

GEOCREs No:  
31G-240-3



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

REPORT  
TO

DELEUW CATHER, CANADA LTD.  
ENGINEERING RECOMMENDATIONS  
SOUTH-EAST CITY  
REGIONAL MUNICIPALITY OF  
OTTAWA-CARLETON  
OTTAWA ONTARIO

VOLUME 4

Distribution:

70 copies - DeLeuw Cather, Canada Ltd.,  
Don Mills, Ontario.

2 copies - H. Q. Golder & Associates Ltd.,  
Mississauga, Ontario.

June, 1974

73908-1

## PREFACE

H. Q. Golder & Associates Ltd. and K. H. King and Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd. Consulting Engineers to the Ontario Housing Corporation, to jointly carry out a geotechnical investigation at the site of a proposed new community which is being planned for a portion of the Township of Gloucester in the south-east section of the Regional Municipality of Ottawa-Carleton. Previous limited investigations had indicated that subsurface conditions would influence the planning for the new community. The purpose of this investigation was to obtain more detailed information on the subsurface conditions. On the basis of the additional information together with previous published and unpublished data relevant to the proposed site, engineering recommendations are made regarding the geotechnical factors which influence the planning of the proposed development.

The geotechnical work incorporated a variety of shallow and deep boreholes, geophysical tests, trial excavations as well as extensive laboratory testing. For convenience the information has been prepared in the following reports:

- Vol. 1    Subsurface Investigations carried out by K. H. King and Associates Ltd. (Report 312-S2-Vol. 1)
- Vol. 2    Geotechnical Mapping prepared by K. H. King and Associates Ltd. (Report 312-S2-Vol. 2)
- Vol. 3    Subsurface Investigations carried out by H. Q. Golder & Associates Ltd. (Report 73908)
- Vol. 4    Engineering Recommendations prepared by H. Q. Golder & Associates Ltd. (Report 73908-1)

Volumes 1 and 3 contain the factual information which was obtained during the field investigations and associated laboratory testing. This information has been incorporated with earlier information in the preparation of the remaining two reports.

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## ABSTRACT

This volume discusses various geotechnical aspects of the site development based on the data given in Volumes 1 to 3 together with published and unpublished information available from studies of soil conditions at sites located near the area under consideration. From the results of foundation analyses and the overall pattern of crust thickness and depth to bedrock across the site obtained from fieldwork results, recommendations are made regarding the optimum location of the various types of structures. Within reasonable constraints concerning such optimum locations, residential and ancillary commercial structures can safely be built on the site using conventional construction methods.

Typical allowable bearing pressures are given for different combinations of soil conditions and foundation types and are discussed in terms of the applied loadings of different types of buildings to be included in the development.

Geotechnical considerations relative to design and installation of services and roads at the site are discussed. Groundwater levels and site drainage are considered and methods of maintaining river bank stability along Bear Brook are recommended.

## INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd., Consulting Engineers to the Ontario Housing Corporation to carry out an appraisal of the geotechnical factors which influence the preliminary planning for the proposed development of a site located in the Township of Gloucester in the south-east area of the Regional Municipality of Ottawa-Carleton, Ontario. The fieldwork for the investigation was carried out jointly by H. Q. Golder & Associates Ltd. and K. H. King and Associates Ltd. and the factual results of those parts of the investigation reported in Volumes 1 and 3. This volume discusses various geotechnical aspects of the site development based on the data given in Volumes 1 to 3 together with published and unpublished information available from studies of soil conditions at sites located near the area under consideration.

## DESCRIPTION OF PROJECT

A site of approximately 6,000 acres located in the Township of Gloucester is to be developed as a satellite city to the city of Ottawa. It is understood that in the long term the proposed community will be largely self-sufficient and that the population will be of the order of 100,000 people. A development of this nature involves the construction of a wide variety of housing from single family residences to high rise apartment buildings. Social facilities, such as schools, community centres, shopping centres and churches together with areas of light industrial structures will be included. It is understood that the existing road system within the site will not likely be retained. As there are no services in the area at present, development will require the installation of all the required services a majority of which are to be placed underground. The existing drainage system of Bear Brook and its associated tributaries is to be used without major modifications to the location of existing channels. However, drainage control measures will be considered for the vicinity of Bear Brook in order to regulate and control the system.

SUMMARY OF SUBSURFACE CONDITIONS

The results of fieldwork and laboratory testing of samples recovered from the site have been reported in Volumes 1 and 3. Geotechnical mapping of the site is reported in Volume 2. Following is a summarized account of the subsurface conditions across the site. Also included is a discussion of the soil properties used in the analyses which form the basis of recommendations relevant to the preliminary planning stage of the development.

Stratigraphy

The site which is under examination extends over an area of some 6,000 acres is basically flat with ground surface elevations ranging between 250 ft. and 270 ft. It is located on Champlain Sea deposits which are common in the Ottawa area. Underlying a desiccated silty clay crust or sand cap of variable thickness is an extensive deposit of highly sensitive soft silty clay, (Champlain Sea clay). This deposit which is the predominant stratum at the site varies in thickness from about 45 ft. to 149 ft. at the locations of the deep borings put down during the fieldwork.

The clay deposit is in turn underlain by a glacial till sheet of varying thickness which in turn overlies shale bedrock. The depth to bedrock from ground surface varies considerably across the site from about 35 ft. to greater than 200 ft. in the south-east and north-west sectors respectively.

The groundwater level across the site, during the period when fieldwork was carried out, was typically about 3 ft. below the ground surface. However, fluctuations in groundwater level have taken place in the past, as evidenced by the existence of the upper desiccated crust. Groundwater levels are as much as 6 ft. lower in later summer (see later).

### Soil Properties

During the course of the present investigation, information on soil properties has been obtained during laboratory and field tests. Additional information is also available from the results of site investigations carried out in connection with the construction of Highway 417 and research carried out by the National Research Council, Division of Building Research, on sites close to the area presently under investigation. This section describes the range of soil properties measured in tests on samples recovered from the site and compares these with the results of the other published and unpublished test results. Emphasis has been placed on the selection of relevant soil properties for use in engineering design.

#### i) Classification Tests

The results of Atterberg limit tests on a large number of samples of desiccated crust and Champlain Sea clay indicate that the marine deposits are highly stratified particularly at shallow depth, with the various strata ranging from non-plastic to highly plastic. The natural moisture content lies between 21 percent to 101 percent. Consequently, the liquidity index of the clay is often in excess of unity, except for the desiccated crust value where it is generally less than one.

The variation of the in situ water content is given in the individual Record of Borehole sheets in the previous volumes, and examples of the variation over short intervals of depth are given in Volume 3. As the liquidity index increases from 'zero' at the plastic limit to the liquid limit value (with a liquidity of unity), the soil becomes more difficult to work in terms of site grading and construction. This factor is discussed in connection with site grading design since it limits



the convenient depth to which site excavated material can be used for grading and landscaping.

ii) Undrained Shear Strength

The results of field vane and triaxial compression tests show that the undrained strength of the desiccated silty clay crust lies between 480 and 2300 lb/sq. ft. with an average value of about 1000 lb/sq. ft. However, because of the fissured nature of the soil in the desiccated zone, the undrained shear strength measured in tests on small samples may overestimate the strength in this zone where the lateral stresses in the soil are reduced by excavation in the field. For the purposes of preliminary foundation design, where field loading causes increased lateral stresses, the undrained shear strength of the desiccated crust has been taken as 800 lb/sq. ft. This minimum design value is consistent with the footing sizes and loads developed from present residential structures on the site in areas where there is shallow crust. Unconsolidated undrained triaxial tests on samples of desiccated crust trimmed at various angles from a block sample indicate that the undrained strength of this material is apparently isotropic, (Fig. 16, Vol. 3).

Underlying the desiccated crust or sand cap, the undrained shear strength of the clay ranges between about 300 and 650 lb/sq. ft. As is typical in this type of deposit, the undrained strength increases with depth to values which are generally in excess of 2000 lb/sq. ft. at a depth of about 100 ft. below ground surface. The overall strength increase with depth can approximately be expressed by a ratio of undrained shear strength to effective overburden pressure,  $S_u/p$ , of about 0.3 to 0.4

Previous work has shown that the undrained strength of lightly overconsolidated post glacial clay deposits may depend on direction, (Aas, 1967). This factor must be taken

into account in the analysis of field loading cases which may cause rupture of the soil along a curved failure plane. However, the results of a series of 10 quick triaxial tests on specimens trimmed from a block sample indicates that the undrained shear strength is largely independent of sampling direction and can therefore be considered as isotropic for the purpose of analysis, (Fig. 16, Vol. 3).

The magnitude of undrained shear strength of some soils may also depend on the rate at which the soil sample is sheared, (Bjerrum, 1972). This factor is important in some soils not only in comparing the results obtained from different test types but also in determining the undrained strength to be used in stability analyses of structures in the field. To assess this effect, the results of unconsolidated undrained tests carried out at strain rates ranging from 0.03 to 1.5 percent/min. were examined, and are plotted on Figs. 1 and 2. The variation in the undrained shear strength of the clay has no consistent trend with strain rate and is also about the same range in magnitude as the variation in strength as determined from the field vane tests (as indicated in Volume 3). These results suggest that within the limited range of available data, the strain rate effect of the clay is of the same importance as the range in natural stratification and that reductions in shear strength are not warranted solely for this effect.

The minimum value of the undrained shear strength as measured in field vane tests and quick triaxial tests on tube samples of the clay under the desiccated crust is about 300 lb/sq. ft. However, the results of unconsolidated and consolidated undrained triaxial tests on specimens trimmed from block samples indicate that the undrained shear strength is about 600 lb/sq. ft. This apparent difference in measured strength may be attributed to the lower degree of disturbance associated with block sampling techniques and is confirmed by previous research on the effect of sampling method on the measured undrained

strength of clay, (Raymond, Townsend and Lojkasek, 1971). Therefore, it is possible that the undrained shear strength immediately below the crust is greater than 300 lb/sq. ft. over most of the site.

The variation in the undrained shear strength for field and laboratory tests carried out on samples below the desiccated crust is shown on Fig. 3. The representative minimum value may be in the order of 300 lb/sq. ft., while the average value, based upon 39 tests is in the order of 450 lb/sq. ft. This average value may partially reflect the amount of sample disturbance during testing as previously mentioned. However, for preliminary planning purposes, the representative minimum value of 300 lb/sq. ft. should be used for design purposes until detailed investigations at the site of proposed specific buildings can be carried out to justify the use of higher strengths.

However, it is suggested that the use of a value of undrained shear strength greater than 300 lb/sq. ft. in analyses for preliminary planning purposes cannot be justified at this time until further detailed information across the complete site is available.

### iii) Preconsolidation Pressure

The variation in preconsolidation pressure with depth in the clay deposit at the site was measured in a series of 21 consolidation tests, (Fig. 7, Vol. 3). The results indicate that the amount by which the preconsolidation pressure ( $p_c$ ) exceeds the existing overburden pressure ( $p_o'$ ) in the unweathered clay deposit varies between 270 and 4,180 lb/sq. ft. and that an apparent minimum value occurs at about elevation 240. However, when combined with the results of other series of consolidation tests carried out on samples recovered from locations at or close

to the proposed site, a more detailed pattern is obtained, Figs. 4 and 5. (The information obtained at Hwy. 417 was obtained by the Ontario Ministry of Transportation & Communications as a part of their routine investigations for overpass structures along the highway. The information at C.F.S. Gloucester, which is located on Regional Road No. 8 about one mile to the south-west of the site, and at the intersection of Boundary Road and Highway 417, was obtained from research studies carried out by personnel from the Division of Building Research, National Research Council). With the exception of the upper desiccated crust the results are reasonably consistent and indicate that there are zones within the deposit which are only lightly overconsolidated. In the design of shallow foundations constructed in a desiccated crust overlying soft clay, the magnitude of  $p_c - p_o'$  in the zone immediately underlying the crust is critical. Taking an average crust thickness across the site of about 8 ft. the elevation of the critical stratum which lies just below the crust ranges between about 242 and 262. With the exception of one test result obtained during the present investigation (at elevation 248.8) and the data obtained from a test fill near C.F.S. Gloucester (Bozozuk and Leonards, 1972), the magnitude of  $p_c - p_o'$  in the critical zone is greater than 400 lb/sq. ft. The single exceptional test result obtained in the present investigation was measured in a sample taken from borehole 5, the closest borehole to the south-west corner of the site. This value of 270 lb/sq. ft. is in good agreement with the results of tests carried out on samples removed from the site of the Gloucester test fill. These results indicate that  $p_c - p_o'$  in the critical stratum underlying the crust in this area typically ranges between 0 and 400 lb/sq. ft. Therefore, although the available data is insufficient to allow a subdivision of the entire site into areas of similar values of  $p_c - p_o'$ , it is possible that this parameter is lower in the south-west corner of the site than in other areas.

A detailed summary of the difference between the present overburden pressure ( $p_o'$ ) and the preconsolidation pressure ( $p_c$ ) is shown on Fig. 6 for all consolidation tests which have been carried out in and adjacent to the site for depths of 60 ft. below the ground surface. With the exception of the south-west corner, the reasonable minimum value of overconsolidation  $p_c - p_o'$  is about 400 lb/sq. ft. while the average value for 49 tests is in the order of 750 lb/sq. ft. In order to minimize post-construction settlements for all structures, the increase in applied bearing stress must be below the value of  $p_c - p_o'$  at the appropriate elevation. Therefore, for preliminary planning purposes, the minimum representative value of 400 lb/sq. ft. has been used for conceptual design purposes until detailed investigations at the site of proposed buildings has been carried out to justify the use of higher values.

Analysis of existing house structures at the site, considering the probable loadings imposed, suggest that, for narrow strip footings at shallow depth, a value of overconsolidation  $p_c - p_o'$  equal to 500 lb/sq. ft. may be used in preliminary design. The higher value is consistent with the fact that the narrow footings used for the present structures on the site will have a relatively shallow depth of influence. However, where footings are greater in width, such that applied stresses will extend to greater depth, it is appropriate to design on the basis of the probable minimum value of  $p_c - p_o'$ , equal to 400 lb/sq. ft. (This design model for analysis of settlement is shown on Fig. 10).

#### iv) Failure Criteria for the Clay

Any applications of measured shear strength must consider failure criteria. This can most conveniently be examined in terms of effective stress. The effective stress parameters for the clay were determined in a series of tests on specimens trimmed from block samples taken at shallow

depth below the desiccated crust. The parameters describing the failure envelope were found to be about  $c' = 180$  lb/sq. ft.,  $\phi' = 26-1/2^\circ$ . However, the three test results which define the effective stress parameters were obtained from samples consolidated at stress levels considerably higher than the anticipated field stress. The remainder of the tests were carried out at stress levels approximately equal to those which would exist during field loading and the test results are summarized in Fig. 46, Vol. 3. During subsequent loading of samples consolidated to about the effective overburden pressure, there was a distinct change in the strain response to loading. It is at this point that cementation bonds between particles is broken down and the soil particle structures become unstable thus causing large strains. In some loading cases, the yielding of the cementation bonds is accompanied by rupture of the sample. The response of the clay to applied stress states lying within the cementation envelope is basically elastic with average values of the porewater pressure parameter  $A_f$ , of 0.33 and Young's Modulus,  $E$ , of 180 tons/sq. ft.

As the behaviour of the clay is largely governed by the cemented soil particle structure, it is not surprising that the results of unconsolidated undrained triaxial tests on specimens trimmed from block samples are in good agreement with the position of the cementation envelope. This agreement confirms the possibility discussed earlier, that the undrained shear strength, which is a measure of the strength of the soil structure, is likely to be greater than the minimum value of 300 lb/sq. ft. at shallow depths below the crust.

It is pointed out that when compared to the published results of other studies on unweathered cemented clays, the cementation envelope indicates that the clay at this site has a fairly low reserve resistance to loading. This finding further supports the design assumption for the

use of the minimum value of undrained strength in preliminary design.

#### PRELIMINARY PLANNING CONSIDERATIONS

One of the major geotechnical factors related to the development of the site is the thickness of desiccated crust or sand cap. Where this upper stratum is thick, the applied stresses carried by shallow footings are distributed over a wide area of underlying unweathered clay and hence the allowable bearing pressure of these footings is increased. On the basis of the results of the fieldwork carried out during the present and a previous site investigation of the area, Fig. 7 has been drawn to show the extent of areas having various ranges of thickness of desiccated crust or sand cap. As mentioned in earlier volumes of this series, the shallow borings were at approximately 2,000 ft. intervals around the perimeter of the site with slightly closer spacing in the south-east corner. The determination of the crust thickness was based upon textural and consistency classification of the soil and apparent changes in sampling performance at generally 2.5 ft. sampling intervals. Detailed soil testing above and below the crust has not been carried out. *Thus it should be noted that the information on which Fig. 7 was compiled is limited and hence the accuracy of the delineated areas is limited.* Further, there are two areas in which no information is available on the thickness of the surficial deposits from the fieldwork carried out in connection with the present investigation. However, it is considered likely that, as shown later in Fig. 14, the thickness of the upper stratum in these areas lies between 4 ft. and 6 ft.

A second important factor related to the preliminary planning of the development is the depth to bedrock across the site. The information on which the bedrock elevation contours which are shown in Fig. 8 have been based is limited. From this drawing, it is clear that there is a sector in the south-east corner of the site in which the depth to bedrock is less than

about 60 ft. This area, is flanked by zones where the depth to bedrock is less than about 85 ft. Combined with a second zone in the south-west sector of the site, the total area in which the depth to bedrock is less than about 85 ft. constitutes about 11 per cent of the total site area. Outside these areas, the depth to bedrock increases to a maximum depth of greater than 200 ft. in the north-west sector of the site.

Based on the soil conditions shown in Figs. 7 and 8 it is clear that there are geotechnical restraints as to the best location of various types of structures. For example, high rise structures which will, in general, be founded on piled foundations can best be located in areas where the depth to bedrock is less than about 80 ft., thus requiring that these structures are concentrated in the two areas described above.

Engineering analyses have been carried out to determine the allowable bearing pressures for various types of structures in the development. (The results of those analyses are described in detail in the following section). The general trend of results indicates that light industrial structures which transmit concentrated column loads to the foundation soil require a minimum crust thickness of about 10 ft. for an adequate foundation support. Consequently, it is clear that the majority of these structures should be located in the area bounded by Highway 417 along the north-east boundary of the site.

On the other hand, a large portion of the total site area is overlain by a crust thickness of between 4 ft. and 6 ft. The results of analyses indicate that the allowable bearing pressure for conventional shallow foundations in these areas is such that they are only suitable for single family residences and low rise multiple family housing units. The social facilities necessary in residential areas, such as



schools, shopping centres, community centres and the like should be carefully located to take advantage of areas where the crust thickness is greater than 6 ft.

The areas discussed above do not include land immediately adjacent to Bear Brook and its major tributaries. Such areas are discussed in a later section.

#### FOUNDATIONS OF STRUCTURE

This section discusses the foundation design for a number of different types of structures which are expected to be constructed during the course of the proposed development. It is emphasized at this point that for reasons discussed earlier, values of recommended bearing pressure should be used for preliminary planning purposes only. The two primary factors controlling the bearing capacity in different combinations of foundation type and soil stratigraphy are the undrained shear strength and magnitude of the overconsolidation,  $p_c - p_o'$ , in the soft clay underlying the desiccated clay crust or sand cap. A secondary factor is the undrained strength of the desiccated clay crust. The values assigned to the significant soil properties in determination of allowable bearing pressures are:

- undrained strength of desiccated clay crust:- 800 lb/sq. ft.
- undrained strength of clay immediately underlying crust:- 300 lb/sq. ft.
- $p_c - p_o'$  in the clay immediately underlying the crust:- 500 lb/sq. ft. (narrow ftgs.), 400 lb/sq. ft. (wide ftgs.)

The allowable bearing pressures have been calculated to incorporate a normal construction factor of safety of 3 against overall shear failure of the foundation soil and also to maintain stress increases at the top of the soft clay deposit below the crust below the limiting overconsolidation value of  $p_c - p_o'$ .

It should be pointed out that in the design of foundations in an area where soft clay underlies a more competent crust of varying thickness, the usual procedure of increasing the width of the footing to reduce the bearing pressure may not be beneficial. The effect of the stronger crust layer is to distribute the foundation loads over a wider area of soft clay. However, when footing widths are increased, the depth to which the soil is significantly stressed increases although the net bearing pressure is reduced. This concept is shown in Figs. 9 and 10 together with the parameters which control allowable bearing pressure.

There are a number of different foundation configurations which can be used depending on the type of structure under consideration. However, one important restriction is that external footings must be placed at sufficient depth below final grade to provide adequate protection against frost heave. In this regard, a preliminary survey, carried out in conjunction with personnel from the Ontario Housing Corporation, of selected structures presently existing at the site, showed that footings for single family residences are normally placed at a depth of about 3 ft. below original grade. The soil excavated for basements is then placed around the basement walls to provide the frost protection and also to permit drainage of surface water away from the buildings. The results of foundation analyses indicate that this procedure gives adequate allowable bearing pressures for the conditions at the site.

For structures which do not require basements, the conventional procedure might be to construct footings for these structures at a depth of about 5 ft. below original grade. However, the crust thickness is of the order of

4 ft. to 6 ft., this design results in applied stresses being transmitted directly to the soft clay which may lead to overstressing of this deposit. Alternative approaches for the site are to use some regrading, structural floors, or styrofoam sheets around the perimeter to provide adequate frost protection.

In the following sections the preliminary foundation design for the various types of structures to be included in the proposed development are discussed in terms of conventional foundation practice. Where necessary alternative foundation configurations are considered.

#### Single Family and Multi-Family Residences

In the proposed development, it is likely that a number of different types of single family residences will be constructed. Therefore the loads transmitted to the foundation soil will depend on the type of structure and method of construction. However, it is understood that the loadings of both one and two storey single family residences with basements will be of the order of 1.2 tons/lineal ft. of footing, with column loadings of up to 9 tons. It is considered that such structures can be successfully constructed in most areas of the site. This is evidenced by the fact that similar buildings exist at the site at present and do not apparently suffer distress due to excessive settlements.

For example, for footings placed at a depth of about 2 ft. to 3 ft. below original grade with excavated soil placed around the basement walls, the results of foundation analyses, based on the soil properties discussed above and for

a crust thickness of 4 ft. to 6 ft. indicate that maximum footing widths of between 2 ft. and 5 ft. are required, (Fig. 11).

It was previously mentioned that the existing single family residences at the site normally have footing widths between 2 and 3 ft. The greater widths suggested for areas where the crust is only 4 ft. thick (as shown on Fig. 11) are a consequence of the conservative preliminary design values, or due to the possibility that these buildings are located where the crust is probably in the range of about 6 ft. thick. The latter value would be consistent with desiccation developed from the seasonal fluctuations in the groundwater level.

In the case of semi-detached structures, loads of up to 1.5 tons/lineal foot of footing can be anticipated under party walls. Loading of this magnitude in areas of shallow crust thickness (4 ft. to 6 ft.) can require footing widths of 5 ft. to 6 ft. Where footings of 6 ft. in width are not practicable in design, such structures may be supported on mat foundations consisting of strip footings structurally connected to the floor slab. If placed at a depth of about 3 ft. below original grade, the resulting net pressure on the foundation soil will be lower than the preconsolidation pressure and consequently only minimal settlements can be expected.

Multiple family residential structures such as row townhouses, if restricted to two storeys with a basement, produce applied loads of about 1.4 tons/lineal ft. of footing

and are therefore similar in loadings to semi-detached structures discussed above.

Where it is required to construct three storey row townhouses with basements or apartment blocks of similar size in areas where the crust thickness is limited to between 4 ft. and 6 ft., the use of conventional strip footings and basement slabs on grade is not recommended. In this case, the net loading on the foundation soil should be reduced by the use of partially compensated foundations in which the basement walls and floor slab act as a structural unit. For three storey structures with basements it is recommended that the underside of such a foundation unit be constructed at a depth of about 5 ft. below original ground level resulting in a net applied pressure on the foundation soil of about 300 lb/sq. ft. The extra construction costs for a continuous structural foundation may not be economical. However, three storey row townhouses can be built with conventional strip footings about 3 ft. wide in areas where the crust thickness is greater than 10 ft.

In the design of row townhouses it is common to construct footings at different elevations for aesthetic reasons. It is pointed out that in the present development, this practice should be discouraged as reduction in the thickness of crust below the footings will increase the likelihood of overstressing of the foundation soil in areas of thin crust. As an alternative, it is suggested that similar aesthetic effect may be obtained by offsetting groups of houses in plan which would not affect the allowable bearing pressure in any given area.

The effect of creating landscaping features which comprise of 4 to 5 ft. of earth fill imposes a loading on the normally consolidated underlying clay which is equivalent to or heavier than the loadings developed from low buildings. As a

result, surface settlements could occur under the earth fill. Therefore it is recommended that earthfill landscaping should not be carried out adjacent to proposed housing and engineered structures.

#### Light Industrial, Commercial and Institutional Structures

The type of building being considered in this section is a single storey light industrial or commercial structure, schools and community centres with loadings on external strip footings of about 1.5 tons/lineal ft. 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 ft. In commercial structures it is anticipated that for the normal type of construction, column loads will be of the order of 25 tons for columns spaced at 30 ft. 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 these loadings will be of the same order of magnitude as in the cases of light industrial or commercial structures, which are discussed below.

In normal use, these structures do not require a basement and as a result the floor slab is usually constructed at about the original ground surface elevation. Consequently, only minor regrading around the walls of the structure will be necessary and as a result conventional strip footings will be at a depth of at least 4 ft. to 5 ft. below original grade. The results of analyses summarized in Fig. 12 indicate that such footings are only practicable where the crust thickness

is greater than 8 ft. to 10 ft. The footing width decreases from about 7 ft. to 2 ft. as the crust thickness increases in the range indicated.

Fig. 13, which shows the allowable bearing loads for spread footings on crusts of varying thickness, indicates that loads of about 25 tons and 33 tons can be carried by 7 ft. square spread footings in areas where the crust thickness is 8 ft. and 10 ft. respectively. These allowable bearing loads are of the same order of magnitude as the expected building loads and emphasize the necessity for careful site investigation and foundation design for individual structures.

In view of the constraints related to crust thickness it will be desirable to locate the majority of light industrial and commercial structures in the area adjacent to Highway 417 where the thickness of the sand cap or desiccated crust is a maximum. However, it may be necessary from a planning standpoint to locate light commercial structures in residential areas where the crust thickness is likely to be less than 8 ft. Where this is the case, it will be necessary to preload the site using imported granular material to loads in excess of those resulting from final structural loadings. This approach may be economically feasible as large quantities of imported granular material will be required for construction of the subbase in the parking areas associated with this type of structure. The application of a preload at a given site causes consolidation and subsequent settlement of the underlying soil. Consequently, to prevent damage to nearby

structures, it is recommended that buildings should not be constructed within 250 ft. of the preload area until after the preload has been removed. Buildings and other facilities within the 250 ft. zone can be constructed when the pore pressures from the preload have dissipated. Normal construction monitoring techniques with piezometers can be used to determine the time period at individual locations.

It is noted that the use of preload techniques have proven successful in minimizing settlements under an approach embankment for bridge structures located close to the site of the proposed development, (Devata and Darch, 1972). Embankments were initially constructed to a height of about 14 ft. and subsequently reduced to the design height of about 9 ft. after about 1 year. During this period the settlement of the compressible clay under the centre of the embankment was of the order of 8 in. In the period of 2 years after the surcharge was removed the settlements under the centre of the embankment were reduced to 2 in.

#### Heavy Institutional Buildings and High Rise Structures

Institutional buildings and high rise apartment structures which are greater than 3 storeys high cannot be supported on spread footings over the major part of the site. Consequently, as suggested earlier in this report, these structures should be located in areas where the depth to bedrock or competent till allows for the economical use of piled foundations. In order to minimize the effect of construction on adjacent structures, it is recommended that non-displacement piles are used such as driven 'H' section piles or bored piles. Further, no structures founded on spread footings should be built within a distance of 250 ft. from the planned location of piled structures, until after the piling work is completed. The allowable loads on the piles



should be established by site investigation at the selected site. However, for preliminary planning purposes the allowable loads for end bearing 'H' piles driven into the shale bedrock can be taken as about 50 tons.

The use of friction piles for high rise structures is not generally recommended, since such friction pile groups, used to transfer building loads to a given elevation within the deposit, could cause overstressing of underlying softer zones, thereby resulting in excessive settlements. A detailed engineering analysis of pile types and effects should be carried out when planning design is complete.

As an alternative to piled foundations in areas where the depth to bedrock is excessive, it is possible to use compensated foundations. This type of foundation has been used successfully in countries where similar soil conditions exist. (Bjerrum, 1967). In this case the applied loads on the foundation soil are reduced either to zero or to values less than  $p_c - p_o$  by excavation and removal of soil to a predetermined depth. In calculating the required depth of basement excavation seasonal variations in groundwater level are taken into account. It is possible that some high rise structures will be designed in which the number of storeys is variable. Where this is the case, differential settlements between the various parts of the structure can be minimized by constructing basements at varying elevations.

In some areas of the site, the depth of large basement excavations may be limited by the possibility of bottom heave. (This is discussed further in later sections of the text). Where site investigations at the site concerned indicate that this is possible, the excavations should be made in narrow sections.

Summary

From the above discussion, it is clear that it is possible to construct the buildings required in the proposed development, provided that appropriate use is made of the land available.

A summary of the foundation types which will be required for the various types of structures is indicated in Table 1. As an example of the use of this Table, structure classification A-3 includes three storey row houses or similar height apartment buildings. These units are typically constructed with basements and founded on strip footings. Such building units are not suitable in areas where the crust is between 4 and 6 ft. thick. These buildings may be constructed with mat foundations in areas where the crust is between 6 and 8 ft. thick. Conventional footings for these units may be used in areas where the crust is greater than 10 ft. thick.

In addition to the categories of A-1 to C which are used in Table 1, an additional building foundation category, Type 'D', has been used to indicate structures which would require piled foundations of economical depth. A preliminary scheme for the use of land for all of the various types of building foundations has been indicated on Fig. 14. The sector designated as 'D' is suitable for piled foundations or lighter structures (whose foundation type will be governed by the crust thickness). The sector designated as structure type A-2 is suitable for semi-detached houses, but these could also be built in sectors A-3, and B. The use of this plan must be done in conjunction with the text of this report.

TABLE I - SUMMARY OF FOUNDATIONS IN RELATION TO CRUST THICKNESS

Type of Structure	Conventional Foundation Type	Crust Thickness, $t_c$ .				Notes
		4' - 6'	6' - 8'	8' - 12'	>12'	
A-1 Single family houses (Fig.11)	Strip ftgs. (with basement)	✓	✓	✓	✓	
A-2 Semi-detached or row houses (1-2 storey only) (Fig.11)	Strip ftgs. (with basement)	? (1)	✓	✓	✓	1. Mat fdns. incorporated in basement floor possible where $t_c \approx 6'$ .
A-3 Three storey row houses or apt. bldgs.	Strip ftgs. (with basement)	No	? (2)	✓ (2)	✓	2. Best located where $t_c > 10'$ otherwise mat fdns. as in (1) above may be used where $t_c \approx 6'$ .
B Light industrial, commercial and institutional struc- tures. (All one to one and half storeys in height) (Figs. 12,13)	Strip ftgs. for external walls, sp. ftgs. for iso- lated column loads (no basement)	No	No (3)	? (4)	✓	3. Pre-load possible where crust >6' but less than 12'. 4. Conventional fdns. only possible where $t_c > 10'$ .
C Heavy institutional bldgs., high rise structures.	Isolated column loads on sp. ftgs. or piles.	No ← —	No (5) —	No — →	? (6)	5. Piled fdns. necessary where $t_c < 12'$ . 6. Allowable fdn. pressures generally $\geq 2$ ton/sq.ft.

General Note: While detailed site investigations are required for all types of structure, allowable fdn. pressures can only be defined for structure types, B & C on the basis of the results of specific studies at actual sites.

As previously mentioned, the peripheral area around piled foundations and preloaded areas should not be utilized for structures until the settlement conditions have stabilized. This construction staging can be incorporated into the planning process, or alternatively, the areas can be used for parking lots and recreational facilities.

#### 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 municipal roads and services both of which involve earth excavation, and the control of the site drainage for storm water and snow melt. Because of the geotechnical factors over the site, these facets of site development must be carefully integrated into a staged construction sequence.

The three major soil divisions within the site consist of the sand cap, the desiccated silty clay crust, and the soft sensitive clay. The sand cap is suitable for earth excavation and general backfill, as is that portion of the desiccated crust where the in situ water contents are close to the plastic limit. Due to the high in situ water content of the sensitive clay, the soil is difficult to handle. If the reworked sensitive clay is used for backfill, settlements will occur under the self-weight of the material. For excavations where the sensitive clay must be removed, the site development should consider a carefully staged construction process, or the use of imported suitable material, or the location of the service trench excavations into areas where settlements can be tolerated.

We understand that the headwaters and tributaries of Bear Brook will be the principal surface drainage control system for the site. In order to control erosion rates in Bear Brook, the rate of run-off for the site should be controlled as much as possible. This may be done by a variety of methods including slope protection and erosion control, and the control of the rate of surface water accumulation. Details of this work will depend upon the eventual design plan. These control measures have the added advantage that they will maintain the present groundwater levels at the site which will have a beneficial effect with respect to vegetation. From these geotechnical considerations, the site drainage must be considered as an integrated system and should be developed in stages similar to the installation of services and major structures.

The particular aspects of the individual factors in site development are discussed in the following sections.

#### Earthworks and Roads

Some regrading will be carried out during the course of development, but because of the flat nature of the site, it is expected that regrading and earthworks will be primarily to a shallow depth within the desiccated crust or sand cap. Deep excavations associated with cuts or deep basement foundations, or earthworks associated with large fills should be minor and because of the soil conditions at the site, the extent of such earthworks should be kept to a minimum.

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 moisture 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 because of its high natural moisture content and sensitive nature. The material can only be used where surface settlements at the trench are not a critical governing criteria.

Because of the lack of quality granular material on the site, emphasis should be placed on keeping the extent of earthworks to a minimum. This will have a significant influence on the planning of locations of buildings and in the design of road and street systems.

For a large part of the year the groundwater level across the site is relatively high. Further, it is likely that there will be areas where the upper 5 ft. of subgrade material is frost susceptible. Because of these factors it is recommended that roads are designed such that cuts are avoided wherever possible. It is also necessary to provide good under-drainage of the road pavement. It is considered that, due to a combination of high water table and unfavourable soil conditions, in terms of surface trafficability, the subbase and base course for the roads should be constructed before development proceeds in a given area, thus allowing easier access of construction traffic. However, when construction has been completed, it may be necessary to examine the suitability of the existing subbase and base course before preparing the final

design specifications.\* For the subgrade conditions existing at the site and based on current local practice, it is suggested that the preliminary design of flexible pavements be based on the criteria of equivalent pavement thicknesses of 30 in. and 24 in. of granular base and subbase for arterial and subdivision roads respectively. In the case of rigid pavements, it is suggested that the thickness of granular material required be 9 in. and 6 in. to 8 in. for arterial and subdivision roads respectively. However, the pavement design will be significantly influenced by the location of service trenches.

In Ontario it is normal practice to construct service trenches under roadways and to backfill the trenches with a well compacted suitable material. With the exception of limited quantities of surficial sands and desiccated crust, there is little or no suitable native material available for this purpose. Hence, if this practice is to be adopted in the present development considerable quantities of imported backfill material will be required. It is possible, however, that during the course of the development this may become economically unviable due to depletion of local resources of this material and as such it may be necessary to consider alternative material for backfill.

\* A suitable method for evaluating subbase and base course conditions, in situ, is the use of the Benkelman Beam. This method of pavement evaluation is recommended when portions of the pavement structures are used for construction purposes prior to the completion of the total design pavement thickness.

One alternative is to use selected desiccated clay crust or surficial sands as backfill in service trenches under roads. The properties of these soils are variable such that the degree of compaction, required to provide a suitable subgrade, may be difficult in some areas. Consequently, if this material is used it is considered necessary to install the services at least 2 years before road construction begins. Problems of differential frost heave between sections of road pavement which overlie service trenches may be minimized by backfilling the service trench with compacted local material to the depth of frost penetration below finished road grade and with suitably compacted quality granular material from this depth to the underside of the pavement structure. Another alternative to the use of select imported material for the backfill of service trenches would be to locate the municipal service trenches away from roads and other structures. At these locations, settlement of the trench backfill can be tolerated so that the local material could be used. It will be necessary to provide for additional fill to compensate for the settlements which will occur.

Regardless of the type of pavement design adopted for the proposed development it is possible to improve the trafficability of the subgrade soil to construction traffic by the use of lime. Recent practice in the Ottawa area has shown that mixing of approximately 5% of lime (by dry weight) to a depth of about 12 in. produces significant reduction in the thickness of overall granular material required in the pavement.



Excavations, Services

The stability of excavations at the site, whether in the construction of deep basements or services is significantly influenced by the strength of the unweathered clay underlying the desiccated crust. The stability of the test pits and test trench excavated at various locations at the site during the course of fieldwork for the present investigation have been analysed and the results included in Fig. 15. From these results it is clear that, based on the design undrained shear strength of 300 lb/sq. ft. in the soft clay underlying the crust, the maximum depth of excavation for service trenches is about 15 ft. to 20 ft. Because of three dimensional effects, the maximum depth of square excavations is higher. The excavations put down for the test samples and the test trench were unsheeted and in some cases had sloping sides (See Vol. 3). As such, these excavations were wider than might be desirable for the installation of services. For narrow excavations carried out in accordance with the Construction Safety Act, bracing and/or sheeting would be required during construction.

It is pointed out that excavations made under winter conditions will probably be advantageous from a stability standpoint and also in terms of good surface trafficability. Trenches excavated during the 'summer' construction period may tend to be less stable and the ease of movement of construction vehicles more difficult. Because of the requirements for reasonable surface trafficability and the possibility that adequate supplies of suitable granular may not be available over the total period of development, it is recommended that excavation of service trenches are kept to a minimum. To this end, it is recommended that where possible

all services should be placed in a common service trench. Further, care should be taken in the design stage to maximize the overall existing grades, thereby reducing the required depth of excavation.

The amount of seepage of groundwater during the construction of service trenches will vary widely depending on soil type. Thus, where service trenches are being installed in areas where there is little sand cover, it is considered that only small quantities of seepage would be expected and that these can easily be handled by pumping from sumps. However, where the sand cap is relatively thick, such as the area bordering Highway 417 on the north-east boundary of the site, it may be necessary to provide a dewatering system in advance of actual construction. Fig. 16 shows the grain size envelope of the surficial sands in the north-west sector of the site and indicates that in most areas, a wellpoint dewatering system can be used.

Although the majority of the services within the site can be installed using normal methods of trench excavation, it will also be necessary to install deep services, such as sewage disposal links to the main Ottawa-Carleton sewer system. These cannot be constructed in open trenches and will require tunnelling methods. This has been common practice in the Ottawa area for some time and at a depth within the clay, is usually carried out under compressed air.

Groundwater, Site Drainage

Previous work at the site and that by K. H. King and Associates Limited (Supplementary Report 209-5-15, dated January, 1973) gives data concerning piezometric conditions. This data is, to a degree, random but suggests that there may be some downward seepage. Piezometric observations during the course of the present investigations indicate that little or no downward seepage is taking place.

The combination of a high groundwater level and the generally flat nature of the terrain at the site cause wide-spread ponding of surface water during periods of prolonged precipitation. Thus it is important in the development of the site to ensure that sufficient regrading of individual housing developments is carried out to prevent surface water ponding around the buildings. Where basements are constructed at depths below the groundwater level it will be necessary to waterproof these structures and to provide sump pumps.

In terms of overall site drainage it is not considered practicable to attempt to lower the groundwater table over the site prior to development. However, it is anticipated that localized depression of groundwater levels will occur as the result of development. It is considered that local depressions in groundwater level of this nature will not induce significant regional settlements, provided that the factors which cause groundwater lowering are controlled. To this end it is suggested that:

- i) surface water run-off is channelled into shallow swales by means of a carefully designed regrading of the area instead of underground storm sewers.

A system of this nature would aid in the replenishment of the groundwater. For example, it would be advantageous to provide ditches along the sides of roadways instead of the 'curb and gutter' method of drainage.

- ii) Flow of groundwater through granular backfill placed in service trenches should be controlled by constructing sections of compacted clay backfill (rather than granular material) at periodic intervals along the length of service pipes and by careful control of the construction of pipe joints through which seepage may also occur.
- iii) The control of groundwater is also, to some extent, affected by vegetation. However, the planting of fast growing trees can cause structural distress in nearby buildings and road pavements due to depletion of water within the clay thus causing consolidation. Therefore, it is recommended that trees with a high water demand are not planted within a distance from buildings or pavements of about the height to which the tree is expected to grow, (Burn, 1973).

#### Stability of Side Slopes of Bear Brook

There is no evidence that major flow slides have occurred in the past at the site. Further it is considered that provided reasonable precautions are taken, as discussed below, there is no reason to assume that flow slides will occur during the course of the proposed development.

From a number of documented case histories of flow slides in the Champlain Sea clay which underlies a significant portion of the Ottawa area, it is apparent that the large majority of flow slides are triggered initially by rotational slope failures in the river banks. These initial rotational failures, which may involve only relatively small volumes of soil, are the result of the continuing process of erosion and undercutting of the river banks. If the discharge of run-off from the site into Bear Brook and its associated tributaries is uncontrolled, then the erosional capacity of the stream would be substantially increased and the possibility of the occurrence of failures of the side slopes could exist.

A semi-empirical method of assessing flow slide potential at a given site has been proposed by Mitchell and Markell, 1974. In this method, the total overburden stress,  $h$ , is compared to a stability factor,  $6 s_u$  with increasing depth from the crest of a river bank. Where the ratio of  $6 s_u / h$ , which is in effect a measure of the factor of safety with regard to flow slides, falls below unity, the possibility of flow slides exists.

Of particular interest in the present development is the data available from this study related to the area on the north-east perimeter of the site, where Highway 417 crosses Bear Brook. This data, combined with information obtained during the present study has been plotted in Fig. 17 and indicates that at present there are areas where the side slopes of Bear Brook are only marginally stable. Therefore,

as part of the proposed development it will be essential to control the amount of water discharged to Bear Brook in a given period. It is further recommended that protection against erosion of the toe of the slopes and subsequent undercutting of the river banks is provided in critical areas. It may also be necessary to flatten the banks in these areas.

On the basis of the above discussion, it is recommended that for preliminary planning purposes, no structures should be located within 250 ft. of the river bank where the height of the bank is in excess of 15 ft. In areas where the bank height is less than 15 ft., it is considered that the distance from the crest of the river bank to the nearest building should be at least 150 ft. In no case should fill be placed within the limits described above. At the time of the geotechnical investigation, detailed contour mapping in the vicinity of Bear Brook was not available. This information is necessary in order to carry out detailed stability studies of the present slopes and any ancillary calculations related to construction of stilling basins and other erosion control features. The recommendations given above should be reviewed when the contour information is available.

When the discharge characteristics of Bear Brook have been established in relation to the planning design, the flow pattern downstream of the proposed site should also be considered. As mentioned above, the detailed contour mapping of the site and its vicinity is not available at the present time. When this information is available, the slope stability of the banks of Bear Brook downstream of the site should be studied in order to determine if hazardous areas exist.

RECOMMENDATIONS FOR DETAILED STUDIES

Prior to final planning and design it is recommended that:-

- i) The piezometric pressures across the site should be monitored to determine the seasonal fluctuations in piezometric conditions. It is considered that the piezometers and standpipes installed during the course of the present investigation are sufficient for this purpose.
- ii) The effect of increased run-off which will be discharged into Bear Brook and associated tributaries at the site should be studied. This work should include a detailed appraisal of the stability of the side slopes of Bear Brook, both within and downstream of the site, in order to define the critical areas in which protective works may be required to maintain or improve the stability of the side slopes.

SUMMARY

The results of the geotechnical investigation indicate that it is possible to develop the 7,000 acre site for the proposed South-East Ottawa community. Due to subsurface conditions, heavy buildings should be founded on end-bearing piles within specific locations on the site, while light commercial, industrial and institutional buildings should be located in areas adjacent to Highway 417. Residential buildings may be constructed on the remaining portions of the site provided that careful attention is given to the pertinent geotechnical factors involved.

June, 1974

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A staged sequence of construction should be developed for both buildings and associated site development work. The final staged development plan should be reviewed by a geotechnical engineer for details of construction methods.

H. Q. GOLDER & ASSOCIATES LTD.

D. L. Townsend, P.Eng.

JHAC/DLT/jb  
73908-1

J. H. A. Crooks

June 14, 1974



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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*

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

#### (b) *Cohesive Soils*

<i>Consistency</i>	<i>c<sub>u</sub>, lb./sq. ft.</i>
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

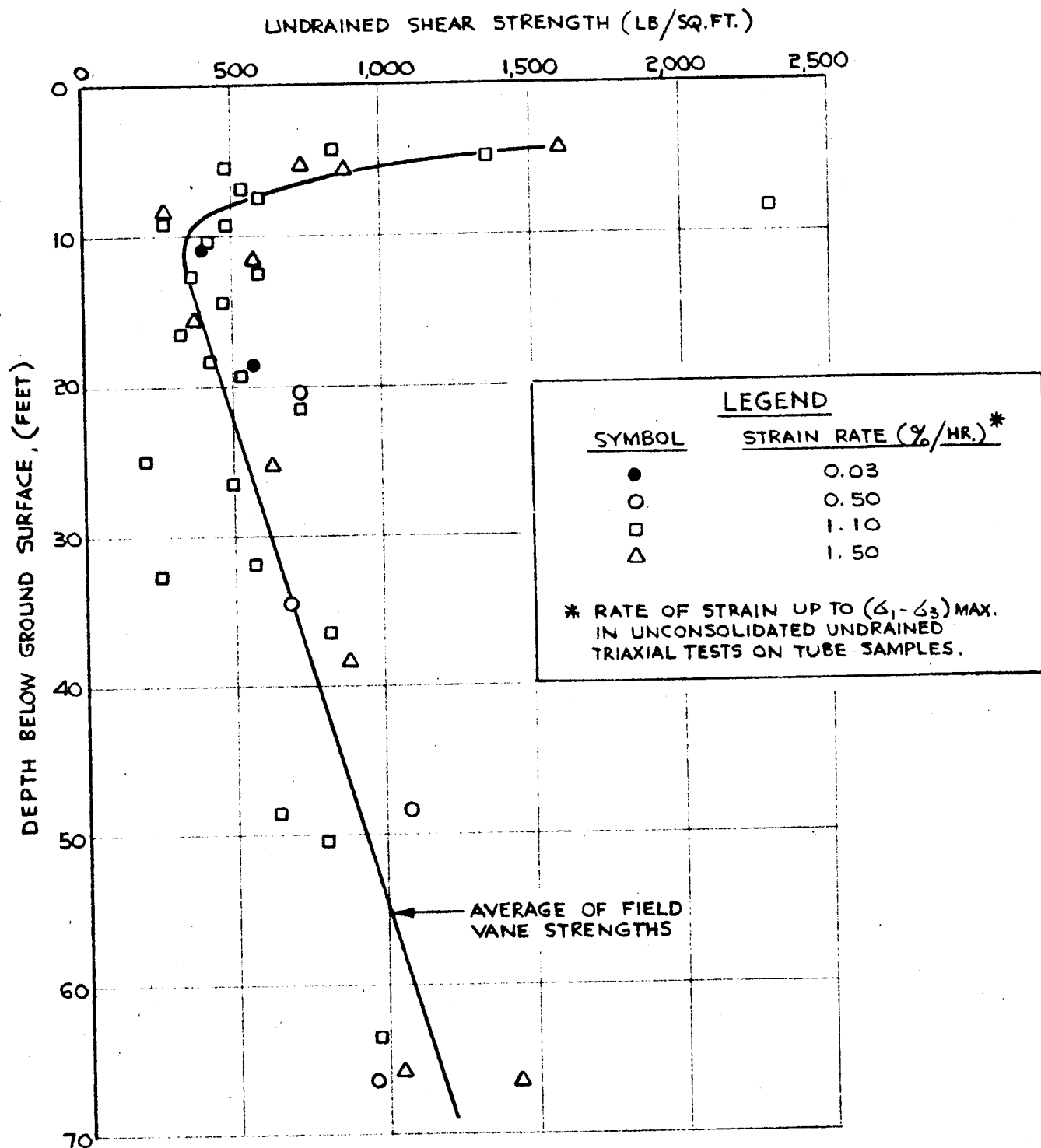
$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

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

\*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.

# SUMMARY OF UNDRAINED SHEAR STRENGTH TEST RESULTS WITH DEPTH BELOW GROUND SURFACE

FIGURE 1



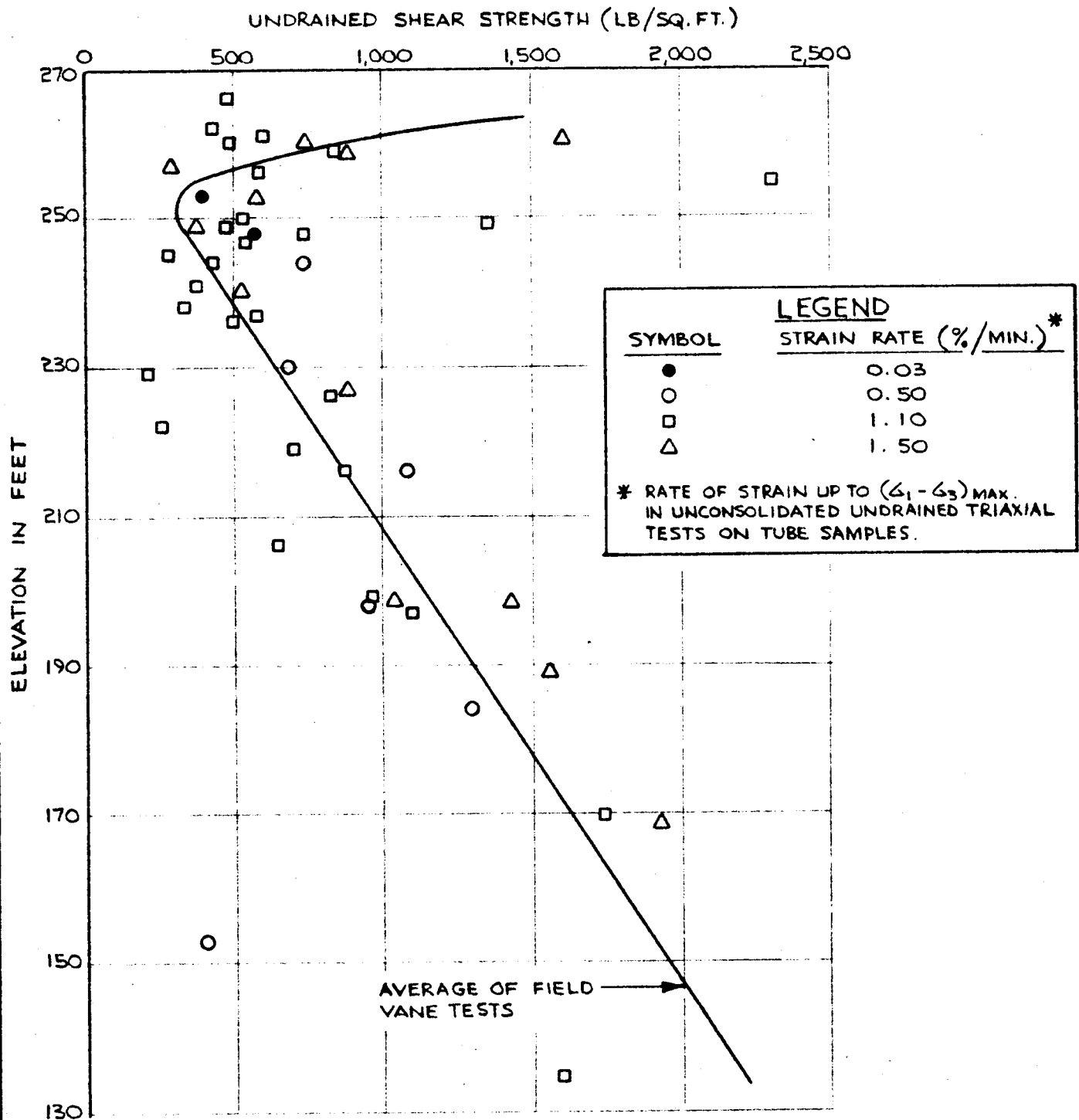
Date MAY 29, 1974

Golder Associates

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

# SUMMARY OF UNDRAINED SHEAR STRENGTH TEST RESULTS WITH ELEVATION

FIGURE 2



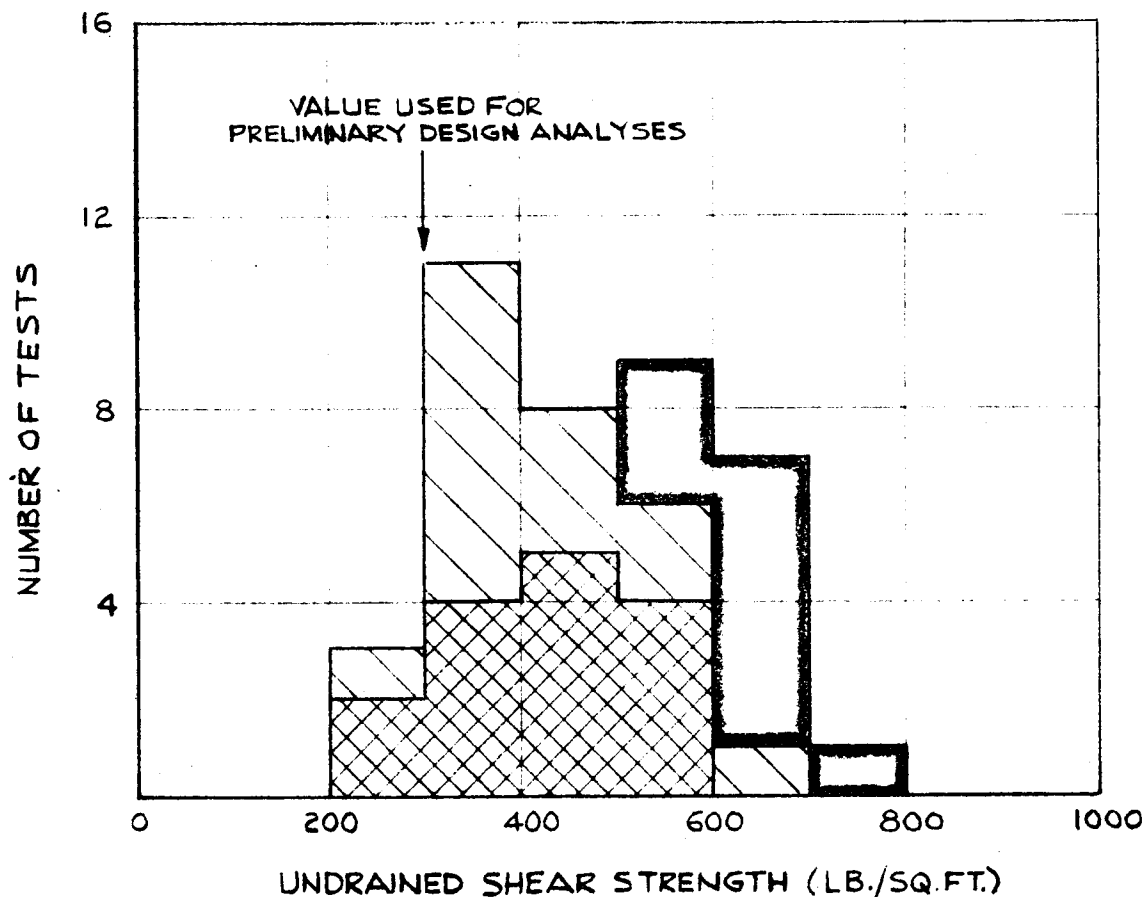
Date MAY 29, 1974

Golder Associates

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

# DISTRIBUTION OF UNDRAINED STRENGTH IN UPPER 10 FT. OF UNWEATHERED CLAY

FIGURE 3



## LEGEND



UNCONSOLIDATED UNDRAINED TEST ON 'TUBE' SAMPLES.



FIELD VANE TESTS.



UNCONSOLIDATED UNDRAINED TEST ON 'BLOCK' SAMPLES.

## NOTE:

DATA TAKEN FROM GOLDER ASSOCIATES REPORT No. 73908, VOLUME 3,  
DATED MAY, 1974.

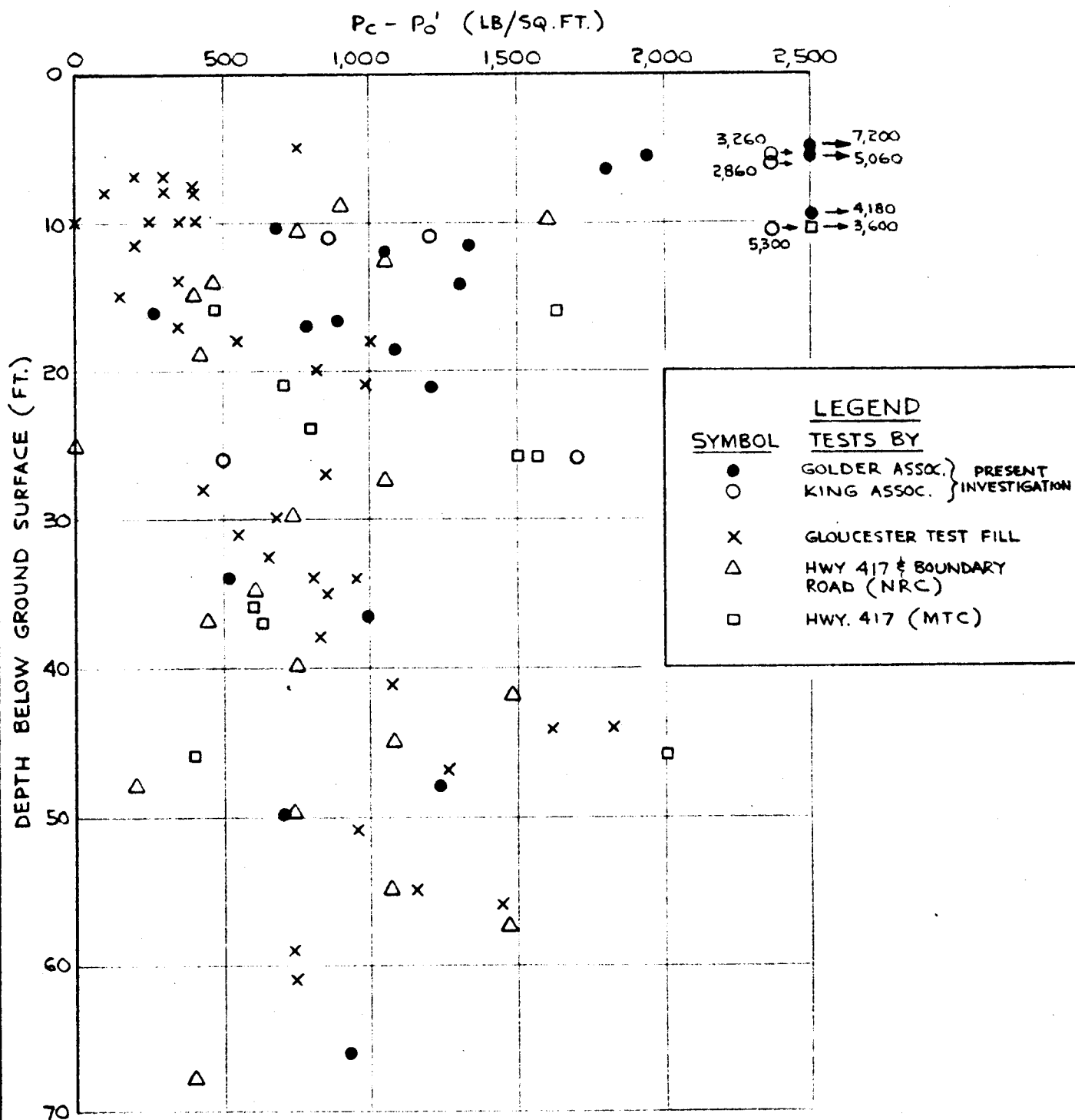
Date MAY 30, 1974.

**Golder Associates**

Drawn *M.Y.B.*  
Chkd *J.H.K.*  
Appd *[Signature]*

# $P_c - P_o'$ VS DEPTH BELOW GROUND SURFACE

FIGURE 4



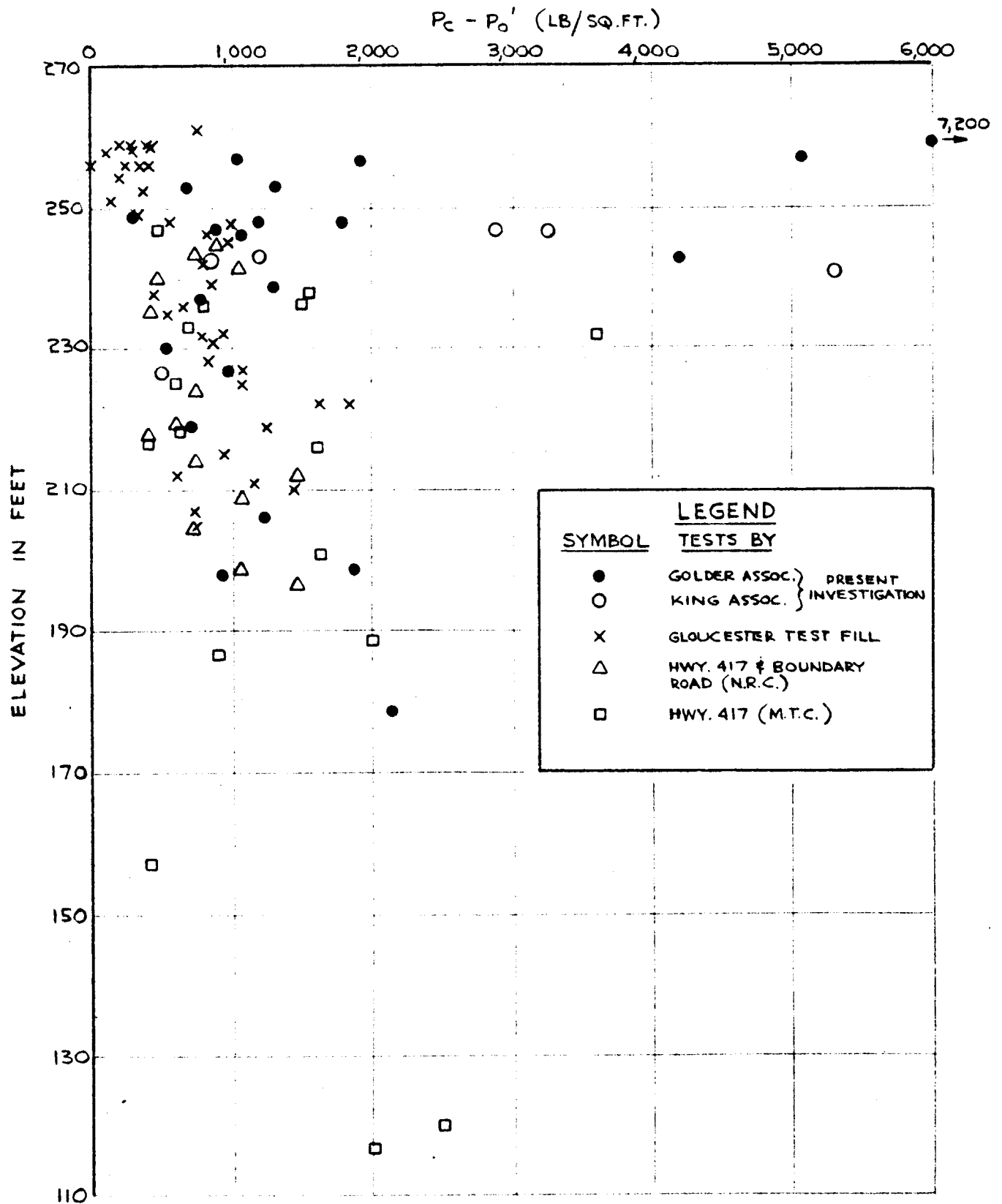
Date MAY 29, 1974

Golder Associates

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

# $P_c - P_o'$ VS ELEVATION

FIGURE 5



Date MAY 29, 1974

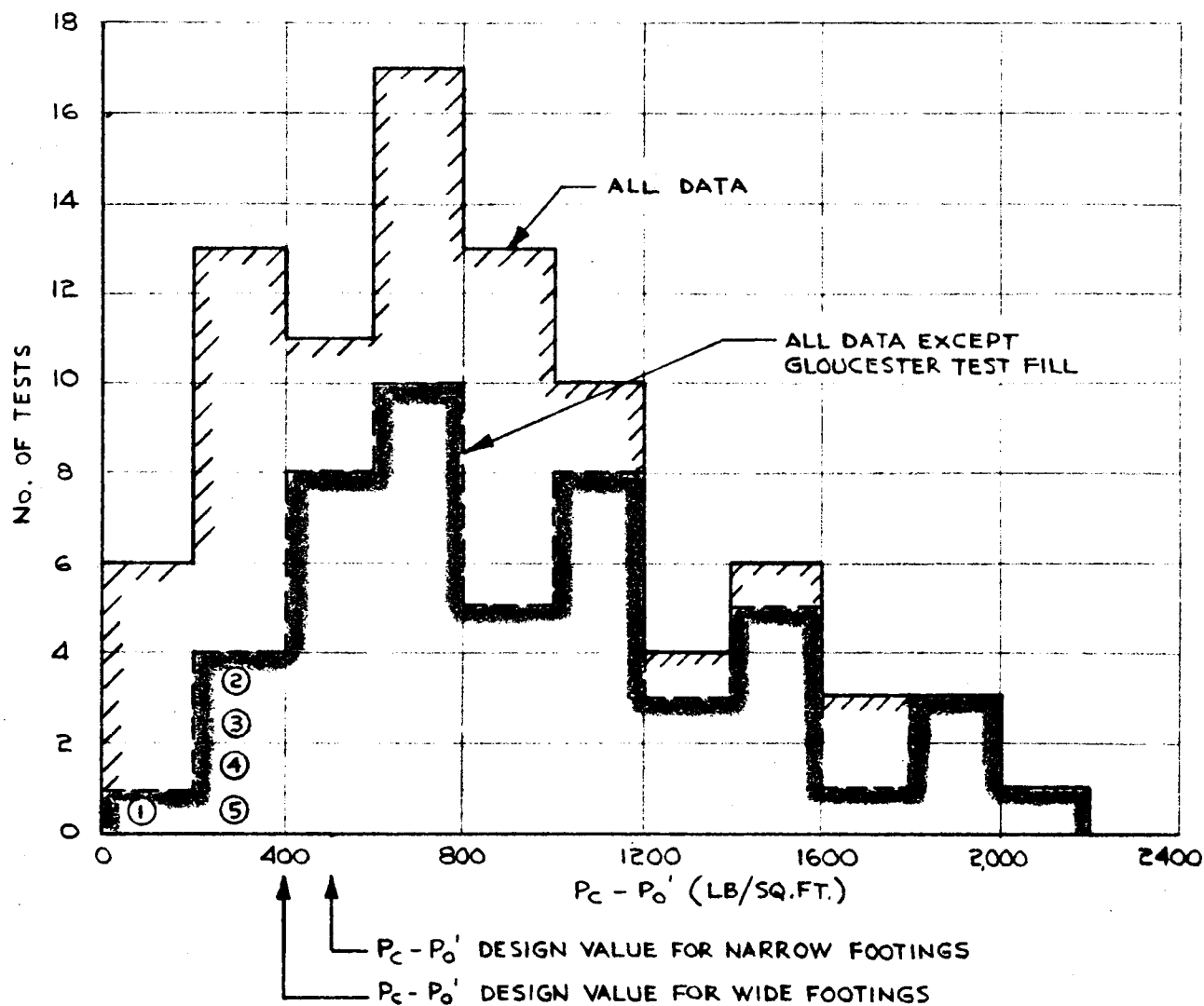
Golder Associates

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Chkd J.A.  
Appd J.A.



# DISTRIBUTION OF $P_c - P_o'$ WITHIN UPPER 60 FEET OF UNWEATHERED CLAY

FIGURE 6



## LEGEND

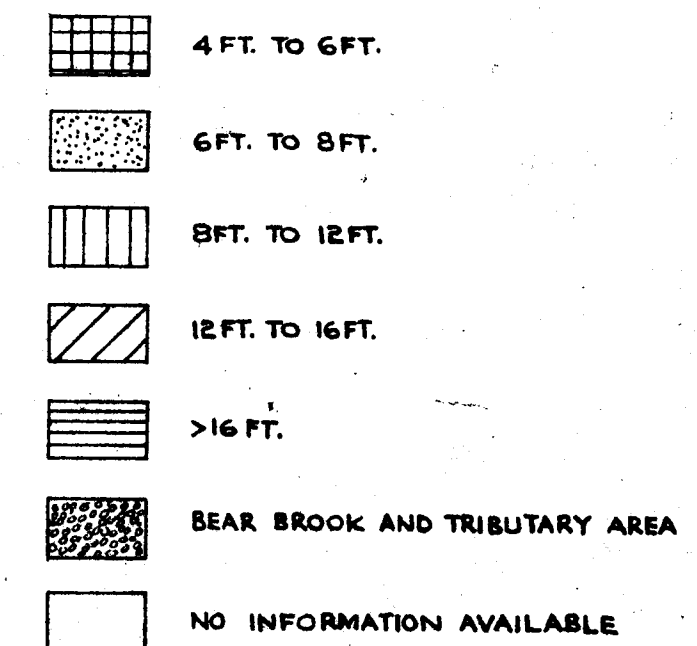
- ① BOUNDARY ROAD/HWY. 417 (N.R.C.) — 25 FT. DEPTH
- ② BH 5, GOLDER ASSOC., PRESENT INVESTIGATION — 16 FT. DEPTH
- ③ BOUNDARY ROAD/HWY. 417 (N.R.C.) — 15 FT. DEPTH
- ④ BOUNDARY ROAD/HWY. 417 (N.R.C.) — 48 FT. DEPTH
- ⑤ ANDERSON ROAD/HWY. 417 (M.T.C.) — 46 FT. DEPTH

Date MAY 30, 1974

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Chkd. J.A.  
App'd. J.A.

## LEGEND



## NOTES

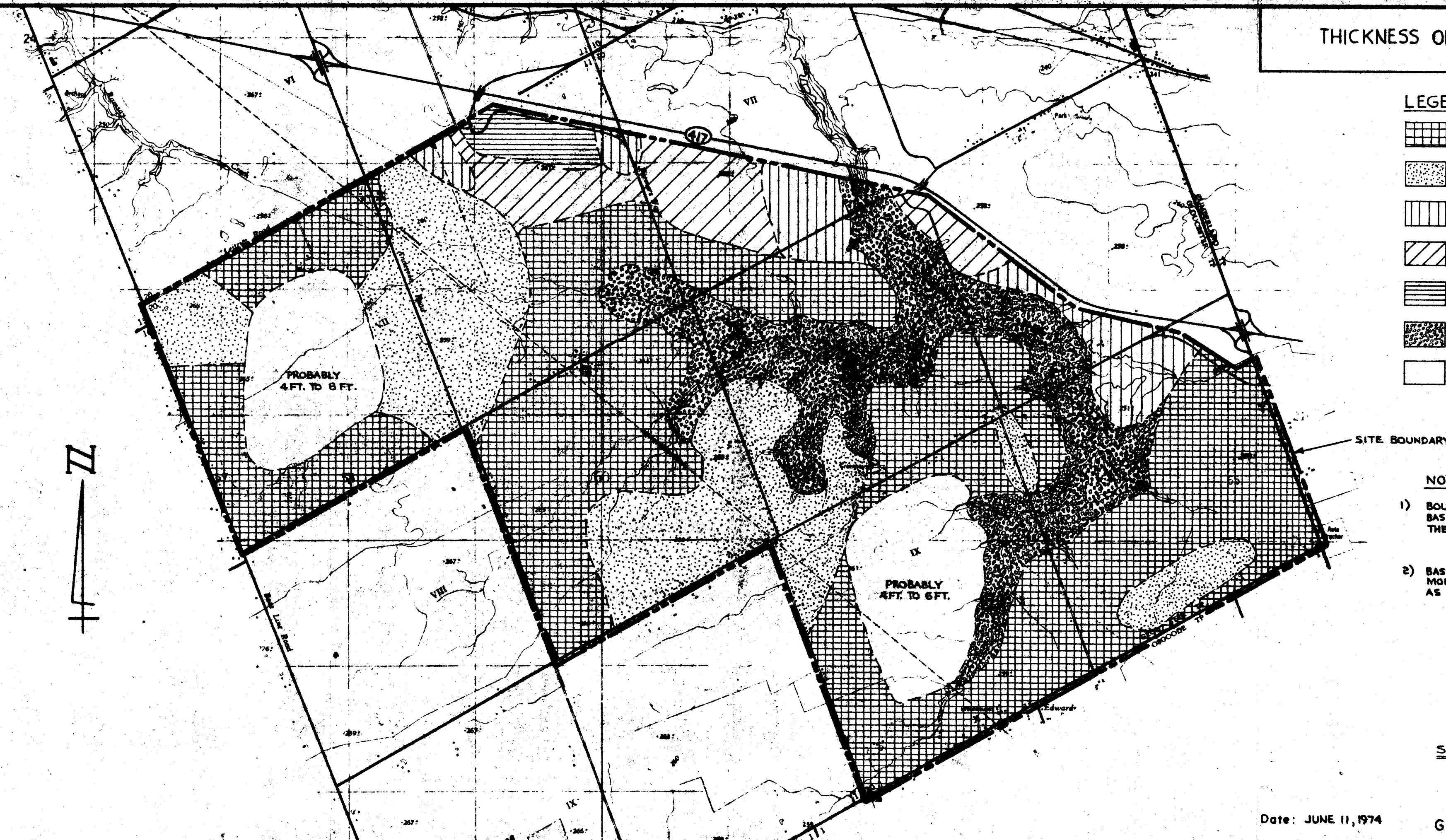
- 1) BOUNDARIES BETWEEN CRUST THICKNESS REGIONS BASED ON WIDELY SPACED LOCATIONS AND SHOULD THEREFORE BE REGARDED AS APPROXIMATE ONLY.
- 2) BASED ON K.H.KING DRAWING No. 312-S.2-3 MODIFIED TO SHOW CRUST THICKNESS GROUPINGS AS DISCUSSED IN THIS REPORT.

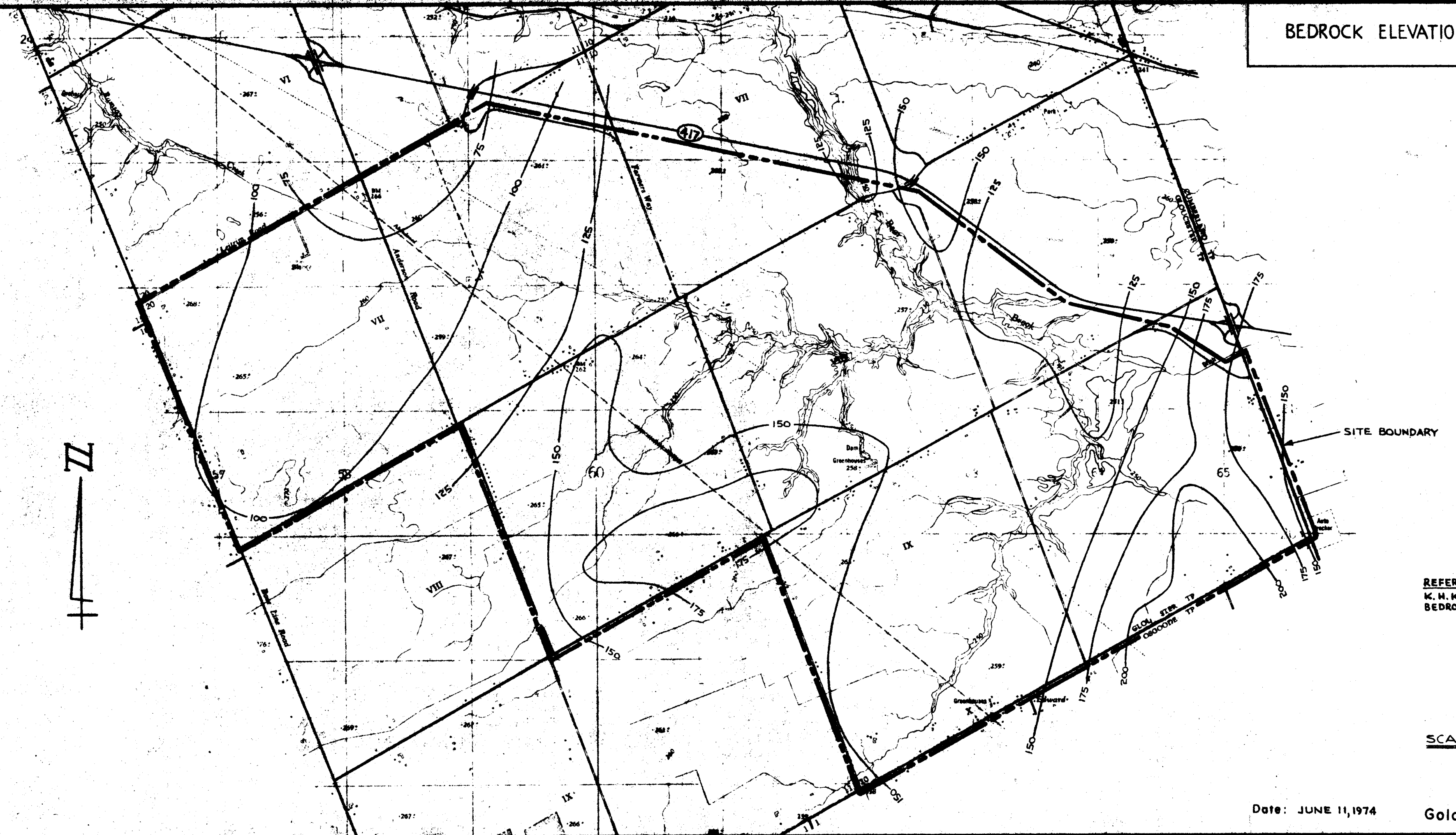
SCALE: 1:25,000

Date: JUNE 11, 1974

Golder Associates

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 Chkd. JHC  
 Appd. J.A.





REFERENCE  
K. H. KING ASSOCIATES LIMITED DRAWING No. 312-S.2-4,  
BEDROCK CONTOURS, DATED MAY, 1974.

SCALE: 1:25,000

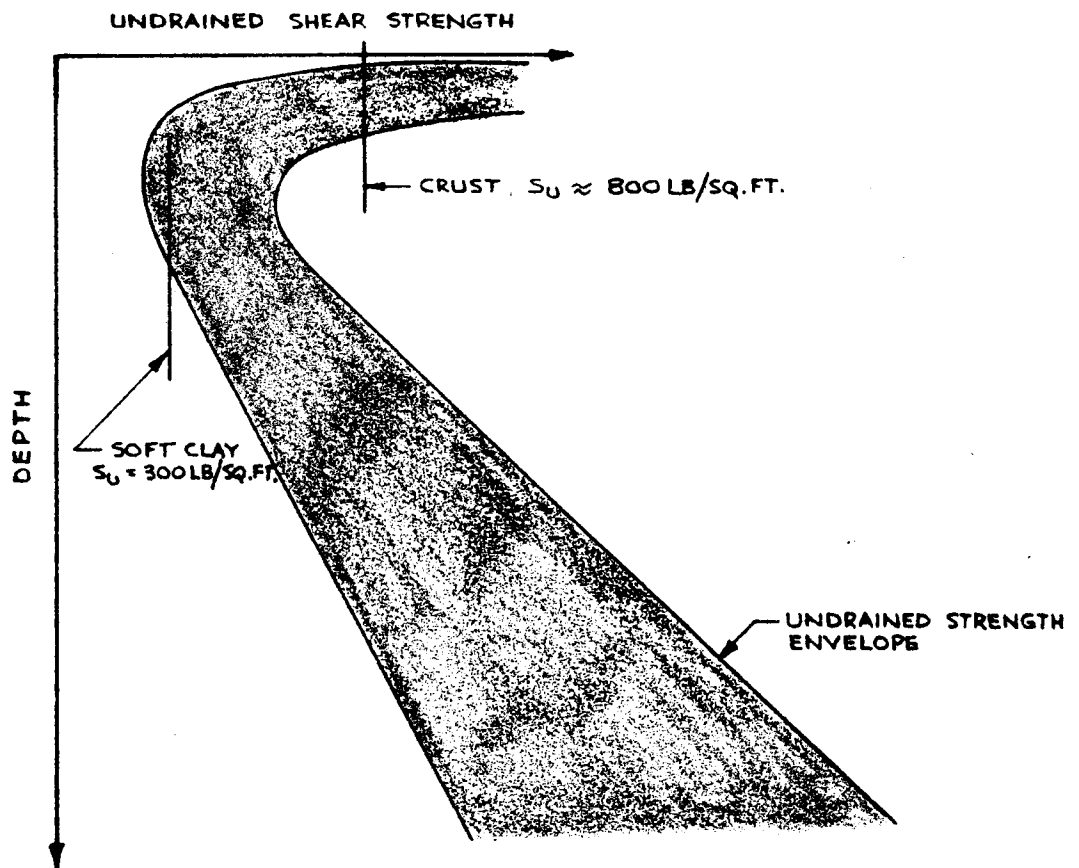
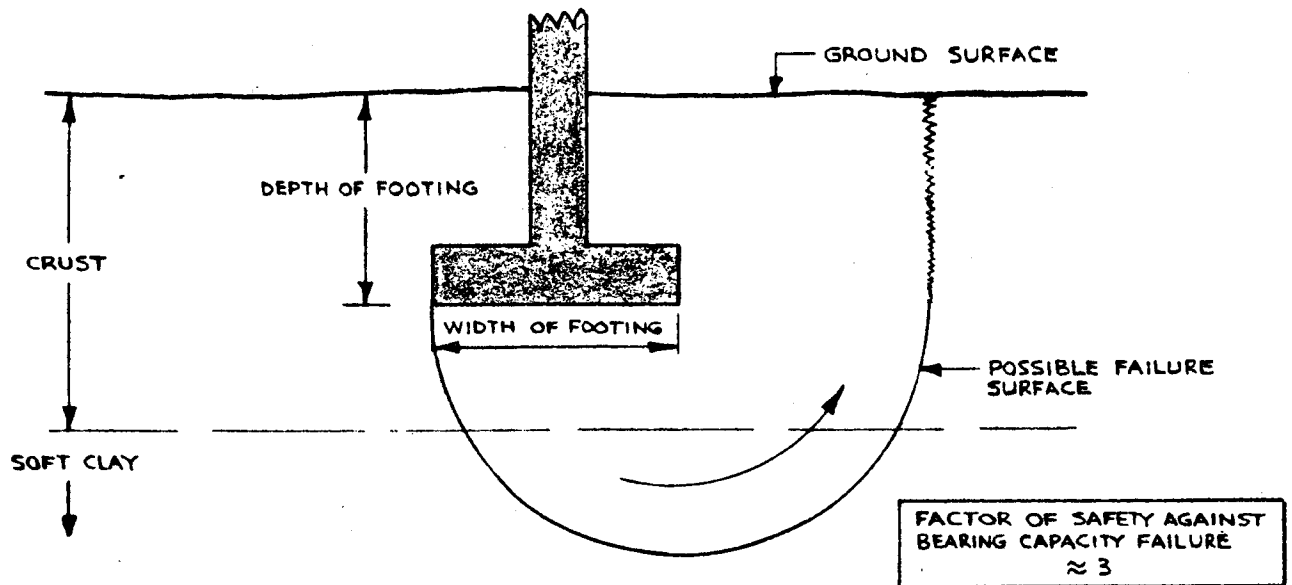
Date: JUNE 11, 1974

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

# SHEAR FAILURE CRITERIA - SHALLOW FOOTINGS

FIGURE 9



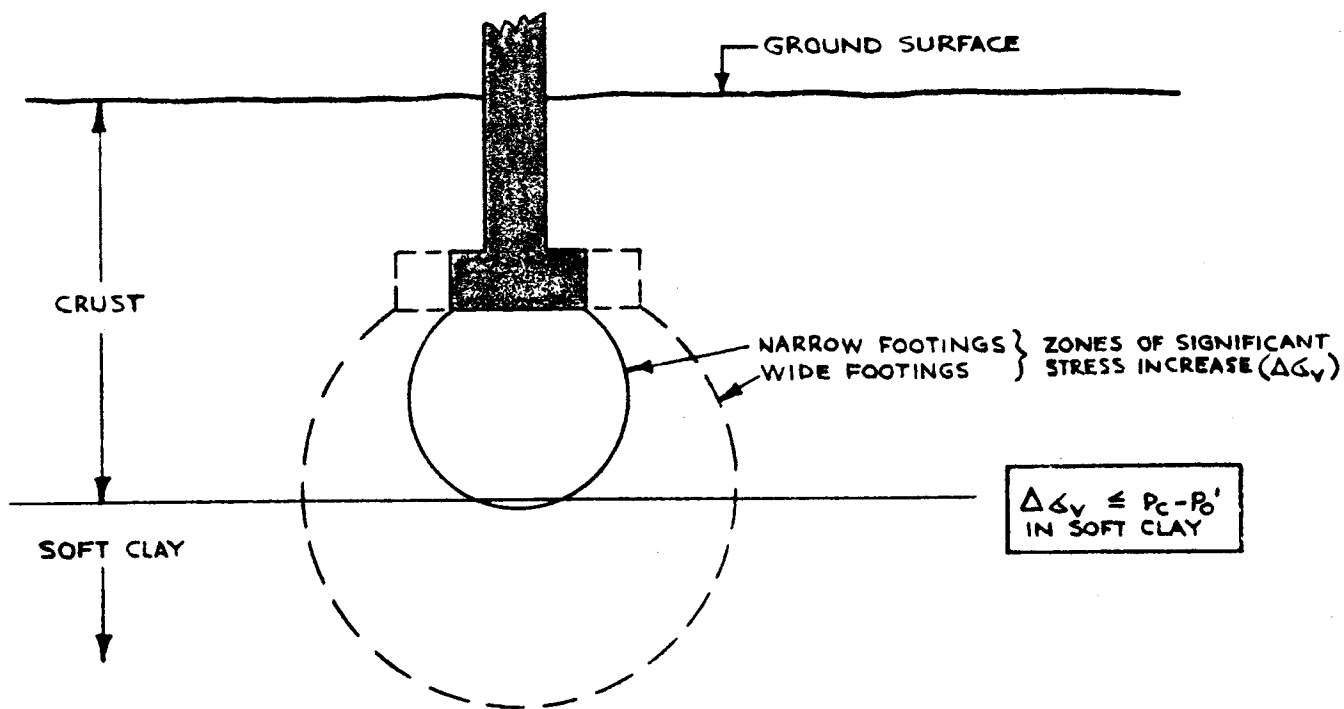
Date MAY 29, 1974

**Golder Associates**

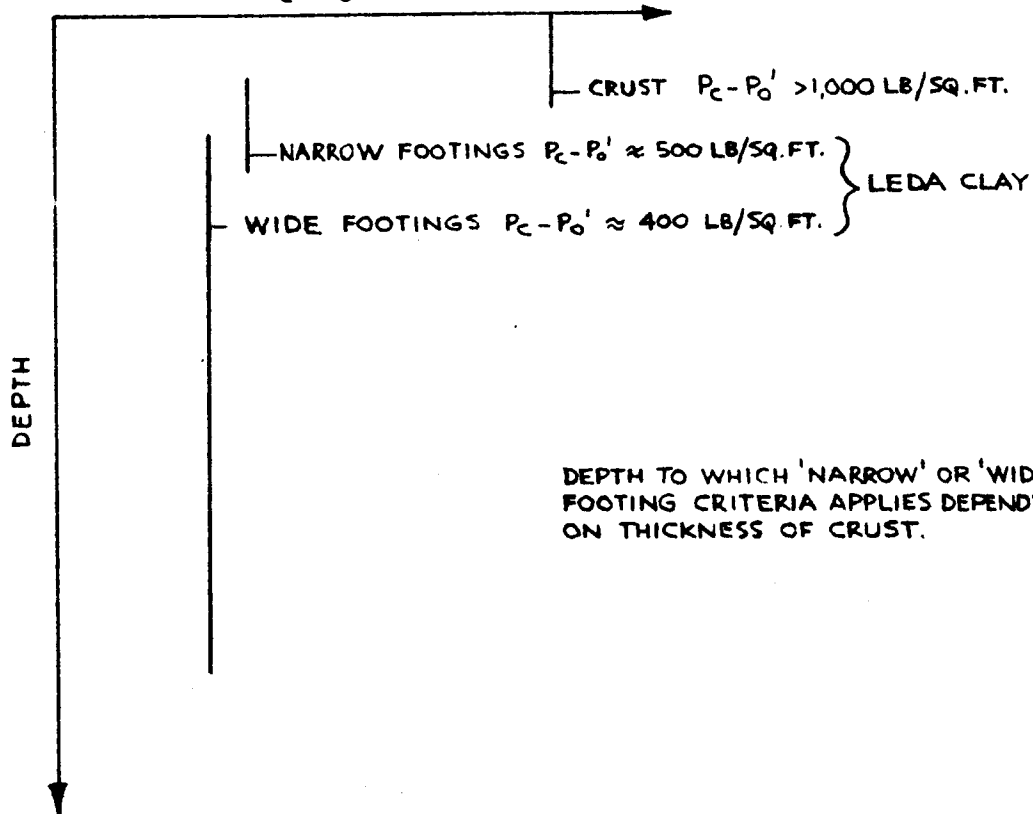
Drawn *J.A.*  
Chkd. *J.H.R.*  
Appd. *D.W.*

# SETTLEMENT CRITERIA - SHALLOW FOOTINGS

FIGURE 10



OVERCONSOLIDATION  
 $p_c - p_o'$



DEPTH TO WHICH 'NARROW' OR 'WIDE' FOOTING CRITERIA APPLIES DEPENDS ON THICKNESS OF CRUST.

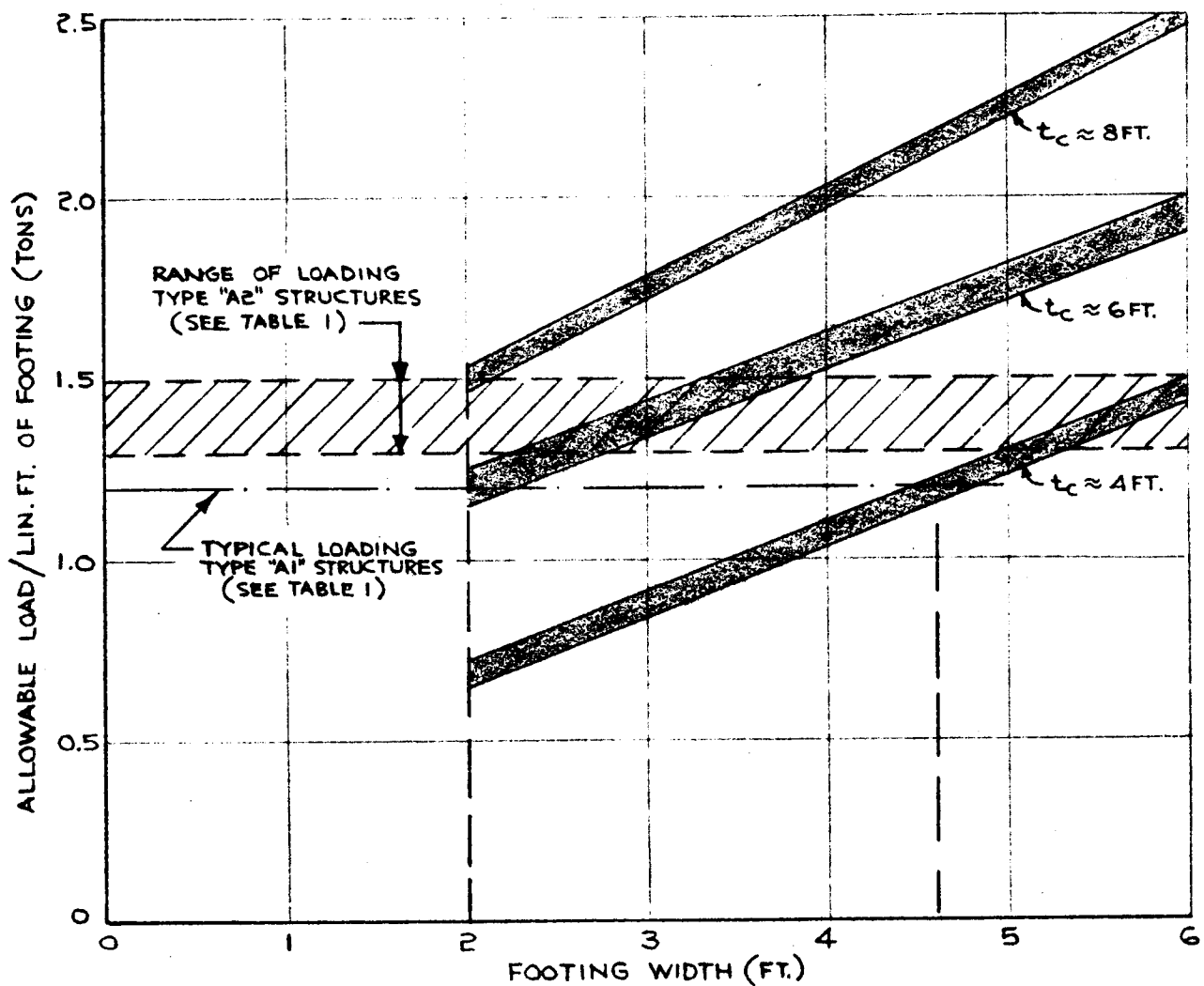
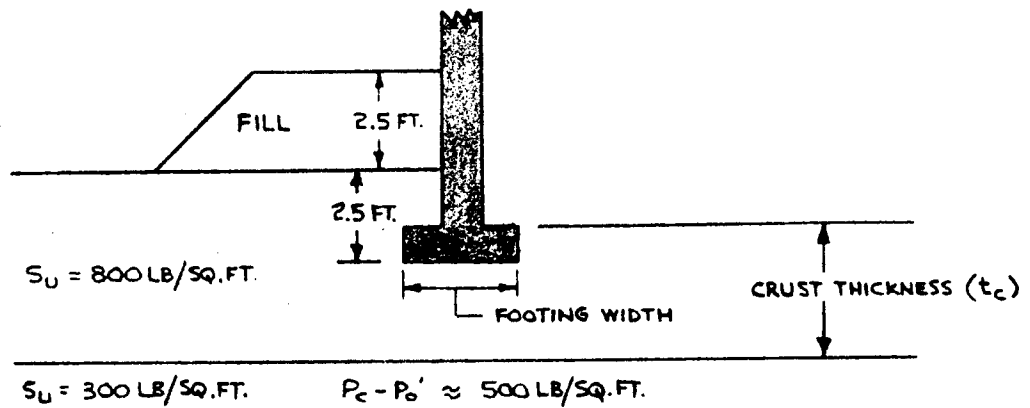
Date MAY 30, 1974

Golder Associates

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

# STRIP FOOTINGS FOR RESIDENTIAL STRUCTURES (TYPE 'A')

FIGURE 11



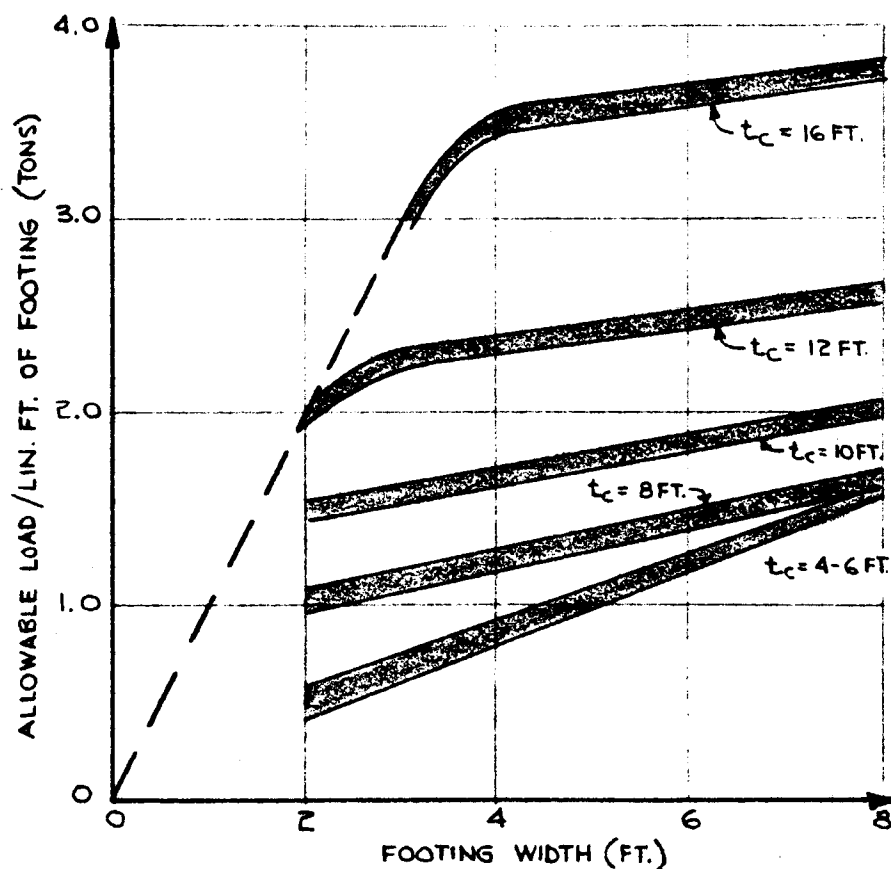
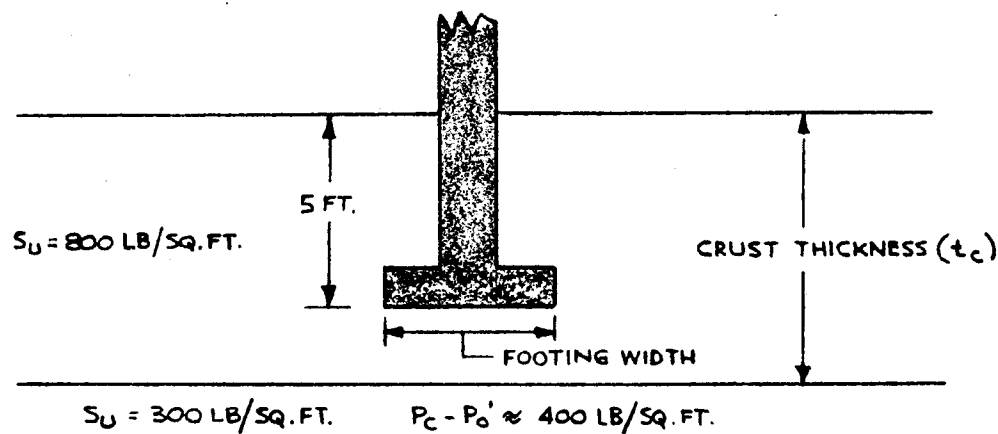
Date MAY 30, 1974

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Chkd. J.A.  
App'd. J.A.

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

FIGURE 12



TYPE 'B' STRUCTURES  
(SEE TABLE I)

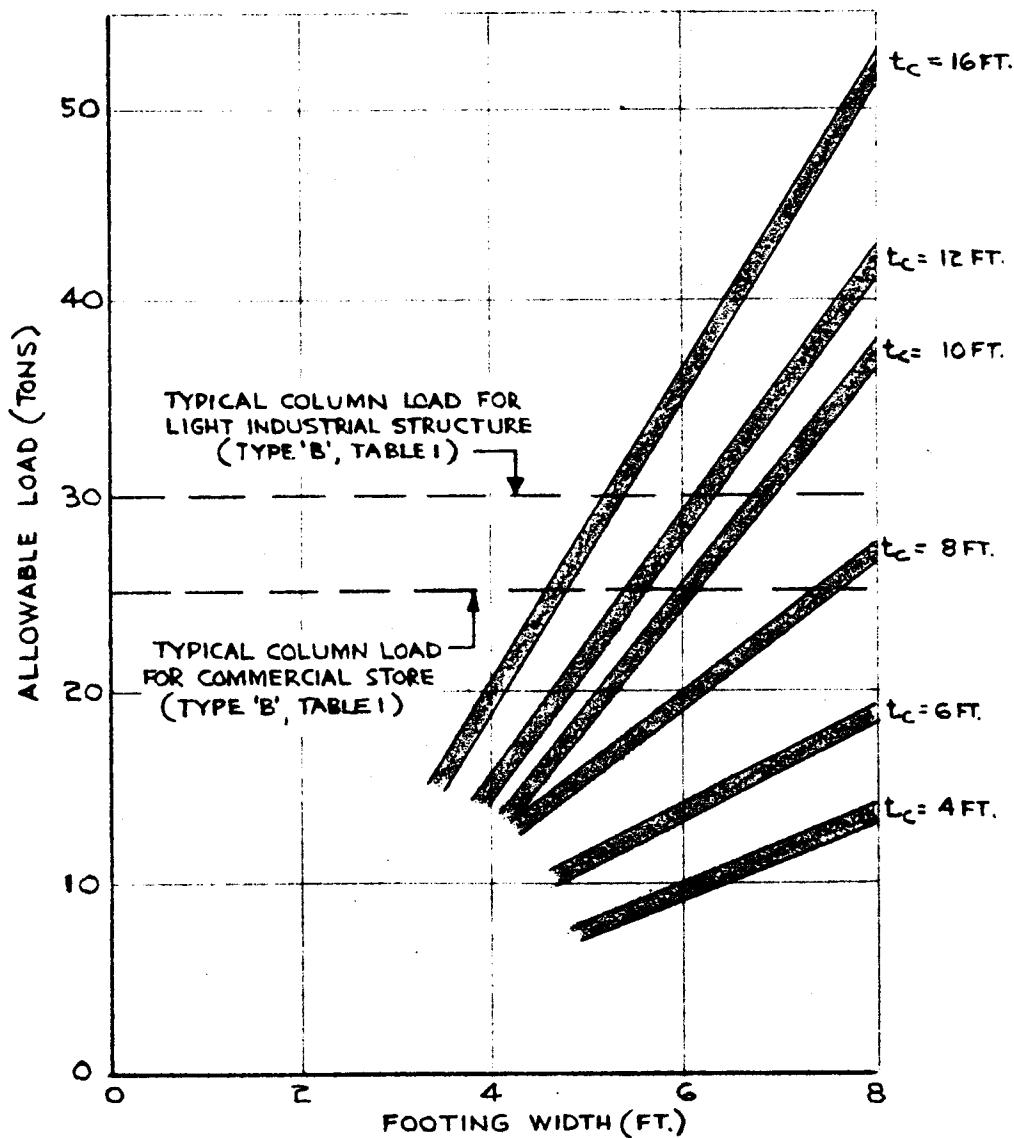
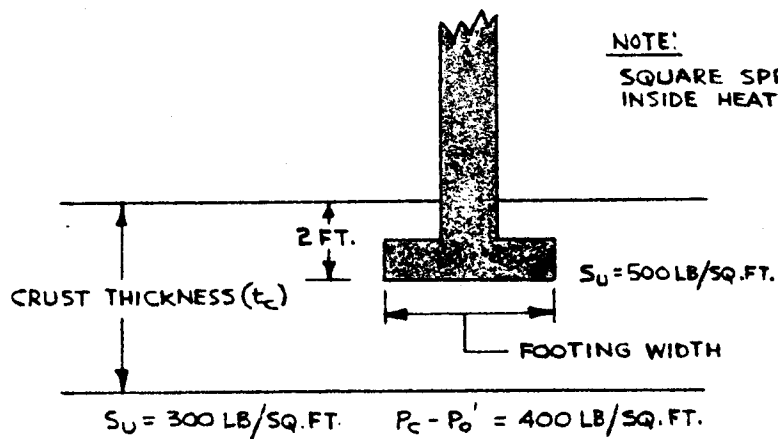
Date MAY 30, 1974

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Appd. J.M.

# ALLOWABLE LOAD FOR INTERNAL SPREAD FOOTINGS

FIGURE 13








Date MAY 30, 1974

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



LEGEND

SYMBOL	TYPE OF STRUCTURE
	A-1
	A-2
	A-3 B
	C
	STRUCTURES WITH PILED FOUNDATIONS (D)

NOTE: FOR DEFINITION OF STRUCTURE TYPES  
REFER TO TABLE I.

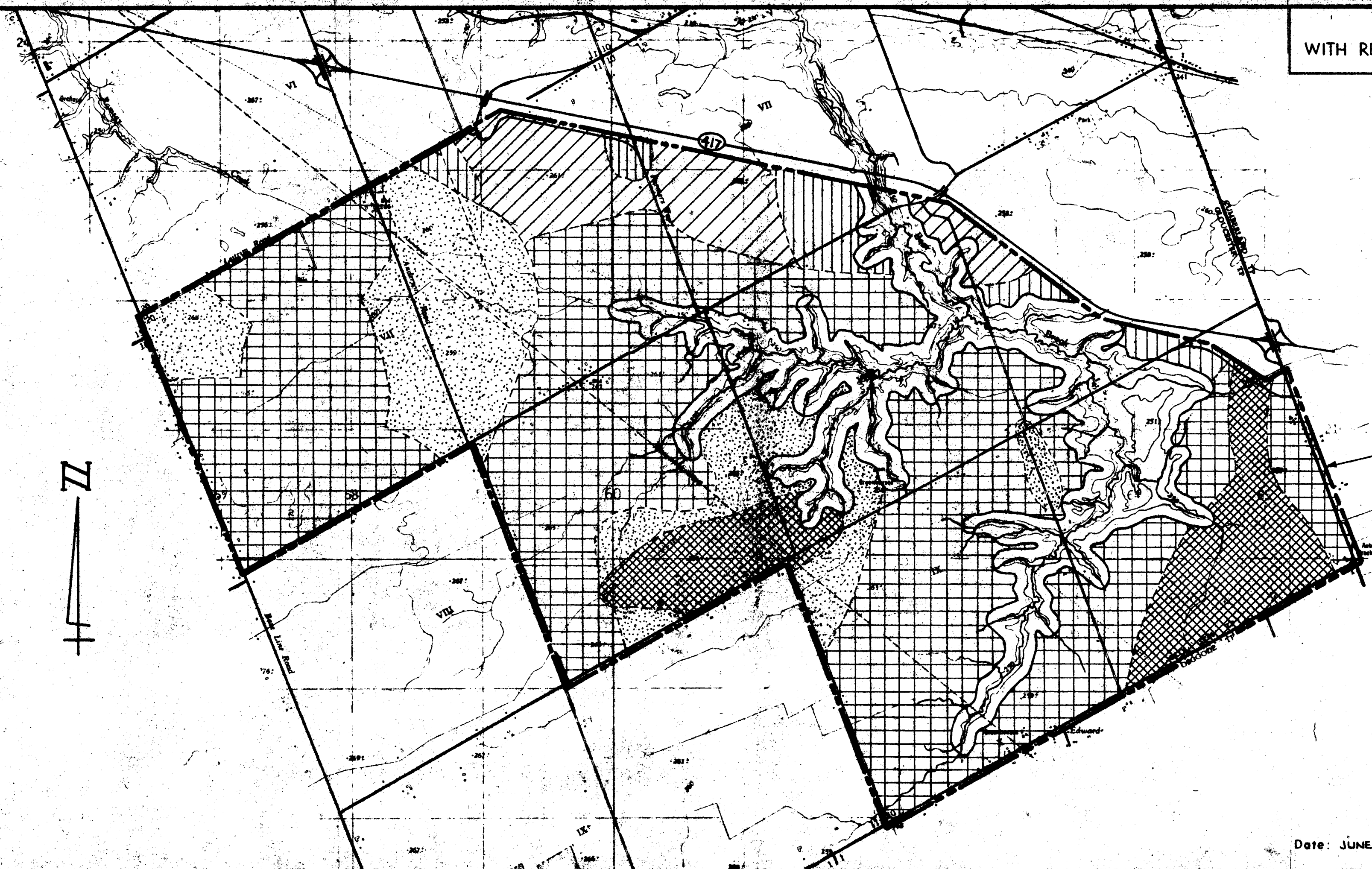
SITE BOUNDARY

SCALE: 1:25,000

Date: JUNE 11, 1974

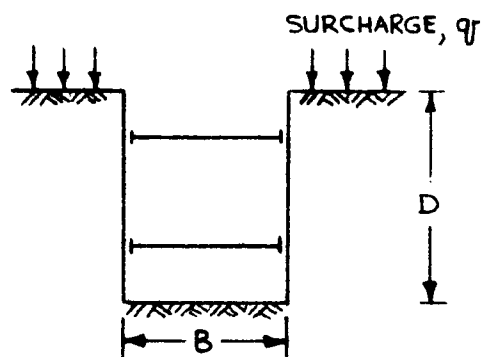
Golder Associates

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Chkd. J.H.S.  
Appd. J.H.S.



# BASAL STABILITY OF EXCAVATIONS

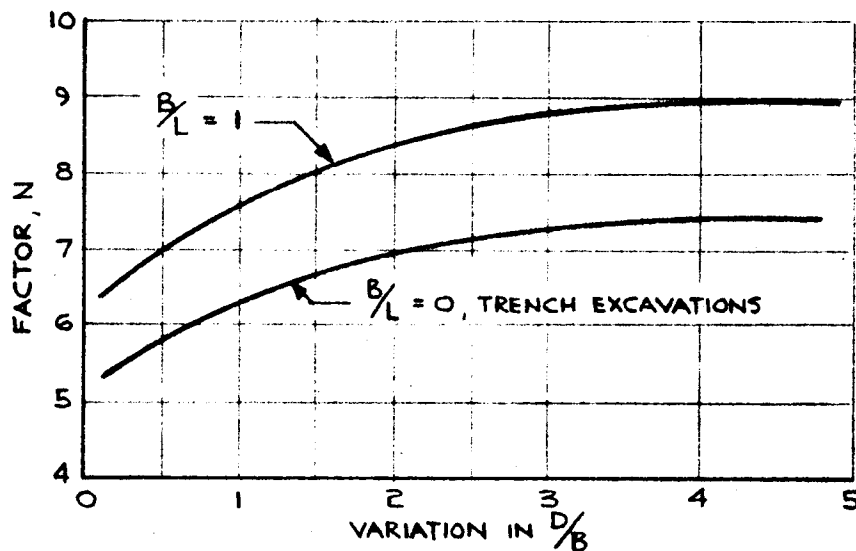
FIGURE 15



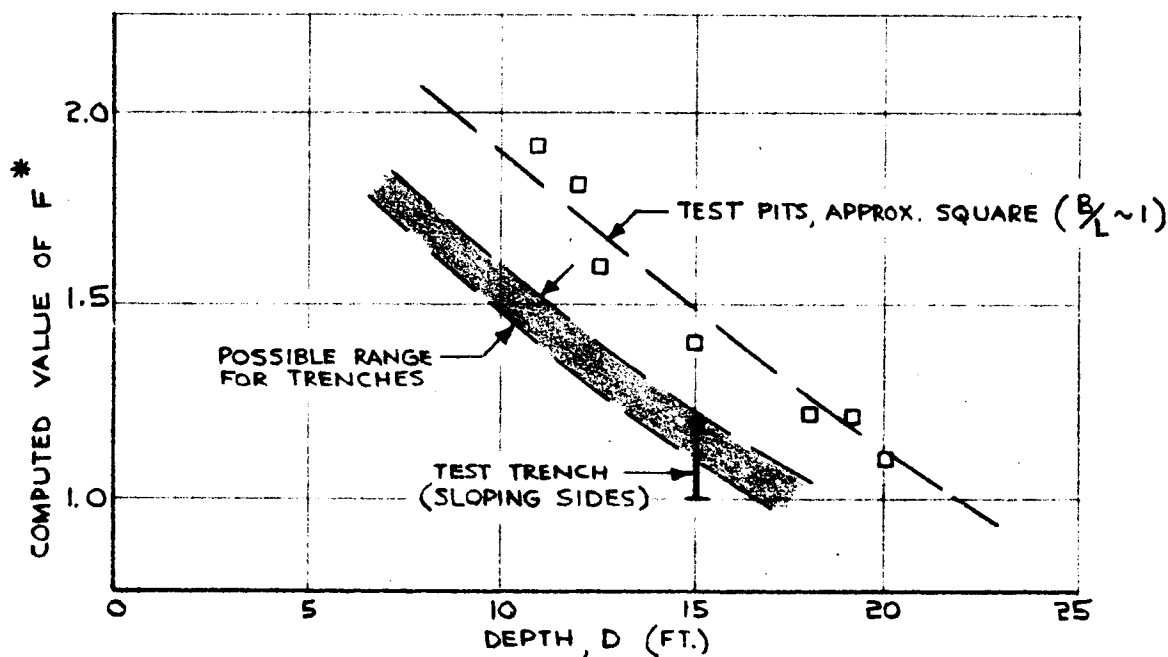
$L$  = LENGTH OF EXCAVATION

FACTOR OF SAFETY,  $F$

$$= \frac{N(s_u)}{\gamma D + q}$$



## GENERAL ANALYSES OF BASAL HEAVE IN EXCAVATIONS



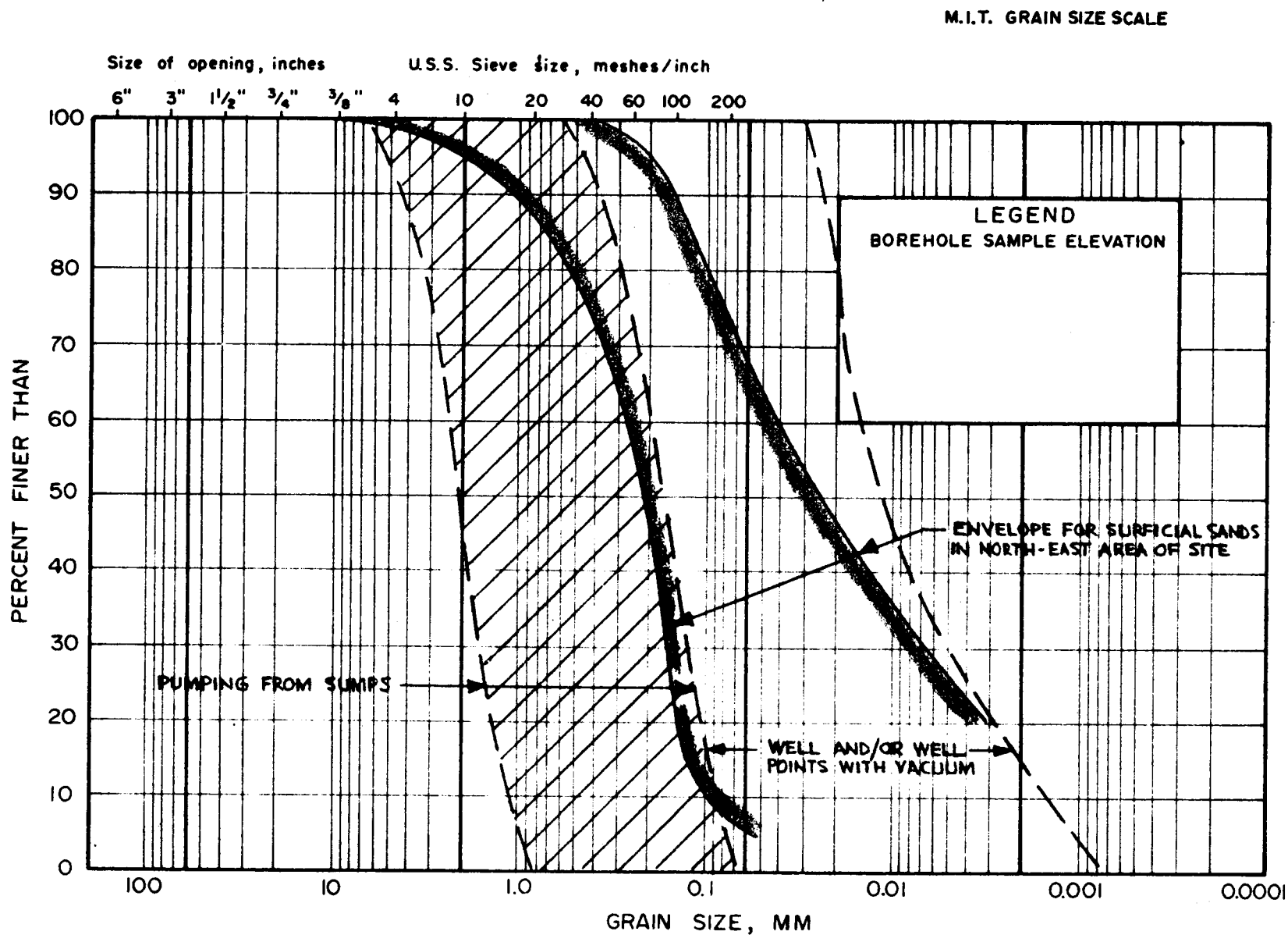
\*  $F$  COMPUTED ASSUMING  $s_u(\text{MIN.}) = 300 \text{ LB/SQ. FT.}$

## ANALYSES OF BASAL STABILITY, TEST PITS AT SITE

Date MAY 29, 1974

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

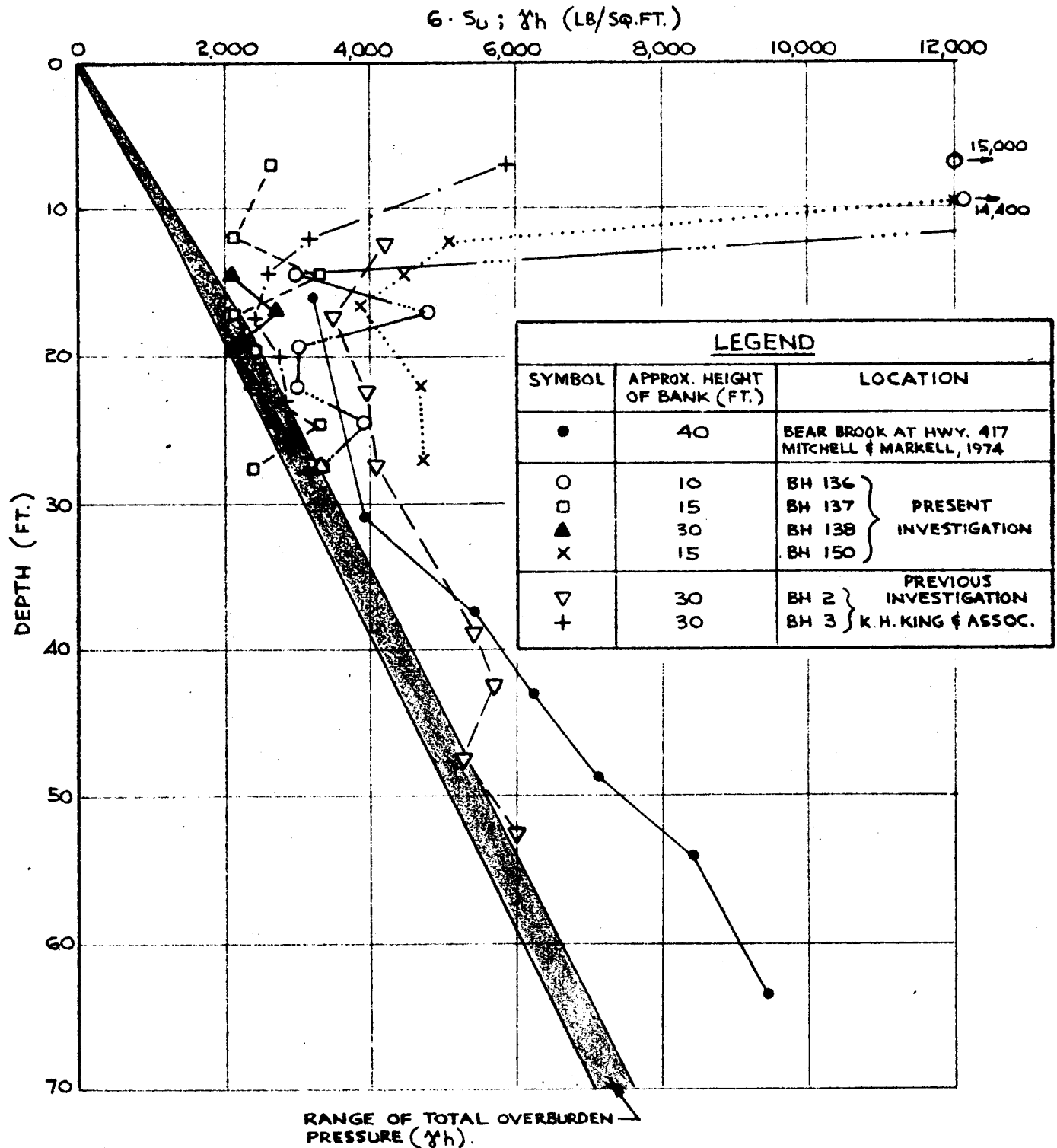


COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

GRAIN SIZE DISTRIBUTION  
(SURFICIAL SANDS - NORTH EAST AREA OF SITE)

# STABILITY OF SIDE SLOPES OF BEAR BROOK

FIGURE 17



Date MAY 30, 1974

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Chkd. J.W.B.  
Appd. J.W.B.