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## ABSTRACT

The results of a preliminary geotechnical investigation at five sites on Bear Brook selected for the possible formation of flood control lakes in the South East City Development area near Ottawa, Ontario are reported.

Based on the results of a shallow boring programme and an associated laboratory testing programme, the sites are covered by surficial deposits of loose silty sand to sand. Underlying these surficial deposits an extensive deposit of firm sensitive silty clay, the upper portion of which has been desiccated to varying depths, was encountered.

The results of stability analyses indicate that, for an adequate factor of safety with respect to overall shear failure and/or local overstress within the foundation subsoil, the maximum safe height of an embankment at any of the sites is of the order of 20 feet. The long term consolidation settlement caused by a 20 foot embankment loading will probably be of the order of 1 to 2 feet.

There do not appear to be any major geotechnical problems associated with the control of seepage through the dam or foundation subsoil at the sites. Subexcavation of the upstream pond area to create additional pond storage is limited both with respect to the proximity to the toe of the dam and with respect to the stability of the existing river banks.

Based on the preliminary results of the laboratory testing programme, it is considered that select portions of the surficial sands may be suitable for use in the construction of the dam. A number of alternative dam designs are considered in the report, but it appears that a rockfill or granular dam section incorporating a synthetic cut-off may be the most economical.

The results of analyses of the stability of the existing natural valley walls of Bear Brook at the proposed dam site locations indicate that relatively steep slopes in excess of about 25 feet high are only marginally stable. On this basis, there may be areas along Bear Brook at which protective measures may have to be provided (or as a minimum an observational programme initiated during the course of the proposed development) to ensure stability over the long term.

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## 1. INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by De Leuw Cather, Canada Ltd., Consulting Engineers to the Ontario Housing Corporation (OHC), to carry out a subsurface investigation at five proposed dam site locations in the South East City development site near Ottawa, Ontario. The purpose of the investigation was to determine the subsurface conditions at the sites and based on the findings to make preliminary engineering recommendations regarding the geotechnical aspects of the design and construction of the proposed dams. Further, the stability of the existing side slopes of Bear Brook at the proposed dam site locations was to be appraised.

## 2. DESCRIPTION OF PROJECT

It is understood that an area covering some 7,000 acres in the Township of Gloucester in the Regional Municipality of Ottawa-Carleton, Ontario is to be developed as a satellite city to the nearby City of Ottawa (for Key Plan, refer to Figure 1).

As part of the preliminary engineering and planning study for the development, previous investigations have been carried out in the area (refer to Golder Associates report number 741230, dated March, 1975). Based on the findings of the previous investigations, it was suggested that a supplementary investigation be carried out to ascertain the effect of increased run-off due to site development on the stability of the side slopes of the main watercourse at the site, namely Bear Brook. It was noted that although there

was no evidence that major flow slides had occurred in the past at the site, the existing stability of the side slopes of Bear Brook had to be maintained. To this end it was recommended that, to avoid erosion and undercutting of the existing banks, the discharge of run-off into Bear Brook and its associated tributaries should be controlled. It was considered that, provided this condition was satisfied and that the critical areas of the river banks were flattened or protected, major flow slides triggered by relatively small slope failures within the riverbanks should not occur as a result of development.

One method of controlling the erosional capability of Bear Brook is to construct a series of stilling ponds or lakes at strategic locations across the site, thereby allowing the quantity of water discharged to Bear Brook to be maintained at an acceptably low level. On the basis of hydrological data and other preliminary planning considerations, five (5) sites were selected as being suitable for the construction of stilling ponds to control the discharge of run-off into Bear Brook. The present investigation was carried out to define the geotechnical factors affecting the construction of dams at the proposed sites and also to assess the general stability of the existing Bear Brook riverbanks at these locations.

### 3. DESCRIPTION OF ALTERNATIVE DAM SITES

The locations of the five selected dam sites (referred to as Site A through E) are shown on Figure 2. Four of the sites are located within the main Bear Brook river channel while the fifth, Site B, is located on the unnamed major

tributary of Bear Brook. Sites A, B and C fall within the limits of the OHC site while the remaining sites fall within lands north of Highway 417 recently acquired by the National Capital Commission (NCC), Ottawa.

The significant characteristics of each of the sites are summarized as follows:

Site A - approximately 15 foot high river banks at dam structure location

- valley side slopes relatively flat (approximately 5 horizontal to 1 vertical)
- upstream ponding area wide and relatively shallow
- watershed extends over the south-central and southern portions of the proposed development area

Site B - approximately 20 to 25 foot high river banks at dam structure location

- valley side slopes steep (approximately 2 horizontal to 1 vertical)
- upstream ponding area consists of narrow steep sided valley
- watershed extends over a major portion of the north and central areas of the proposed development area

Site C - approximately 25 to 30 foot high river banks at dam structure location

- valley side slopes relatively steep (approximately 3 horizontal to 1 vertical)
- upstream ponding area consists of narrow steep sided valley

- watershed includes all of proposed development area

Site D - approximately 20 to 25 foot high river banks at proposed dam structure location

- valley side walls relatively steep (approximately 2 to 3 horizontal to 1 vertical)
- located north of Highway 417 at the head of an extensive downstream flood plain
- upstream ponding area consists of narrow, steep sided valley
- watershed includes all of proposed development area

Site E - approximately 15 foot high river banks at proposed dam structure

- valley side walls extremely flat (approximately 10 horizontal to 1 vertical)
- located on major flood plain in relatively mature section of river
- widespread shallow upstream ponding area
- watershed includes all of proposed development area

#### 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 re-deposition cycles.

In time the flow of water into the Champlain Sea decreased as glacial meltwater was channelled 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.

##### 5. FIELDWORK PROCEDURES

The fieldwork for the present investigation was carried out between November 22, 1974 and January 8, 1975. During this period one borehole was put down at the crest of the river bank and a second borehole was located within the valley floor at each dam site.

To differentiate between borings put down during other investigations in the general area, the boreholes put down in the present investigation were numbered 301 to 310, inclusive. Because of difficulty in gaining access to the dam sites which were generally densely treed on the river banks and swampy within the valley floor, a bombardier mounted power auger supplied and operated by a local drilling contractor was used to advance the borings through the overburden soils. The boreholes on the crest of the river banks were put down to depths of between about 35 to 55 feet below ground surface. Within the valley floor, the boreholes were advanced to depths of between 30 and 35 feet.

Samples of the overburden were obtained at depth intervals of 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 2 and 3 inch diameter thin walled piston tube and standard Shelby tube samples. During split spoon sampling operations, standard penetration tests were carried out. Between samples in the cohesive overburden soils, in situ vane shear tests were carried out. At periodic intervals during the course of the fieldwork, the samples were transported to our laboratory for detailed examination and testing.

Following completion of each boring, a piezometer and/or standpipe was installed in the hole to establish the stabilized groundwater levels at the sites and to permit monitoring of these installations as shown on the Record of Borehole Sheets following the text of this report.

The fieldwork was supervised throughout by a member of our engineering staff who located the borings in the field,

directed the drilling, sampling and field testing operations, logged the borings and cared for the samples obtained. The detailed plan locations and ground surface elevations at the individual boreholes were established in the field by personnel from De Leuw Cather, Canada Ltd.

#### 6. SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the borings is given on the Record of Borehole sheets. Stratigraphic sections at each of the proposed dam sites are shown on Figure 3 to 7, inclusive. Following is a summarized account of the subsurface conditions at the proposed sites.

The stratigraphy encountered in the borings is basically similar to that determined in previous investigations in the general area. Underlying thin surficial topsoil or organic desposits, an intermittent sand cap overlies an extensive deposit of sensitive silty clay. Although none of the borings put down during the present investigation fully penetrated through the Leda clay deposit, it is known from the results of previous investigations that the clay is underlain by glacial till which, in turn, overlies shale bedrock. From the previous investigations (see for example Golder Associates report number 73908, Vol. III, dated May, 1974), it is considered that the surface of this till or bedrock is close to or in excess of 100 feet below the ground surface at the proposed dam sites.

Because of the erosion and redeposition processes associated with the development of Bear Brook, the subsurface conditions within the valley floor and on the crests of the

slopes forming the valley banks are slightly different and are discussed separately below.

Valley Floor: The upper 0.5 to 1 foot below ground surface consists of organic sandy or clayey topsoil. Except at Site E, the surficial organic deposits are underlain by between about 7 and 15 feet of silty sand or desiccated silty clay. The silty sand layer generally contains a trace to some organic matter and a trace of clay. Overlying the silty sand layer at Site A is a 6 foot thick layer of desiccated brown sandy clayey silt to silty clay. Within the valley floor and underlying the upper sand and desiccated clay strata, the sites are underlain by unweathered sensitive grey silty clay.

Crests of Slopes: The upper 0.5 foot thick topsoil layer is underlain by between about 3 to 7 feet of brown sand to silty sand at sites A, B and C. The surficial sand layers at Site B and the topsoil layers at Sites D and E are underlain by between 4 and 11 feet of desiccated brown to grey brown silty clay containing some thin coarser silt or sand layers.

The surficial sand deposits appear to contain an increasing proportion of fines (silt and clay size particles) with depth. This gradual change in the nature of the deposit is reflected in the water content which increases with depth from about 20 percent to about 40 percent. The standard penetration resistance in the upper portion of deposit lies between about 4 and 19 blows/foot and decreases to about 2 to 3 blows/foot with depth.

Where present in the crests of the river banks, the upper brownish silty clay is the result of desiccation of the

underlying unweathered silty clay. The consistency of the crustal soil is stiffer than the parent material with undrained shear strengths of the order of about 1000 pounds per square foot. The natural water content of the desiccated crustal soil is about 40 percent, associated with average liquid and plastic limits of about 55 and 21 percent, respectively. It should be noted that the desiccated clayey crust contains coarser material consisting of layers of silt and sand (see Figures 9 and 10). As a result of environmental conditions during deposition, the unweathered silty clay is highly sensitive. Because of the high average water content of about 80 percent, which is often in excess of the liquid limit (Figure 11), and the sensitive nature of the material, the silty clay is almost "liquid like" when remoulded. However, in a relatively undisturbed state, the undrained shear strength of the material ranges from about 500 to 1000 pounds per square foot. Within individual boreholes there is a tendency for the undrained shear strength to increase with depth (refer to Record of Borehole 306). Because of changes in ground surface elevations at the borehole locations, there is a considerable scatter in the results of strength tests carried out in the crest and valley boreholes (see Figures 12 and 13). As found in previous investigations, in the general area, (Golder Associates report number 741230, dated March, 1975) there is a consistent correlation between undrained shear strength and elevation. The test results obtained during the present investigation fall within the range of results determined by the previous work (see Figure 14).

Within the valley floor where the ground surface elevation is lower, the undrained shear strength is higher than that encountered at corresponding depths below ground surface

in the boreholes put down at the crest of the slopes. For preliminary design purposes however, the increase in strength with depth has been neglected and the following values of undrained shear strength ( $S_u$ ) have been used in the analyses:

$S_u$ - crest of river bank (Figure 13)	= 600 lb/sq. ft.
$S_u$ - valley floor (Figure 12)	= 750 lb/sq. ft.

The results of consolidation tests carried out on relatively undisturbed samples of unweathered silty clay are shown on Figures 15 to 19, inclusive. From these results, it appears that the unweathered silty clay at relatively shallow depth below the valley floor is lightly overconsolidated with the preconsolidation pressure,  $P_c$ , exceeding the existing overburden pressure,  $P_o$ , by about 1000 pounds per square foot. Because of the "brittle" nature of the soil, the compressibility at stresses below the preconsolidation pressure is low, while at stresses in excess of  $P_c$  the compressibility increases markedly.

## 7. RECOMMENDATIONS FOR PRELIMINARY DAM DESIGN

### 7.1 General Considerations

It is understood that one or a number of the sites under consideration are possible locations for the formation of ponding lakes. The purpose of the lakes is to control the discharge of run-off into Bear Brook resulting from future development of the surrounding South East City Site. By controlling the rate of flow in the existing Bear Brook watercourse, the possibility of excessive erosion and subsequent failures within the river banks, which, if unchecked could trigger flow slides, can be greatly reduced.

The lakes are to be formed by constructing earth embankment dams and it is understood that consideration is being given to increasing the natural capacity of the lakes by subexcavation of the upstream pond area. For preliminary planning purposes, it is understood that the total embankment height will not exceed 20 to 30 feet.

In general, there are four basic geotechnical factors which significantly affect the design of earth dams at this site. These are:

- (i) Stability of the material used to construct the dam and of the foundation subsoil with respect to shear failure.
- (ii) Cracking of the dam structure caused by excessive settlement of compressible foundation subsoils.
- (iii) Excessive seepage through the dam or foundation subsoil which may result in a "functional" failure of the dam or "outright" failure of the dam by internal erosion or "piping".
- (iv) Economical use of native materials in the construction of the dam.

## 7.2 Stability

For the purpose of this preliminary study, it has been assumed that the dam consists of a granular embankment constructed on a homogeneous cohesive foundation subsoil. The geometry of the embankment, (i.e. the height and slope angle) was varied in the analyses to determine its effect on the

stability of the dam. The shear strength of the foundation subsoil has also been treated as a variable but has been considered to be isotropic and constant with depth. It should be noted that analyses of the stability of a dam with respect to failure of the foundation subsoil have been carried out in terms of total stresses (i.e. using undrained shear strengths). This method, which is applicable to the "end of construction" case, is considered to better reflect the nature of the "failure" envelope of the unweathered silty clay in the working stress range (see Golder Associates report number 73908, Vol. III, dated May, 1974). During final design, the stability of the proposed embankment structure will also have to be analysed in terms of effective stresses to take into account changes in pore water pressures associated with variations in the level of the pond and seepage pressures within the dam.

The results of our stability analyses are summarized on Figure 20 and indicate that for an average minimum undrained shear strength of 750 pounds per square foot within the foundation subsoil, the maximum permissible height of embankment (side slope 2.5:1) is about 20 to 25 feet. It should be noted that this range in height is based on a factor of safety between 1.5 and 1.7 and that the upper limit of this range is somewhat higher than values normally associated with dam design. However, recent recorded instances of embankment failures on soft sensitive clays indicate that the use of a factor of safety of this magnitude is appropriate for foundation subsoil conditions similar to those at the present site.

Consideration has been given to the consequence of local overstressing of the foundation subsoil (refer to Figure 21). Because of the sensitive nature of the Leda clay deposit underlying the dam sites, the use of a factor of safety of at least



1.2 would be prudent to prevent excessive local overstressing with consequent large progressive deformations/failure within the foundation subsoil. On this basis, therefore, it is considered that the maximum embankment height should be restricted to 20 feet for preliminary design purposes.

Depending on the material used to construct the embankment, the side slopes may vary. For example, a rock fill embankment could be constructed with side slopes steeper than 2 horizontal to 1 vertical. However, as shown on Figure 20, the factor of safety of a 20 foot high embankment with respect to shear failure of the foundation subsoil decreases as the slope becomes steeper. Therefore, unless the maximum permissible height is reduced, the steepest slope which should be used for preliminary design is 2 horizontal to 1 vertical.

### 7.3 Settlement

Based on previous experience in the general area and the results of preliminary laboratory consolidation tests, it is estimated that, at the proposed sites, the magnitude of the long term consolidation settlement which will occur due to the construction of a 20 foot high embankment will be of the order of about 1 to 2 feet. About 75 per cent of the total settlement will probably occur within 10 years after completion of construction. It is essential that the dams be designed to accommodate this anticipated magnitude of settlement without cracking or detrimental deformation.

### 7.4 Seepage

As the purpose of the proposed dams is to control the flow of water in Bear Brook and not primarily to act as

water supply reservoirs, it is likely that some seepage of ponded water can be tolerated. However, to minimize seepage losses and, more important, seepage pressures within the dam and the foundation subsoil, consideration should be given to the provision of an impervious core or other form of cut-off.

It is considered that if the dams are located in an area underlain by a sandy layer, a seepage barrier (cut-off) which extends into the impervious silty clay subsoil should be provided as an integral part of the final dam design.

#### 7.5 Materials

For economic reasons, consideration has to be given to the use of native materials in the construction of the proposed dams. In this respect, it should be noted that because of its high sensitivity and natural water content, the unweathered silty clay material which underlies all of the present sites is unsuitable for use as a construction material. Further, based on the results of laboratory tests it appears that the water content of the "desiccated" silty clay crust is not sufficiently low to allow proper workability and compaction of this material during construction.

However, it is possible that, depending on detailed investigations at the finally chosen locations, the surficial sands may be suitable for use in embankment construction. However, as noted in Section 6, this material varies from sand to silty sand and, depending on the percentage of fines present, some of the finer material may be difficult to compact adequately (see Figure 22).

Finally, based on the results of previous investigations in the South East City development area, there are no significant quantities of granular or rock fill material available within the proposed development site.

#### 7.6 Alternative Methods of Dam Construction

There are a number of possible methods of constructing short duration and low head water retaining structures at the sites under consideration. From the outset, however, it is recognized that for economic and technical reasons, concrete dam structures are not feasible and that "earthfill" dams will be used.

Five alternative types of earthfill dam sections, which can be considered for preliminary design purposes are shown on Figure 23. The advantages and disadvantages of the various alternatives are discussed below:

(i) Homogeneous earthfill section - no source of necessary quantity of suitable material available in general area to construct an "engineered fill"; - subject to cracking (and subsequent internal erosion) due to anticipated settlements.

(ii & iii) Impervious core sections - greater possibility of using available native materials than in case (i) above; core subject to cracking due to anticipated settlements.

(iv) Granular or Rockfill section with sheet pile cut-off - suitable material in limited quantities may be available at site; section capable of withstanding some

deformation particularly if sheeting is driven zig-zag style; relatively easy to construct and less dependent on construction procedures. (Properly designed, a rockfill embankment may not require sophisticated spillway structure to prevent failure by overtopping).

(v) Granular or Rockfill section with synthetic cut-off membrane - as in (iv) above but requires careful control during construction of the membrane padding zone; section capable of withstanding relatively large deformations caused by settlement.

#### 7.7 Sub-Excavation of Upstream Pond Area

It is understood that, in order to provide the required storage capacity, it is proposed to sub-excavate portions of the upstream pond area. An excavation operation of this nature could affect the stability of the completed dam structure and the existing river banks. The results of analyses shown on Figure 24 indicate that to maintain stability of a 20 foot high dam, the downstream limit of a 10 foot sub-excavation should not be closer than about 50 feet from the upstream toe of the dam.

The effect of increasing the effective height of slopes on the overall stability of the existing banks is shown on Figure 25. From these results it can be seen that at sites A and E where the valley slopes are flat and about 15 feet high, it is possible to sub-excavate to a depth of about 10 feet below the existing valley floor without jeopardizing the stability of the river banks. At site B, where the bank height is between about 20 and 25 feet, it may be possible to sub-excavate to a depth of 5 feet below the valley floor

level, if the slopes are flattened from the existing 2 to 3 horizontal to 1 vertical to about 4 to 5 horizontal to 1 vertical. However, at the remaining two locations, (i.e. sites C and D) the results of analyses indicate that there is a lower factor of safety against instability of the existing slopes and therefore no sub-excavation should be carried out in these areas.

It should be further noted that the sub-excavation of the surficial sands will be carried out below the groundwater level. Consequently, because of poor trafficability, it will be necessary to carry out the excavation using draglines.

#### 7.8 Summary

Based on the above, the following conclusions regarding preliminary geotechnical design of the dams are made:

- (i) To ensure stability of the dams, the maximum height should be taken as no greater than 20 feet for preliminary design purposes.
- (ii) The long term settlement of a 20 foot high embankment dam will probably be between about 1 and 2 feet.
- (iii) Seepage losses can be controlled by careful design of the dam section and are, therefore, not considered to be a major geotechnical problem at the sites under consideration.
- (iv) Depending on the results of detailed investigations at the finally selected dam site locations, the native surficial sands, although limited in quantity, may be used

for construction of the dams.

(v) Of the alternative schemes outlined in Section 7.6 (see Figure 23) it is considered that, from a geotechnical standpoint, a rockfill or granular embankment incorporating some form of flexible cut-off section would be suitable for preliminary design purposes.

(vi) To prevent instability of the existing river banks, the upstream pond areas at sites C and D should not be sub-excavated. At site B, the valley can be deepened by only 5 feet provided that the existing valley slopes are flattened. The upstream pond area at sites A and E can be safely sub-excavated to a depth of 10 feet below the existing valley floor.

#### 8. STABILITY OF BEAR BROOK SIDE SLOPES

A preliminary assessment of the stability of the valley walls at the proposed dam sites has been made on the basis of the undrained strength data obtained from the boreholes put down through the crests of the river banks at the individual sites (boreholes 306 to 310, inclusive). The method used was to compare the values of  $6.S_u$  (where  $S_u$  is the in situ undrained shear strength) and the total overburden pressures ( $\gamma \cdot h$ ) with depth below the crest of the slope. Previous published studies of various sites in the Ottawa area susceptible to flow slides, indicate that where the ratio  $6.S_u$  to  $\gamma \cdot h$  falls below unity, the occurrence of flow slides is possible. Figure 26, which has been drawn up on the basis of the results of the present investigation, indicates that the large majority of the  $6.S_u$  values are in excess of the approximate overburden stresses. However, the lower range

of the  $6.S_u$  envelopes shown on Figure 26 indicates that in some areas, the stability of the existing river banks may be marginal. These results are confirmed by the analyses carried out on the effect of sub-excavation on the stability of the existing river banks (Figure 25).

Based on the above data, it is considered that for preliminary design purposes, river banks which are relatively steep (i.e. 2 to 3 horizontal to 1 vertical) and in excess of 25 feet high should be considered as only marginally stable. Depending on the results of detailed investigation of these areas, protective measures (such as erosion protection or slope flattening) may be necessary in certain sections to ensure the stability of these slopes during future development.

It should be noted that the existing Bear Brook water-course has evolved by erosion of the surficial sands and silty clay deposits below the level of the surrounding table land. In most areas, and particularly those delineated on Figure 27, the actual profile of the river valley is typically "broad and U-shaped" with the course of the river bed being extremely contorted within the valley floor. In order to attain its present shape, the river has continuously undercut its banks causing localized small scale rotational failures to occur within the river banks. Scars of such localized failure zones are evident from an examination of 1:1250 scale topographical maps of the relevant areas. However, there is no evidence to suggest that large scale flow slides have occurred as a result of such localized failures. This is considered to be significant and suggests that, provided there is no significant increase in the erosional capability of Bear Brook, major flow slides should not occur as a result of

development. However, the need for protective measures in some areas and restriction of the proximity of structures to the crest of slopes should be taken into account during preliminary planning.

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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	<i>N</i> , 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	<i>c<sub>u</sub></i> , 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<sup>1</sup>  
Q undrained triaxial<sup>2</sup>  
R consolidated undrained triaxial<sup>2</sup>  
S drained triaxial  
U unconfined compression  
V field vane test

### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>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
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	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

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_i$	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

\*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.

DATUM      GEODETIC

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

DRAWN m.j.B.  
CHECKED BG

# RECORD OF BOREHOLE 302

DATUM      GEODETIC

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

[illegible]

DRAWN m.j.B.  
CHECKED BG

RECORD OF BOREHOLE 303

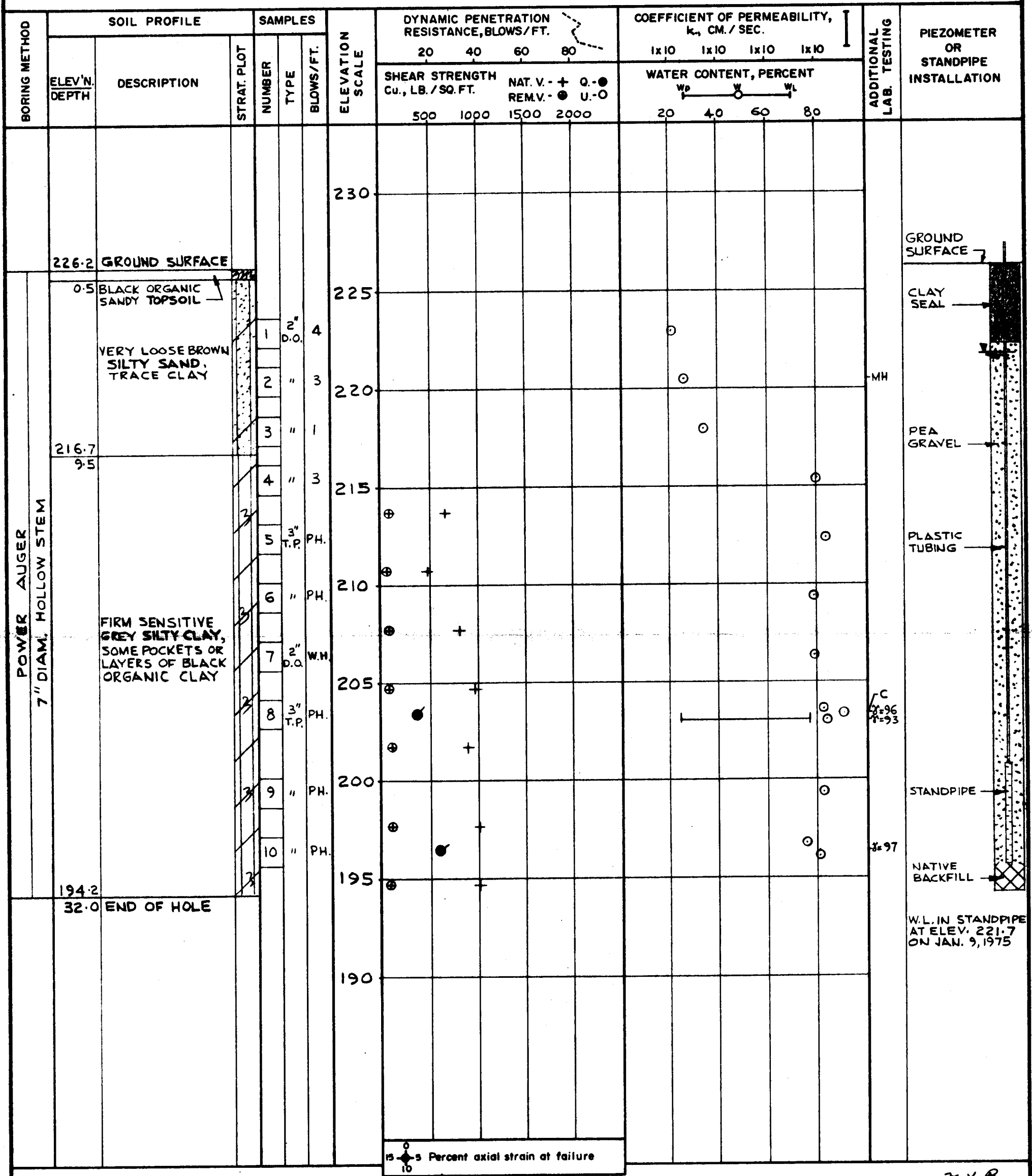
LOCATION See Figure 2

BORING DATE DEC. 4 & 5, 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 *m.j.B.*  
CHECKED *RJ*

RECORD OF BOREHOLE 304

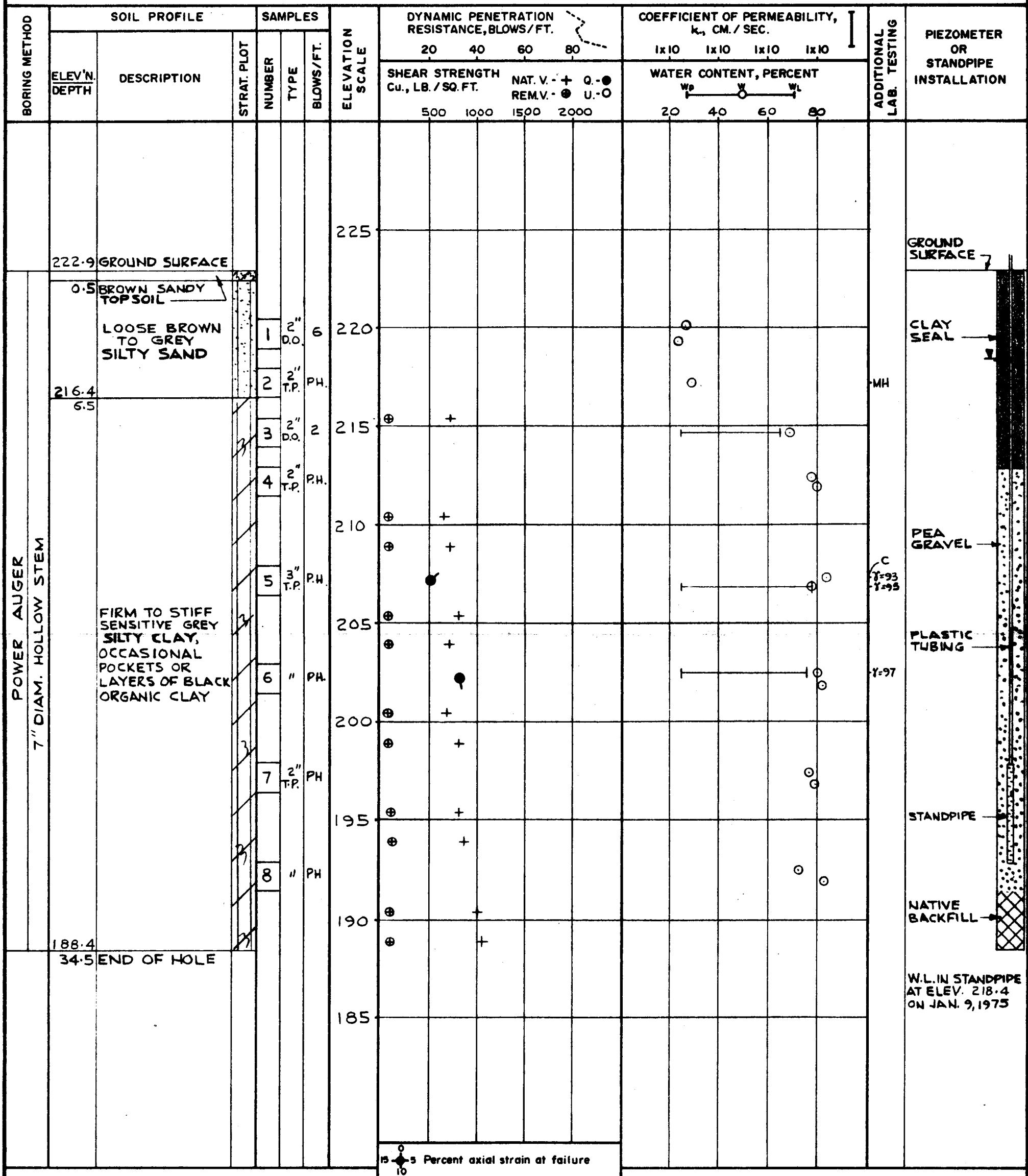
LOCATION See Figure 2

BORING DATE JAN. 6, 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 *214-B*  
CHECKED *B.G.*



RECORD OF BOREHOLE 306

LOCATION See Figure 2

BORING DATE

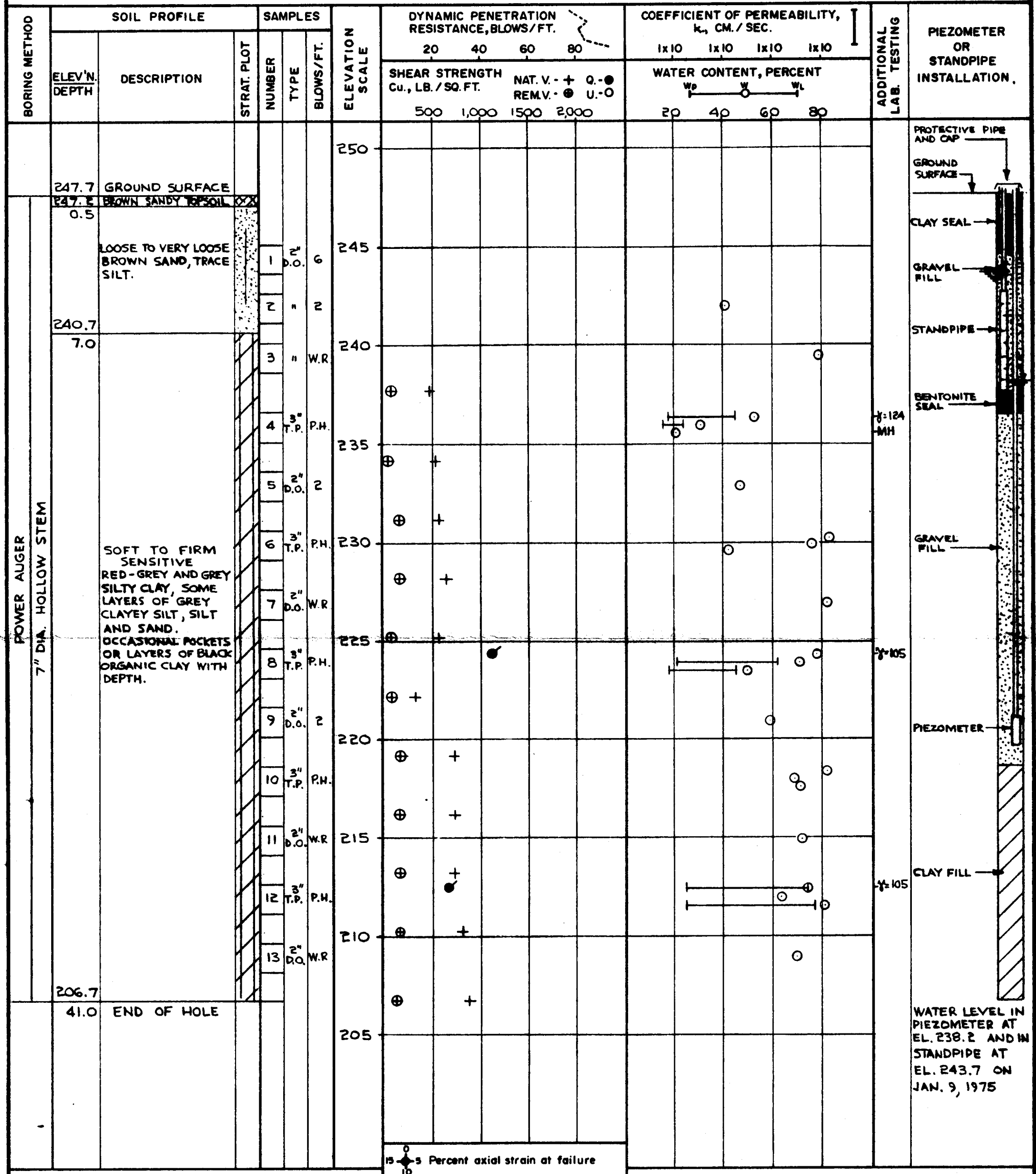
NOV. 25 & 26, 1974

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 J.A.  
CHECKED BG



## RECORD OF BOREHOLE 307

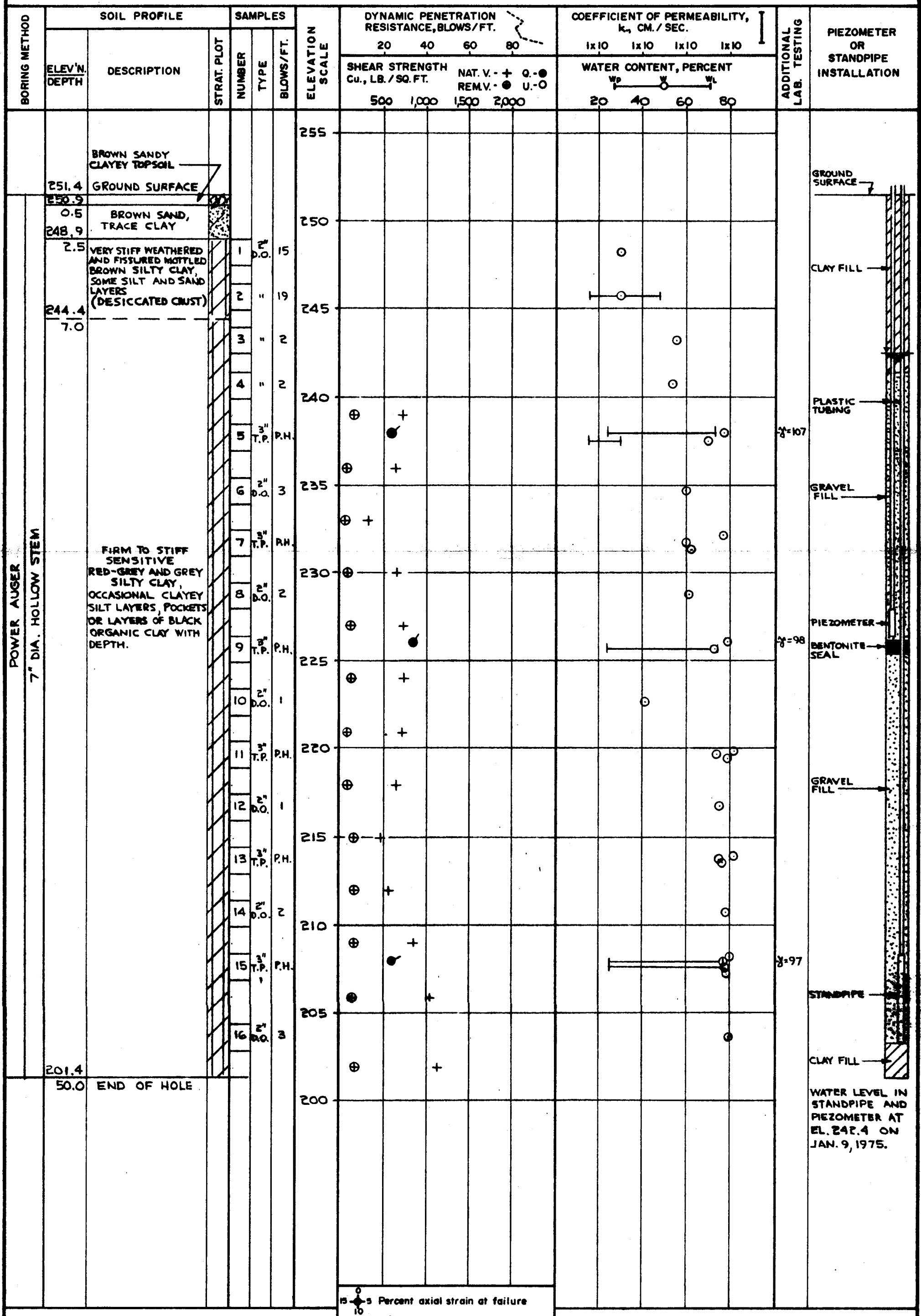
LOCATION See Figure 2

BORING DATE NOV. 28, 29 &amp; DEC. 1, 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 J.A.  
CHECKED B.G.

## RECORD OF BOREHOLE 308

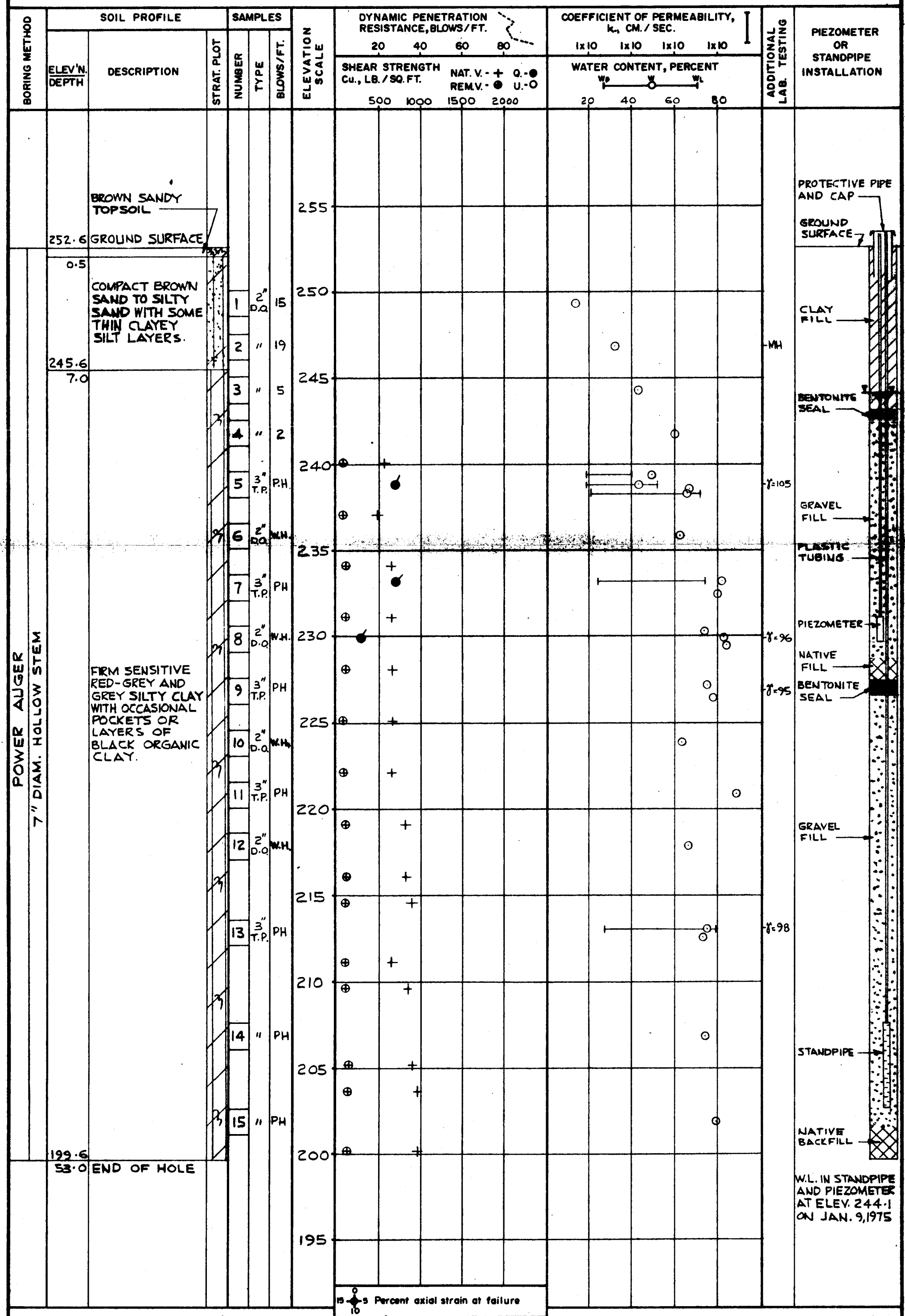
LOCATION See Figure 2

BORING DATE DEC. 18-4, 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 *mt.B*  
CHECKED *BL*

# RECORD OF BOREHOLE 309

**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	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS / FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION									
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE																		
								20		40		60		80			1x10		1x10		1x10		1x10		
								SHEAR STRENGTH Cu., LB. / SQ. FT.				NAT. V. - + Q.-● REM.V. - ● U.-○					WATER CONTENT, PERCENT								
								500    1000    1500    2000									20    40    60    80								
		BROWN CLAYEY TOPSOIL					240										GROUND SURFACE								
	238.4	GROUND SURFACE																							
	0.5																								
		VERY STIFF WEATHERED BROWN TO GREY-BROWN SILTY CLAY, SOME SILT LENSES. (DESICCATED CRUST)		1	2" D.O.	15	235										CLAY FILL								
				2	2" T.P.	PH																			
							230										BENTONITE SEAL								
	226.4			3	" PH											H									
	12.0						225										PLASTIC TUBING								
				4	3" T.P.	PH										R-98									
							220										PIEZOMETER								
				5	" PH												BENTONITE SEAL								
		FIRM SENSITIVE RED-GREY AND GREY SILTY CLAY, OCCASIONAL THIN SILT SEAMS AND LAYERS OR POCKETS OF BLACK ORGANIC CLAY WITH DEPTH.					215																		
				6	2" T.P.	PH											GRAVEL FILL								
							210																		
				7	3" T.P.	PH										R-98									
							205																		
				8	2" T.P.	PH																			
							200										STANDPIPE								
	196.9			9	3" T.P.	PH										R-97									
	41.5	END OF HOLE					195																		
																	W.L. IN STANDPIPE AT ELEV. 224.9 AND IN PIEZOMETER AT ELEV. 232.9 ON JAN. 9, 1975								

Percent axial strain at failure

**VERTICAL SCALE  
1 IN. TO 5 FT.**

## Goldier Associates

DRAWN m.j.B.  
CHECKED BG

RECORD OF BOREHOLE 310

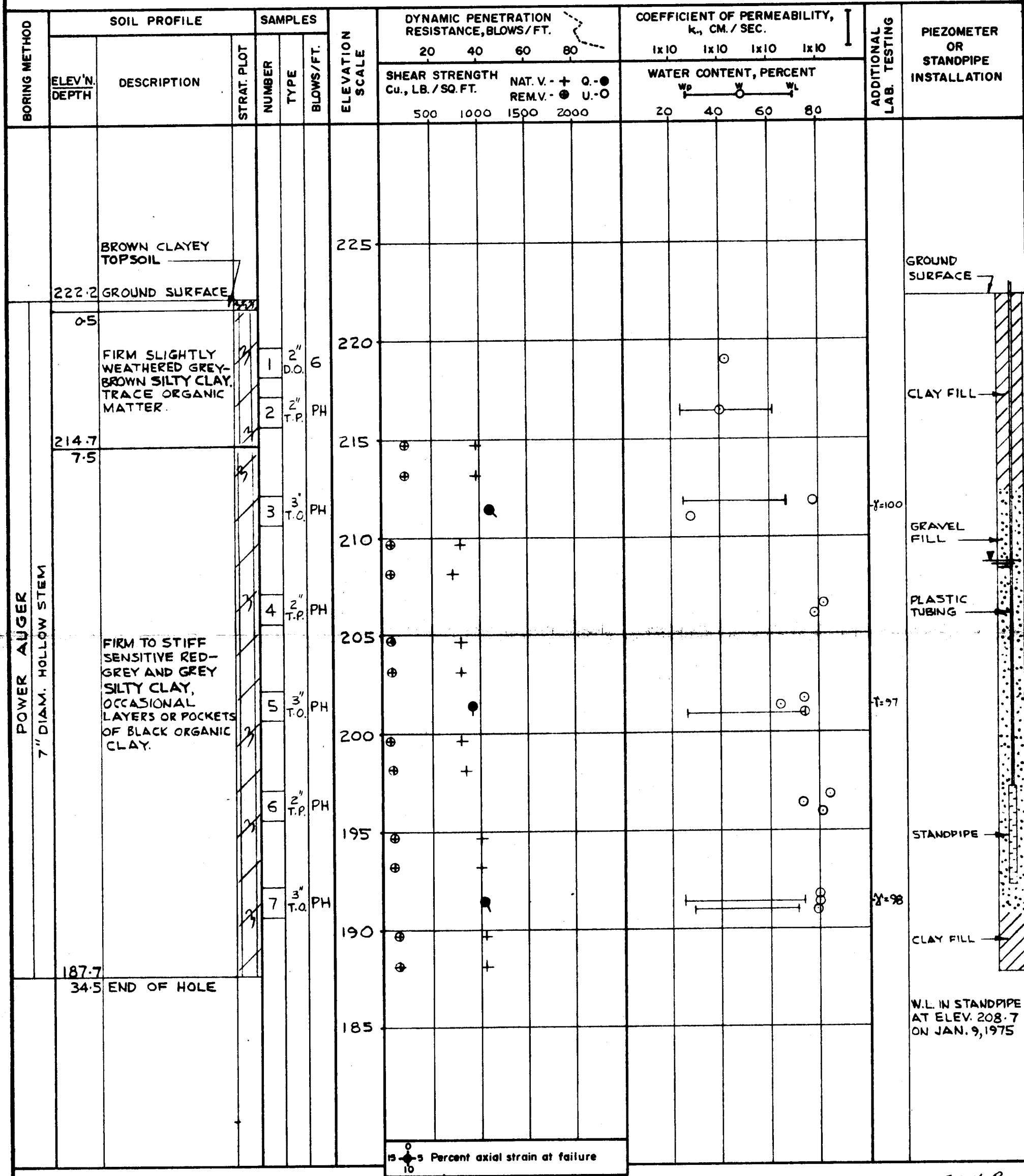
LOCATION See Figure 2

BORING DATE JAN. 8, 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

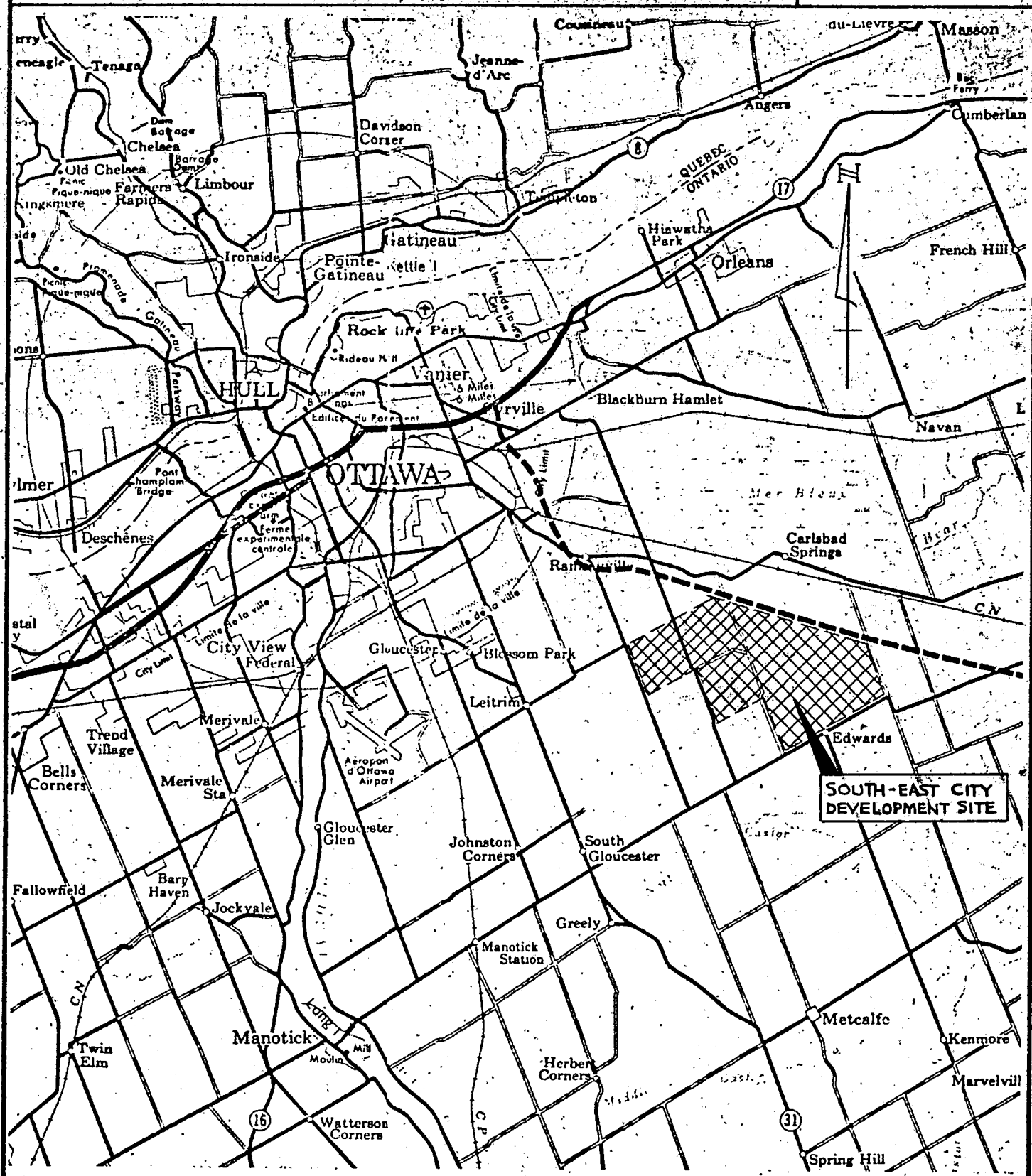
DRAWN *m.j.b.*  
CHECKED *B.G.*

PROJECT No. 741231

Form G.A.-D.-4

# KEY PLAN

FIGURE 1



SCALE: 1 INCH TO 3 MILES

Date MARCH 27, 1975

Golder Associates

Drawn - J.A.  
Chkd. - B.G.  
Appd. - J.H.W.





LEGEND

BOREHOLE IN PLAN - PRESENT INVESTIGATION  
(LOCATIONS APPROX. ONLY)

APPROXIMATE OUTLINE OF PROPOSED DAM POND AREA

NOTE:  
FOR STRATIGRAPHIC SECTIONS AT DAM SITES 'A' TO 'E' REFER  
TO FIGS. NO. 3 TO 7 INCLUSIVE.

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

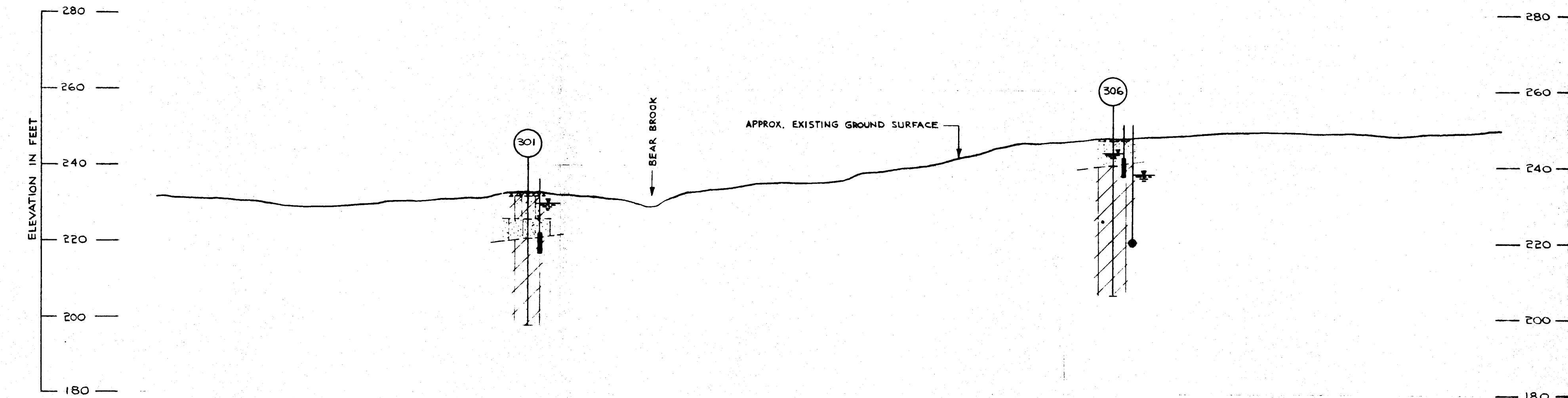
SPECIAL NOTE  
THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT.

Date: JAN. 14, 1975

Golder Associates

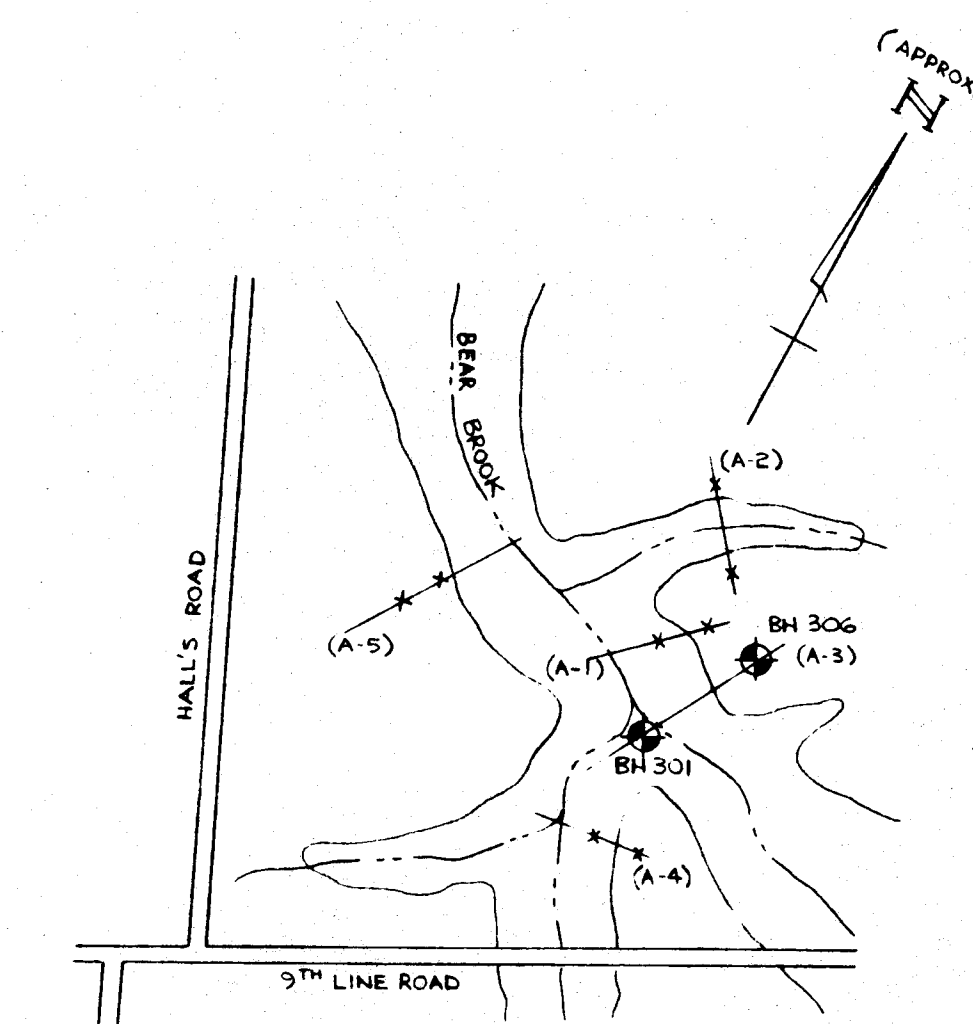
Drawn J.A.  
Chkd. B.L.  
Appd. J.H.C.





STRATIGRAPHIC SECTION ALONG LINE A-3  
SCALE: 1" TO 20'

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



LOCATION SKETCH  
(NOT TO SCALE)

NOTE: SURVEY DATA SUPPLIED BY DELEUW CATHER, CANADA LTD.

**LEGEND**

- ◆ BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL IN PIEZOMETER OR STANDPIPE, JAN 9, 1975
- STANDPIPE
- PIEZOMETER

**STRATIGRAPHY**

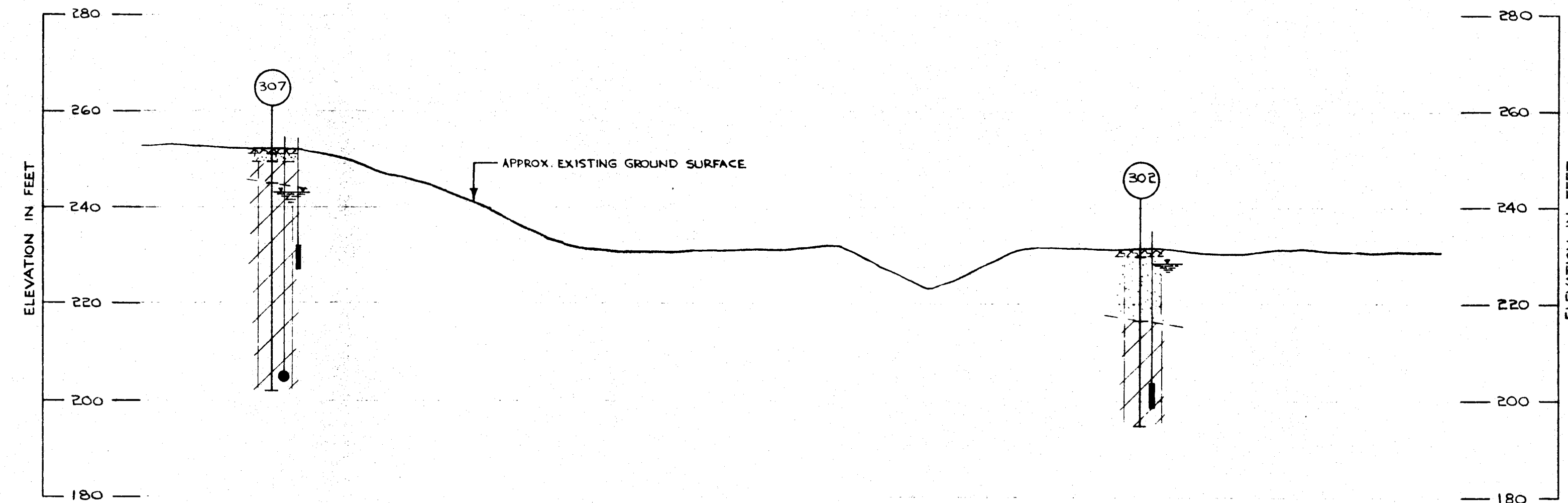
- ▨ TOPSOIL
- ▨ LOOSE GREY SILTY SAND, TRACE CLAY AND ORGANIC MATTER
- ▨ LOOSE TO VERY LOOSE BROWN SAND, TRACE SILT
- ▨ STIFF TO FIRM BROWN SANDY CLAYEY SILT TO SILTY CLAY, TRACE ORGANIC MATTER
- ▨ SOFT TO FIRM SENSITIVE RED GREY AND GREY SILTY CLAY

**NOTE**  
Data concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.  
For detailed stratigraphy at each borehole location refer to the record of borehole sheets.

Date MARCH 24, 1975

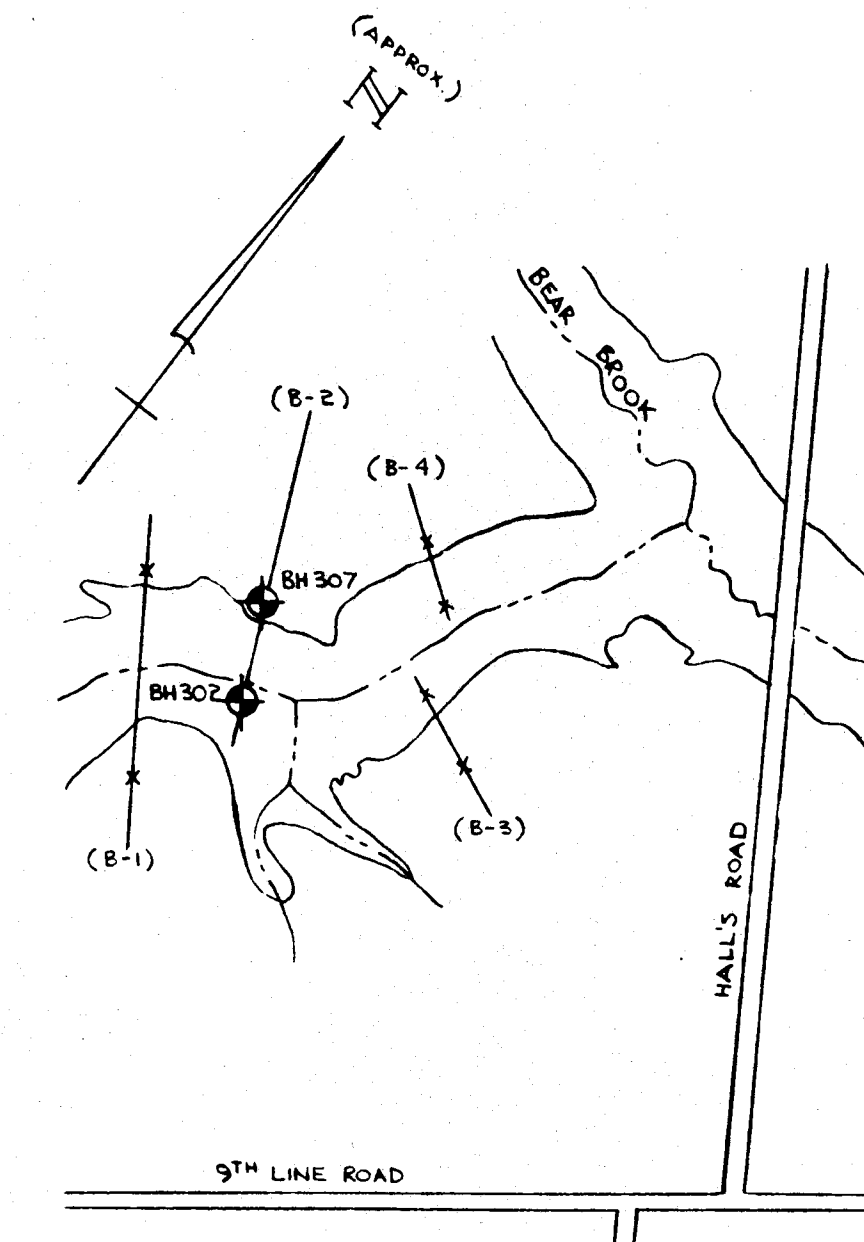
Golder Associates

Drawn J.A.  
Chkd. J.A.  
Appd. J.A.



STRATIGRAPHIC SECTION ALONG LINE B-2  
SCALE: 1" TO 20'

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



LOCATION SKETCH  
(NOT TO SCALE)

NOTE: SURVEY DATA SUPPLIED BY DE LEUW CATHIER, CANADA LTD.

**LEGEND**

- ◆ BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL IN PIEZOMETER OR STANDPIPE, JAN. 9, 1975
- STANDPIPE
- PIEZOMETER

**STRATIGRAPHY**

- ▨ TOPSOIL
- ▨ COMPACT TO VERY LOOSE SILTY FINE SAND, TRACE CLAY AND ORGANIC MATTER
- ▨ BROWN SAND
- ▨ VERY STIFF MOTTLED BROWN SILTY CLAY, SOME SILT AND SAND LAYERS (DESICCATED CRUST)
- ▨ FIRM TO STIFF SENSITIVE RED-GREY AND GREY SILTY CLAY

**NOTE**

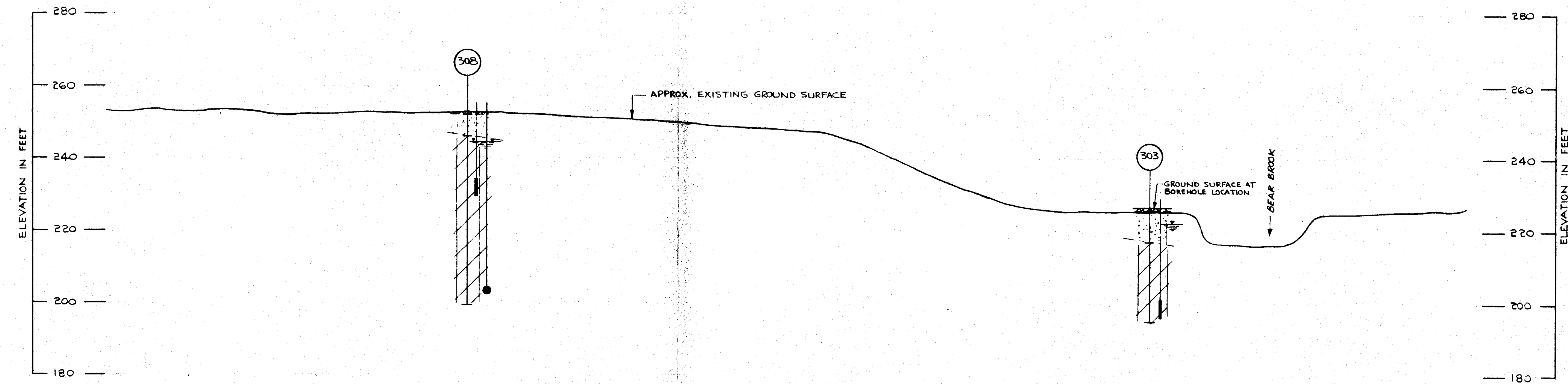
Date concerning this various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.  
For detailed stratigraphy of each borehole location refer to the record of borehole sheets.

Date MARCH 24, 1975

Golder Associates

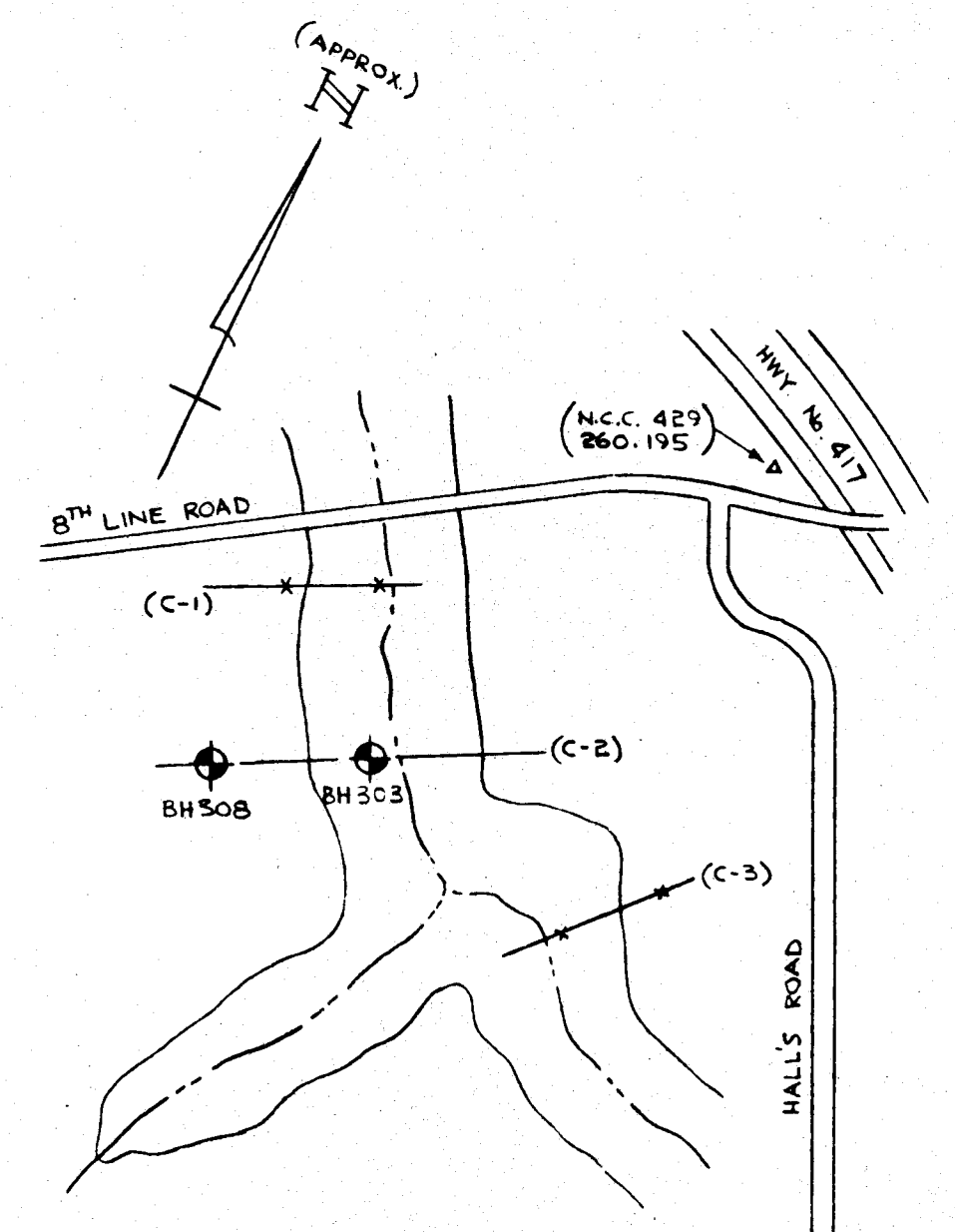
Drawn J.A.  
Chkd. J.A.  
Appd. J.A.





STRATIGRAPHIC SECTION ALONG LINE C-2  
SCALE: 1" TO 20'

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



LOCATION SKETCH  
(NOT TO SCALE)

NOTE: SURVEY DATA SUPPLIED BY DE LEUW CATHAR, CANADA LTD.

LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL IN STANDPIPE OR PIEZOMETER, JAN. 9, 1975
- STANDPIPE
- PIEZOMETER

STRATIGRAPHY

- TOPSOIL
- VERY LOOSE BROWN SILTY SAND, TRACE CLAY
- COMPACT BROWN SAND TO SILTY SAND
- FIRM SENSITIVE RED-GREY AND GREY SILTY CLAY






**NOTE**  
Data concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.  
For detailed stratigraphy of each borehole location refer to the record of borehole sheets.

Date MARCH 24, 1975





Golder Associates

Drawn J.A.  
Chkd B.H.  
Appd J.H.A.

LEGEND

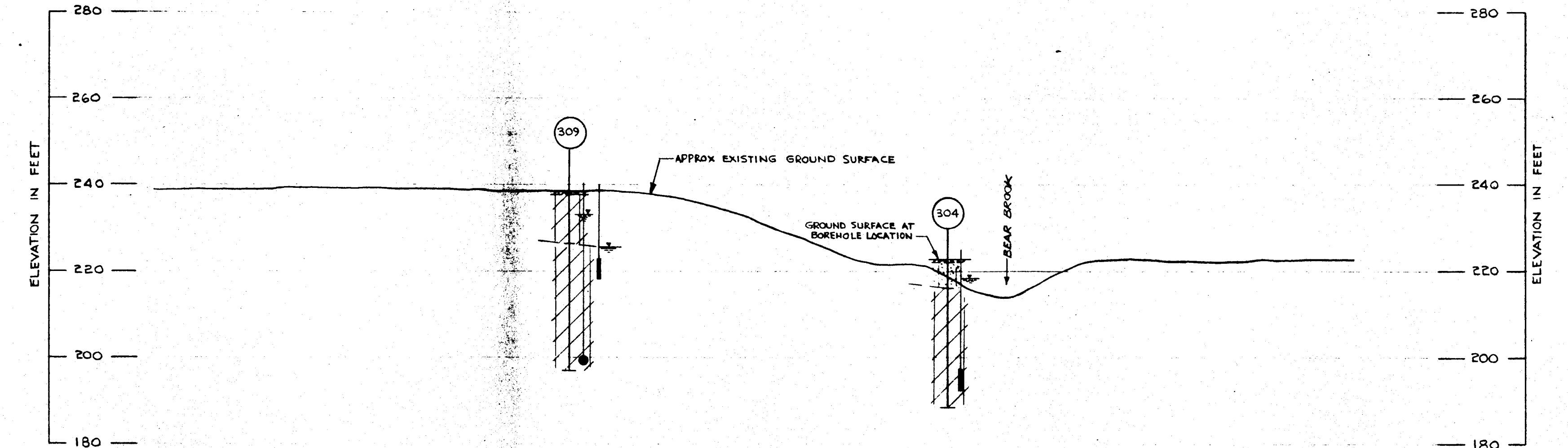
-  BOREHOLE IN PLAN
-  BOREHOLE IN ELEVATION
-  WATER LEVEL IN STANDPIPE OR PIEZOMETER, JAN. 9, 1975
-  STANDPIPE
-  PIEZOMETER

STRATIGRAPHY

-  BROWN SANDY TOPSOIL
-  LOOSE BROWN TO GREY SILTY SAND
-  VERY STIFF BROWN TO GREY-BROWN SILTY CLAY, SILT LENSES (DESICCATED CRUST)
-  FIRM TO STIFF SENSITIVE RED-GREY AND GREY SILTY CLAY

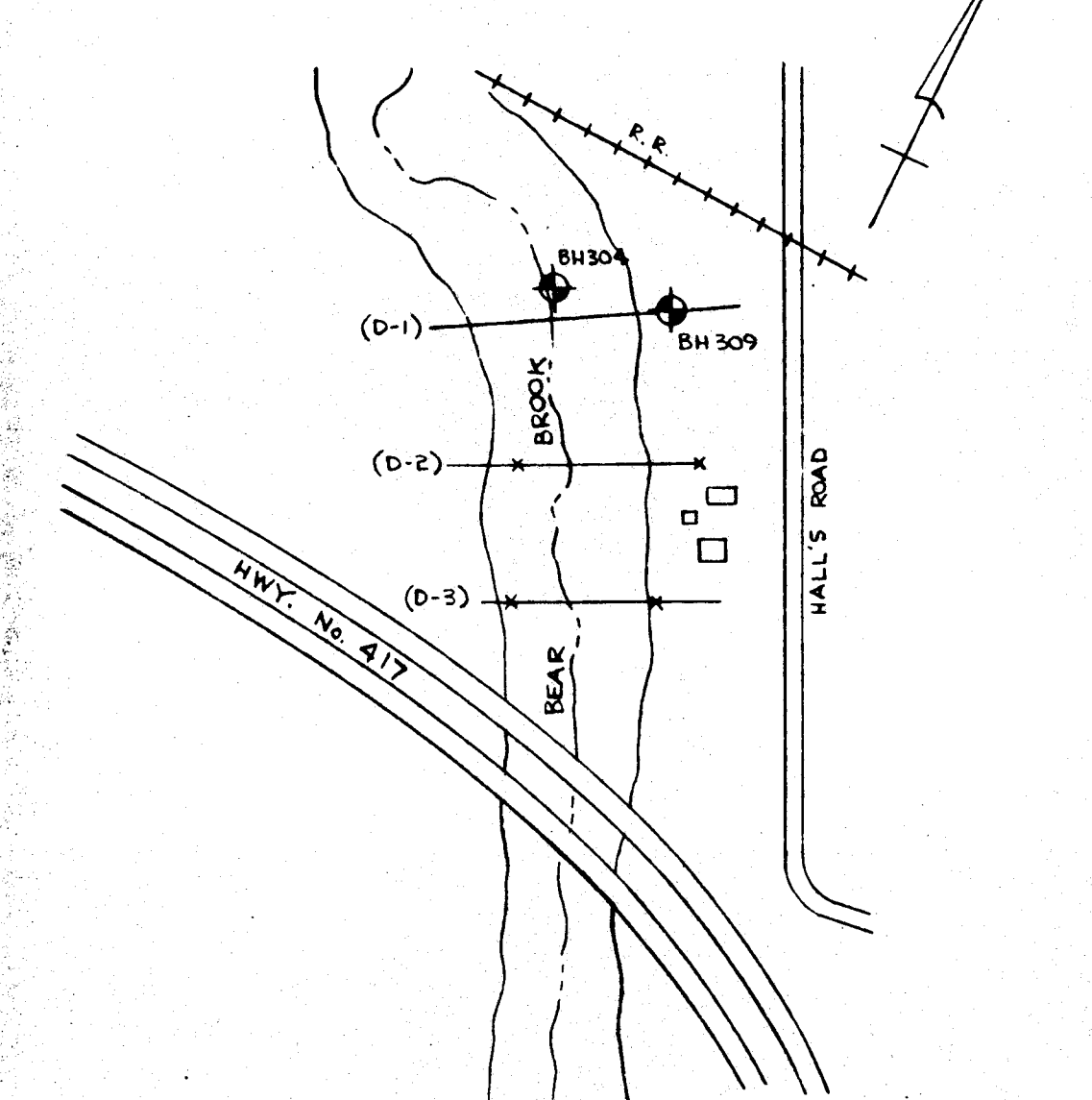
**NOTE**  
Data concerning the various strata were obtained at various locations. The soil strata shown in the borehole logs were inferred from geophysical logs and may vary from the actual.  
For detailed stratigraphic data, see Section Location Data to the record of borehole sheets.

**NOTE:** SURVEY DATA SUPPLIED BY DE LEUW CATHER, CANADA LTD.



STRATIGRAPHIC SECTION ALONG LINE D-1  
SCALE: 1" TO 20'

**SPECIAL NOTE**  
THIS BOREHOLE IS TO BE USED FOR MONITORING  
WITH ACCOMPANYING REPORT





LOCATION SKETCH  
(NOT TO SCALE)

Date MARCH 24, 1975




Golder Associates

Drawn J.A.  
Chkd. J.A.  
Appd. J.A.

LEGEND

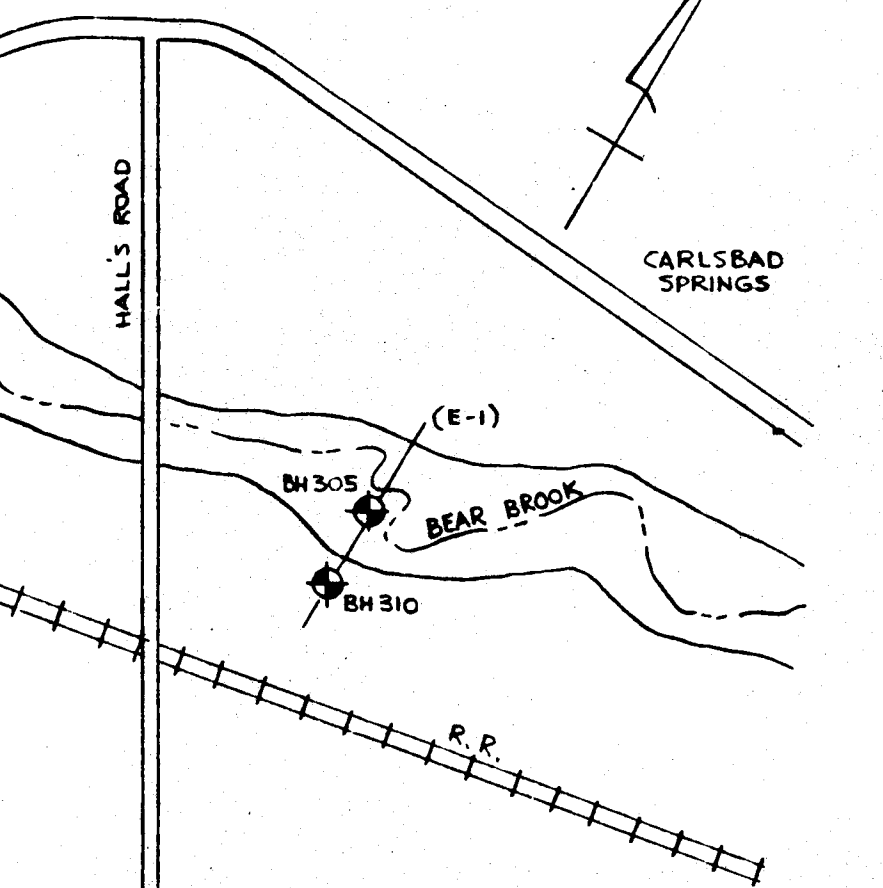
-  BOREHOLE IN PLAN
-  BOREHOLE IN ELEVATION

STRATIGRAPHY

-  TOPSOIL
-  FIRM WEATHERED GREY-BROWN SILTY CLAY, TRACE ORGANIC MATTER
-  FIRM TO STIFF SENSITIVE GREY SILTY CLAY

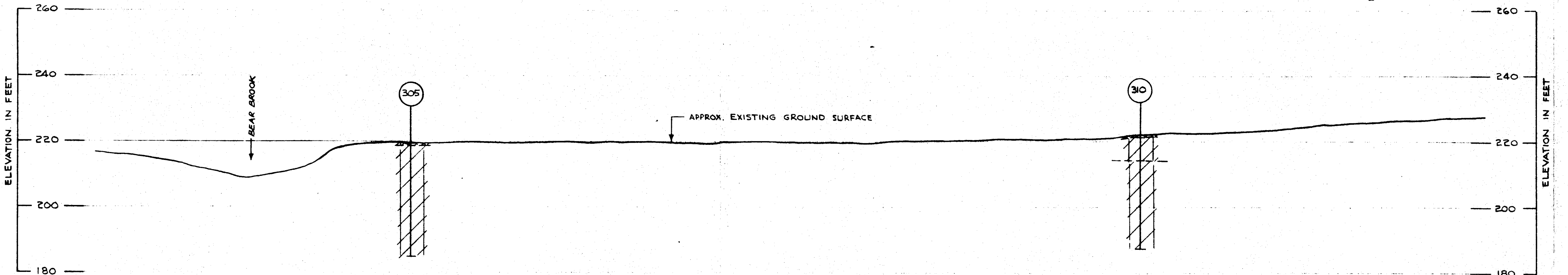
NOTE

Date: December 1974. The above data were obtained from a review of the borehole logs and the soil stratigraphy between the boreholes. The data were obtained from a review of the borehole logs and the soil stratigraphy between the boreholes. The data were obtained from a review of the borehole logs and the soil stratigraphy between the boreholes.



LOCATION SKETCH  
(NOT TO SCALE)

NOTE: SURVEY DATA SUPPLIED BY DE LEUW CATHER, CANADA LTD.



STRATIGRAPHIC SECTION ALONG LINE E-1  
SCALE: 1" TO 20'

SPECIAL NOTE  
THIS DRAWING IS TO BE USED IN CONNECTION  
WITH ACCOMPANYING REPORT

Date: MARCH 24, 1975

Golder Associates

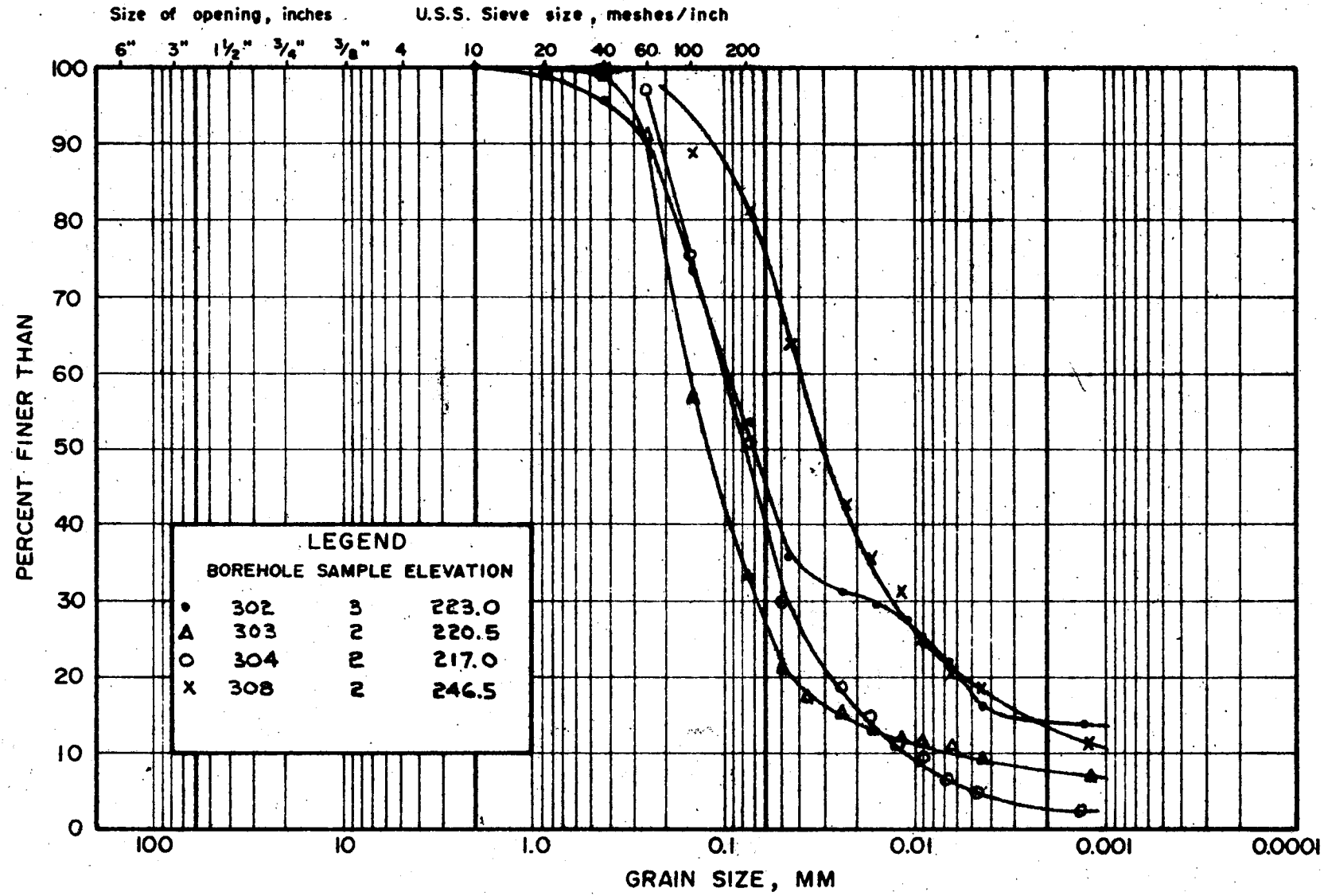
Drawn: J.A.  
Chkd: J.A.  
Appd: J.A.

M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION  
SURFICIAL SANDS

FIGURE 8

Golder Associates



Golder Associates

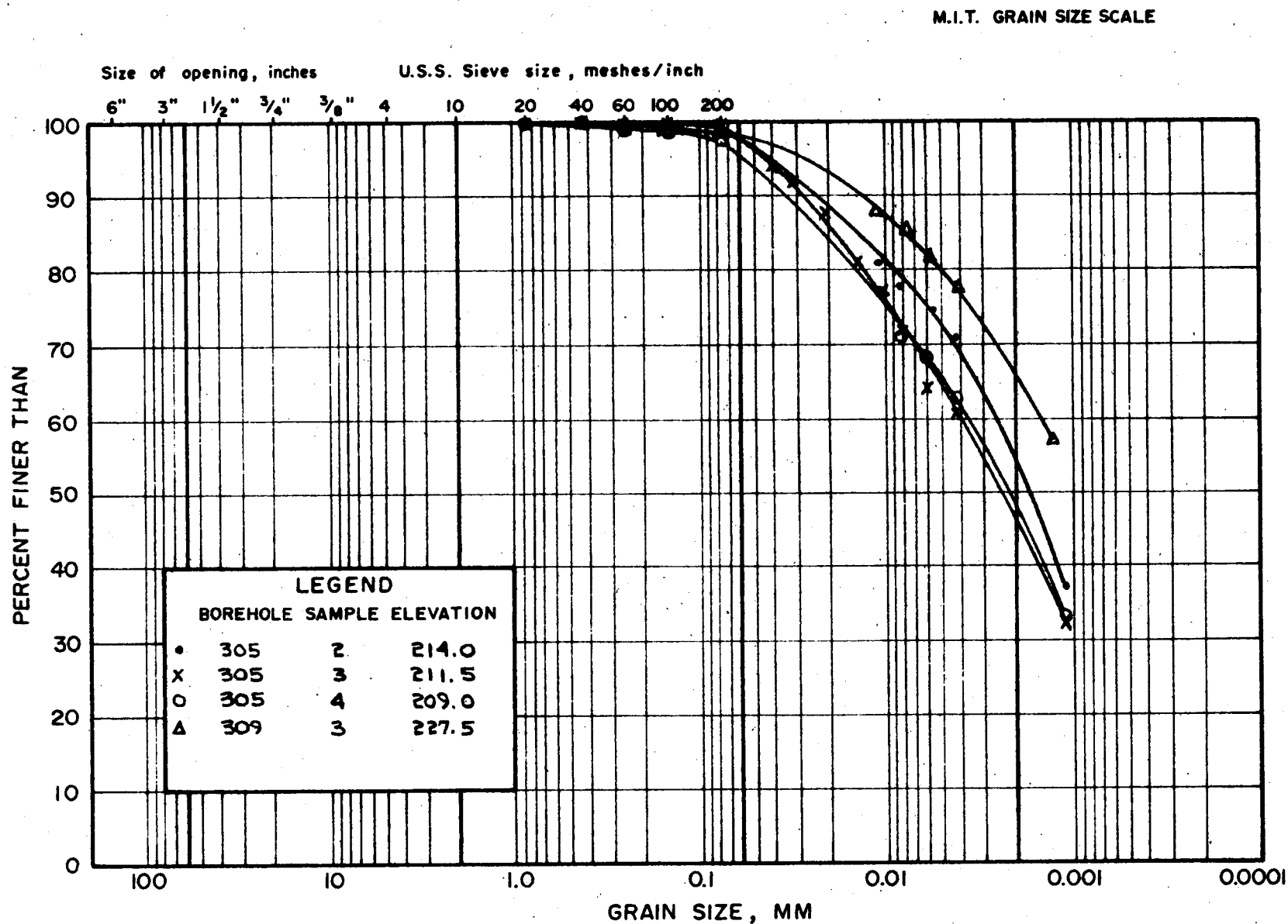
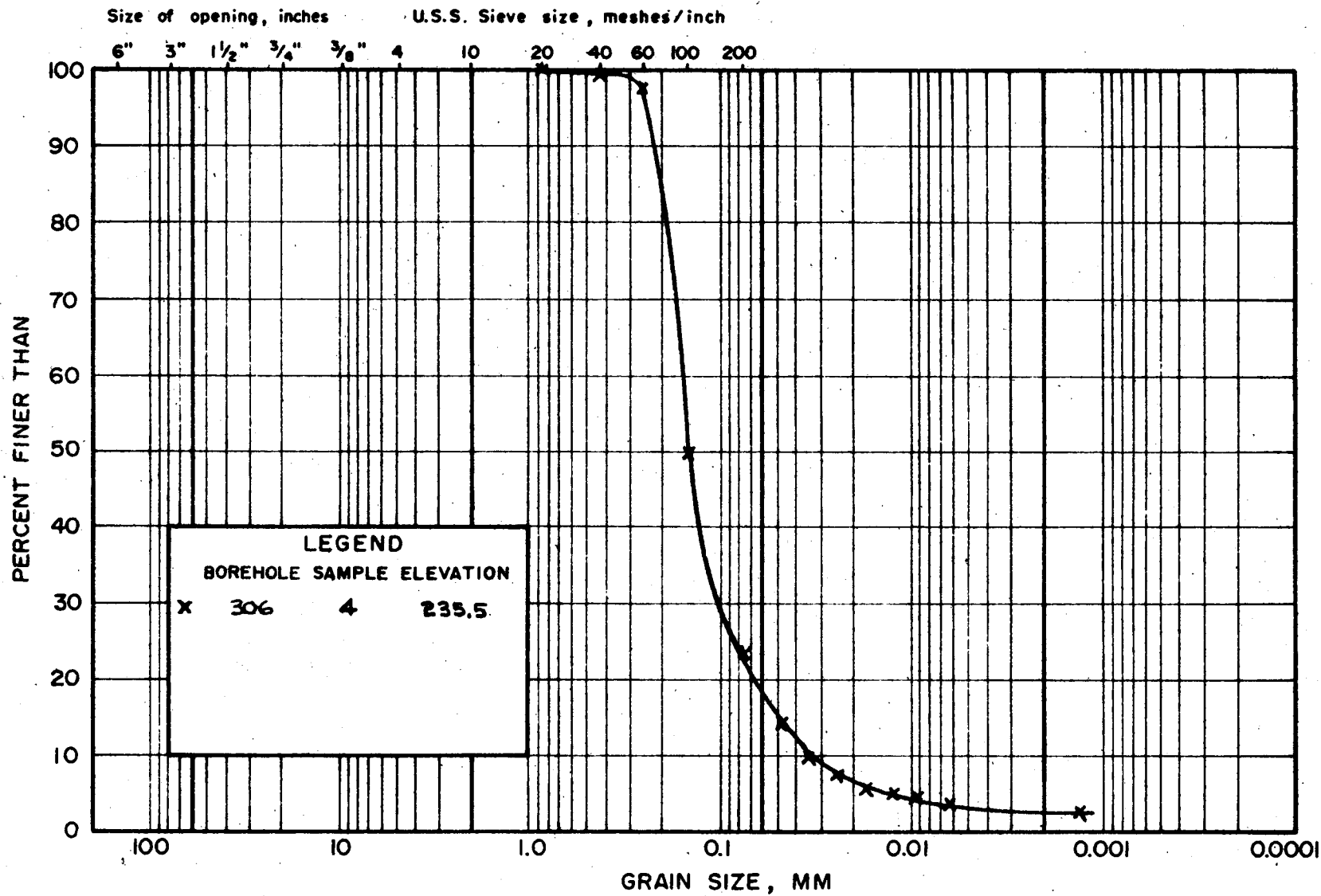
GRAIN SIZE DISTRIBUTION  
SILTY CLAY

FIGURE 9

## M.I.T. GRAIN SIZE SCALE



Golder Associates

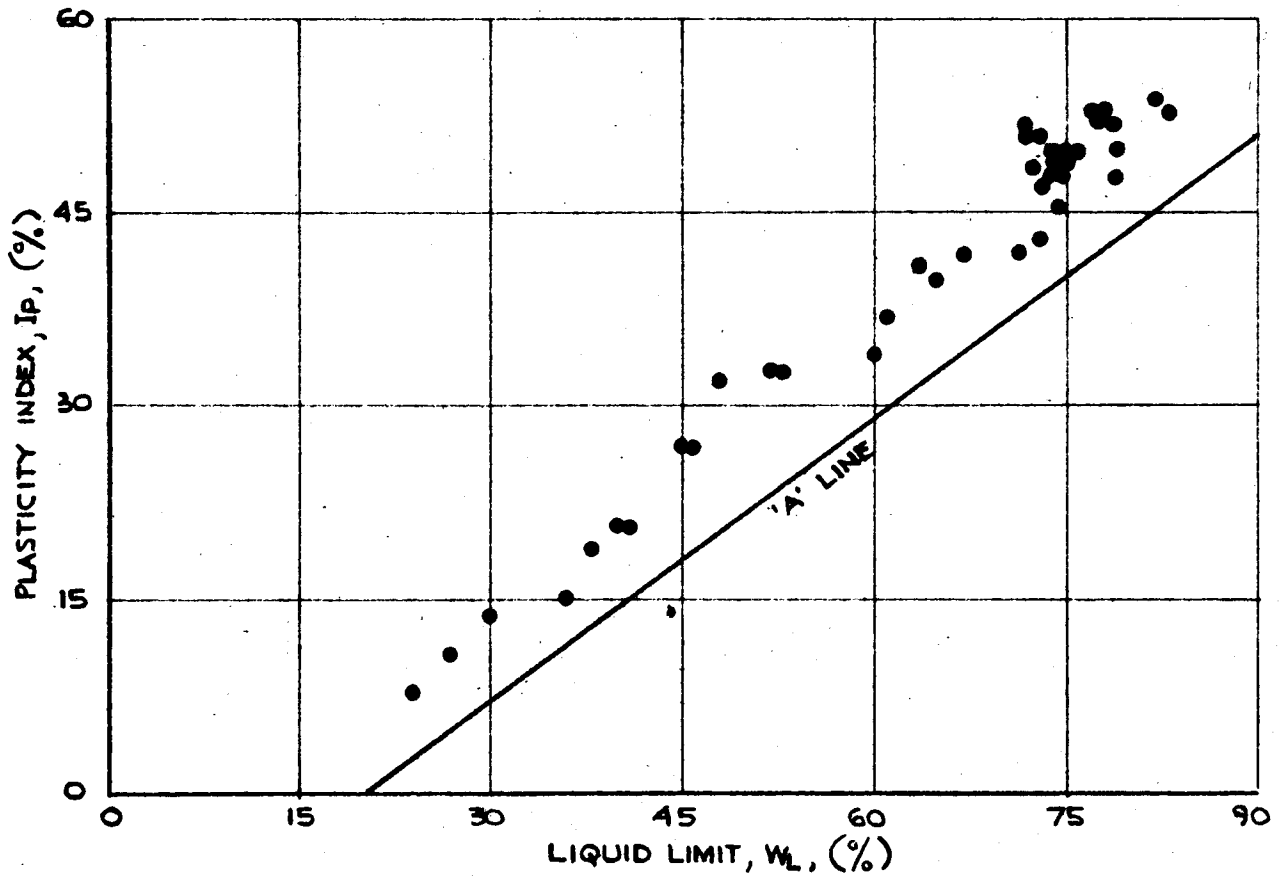
GRAIN SIZE DISTRIBUTION  
SAND LAYER IN SILTY CLAY

FIGURE 10

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

# PLASTICITY CHART

FIGURE II



Date MARCH 25, 1975

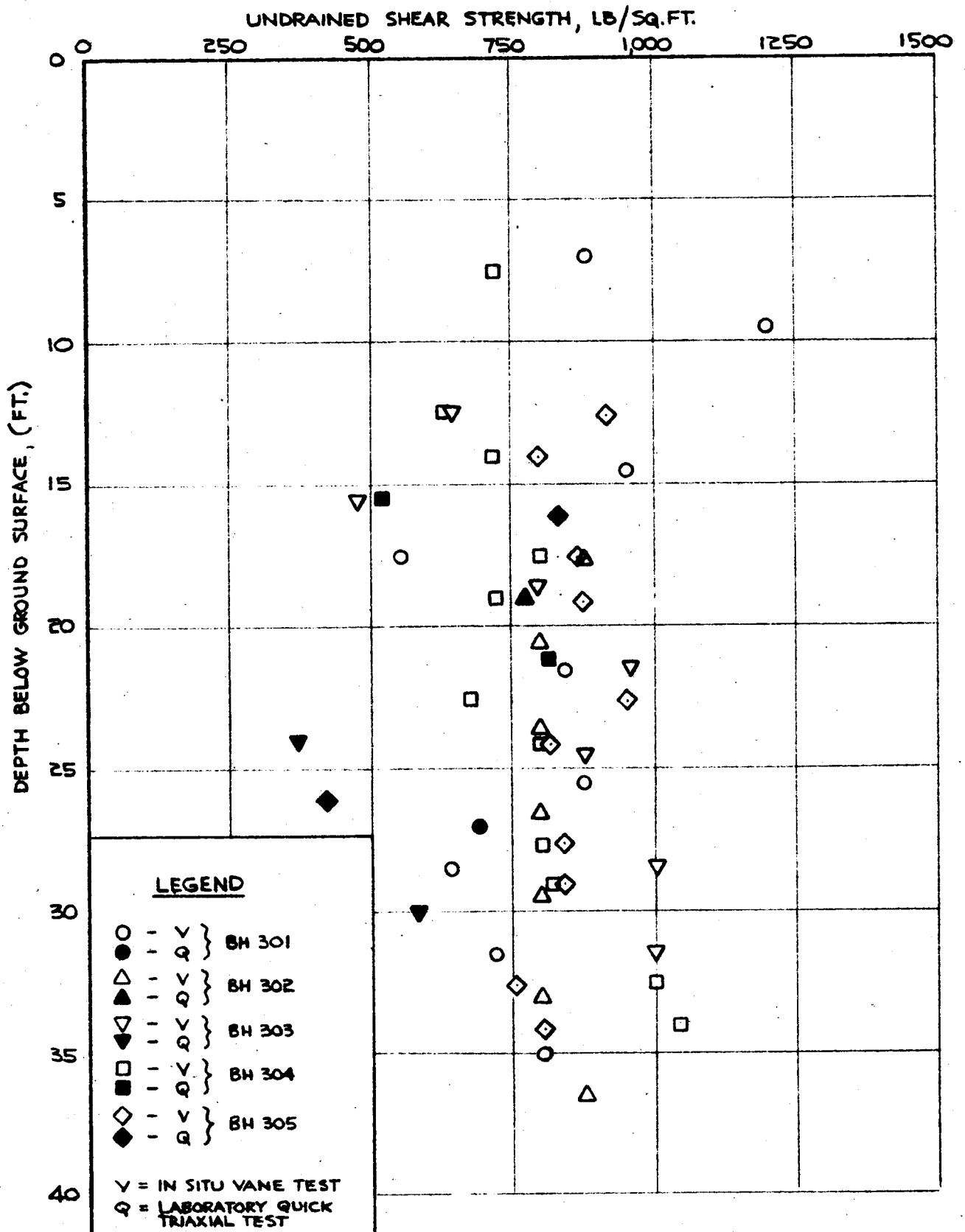
**Golder Associates**

Drawn J.A.  
 Chkd. EG  
 Appd. J.H.

PROJECT No. 7411231  
 Form G.A. - D - 4

# UNDRAINED SHEAR STRENGTH VS DEPTH BELOW GROUND SURFACE ( VALLEY BOREHOLES 301-305 )

FIGURE 12



Date MARCH 25, 1975

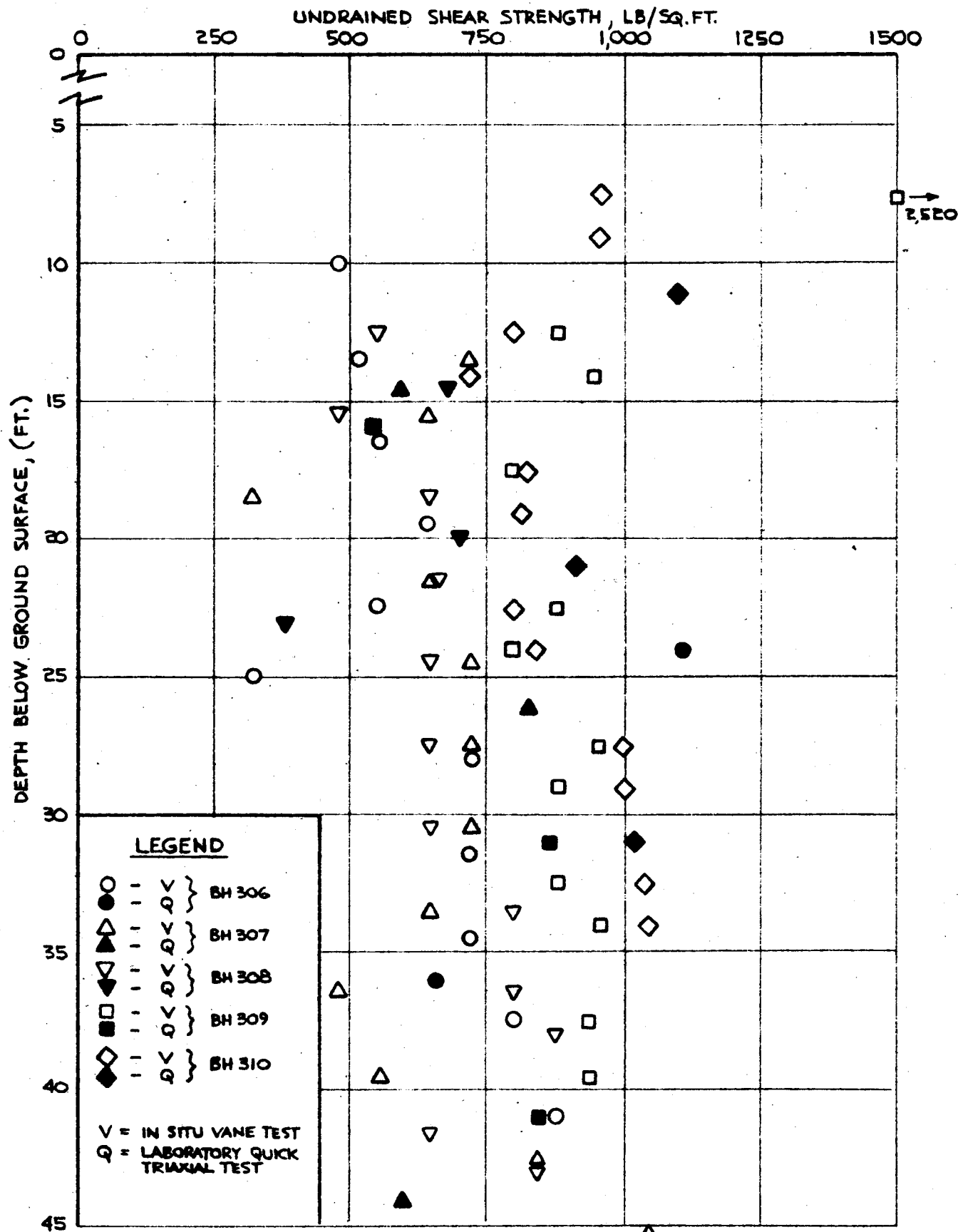
Golder Associates

Drawn J.A.  
Chkd. B.G.  
Appd. J.L.H.



# UNDRAINED SHEAR STRENGTH VS DEPTH BELOW GROUND SURFACE (CREST BOREHOLES 306 - 310)

FIGURE 13



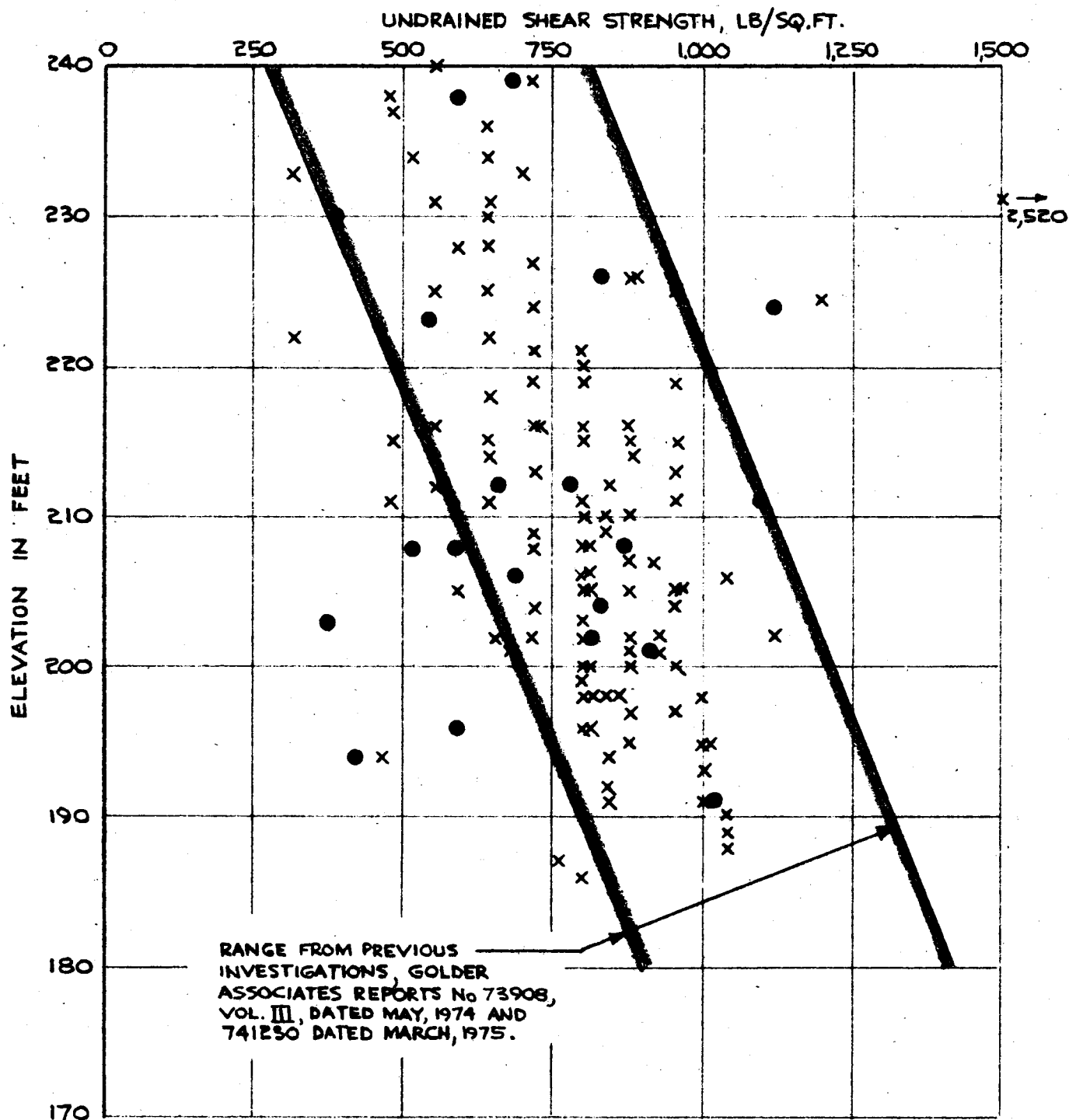
Date MARCH 25, 1975

Golder Associates

Drawn J.A.  
Chkd. J.A.  
Appd. J.A.

# UNDRAINED SHEAR STRENGTH VS ELEVATION (ALL BOREHOLES)

FIGURE 14



## LEGEND

- X IN SITU FIELD VANE TEST
- LABORATORY QUICK TRIAXIAL TEST

Date MARCH 25, 1975

**Golder Associates**

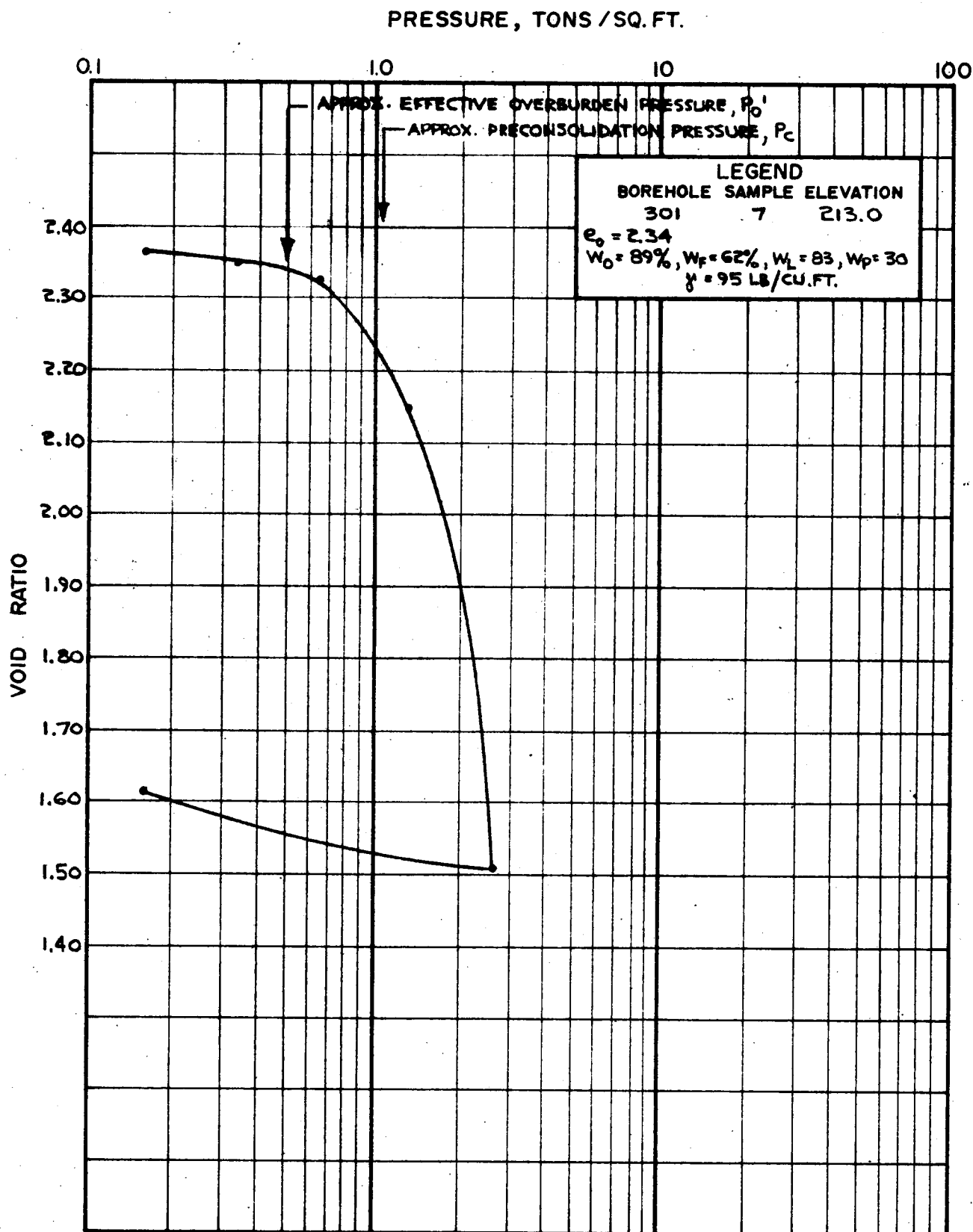
Drawn J.A.  
Chkd. B.B.  
Appd. J.H.K.

PROJECT No. 741231

Form G.A. - D - 4

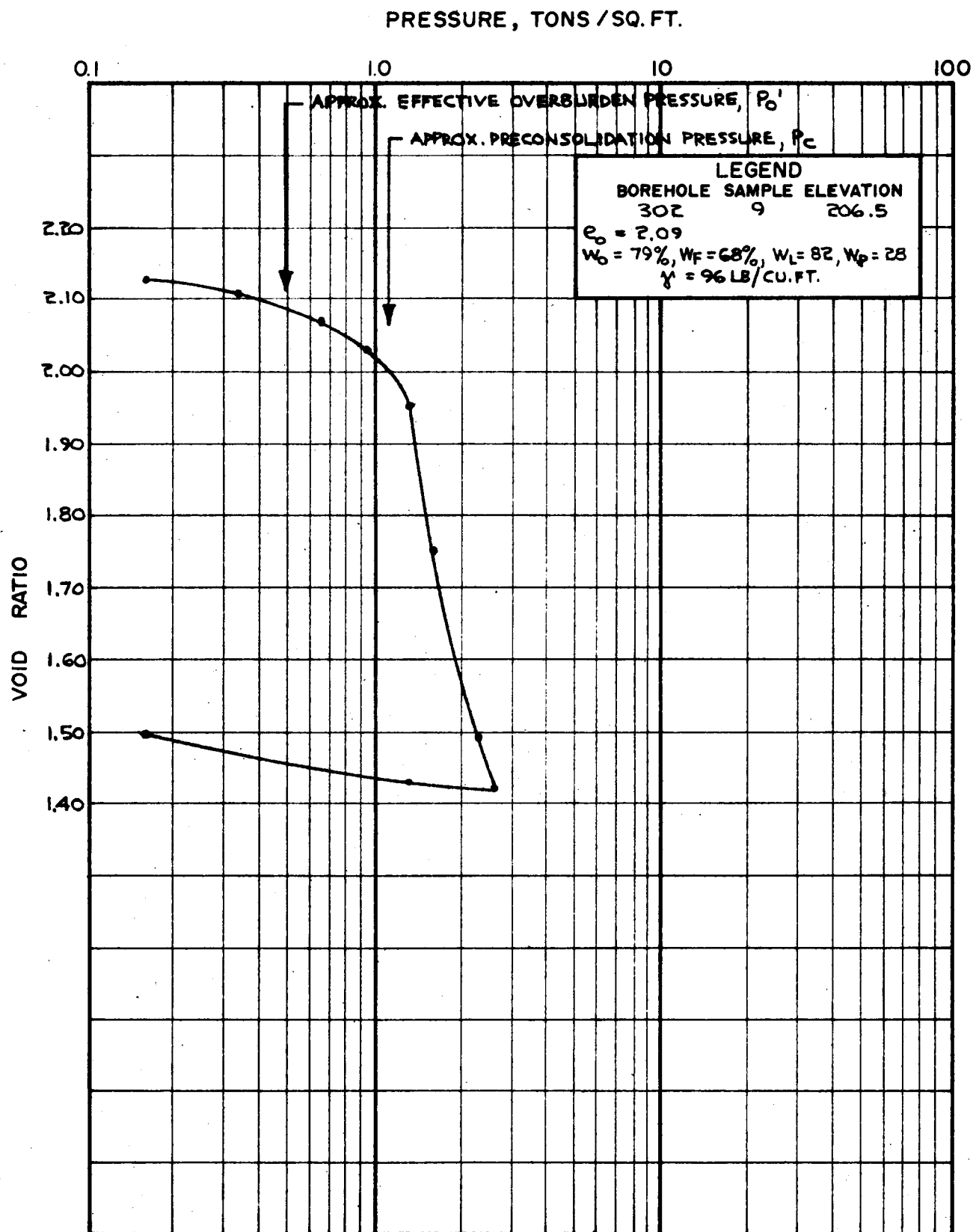
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 15



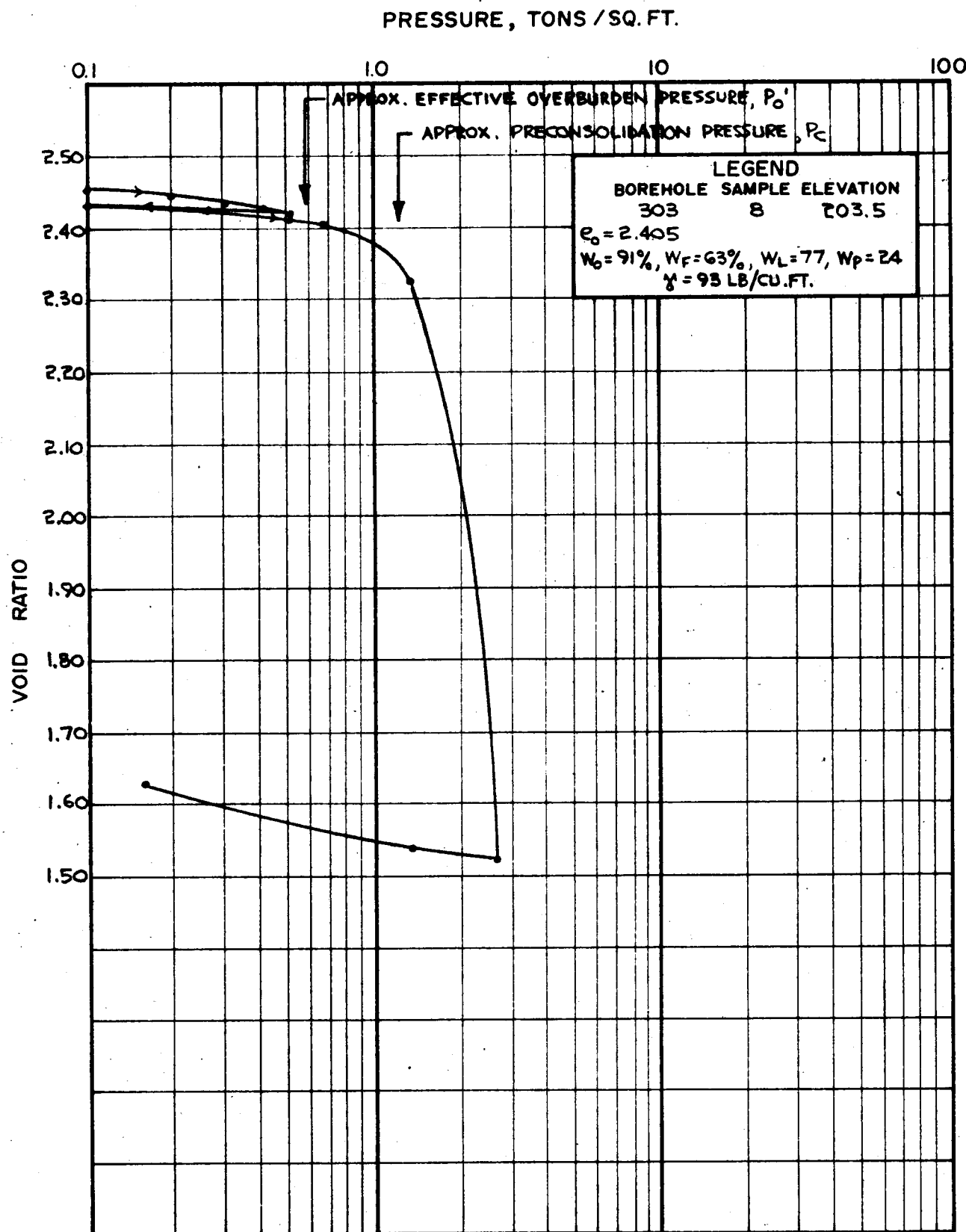
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 16



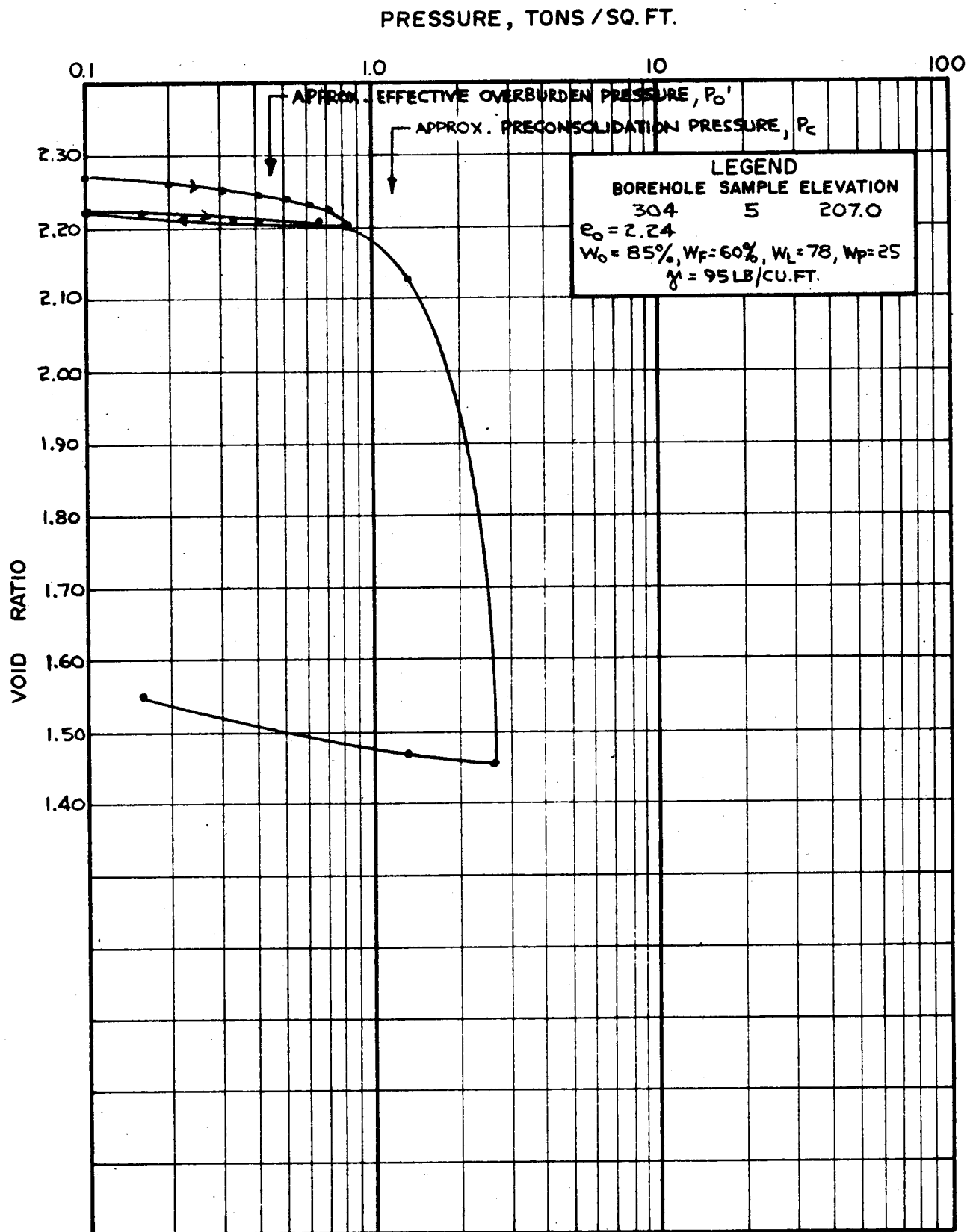
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 17



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

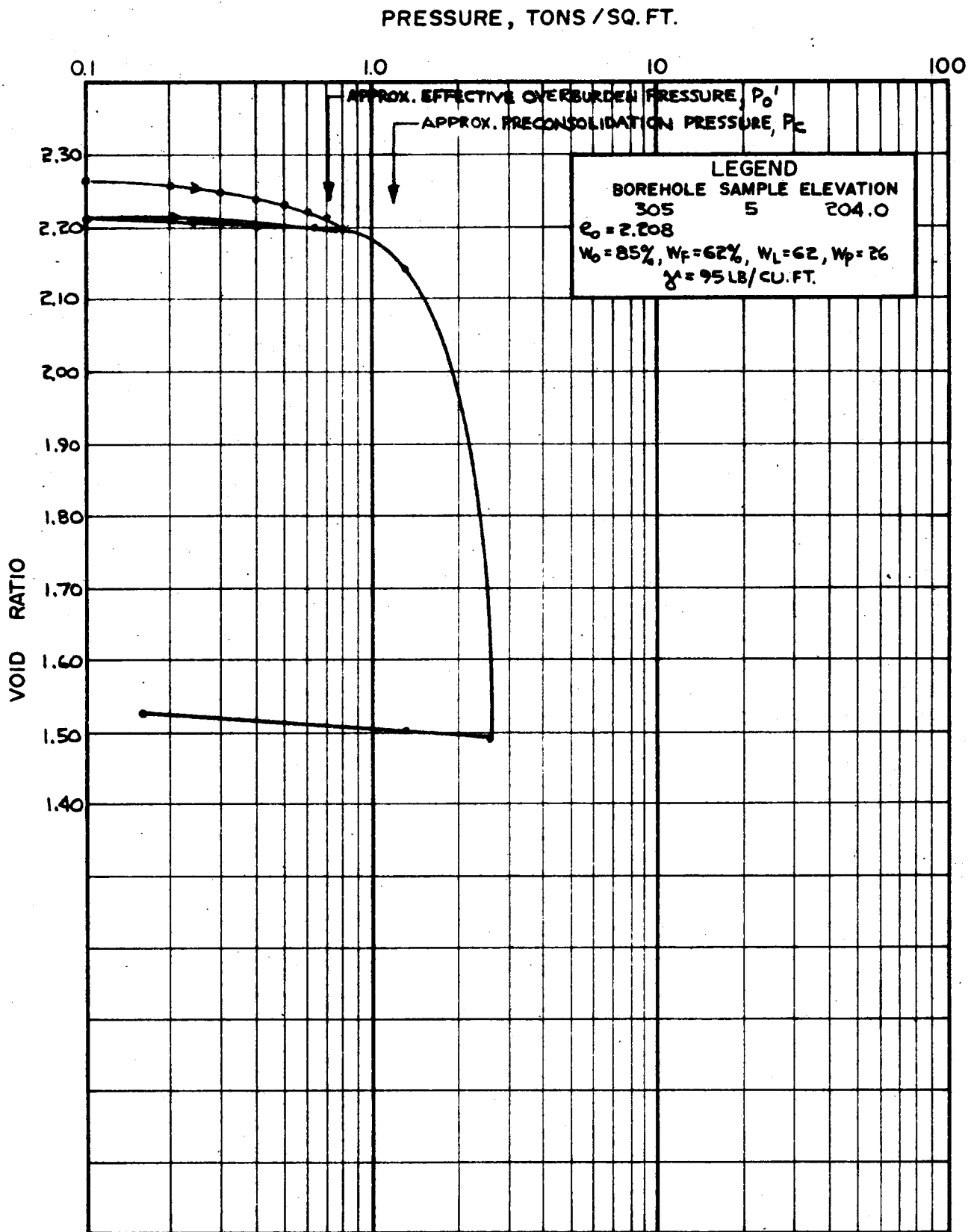
FIGURE 18



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# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

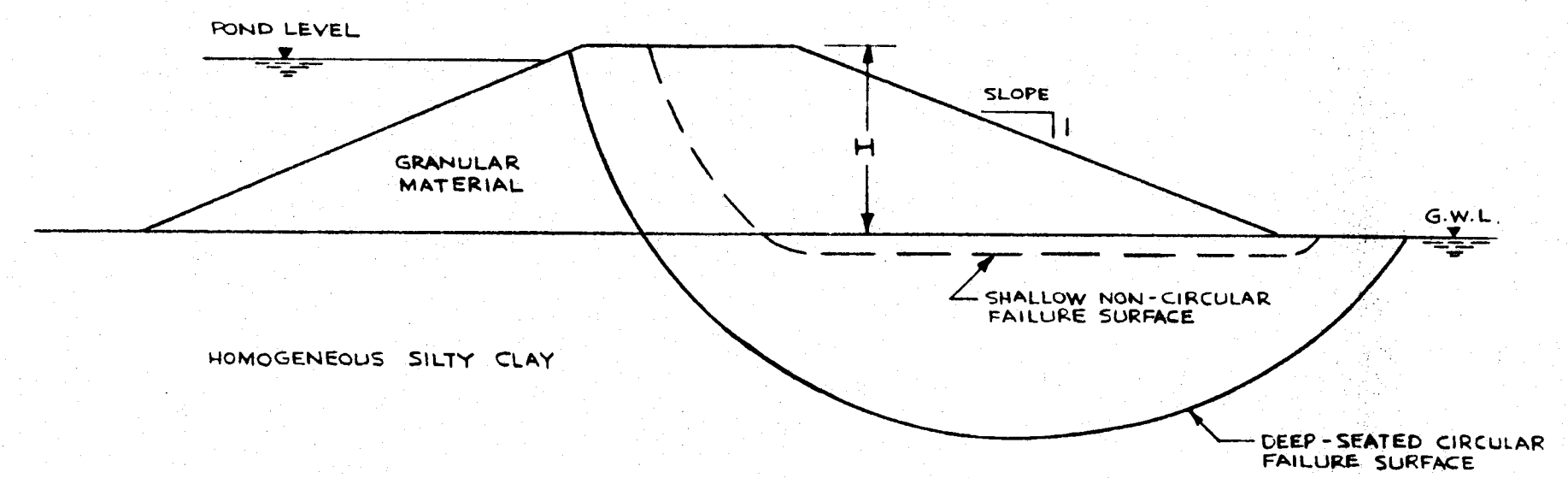
FIGURE 19



741231

OVERALL STABILITY OF DAM  
(NO SUBEXCAVATION UPSTREAM)

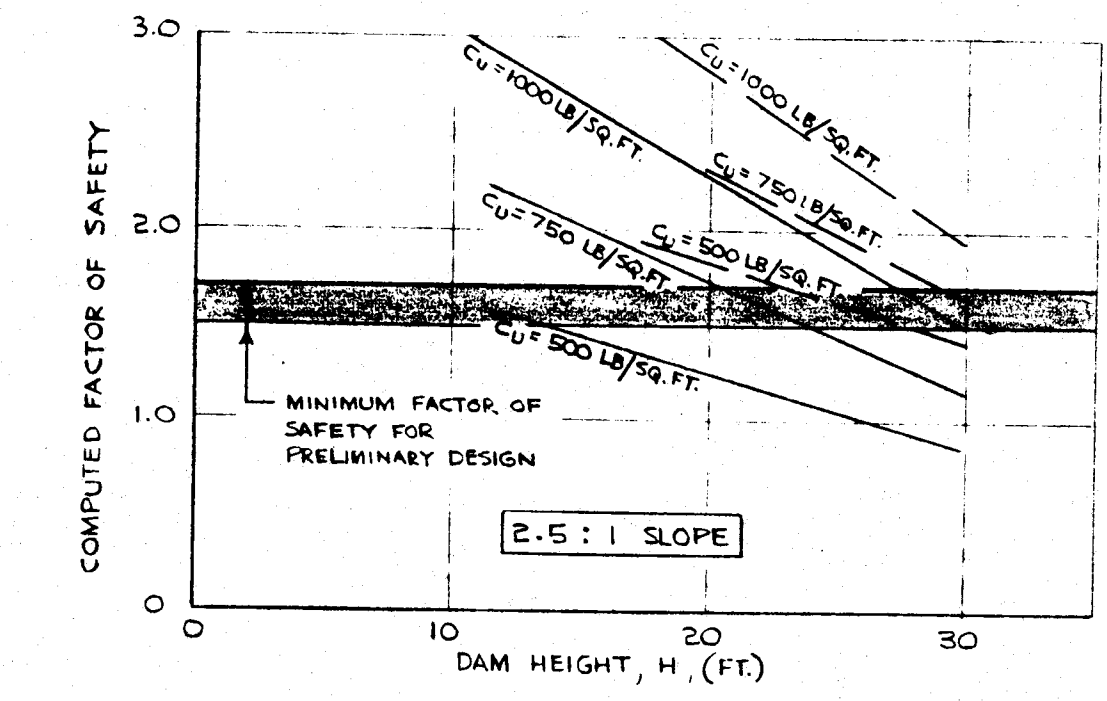
FIGURE 20



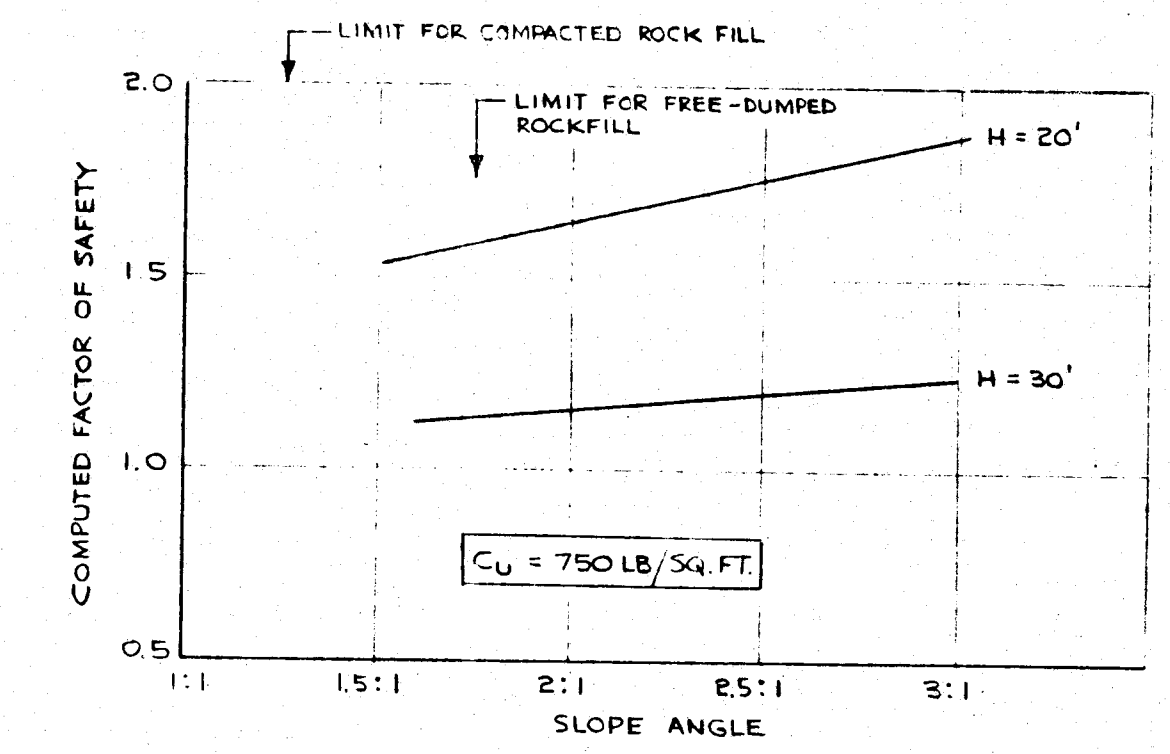
SOIL PROPERTIES

GRANULAR MATERIAL	$\gamma' = 130 \text{ LB/CU. FT.}$ $C' = 0$ $\phi' = 30^\circ$
SILTY CLAY	$\gamma' = 100 \text{ LB/CU. FT.}$ $C_u = \text{VARIABLE UNIFORM SHEAR STRENGTH}$ $\phi_u = 0$

CROSS SECTION AND SOIL PROPERTIES USED IN ANALYSES



LEGEND  
— DEEP SEATED CIRCULAR FAILURE SURFACE  
--- SHALLOW NON-CIRCULAR FAILURE SURFACE



EFFECT OF VARYING HEIGHT AND SLOPE ANGLE ON STABILITY OF DAM

Date MARCH 26, 1975

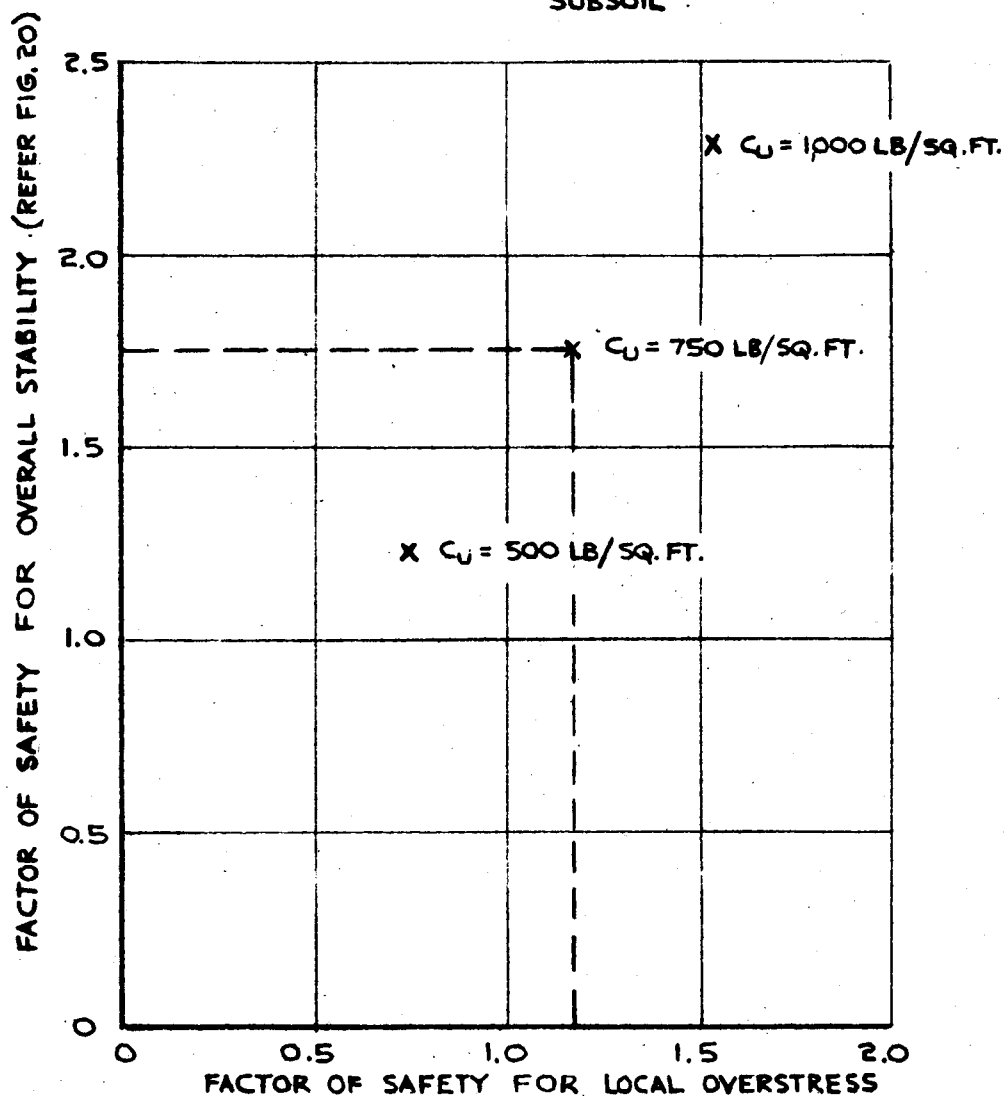
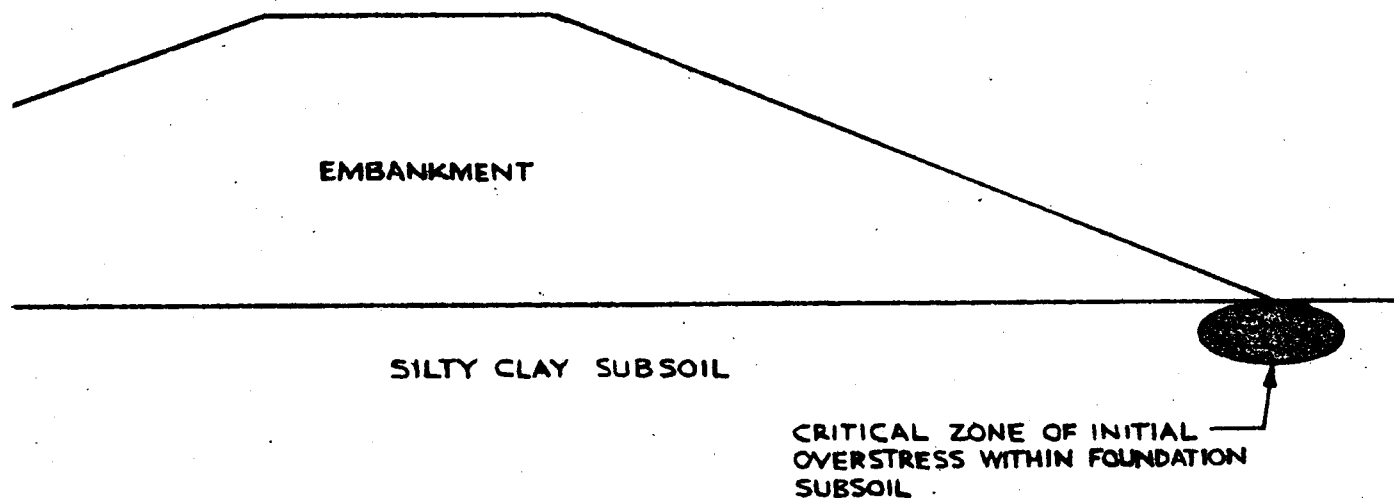
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Drawn J.A.  
Chkd. J.A.  
Appd. J.A.



# COMPARISON OF OVERALL STABILITY WITH LOCAL OVERSTRESS OF SUBSOIL

FIGURE 21



Date APRIL 2, 1975

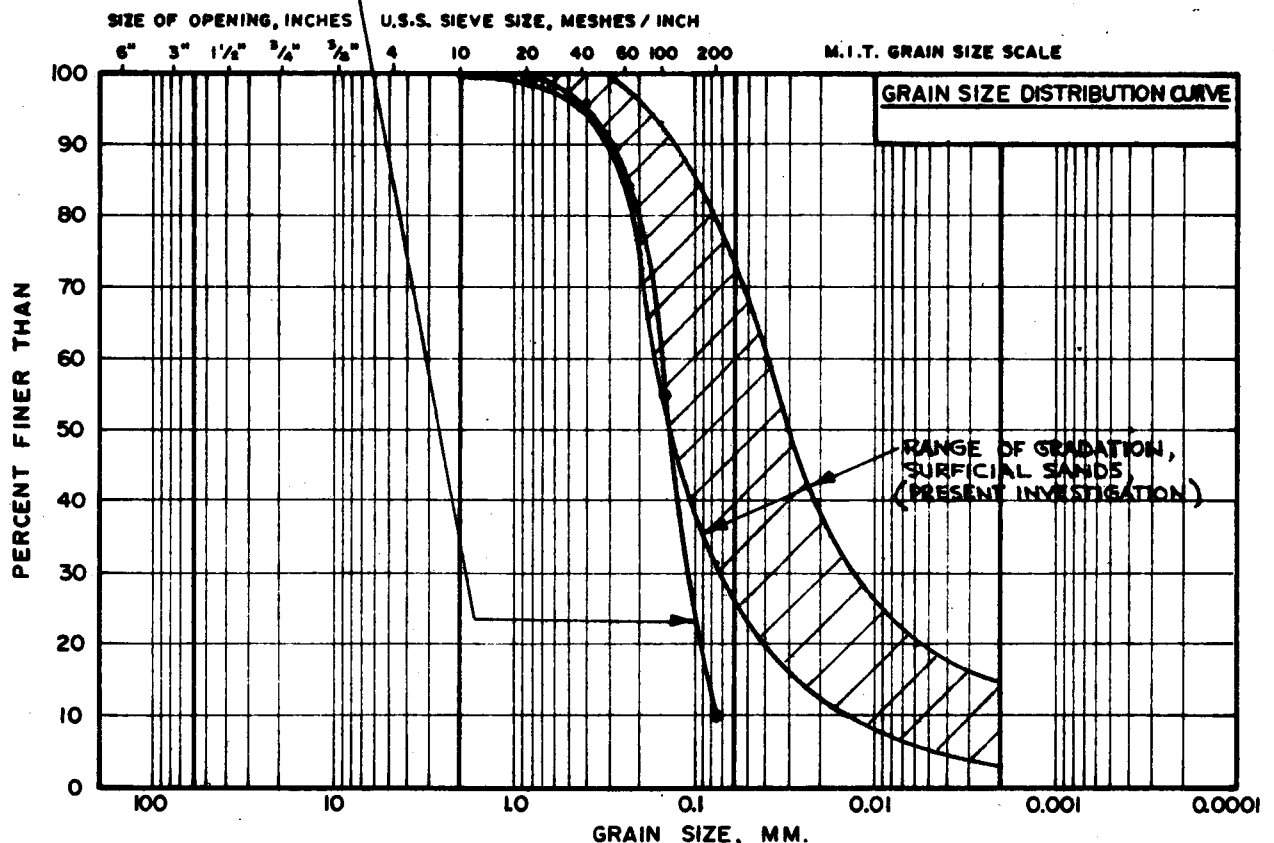
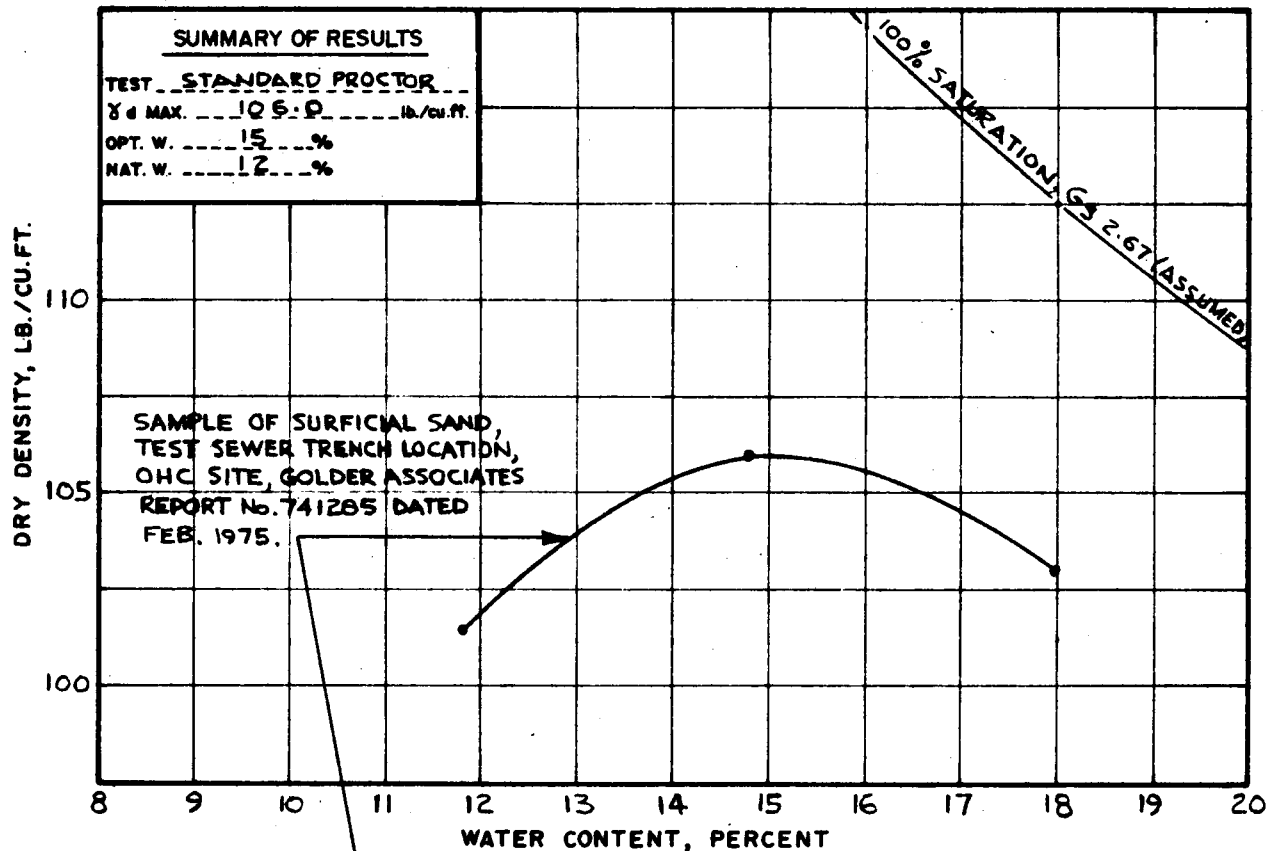
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Chkd. RS  
Appd. \_\_\_\_\_

PROJECT No 741231  
Form G.A. - D-4

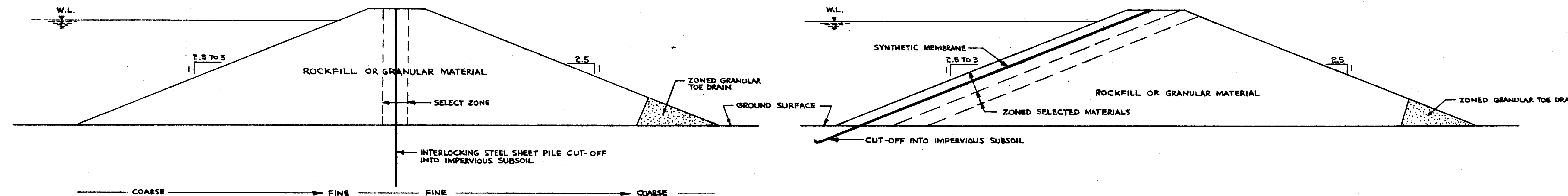
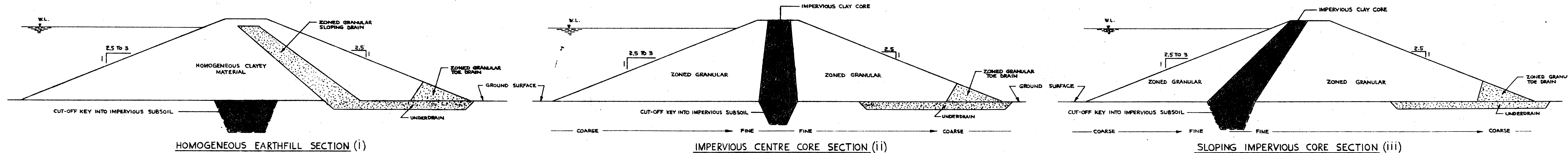
# LABORATORY COMPACTION TEST RESULTS SURFICIAL SANDS

FIGURE 22



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

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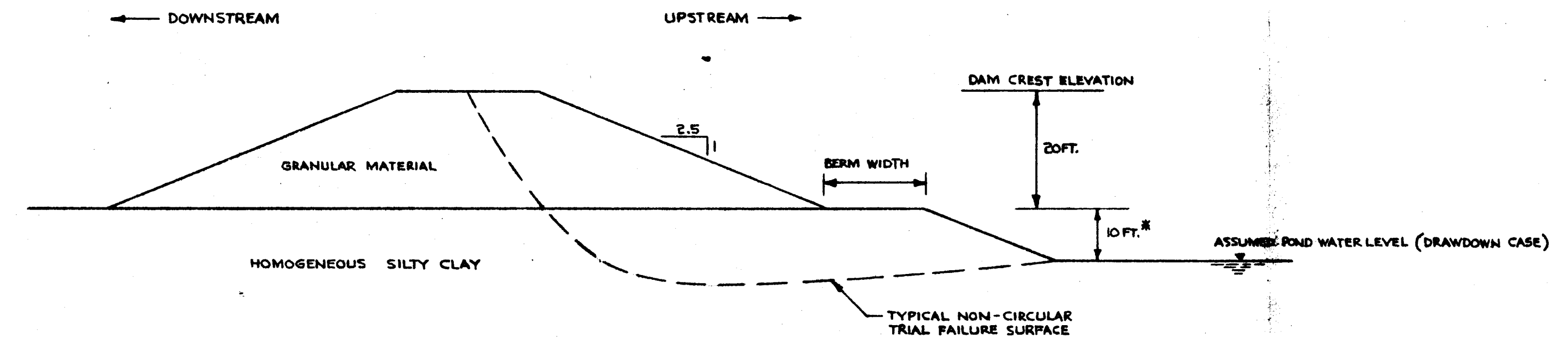
NOTE: SLOPE OF 2.5 (HORIZONTAL) TO 1 (VERTICAL) USED FOR PRELIMINARY ANALYSES AND SHOWN FOR ILLUSTRATION PURPOSES ONLY

NOT TO SCALE

Date MAR. 25, 1975

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Appd. J.H.C.



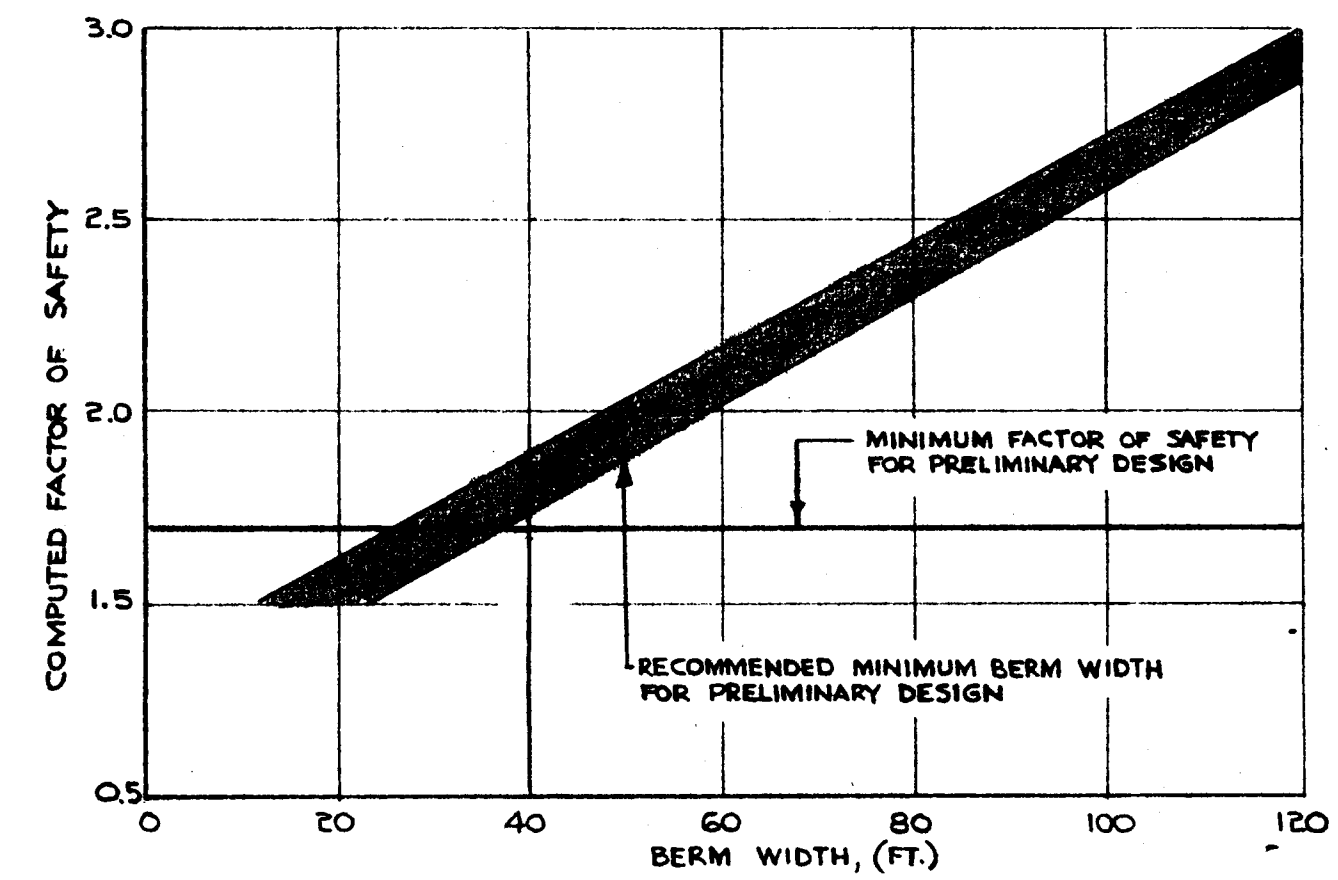
SOIL PROPERTIES

GRANULAR MATERIAL:  $\gamma = 130 \text{ LB/CU. FT.}$   
 $C = 0$   
 $\phi = 30^\circ$

SILTY CLAY:  $\gamma = 100 \text{ LB/CU. FT.}$   
 $C_u = 750 \text{ LB/SQ. FT.}$   
 $\phi_u = 0$

\* DEPTH OF SUBEXCAVATION MAY BE LIMITED BY  
CONSIDERATION OF STABILITY OF RIVER BANKS  
WITHIN POND AREA.

CROSS SECTION AND SOIL PROPERTIES USED IN ANALYSES



SUMMARY OF RESULTS  
EFFECT OF BERM WIDTH ON STABILITY OF DAM

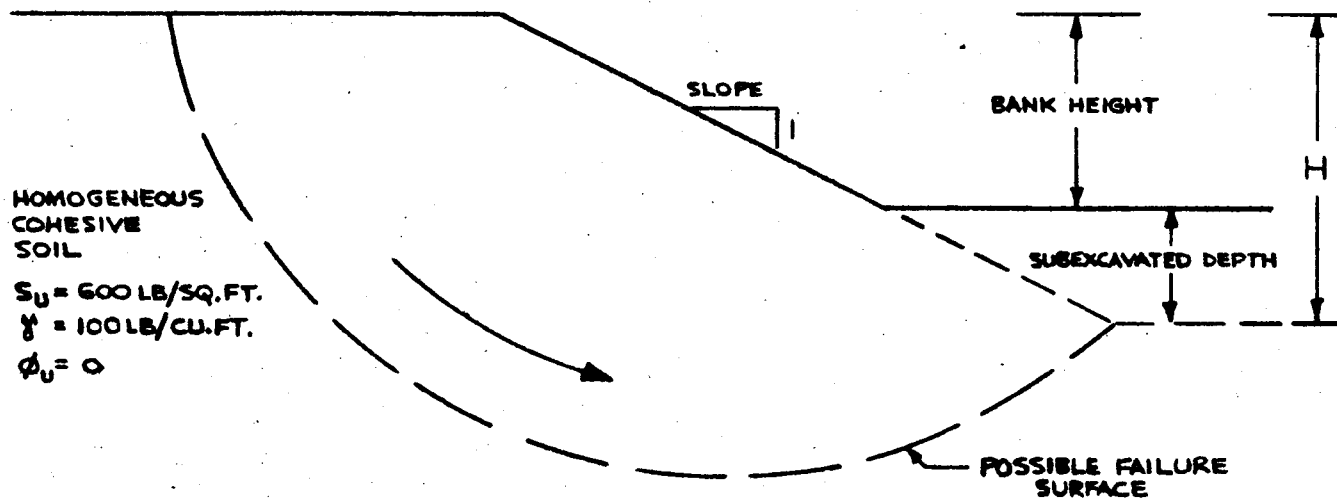
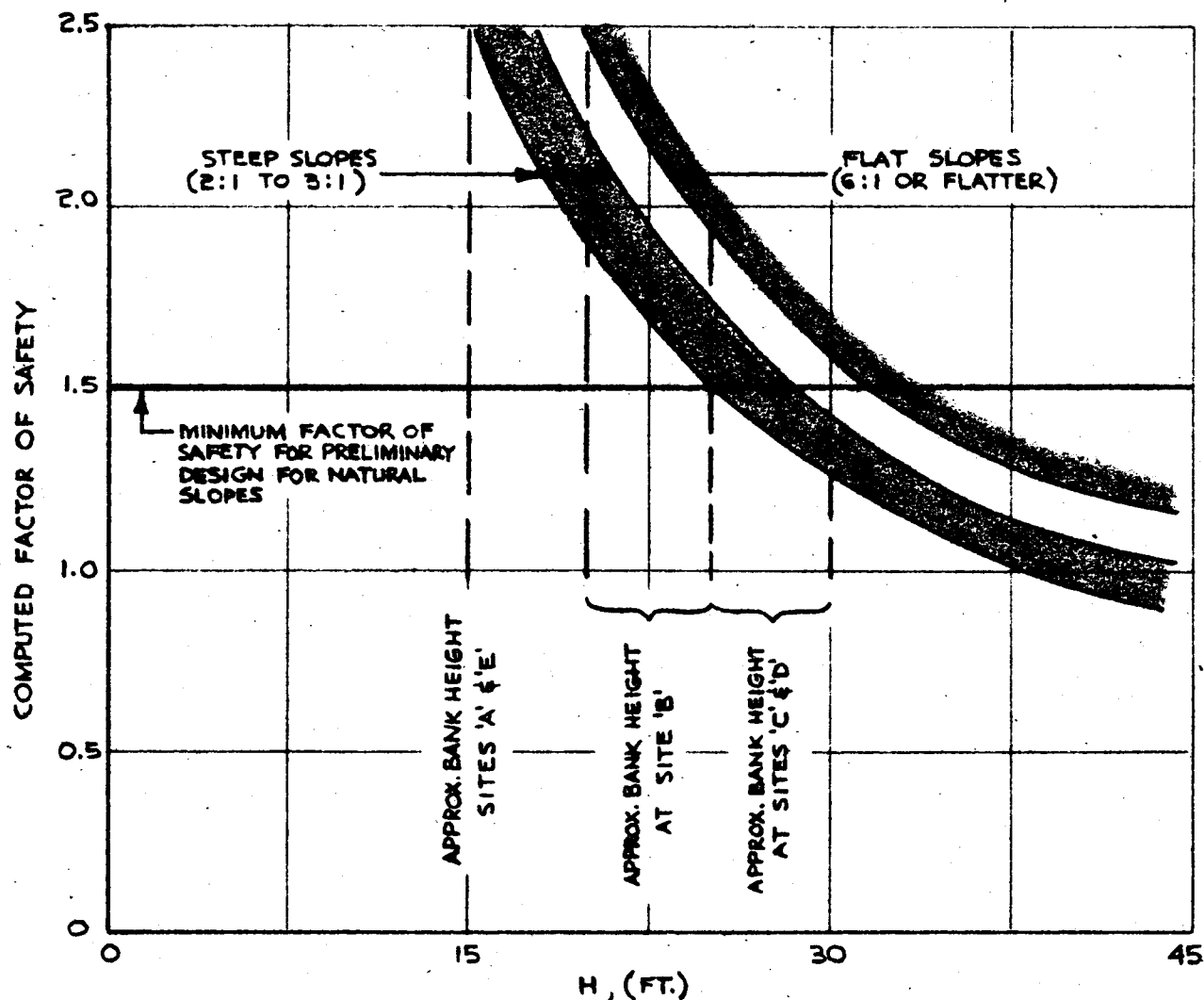
Date MARCH 26, 1975

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Appd. J.M.

# EFFECT OF SUBEXCAVATION ON STABILITY OF EXISTING SLOPES

FIGURE 25



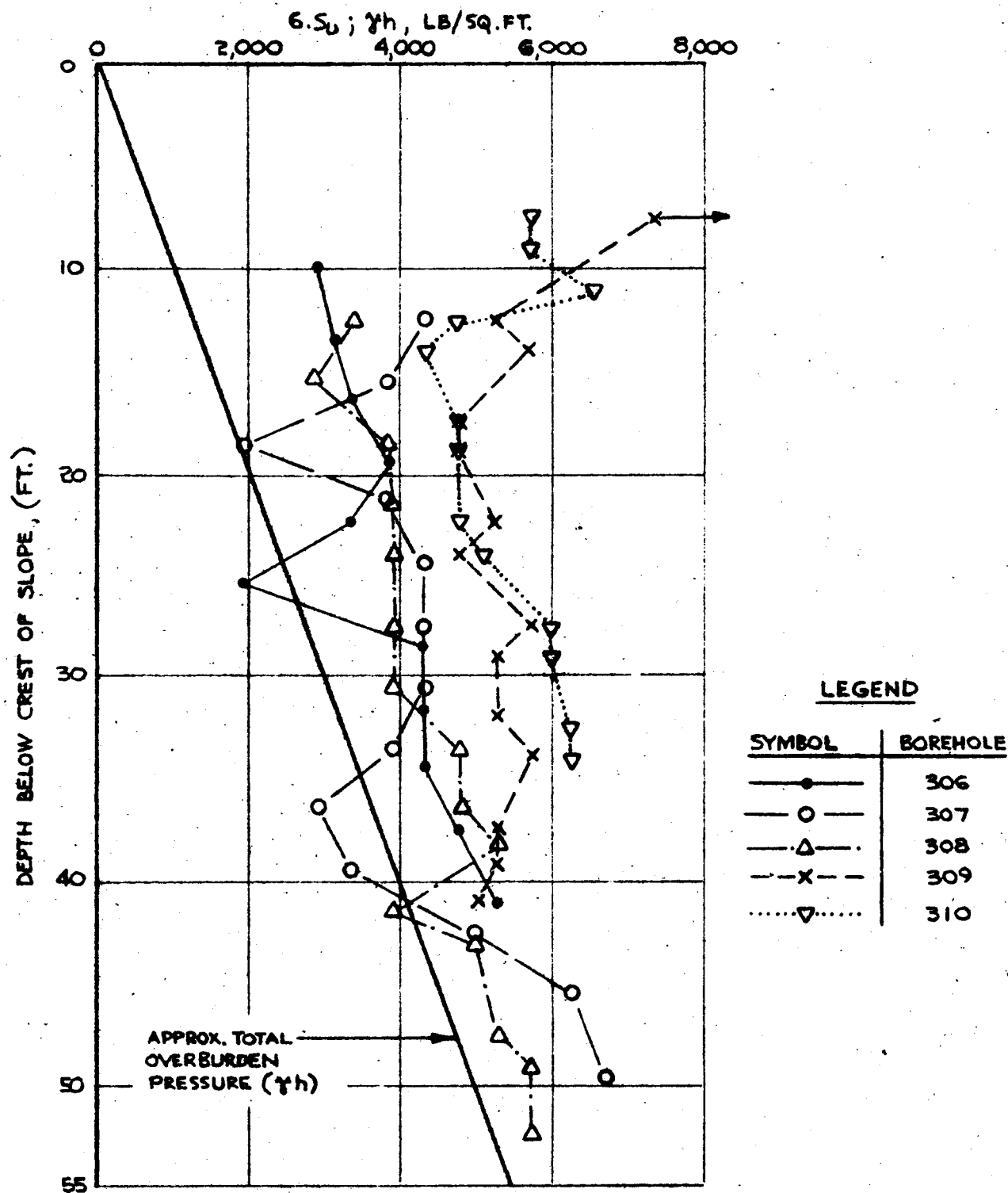
Date MARCH 25, 1975

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 Chkd. BE  
 Appd. \_\_\_\_\_

# STABILITY OF EXISTING SLOPES

FIGURE 26



STABILITY CRITERION:  $6.S_u > \gamma h$  (MITCHELL & MARKELL, 1974)  
( $S_u$  - VANE STRENGTH, LB/SQ. FT.)

Date MARCH 25, 1975

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Appd. \_\_\_\_\_