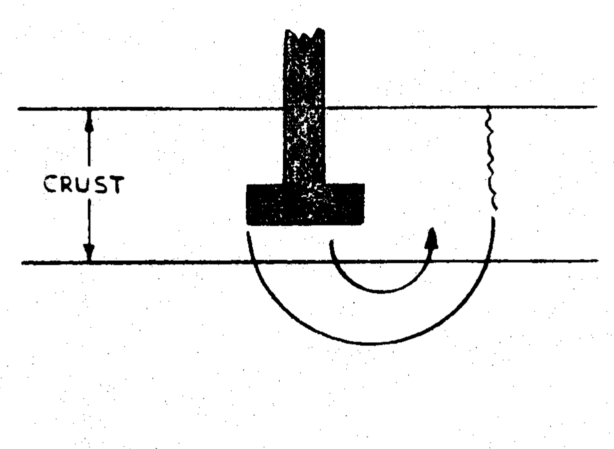
Form G.A. - D. - 4.

FACTORS AFFECTING ALLOWABLE BEARING CAPACITY OF SHALLOW FOUNDATIONS

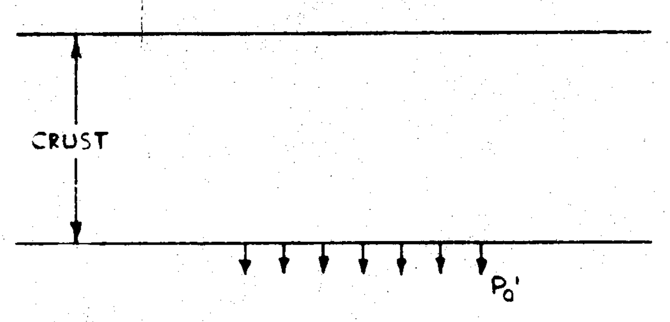
FIGURE 32



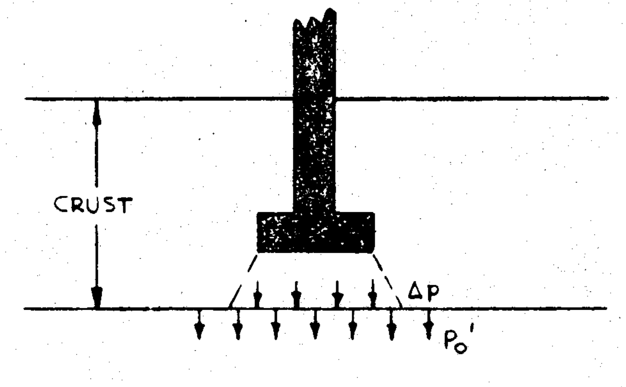
I - BEARING CAPACITY FAILURE IN CRUSTAL SOIL



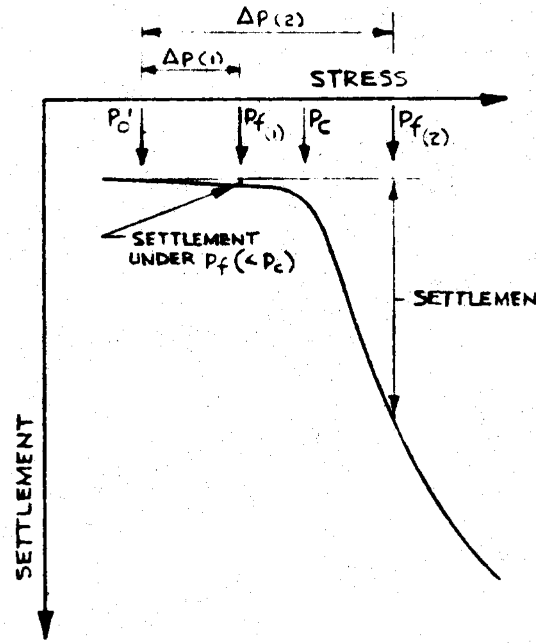
II - BEARING CAPACITY FAILURE IN "SOFT" UNWEATHERED CLAY



INITIAL STRESS ON UNWEATHERED CLAY DUE TO WEIGHT OF SOIL (P_o')



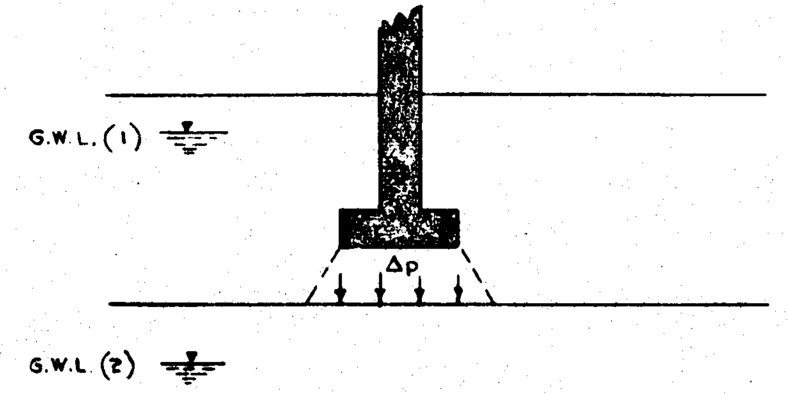
FINAL STRESS ON UNWEATHERED CLAY (P_f) DUE TO WEIGHT OF SOIL (P_o') AND FOOTING LOADING (Δp)
(i.e. $P_f = P_o' + \Delta p$)



NOTE: - AT $P_{f(1)} < P_c$ RESULTING SETTLEMENT IS SMALL
- AT $P_{f(2)} > P_c$ RESULTING SETTLEMENT IS LARGE

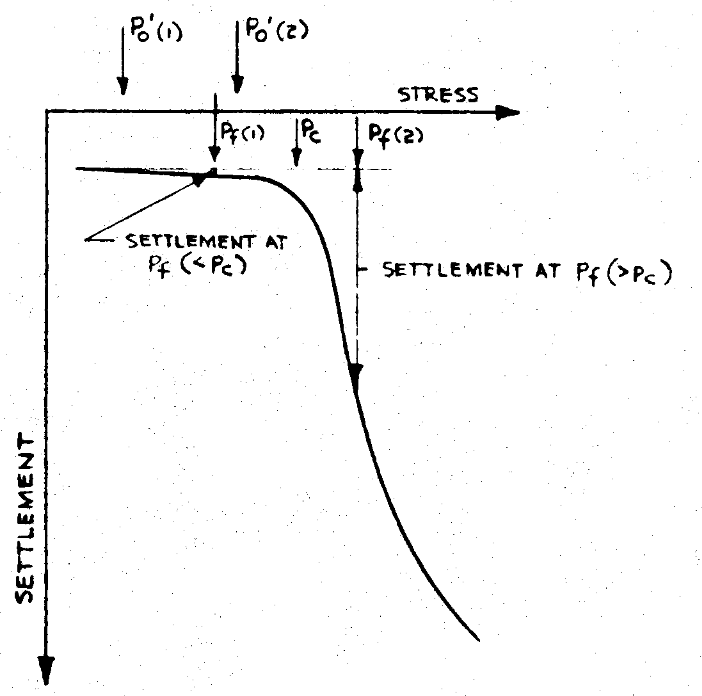
III - "SETTLEMENT" CRITERIA

A - SOIL PROPERTIES

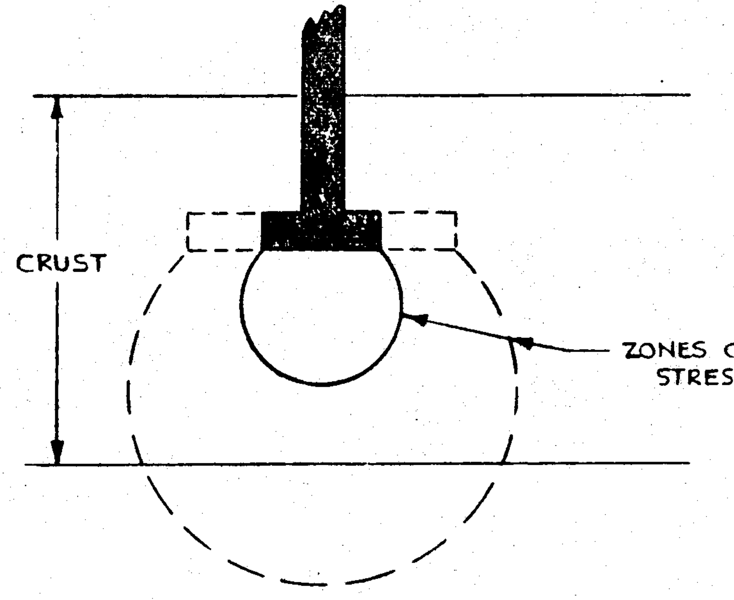


NOTE: - LOWER GROUNDWATER LEVEL INCREASES P_o', THEREFORE (WITH CONSTANT Δp) FINAL STRESS (P_f) IS INCREASED

IF $P_{f(1)} < P_c$ THEN SETTLEMENT IS SMALL
IF $P_{f(2)} > P_c$ THEN SETTLEMENT IS LARGE

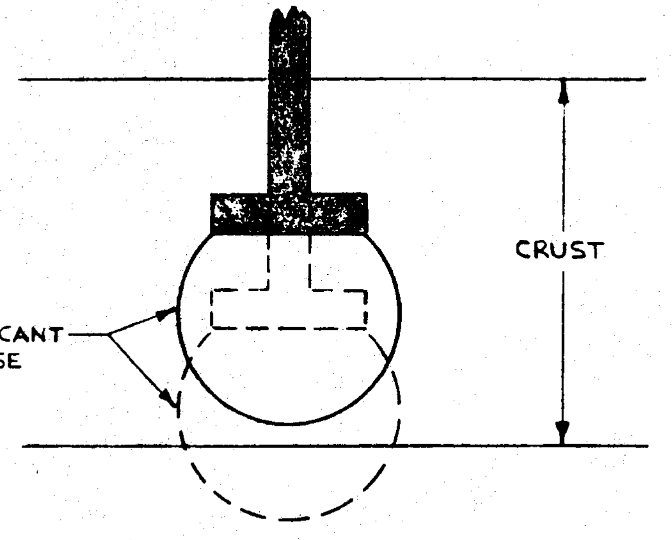


B - GROUNDWATER LOWERING



NOTE: - AT CONSTANT FOOTING ELEVATION INCREASING FOOTING WIDTH INCREASES THICKNESS OF STRESSED ZONE OF UNWEATHERED CLAY.

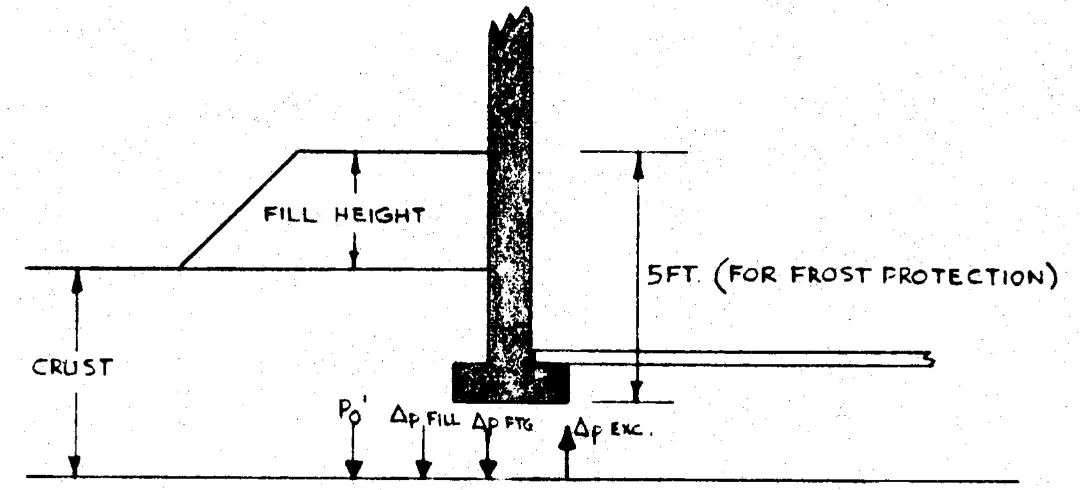
I - EFFECT OF WIDTH OF FOOTING



NOTE: - FOR GIVEN FOOTING WIDTH, LOWERING FOUNDING ELEVATION CAUSES INCREASED STRESSES TO BE APPLIED TO UNWEATHERED CLAY.

II - EFFECT OF FOOTING DEPTH

C - FOOTING LOCATION AND GEOMETRY



NOTE: - TOTAL FINAL STRESS (P_f) ON UNWEATHERED CLAY ZONE $= P_o' + \Delta p_{FILL} + \Delta p_{FTG.} - \Delta p_{EXC}$

FOR SMALL SETTLEMENT FINAL STRESS MUST BE LESS THAN PRECONSOLIDATION PRESSURE (P_c)

D - STRUCTURE TYPE

NOTE:

P_o' = EFFECTIVE OVERBURDEN PRESSURE

P_c = PRECONSOLIDATION PRESSURE

Δp = STRESS INCREASE

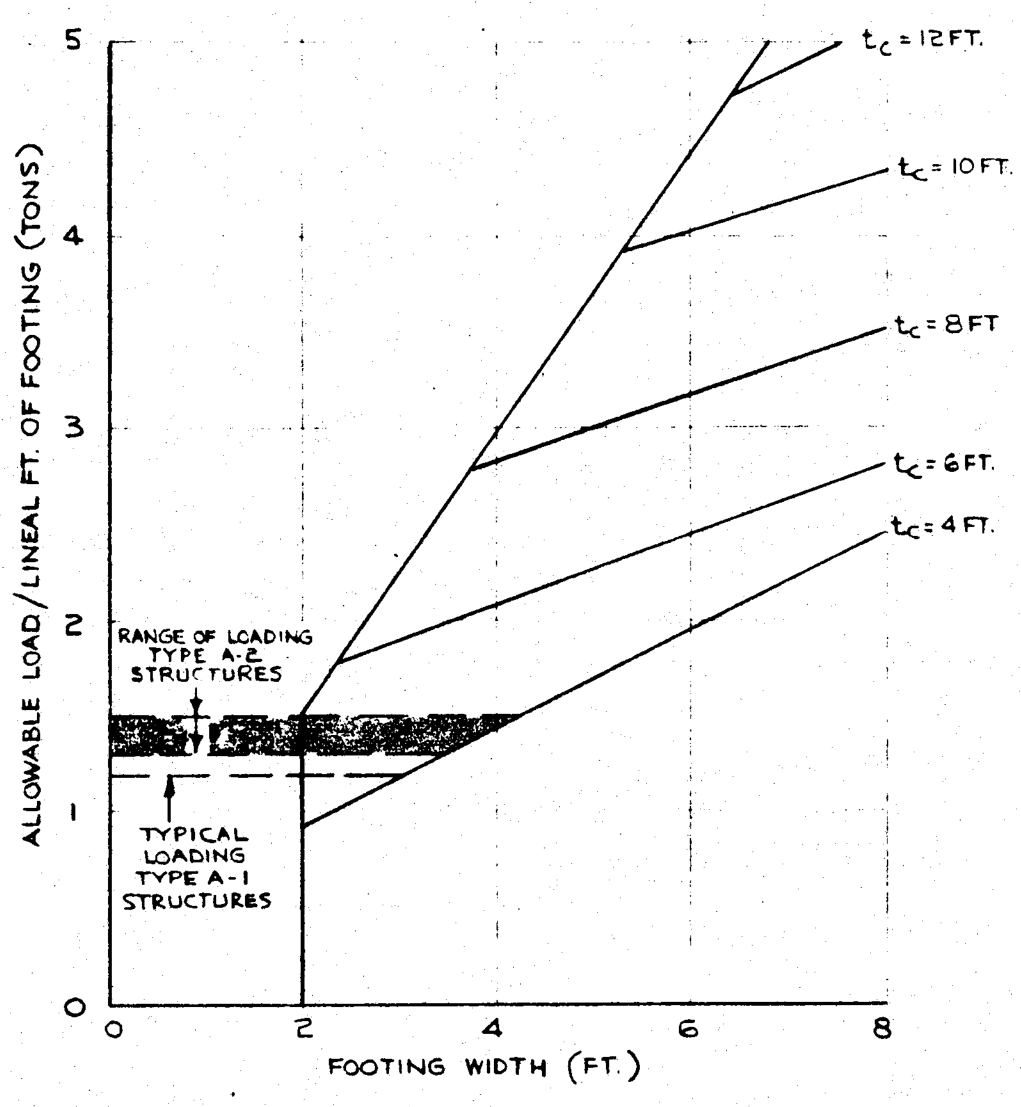
Date MARCH 12, 1975

Golder Associates

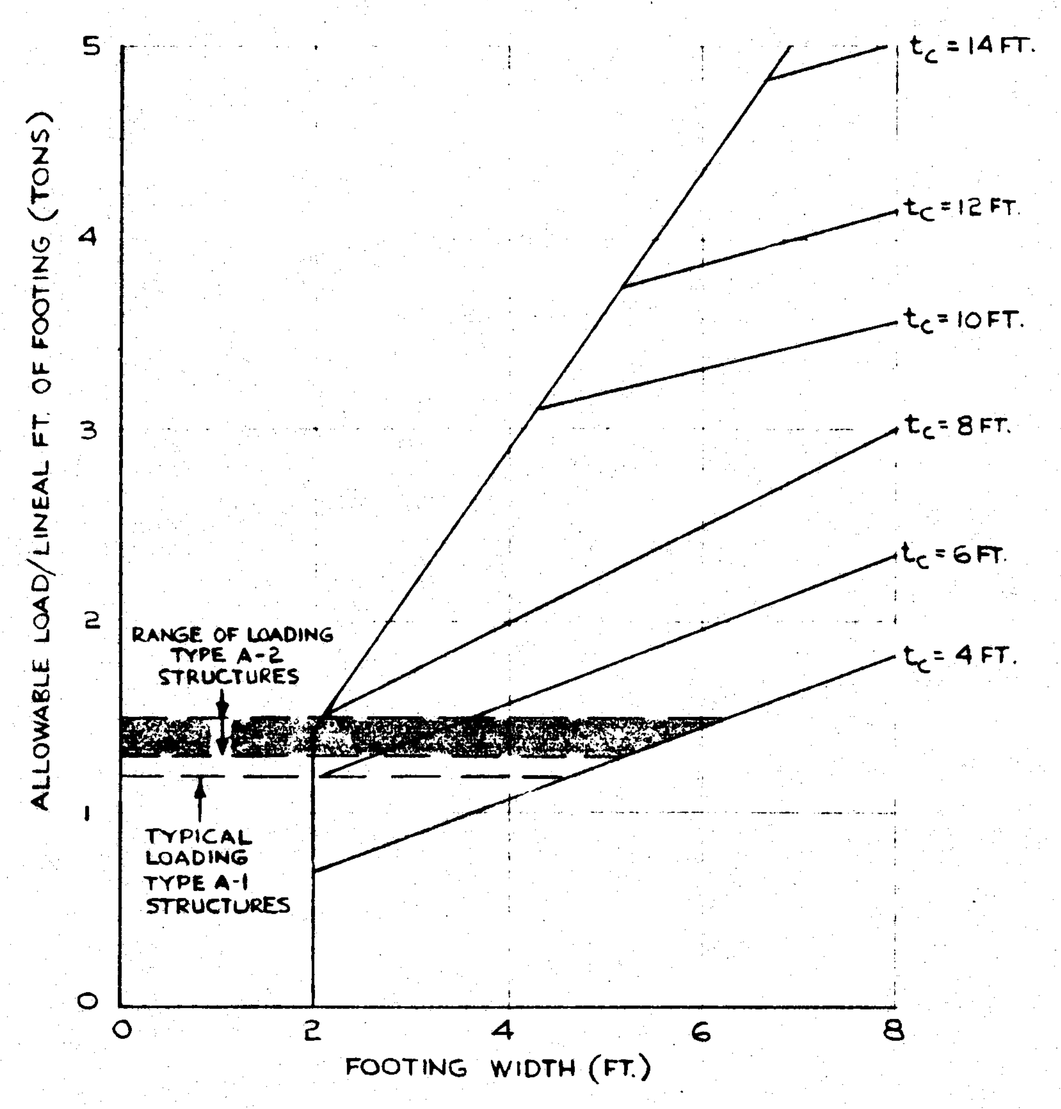
Drawn J.A.
Chkd. [illegible]
Appd. [illegible]

STRIP FOOTINGS FOR RESIDENTIAL STRUCTURES (TYPE 'A')

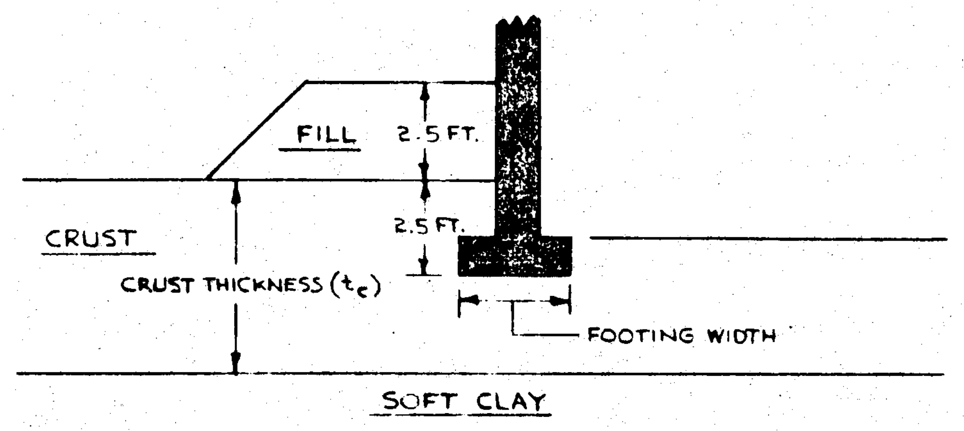
FIGURE 33



AREA NORTH OF HIGHWAY No. 417



AREA SOUTH OF HIGHWAY No. 417



FOUNDATION GEOMETRY AND DESIGN PARAMETERS

CRUST – S_u = 800 LB/SQ. FT.

SOFT CLAY – S_u = 300 LB/SQ. FT., $P_c - P_0' \approx$ 500 LB/SQ. FT. — AREA SOUTH OF HIGHWAY No. 417

S_u = 400 LB/SQ. FT., $P_c - P_0' \approx$ 600 LB/SQ. FT. — AREA NORTH OF HIGHWAY No. 417

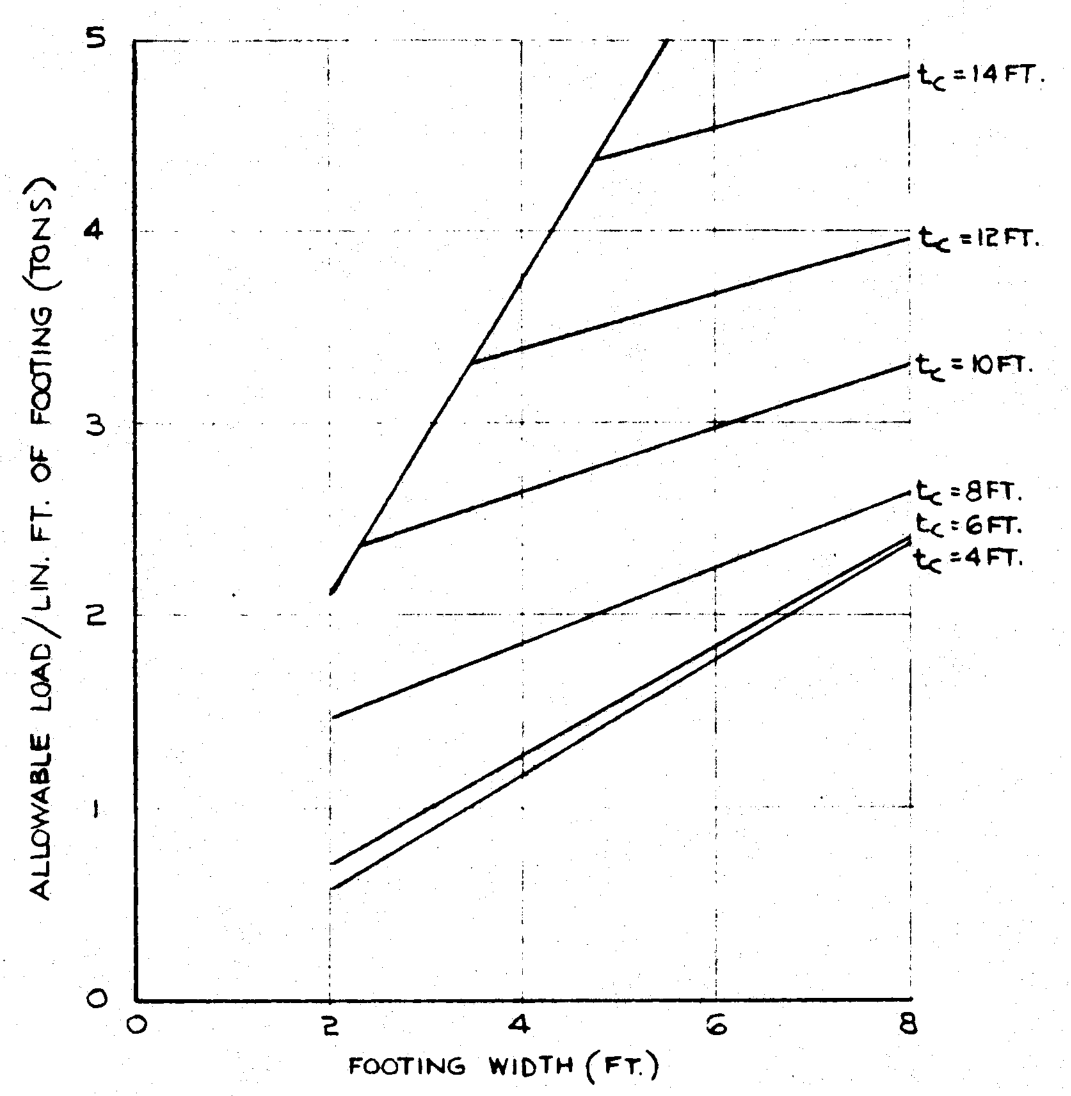
Date FEB. 26, 1975

Golder Associates

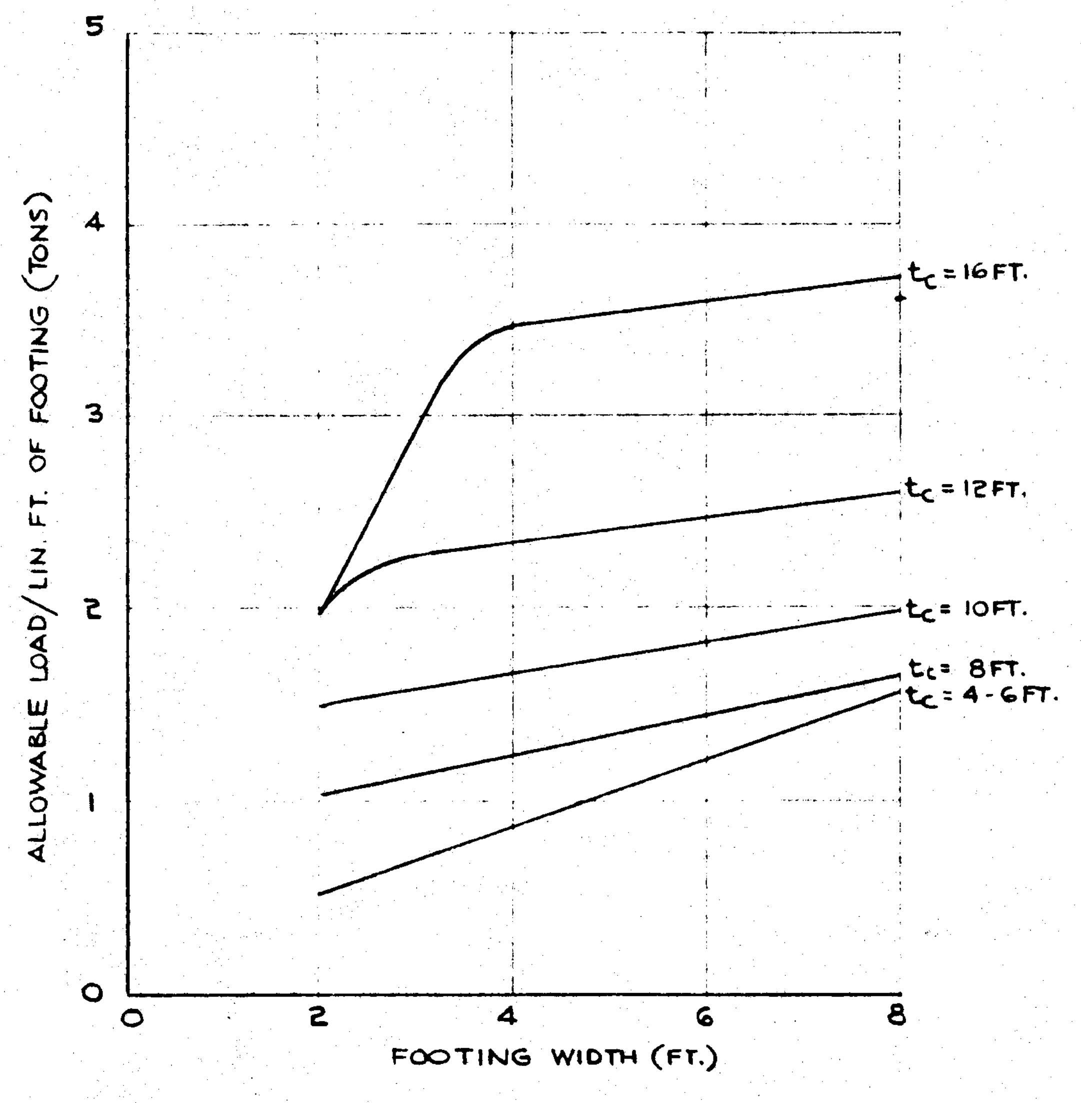
Drawn J.A.
Chkd. [illegible]
Appd. [illegible]

STRIP FOOTINGS FOR STRUCTURES NOT REQUIRING BASEMENTS (TYPE 'B')

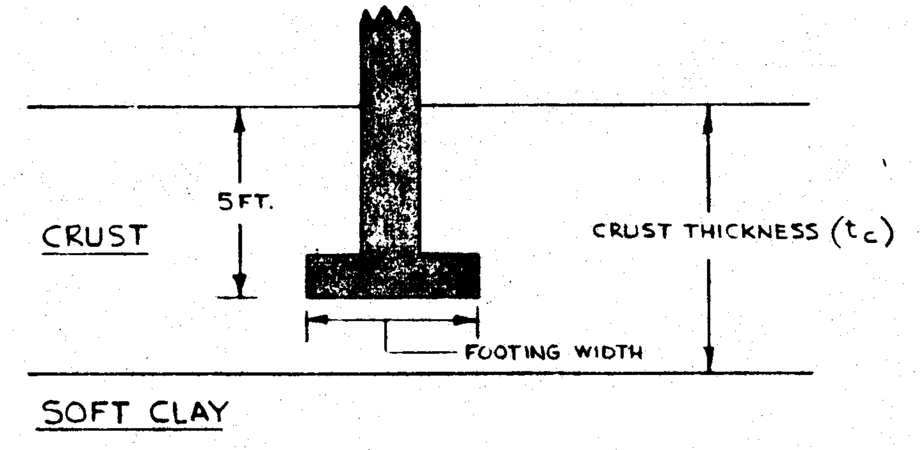
FIGURE 34



AREA NORTH OF HIGHWAY No. 417



AREA SOUTH OF HIGHWAY No. 417



FOUNDATION GEOMETRY AND DESIGN PARAMETERS

CRUST S_u = 800 LB/SQ.FT.

SOFT CLAY
S_u = 300 LB/SQ.FT.
$P_c - P_o'$ ≈ 500 LB/SQ.FT.
AREA SOUTH OF HIGHWAY No. 417

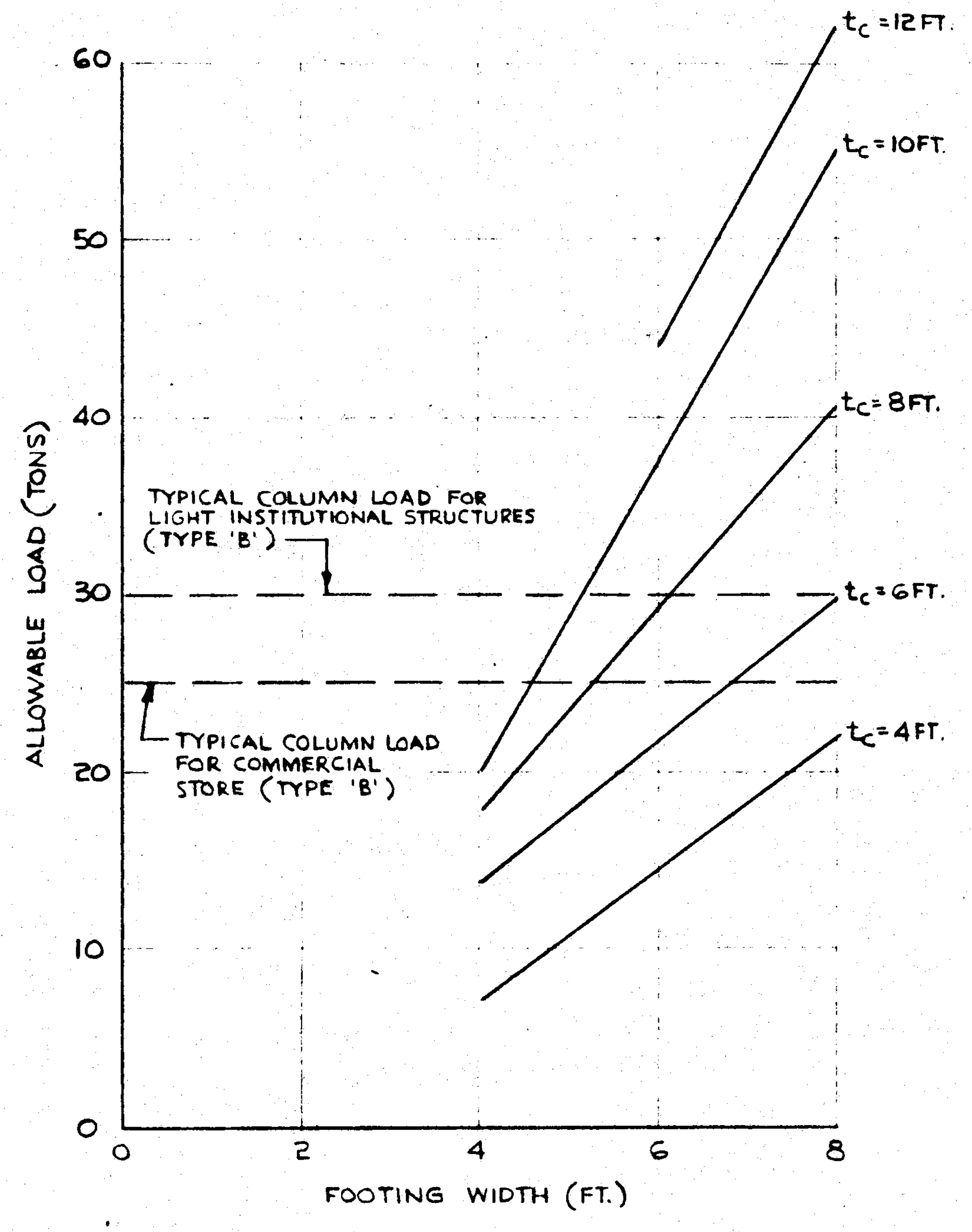
S_u = 400 LB/SQ.FT.
$P_c - P_o'$ ≈ 600 LB/SQ.FT.
AREA NORTH OF HIGHWAY No. 417

Date FEB. 27, 1975

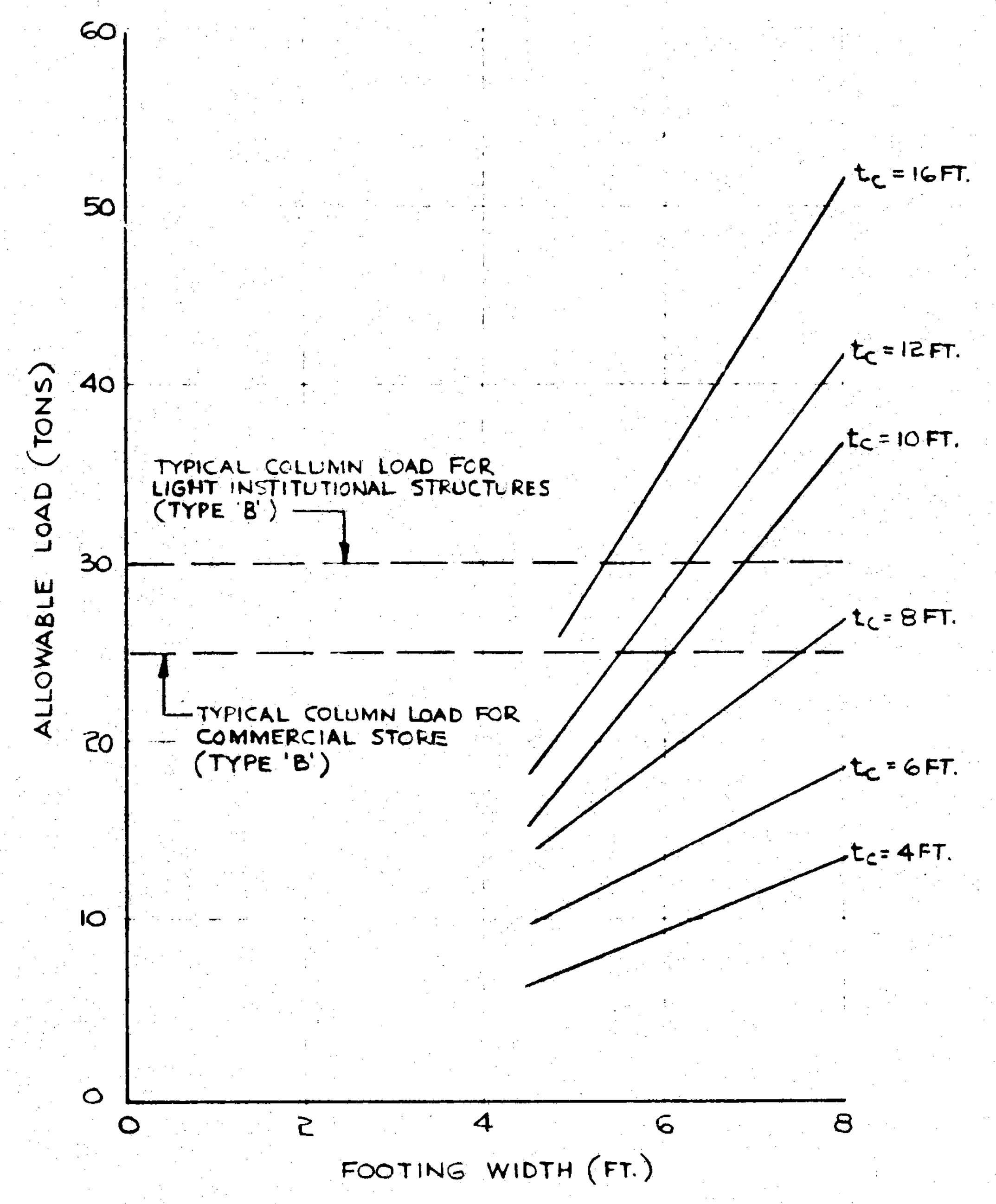
Golder Associates

Drawn J.A.
Chkd. BG
Appd. HAC

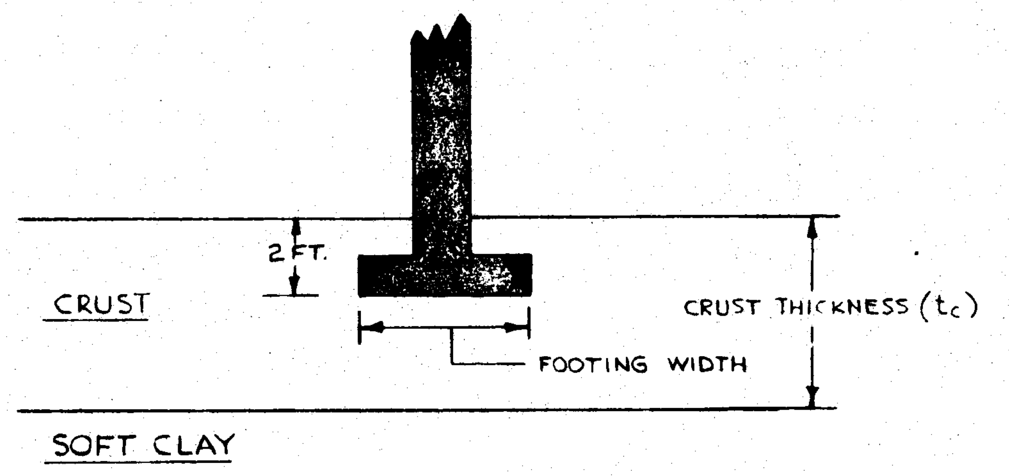
ALLOWABLE LOADS FOR INTERNAL SPREAD FOOTINGS	FIGURE 35



AREA NORTH OF HIGHWAY No. 417



AREA SOUTH OF HIGHWAY No. 417



FOUNDATION GEOMETRY AND DESIGN PARAMETERS

CRUST S_u = 800 LB/SQ.FT.

SOFT CLAY S_u = 300 LB/SQ.FT., $P_c - P_o'$ ≈ 500 LB/SQ.FT. } AREA SOUTH OF HIGHWAY No. 417

S_u = 400 LB/SQ.FT., $P_c - P_o'$ ≈ 600 LB/SQ.FT. } AREA NORTH OF HIGHWAY No. 417

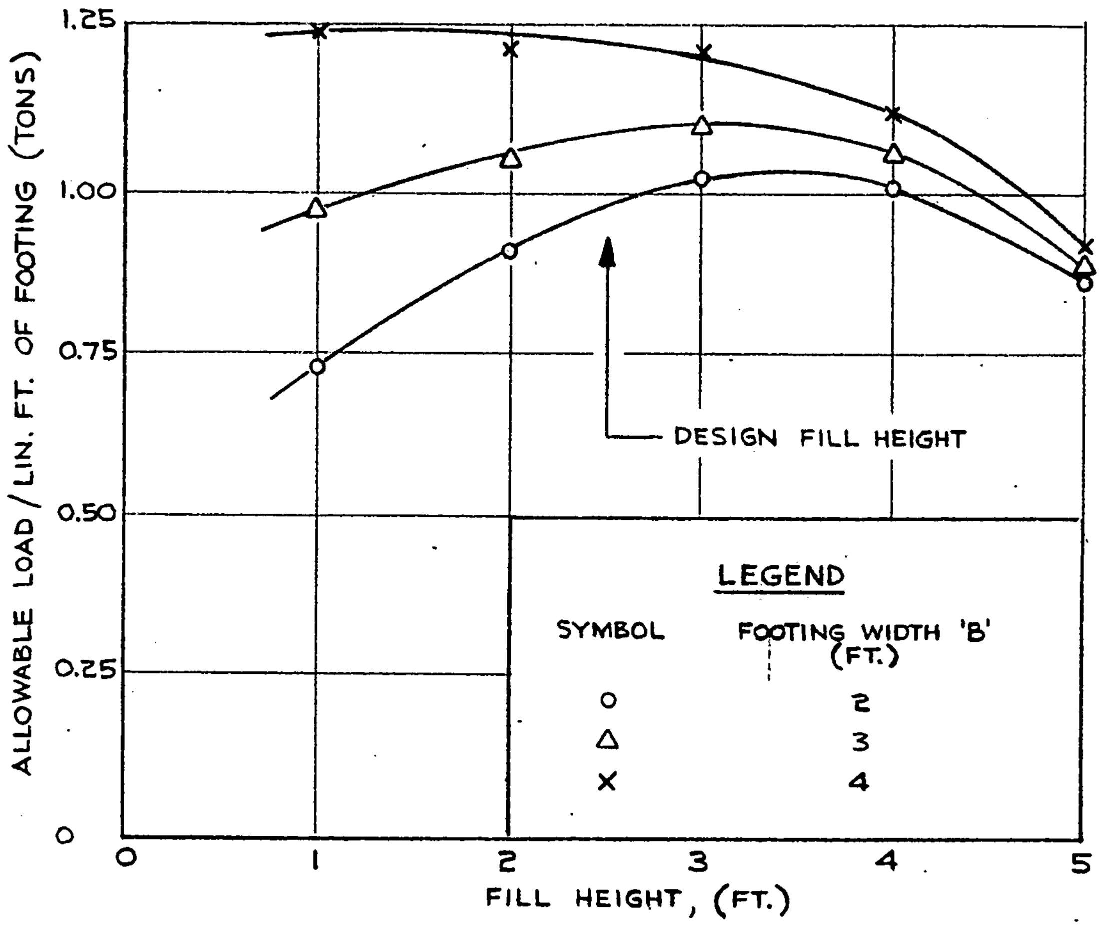
Date FEB. 27, 1975

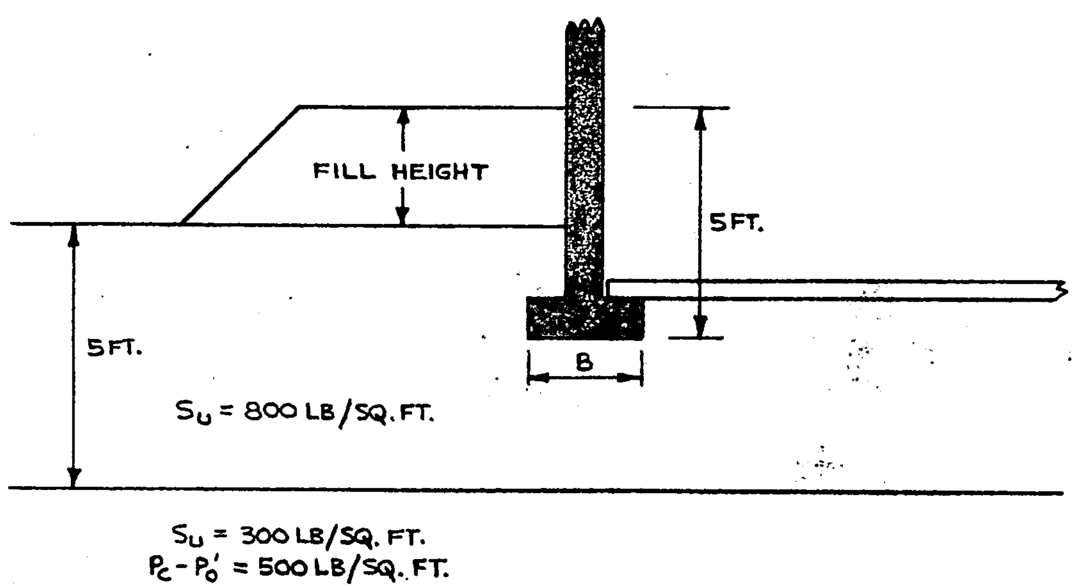
Golder Associates

Drawn J.A.
Chkd. R.G.
Appd. J.H.K.

EFFECT OF FILL HEIGHT ON ALLOWABLE BEARING PRESSURE OF SHALLOW STRIP FOOTINGS

FIGURE 36





Date MARCH 11, 1975

Golder Associates

Drawn J.A.
Chkd. BG
Appd. [illegible]

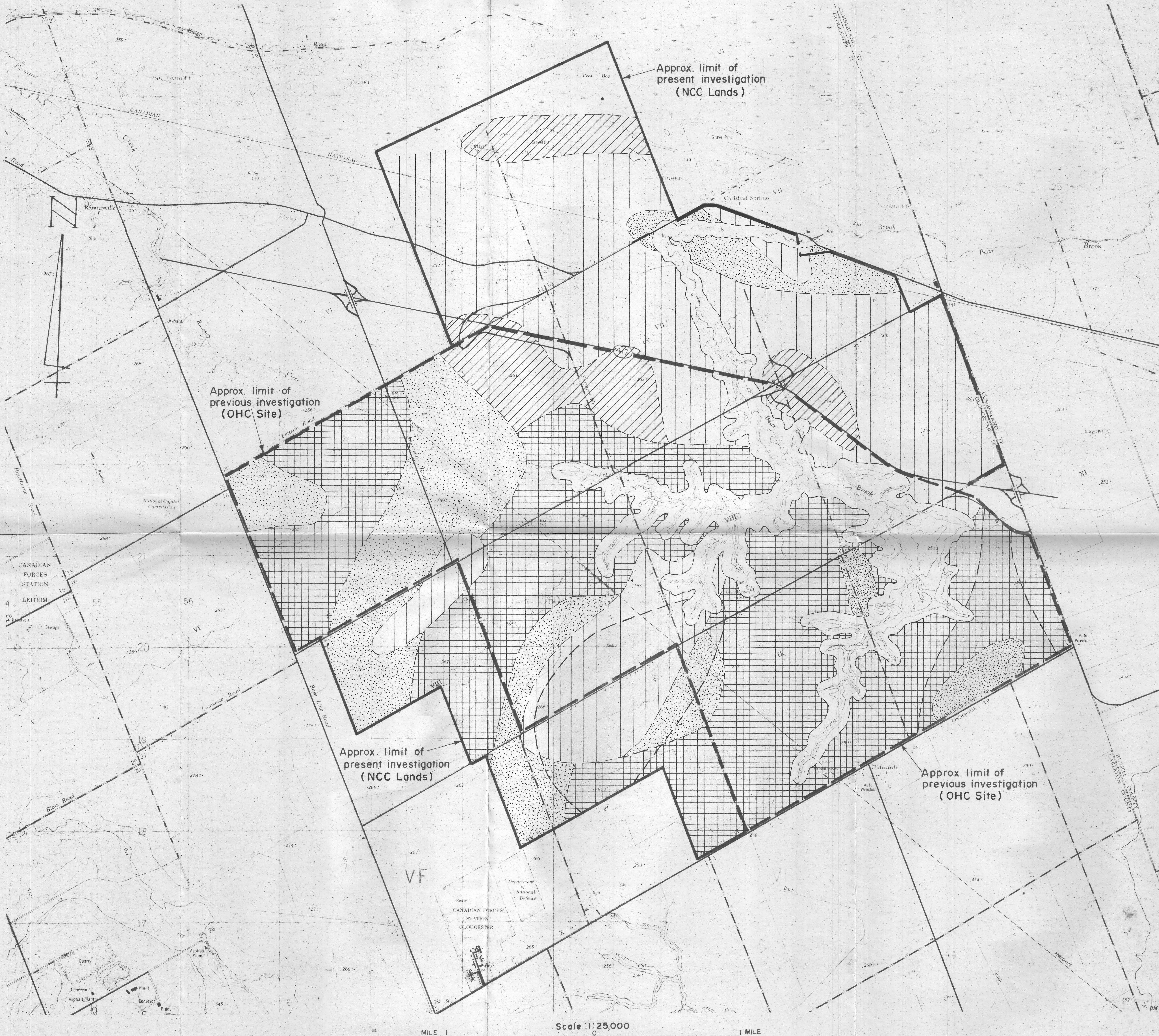
PROJECT No. 741230

Form G.A - D - 4

741230

LAND USE WITH RESPECT TO BUILDING FOUNDATIONS

FIGURE 37



LEGEND

TYPE OF STRUCTURE

- A-1
- A-2
- A-3 & B
- C
- STRUCTURES WITH PILED FOUNDATIONS D
- BEAR BROOK AND TRIBUTARY AREA

NOTE

FOR DETAILED SUMMARY OF STRUCTURE TYPES REFER TO TABLE I

TOPOGRAPHIC INFORMATION based on national topographic system maps EDWARDS 31G/6d, NAVAN 31G/6e, ed. 2 SOUTH GLOUCESTER 31G/5a, BLACKBURN 31G/5h ed. 3

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

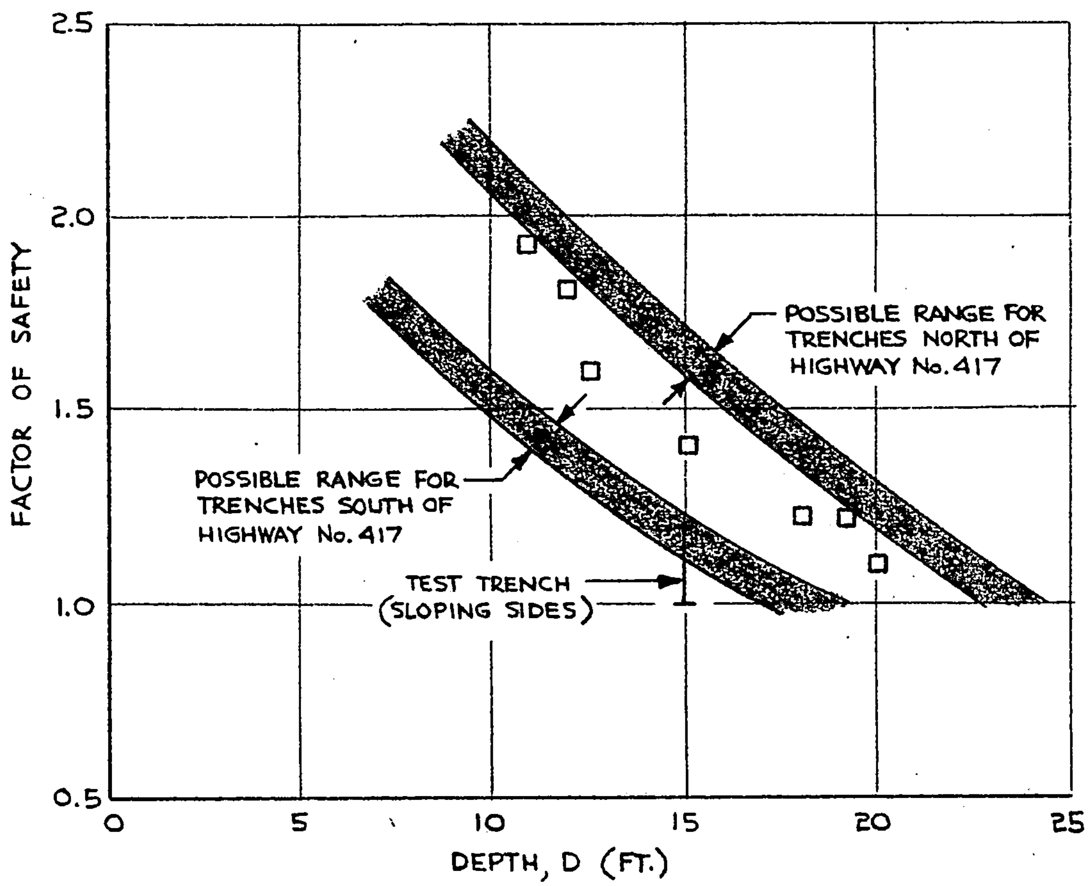
Date: MARCH 18, 1975

Golder Associates

Drawn J.A.
Chkd. BG
Appd. JHKC

BASAL STABILITY OF TRENCHES

FIGURE 38



NOTES: FACTOR OF SAFETY COMPUTED ASSUMING S_u (MIN.) = 300 LB/SQ.FT. IN AREA SOUTH OF HWY. No. 417
= 400 LB/SQ.FT. IN AREA NORTH OF HWY. No. 417

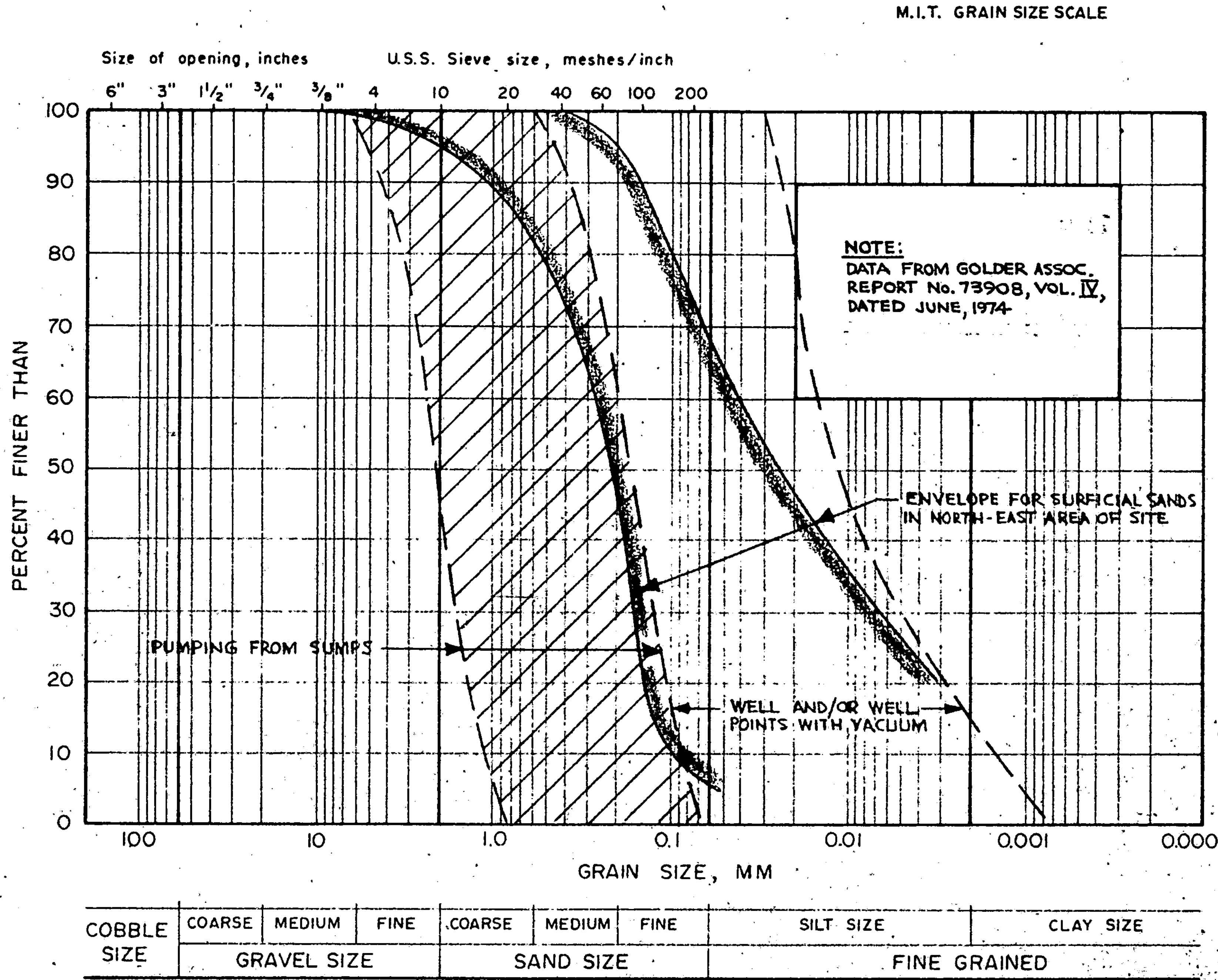
□ - TEST PITS (APPROXIMATELY SQUARE) PUT DOWN ON OHC SITE (SEE GOLDER ASSOCIATES REPORT No. 73908, VOL. III, DATED MAY, 1974.)

TEST TRENCH PUT DOWN USING CONSTRUCTION SIDE SLOPES APPROX. 1 HORIZONTAL TO 1 VERTICAL

Date MARCH 20, 1975

Golder Associates

Drawn J.A.
Chkd. BG
Appd. JHK



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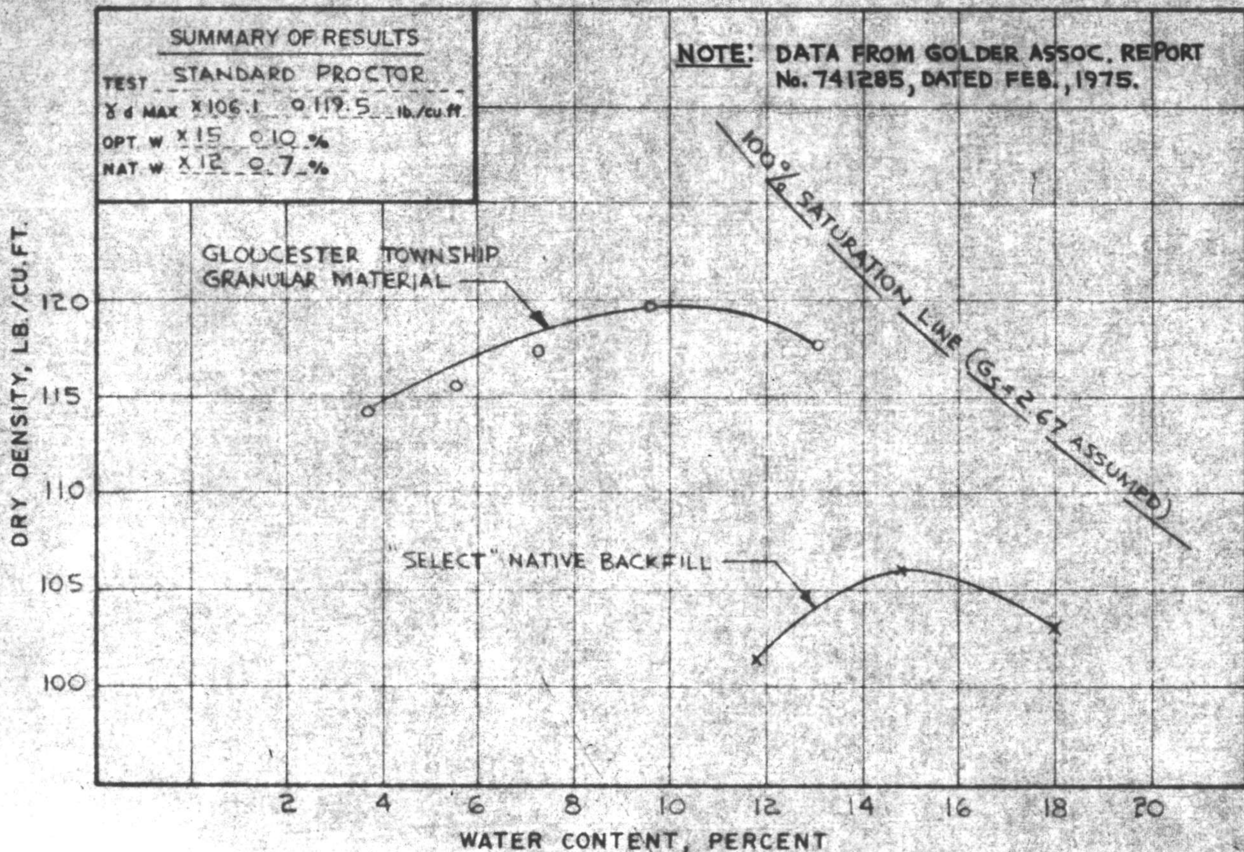
GRAIN SIZE DISTRIBUTION

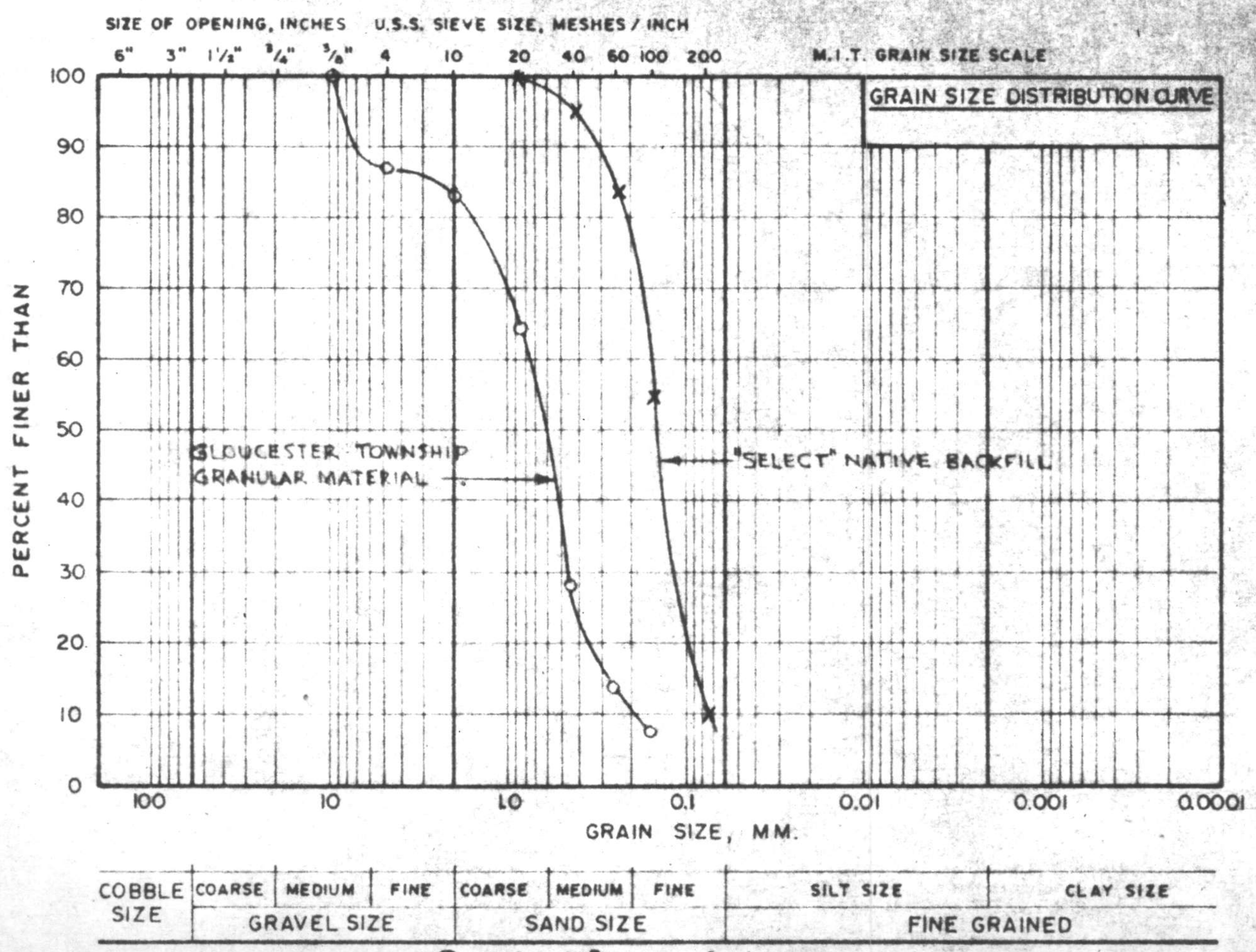
DEWATERING TECHNIQUES APPLICABLE TO SURFICIAL SANDS

FIGURE 39

LABORATORY COMPACTION TEST RESULTS

FIGURE 40





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PROJECT No. 741230

Form G.A.-D-9