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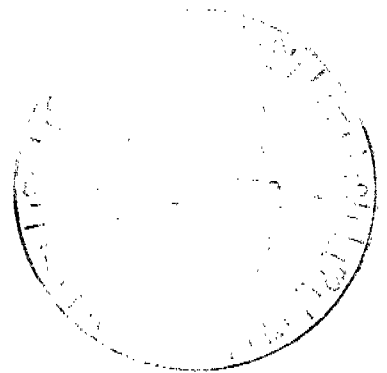
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**Golder Associates**  
CONSULTING GEOTECHNICAL AND MINING ENGINEERS



REPORT TO  
DELEUW CATHER, CANADA LTD

GEOTECHNICAL INVESTIGATION  
AND  
REVIEW OF GEOTECHNICAL DESIGN ASPECTS  
ORMONT DRIVE GRADE  
SEPARATION PROJECT

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ABSTRACT

This report presents the result of a geotechnical investigation and review of an underpass design for the Ormont Drive Grade Separation Project.

After construction had commenced, some concern was expressed as to the properties of the subsoil and adequacy of the design. Golder Associates were asked to review the design and as part of this review a geotechnical investigation was carried out. In general the soil conditions were found to be consistent with those previously reported and consist of fill over stiff to firm clay which in turn overlies loose silt and sand.

Analyses of the retaining wall foundations have been made for different earth pressure conditions and by various established methods. It is concluded that only in the case of the low walls are the designed foundations adequate to provide normal security against bearing failure and wall movements. Foundations for the low walls and bridge abutments are considered to be adequate.

Recommendations for support of the high retaining walls on piled foundations are given in the report.

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## 1. INTRODUCTION

Golder Associates has been retained by DeLeuw Cather Canada Ltd. (DeLCan) to carry out a review and provide a second opinion on geotechnical aspects of the underpass design for Ormont Drive Grade Separation Project. Construction of the underpass had already commenced when some concern was expressed as to the properties of the subsoil and adequacy of the design. In order to answer these questions a geotechnical investigation and design reviews were carried out between November 9 and December 4, 1981.

On December 14th a draft report, outlining the findings, was issued to DeLeuw Cather for comment. A copy of the draft was forwarded to the Trow Group and a meeting between Deleuw Cather, The Trow Group and Golder Associates was convened on December 17th to discuss the report.

The main conclusion of the draft report was that the designed spread footings were inadequate for all but the lowest sections of wall. The meeting discussed the effect of assuming lower earth pressures in the analysis due to the contractor having installed the temporary soldier pile wall further back from the proposed location of the permanent wall than shown in the design drawings. In addition various methods of analysis for the calculation of bearing capacity were considered. The method used by Golder Associates in the analysis presented in the draft report had been based on the Canadian Foundation Engineering Manual (Ref.1). Other

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1. 'Canadian Foundation Engineering Manual' (1978) Canadian Geotechnical Society.

methods, as outlined by Teng Bowles and Vesic (Refs. 2 to 5) could also be adopted and these might give less conservative results.

It was considered that several of the alternatives for improving the stability of the retaining walls as discussed in the draft report were unacceptable from considerations other than geotechnical. These included reducing the surface grade in the vicinity of the wall and placing horizontal struts across the excavation.

This report supercedes the draft report and previous correspondence on the question of wall stability. It outlines the investigation carried out and contains the results of calculations made taking account of the comments made subsequent to the issue of the draft report.

## 2. PROJECT DESCRIPTION

The proposed roadway beneath the existing CP tracks is an extension of Ormont Dr. connecting Fenmar Dr. and Toryork Dr. located in the City of North York, Ontario. The proposed underpass is a reinforced concrete structure consisting of a bridge supporting the CP tracks and grade separating cantilever retaining walls extending some 80 m west and about 15 m east of the tracks. The retaining walls east of the tracks are of limited extent since beyond the railway embankment

2. Teng 1962 "Foundation Design" Prentice Hall Inc. New Jersey.
3. Bowles 1974 "Analytical and Computer Methods in Foundation Engineering" McGraw Hill Inc.
4. Bowles 1977 "Foundation Analysis and Design" McGraw Hill Inc.
5. "Bearing Capacity of Shallow Foundations" Chapter 3 in "Foundation Engineering Handbook" Winter Korn and Fang-Editors. Van Nostrand Reinhold Company.

the road allowance is sufficient to cut back the side slopes to a gradient of 3:1 (Contract Drawing No. 2). The road allowance west of the tracks is limited and the design utilizes full height retaining walls from the bridge abutment to the roadway at Toryork Dr. The south retaining wall west of the tracks is in close proximity to a light industrial building occupied by Bona Foods Ltd. located 3.5 m from the south edge of the retaining wall footing. On the north side a similar building occupied by 'International Auto Collision' is situated some 10 m from the north retaining wall.

The design drawings indicate that only the bridge abutments are to be founded on pile foundations. Wing walls and retaining walls are designed to be founded on spread footings 1.2 to 2.2 m below proposed road level.

Prior to the design a site investigation was carried out by the Trow Group Ltd. The results of this are contained in their report T 1625-G dated November 22, 1979.

### 3. SITE AND GEOLOGY

The site is located between Fenmar Dr. and Toryork Dr. near Ormont Dr., North York, Ontario (Figure 1). At the time of the geotechnical investigation carried out by Golder Associates, steel H-piles for temporary soldier pile walls had been installed outside the proposed retaining wall alignments. Excavation had commenced between the soldier pile walls and the site grade had been reduced by up to 2 m.

Available geological records indicate that this site is underlain by a succession of glacio-lacustrine clays, silts and sands deposited following the recession of the Wisconsin glacier.



The soils in the area typically consist of a thin veneer of sand underlain by clay till; although above the till, layered clays and silts with some gravel inclusions are often encountered. The depth to bedrock in the general area is highly variable.

#### 4. SITE INVESTIGATION PROCEDURE

The field work for the investigation was carried out during the period from November 9 to 13, 1981 with some additional work done on November 19, 1981. Eight boreholes were put down to an average depth of 9.6 m at locations along the north and south retaining walls as indicated on Figures 2 and 3 (Boreholes 101 to 108).

The borings were put down using a truck-mounted CME 55 power auger supplied and operated by a specialist drilling contractor. In the borings, samples of the overburden soils were obtained using a conventional split spoon sampler at depth intervals of 0.75 to 1.5 m. Standard Penetration Tests were carried out during sampling operations. Where cohesive soils were encountered, the split-spoon samples were augmented with thin-walled Shelby tube samples. Field vane tests were carried out in cohesive soils between sample elevations to determine undrained shear strength. Standpipes were installed in each boring to permit monitoring of the groundwater levels. Borehole elevations and locations were surveyed by a member of Golder Associates staff. Subsequent to the borings, dynamic cone penetration tests were carried out at locations adjacent to Boreholes 102 to 105 inclusive.

## 5. SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the boreholes, together with the results of the laboratory tests carried out on representative samples, are given on the attached Record of Borehole sheets and figures. It should be noted that the stratigraphic boundaries indicated on the borehole logs are inferred from non-continuous sampling. The boundaries typically represent a transition from one soil to another and do not necessarily indicate an exact plane of geological change. Furthermore, the subsurface conditions will vary between boreholes.

In general, the findings of the investigation are in agreement with those of the previous investigation. The site is underlain by a firm to stiff silty clay which extends to variable depths and is about 6 m below ground surface in the area of the railway embankment. West of the railway tracks, in the area where the retaining walls are present, the depth of clay increases to about 7.5 m.

The clay strata was found to be underlain either by loose to compact silt or very loose to dense fine silty sand.

The current investigation was primarily concerned with the proposed founding stratum for the retaining walls and the strata below elevation 144 m were not investigated. The previous work by the Trow Group Ltd. had found dense sandy silt underlying the above strata (The Trow Group Ltd. Report T1625-G).

A detailed description of the soils encountered follows:

### 5.1 Topsoil and Fill

Topsoil was encountered in Boreholes 108 and 107 located east of the railway tracks on opposite sides of the proposed roadway. The depth of topsoil is 0.45 m and about 0.3 m respectively. West of the tracks between 0.3 m and 1.8 m of fill material was encountered in all boreholes put down from the original ground level. The fill was found to be generally very stiff to hard grey and brown silty clay and clayey silt, occasionally mottled. Natural moisture contents varied from 12 to 21 per cent.

### 5.2 Silty Clay

Below the surficial deposits of topsoil and/or fill, a 4.9 m to 6.7 m thick deposit of clayey silt to silty clay was encountered. The bottom of the clay stratum is at elevations 147.3, 147.4, 148.1, 145.5, 146.6, 146.2, 148.3, 148.1 m at the locations of Boreholes 101 to 108 respectively. The soil is variable in composition. In the upper 0.7 m to 3.5 m the material is mottled in appearance and is generally of a very stiff to hard consistency. This layer may constitute a weathered crust and it is underlain by a firm to stiff grey silty clay which at certain locations gradually becomes a clayey silt with depth.

Several of the samples indicated that the clay contained fine laminations with mainly horizontal fissuring. Lenses of sand and silt were noted at various horizons. Natural moisture contents of the clay strata vary between 19 and 38 per cent with lower moisture contents measured in the upper layers. Laboratory Atterberg Limit tests are summarized on the Plasticity Chart (Figure 4) and indicate clays of low plasticity (CL type according to the Unified Soil Classification System ASTM D 2487).

In situ vane tests carried out between elevations 146 and 150 m (i.e. below the upper crust) indicate an undrained shear strength of 25 to 60 kPa with a sensitivity based on remoulded strengths generally between 2 and 4.

Laboratory unconfined compression tests carried out on "undisturbed" Shelby tube samples from between the same elevations resulted in undrained shear strengths between 7 and 22 kPa. Similar tests on samples from the upper zone of clay indicated shear strengths of 55 and 112 kPa.

The results of both the in situ and laboratory strength tests have been plotted against elevation on Figure 5. It is apparent that the laboratory tests give significantly lower values of shear strength than those obtained from the in situ vane test. The clay material is of fairly low plasticity and although it contains occasional lenses of sand and silt it is felt that these are not of sufficient frequency to invalidate the vane test results. It is interesting that the laboratory test results give values close to the remoulded shear strength measured by the vane (refer to Record of Borehole (sheets)). This implies that the samples tested in the laboratory had experienced some disturbance and it is concluded that the vane strengths provide the best estimate of undrained shear strength of the deposit.

A zone representing the range of shear strengths measured by the vane tests is indicated on Figure 5. Although the zone is fairly wide it appears that the strength gradually reduces with depth to about elevation 148.5 m and thereafter starts to increase. This is consistent with a deposit in which the upper zone has been overconsolidated due to weathering while the lower material is only lightly overconsolidated and has properties which are a function of the overburden stress.

A consolidated undrained triaxial test (Figure 7) was carried out on a sample from 150 m elevation in Borehole 104. This gave effective stress parameters of:

$$c' = 11 \text{ kPa} \qquad \phi' = 22.5^\circ *$$

It should be noted that the sample was taken from an elevation at which some overconsolidation is apparent. At lower elevations it is possible that the cohesion intercept ( $c'$ ) would reduce to zero.

### 5.3 Silt

At the location of Boreholes 101 and 102 the silty clay is underlain by loose to compact silt to sandy silt. The thickness of the stratum at these locations is 2.7 and 2.0 m respectively and Standard Penetration 'N' \*\*values were found to range from 3 to 8. In Borehole 108, east of the railway, the silt underlying the clay layer was found to have an appreciable clay content and to be of firm consistency. Grain size distributions of samples from Borehole 101 are shown on Figure 6.

### 5.4 Fine Sand

Very loose to loose fine sand was encountered below the clay stratum in Boreholes 103 to 106. The thickness of the strata varies between 1.1 and 2.5 m and Standard Penetration values of 2 and 3.

It was considered possible that some disturbance of the fine sand and silt (Section 5.3 above), caused by an upwards flow of groundwater at the bottom of the borehole, had taken place during drilling. While there was no evidence of such

\* Refer to List of Symbols

\*\* Refer to List of Abbreviations.

'blowing' of the cohesionless deposits it was considered necessary to check the penetration results. Holes were augered to within a few feet of the cohesionless layers about 3 ft. from Boreholes 102 to 105 inclusive and a continuous cone penetration test carried out. Where there is no risk of significant friction on the rods this test gives values consistent with 'N' values from a Standard Penetration Test. The results are plotted on the relevant Record of Borehole sheet and confirm the loose state of the cohesionless layers.

In Borehole 107, east of the railway, the fine sand layer was found to be in a compact to dense state (N values between 11 and 46). In Borehole 108 the sand was found underlying the silt and again it appeared to be compact.

#### 6. GROUNDWATER CONDITIONS

Groundwater levels recorded in the standpipes are indicated on the Record of Borehole sheets and in the stratigraphic section (Figure 3). Prior to commencing the excavation the contractor had proceeded to lower the water level in the sand aquifer and the levels recorded are significantly below the level of about 153 m recorded by Trow Ltd. in 1979. It is worthy of note that the groundwater conditions recorded in the present investigation are temporary and do not necessarily reflect those which will be established during the working life of the underpass.

#### 7. ANALYSIS OF EARTH PRESSURES ON THE PROPOSED RETAINING WALLS

Earth pressure calculations have been based on the conventional assumption that small lateral wall movements (inevitable for unrestrained walls) will result in "active" conditions in the soil mass behind the wall.

Under "active" conditions the earth pressure which has to be supported by the wall at ultimate limit state is determined by the pressure necessary to support the wedge of soil which is in the most critical condition of failure. Where a considerable extent of cohesionless backfill is present behind the wall this critical wedge and therefore 'active' pressure will be dependent only on the properties of the backfill. Where the backfill is limited the critical wedge can extend into the natural soil behind the backfill.

For the Ormont Drive retaining walls the contract drawings indicate backfill to extend only to a vertical line close the back of the footing. The contractors shoring details, (S. McNally & Sons Ltd. Drawing No. C-246-3) show the soldier pile wall to be further back from the proposed retaining wall than indicated on the contract drawings. There is therefore greater space for granular backfill to be placed and this would have an effect on the earth pressures. It is understood that some discussions are taking place on the acceptability of the soldier pile wall in the position indicated. Calculations of earth pressure have therefore been made for two conditions (Refer to Figure 8):

- a) "Design Condition" in which backfill extends only to a vertical line from the back of the retaining wall footing.
- b) "Assumed As-built Condition" in which specified granular backfill extends to soldier pile wall as indicated on S. McNally & Sons Ltd. Drawing No. C-246-3.

Lateral thrusts obtained from a wedge analysis of the backfill and the natural ground behind it have been reduced to an 'equivalent earth pressure coefficient' ( $K_{ae}$ ) so that

a design value can be assigned for the wall. The total earth pressure (P) obtained from the earth pressure calculations is assumed to be obtainable from the expression:

$$P = K_{ae} \left( \gamma \frac{H^2}{2} + qH \right)$$

Thus for any retained height H, surcharge q and unit weight the equivalent earth pressure coefficient  $K_{ae}$  can be calculated. The values obtained are given in Table I.

## 8. ANALYSIS OF RETAINING WALL FOUNDATIONS

Retaining wall foundations must provide adequate security against sliding of the base and bearing failure in the subsoil. At the same time deformations must be acceptable considering the purpose of the structure.

### 8.1 Design Parameters

Footing elevations along the wall vary from 147.3 m for the high wall adjacent to the bridge to 150.0 m at Toryork Drive (Figure 3). Apart from the high walls (Panels N-G, N-H, N-K, S-D, S-C and S-B) which are founded in the loose silt all the walls are founded in the overlying firm to stiff clay.

For short term loading a design undrained shear strength of 30 kPa has been selected for the clay layer (Figure 5). This corresponds to about the lower quartile of vane strengths between elevations 149 and 147 m. Since the footings are from 3 to 4.75 m wide it is this layer which will have the most influence on the bearing capacity of the clay.

For long term stability, in which it is assumed that the pore pressures in the subsoil have stabilized at a steady state



value, an effective stress analysis has been carried out. As discussed in Section 5.2 effective stress parameters of a sample from 150 m elevation are:

$$c' = 11 \text{ kPa} \qquad \phi' = 22.5^\circ$$

The presence of a cohesive intercept is normally associated with overconsolidation and reference to the undrained shear strength plot (Figure 5) suggests that between elevation 149 and 147 m this may reduce to zero. The long term analysis has therefore been carried out for two cases. The first assumes  $c' = 0$  and  $\phi' = 25^\circ$  which gives an overall shear strength approximately equal to that measured. The second case is considered to be a lower bound and assumes  $c' = 0$  and  $\phi' = 22.5^\circ$ .

For the loose silt or silty fine sand on which the high walls are founded it is assumed that steady state pore pressures establish quickly and only an effective stress analysis has been carried out. The shear strength has been assumed to be represented by:

$$c' = 0, \quad \phi' = 28^\circ.$$

The saturated unit weight of the soil has been taken as 20 kN/cu.m. Consistent with the design criteria provided by DeLeuw Cather, an overall surcharge of 10 kPa has been assumed to account for traffic and adjacent building loads.

## 8.2 Resistance Against Sliding

The Canadian Foundation Manual (Ref.1) requires a factor of safety of 2.0 against sliding where passive pressure against the toe of the wall is included in the analysis. Where this

is ignored a factor of safety of 1.5 is sufficient. In cases where drains and services are located at the toe of the wall as at Ormont Drive, the passive pressure is not included and so the second criterion is considered to be relevant.

Sliding resistance is considered to be provided by friction along the base but in the short term this cannot be greater than the undrained shear strength of the clay. Safety factors have been calculated for the 'design condition' assuming a perfectly rough contact between the base and the soil and these are tabulated in Table II. The frictional resistance is tabulated under long term condition while the undrained shear resistance (for the walls on clay) is listed under short term condition. For the frictional calculation it is assumed that the piezometric level at the base of the wall is at the finished road level.

The high walls founded on silt have been provided with a 0.6 m deep shear key. The contribution of this to sliding resistance has been investigated by various trial wedges in front of the key. The overall effect is similar to lengthening the potential failure surface from the width of the base to the length of a line extending from the heel to the toe and extending under the key. This results in an increase in resistance of about 5 per cent. In practical terms the key is useful in ensuring that failure will take place within the soil mass and not along a concrete/soil contact which may have a lower coefficient of friction or have been disturbed during construction.

The results of the analyses indicate that the factors of safety for all panels apart from S-H are less than 1.4 in the short term and as low as 1.2 for the high wall (S-C). In

the long term the lower bound strength parameter ( $\phi' = 22.5^\circ$ ) gives similar values.

### 8.3 Bearing Capacity

Due to the combined effect of self weight (including the soil over the heel and toe of the wall) and the lateral thrust from the soil the resultant load on the foundation is inclined to the vertical. Depending on the particular proportions of the wall the load may also be eccentric to the centre of the footing. The effect of these factors is taken into account in the methods of calculating bearing capacity recommended by the most recent foundation engineering publications (Refs. 1 to 5). The methods quoted are based on the work by Meyerhof (Ref. 6) and Brinch Hansen (Ref. 7).

The values of safety factors given in Table II are for the 'design condition' defined in Section 7 and are based on the method given in the Canadian Foundation Engineering Manual (Ref. 1).

It can be seen that only in the case of the short wall is the factor of safety greater than 2.0. For the other wall panels the factors range from 1.2 to 1.7.

- 
6. Meyerhof (1953) "The Bearing Capacity of Foundations under Eccentric and Inclined Loads" 3rd Int. Conf. SMFE Zurich.
  7. Brinch Hansen (1961) "A General Formula for Bearing Capacity" Bulletin No. 11, Danish Technical Institute, Copenhagen.

#### 8.4 Effects of 'Assumed As-built Condition' and Method of Analysis

A calculation has been made of the effect of the reduced earth pressure coefficient arising from the possibility of a greater extent of granular backfill placed behind the wall. (Assumed As-built Condition - Table I). At the same time the effect of the method of analysis has been investigated by carrying out the calculations according to the methods presented in References 1 to 5. Table III presents the results of the calculation for the short term condition. The detailed calculations for Panel S-E are given in the Appendix.

Where specific recommendations on minimum safety factors are included in the publications referred to, these are also given in the Table. The final column expresses the calculated factor of safety as a ratio of the minimum recommended factor of safety.

Only in the case of the low wall (Panel S-H) is the minimum factor of safety achieved in both sliding and bearing by any one of the methods adopted. It is interesting that the method which provides the highest factor of safety against bearing failure (Ref. 4) recommends a higher minimum factor for sliding on cohesive soil and none of the panels meet this requirement.

#### 8.5 Effect of Water Pressures on Factors of Safety

The calculation of factor of safety in the long term (Table II) is based on an effective stress analysis which requires some knowledge of pore pressures in the soil. Normal bearing capacity equations are based on no flow conditions in the subsoil and by using the buoyant unit weight in the calculations it has been implicitly assumed that the piezometric

level in the subsoil is at the road elevation. This assumption allows for the possibility of the roadway flooding.

In the long term the actual porewater pressures in the subsoil at Ormont Drive will be complicated and difficult to predict. The present natural groundwater level is at about elevation 153m which is more than 3 m above the roadway level in the centre of the underpass. In order to provide water pressure relief in the sand aquifer under the road a drainage system connected to a storm sewer has been provided in the centre of the road. It is understood that the design is based on the results of a pumping test carried out in 1979 by the Trow Group Ltd. If the drainage is only designed to ensure that the road does not experience uplift, the piezometric level in the aquifer could rise to more than 2 m above the road level (Ref. to Figure 9). Under these conditions the pore pressure in the soil supporting the wall foundations would be higher than assumed for the calculations presented in Table II. The safety factors for long term bearing capacity and sliding resistance would therefore be substantially lower than those given in Table II.

The normal foundation analysis allows for the possibility of the road flooding by assuming buoyant unit weight for the soil. However if the drainage system is not adequate to lower the piezometric level in the founding stratum to below the road level the factors of safety in the long term would be below those given in Table II.

#### 8.6 Deformations and Acceptable Factors of Safety

Conventional minimum factors of safety such as those quoted in Table III are based on normally accepted degrees of risk for bearing failure of the footing. When applying any particular

method of design it is important to adopt the safety factors recommended in that method. Acceptance of lower factors carries with it the acceptance of a greater risk of failure.

It is apparent that the wall foundations east of panels S-H and N-B and west of S-C and N-G are inadequate to provide normal security against bearing failure; even allowing for the lower earth pressures obtained from the 'assumed as-built condition' (Section 7 - Figure 8). For panels east of S-C and N-G the factors of safety are acceptable from consideration of bearing failure but fall below criteria for sliding resistance even with the lower earth pressures. In addition these walls are founded on very loose to loose silts and fine sands (N values typically 3 and less), which are considered to be liable to significant and non uniform settlement under load. Bearing in mind the potential excess porewater pressure discussed in Section 8.5 it is considered that spread footings should not be adopted for this part of the wall.

Deformations of the wall are dependant on the construction technique and the level of stress in the foundation (i.e. the factors of safety). It is likely that conditions approximating to the 'active' state will be established in the soil behind the soldier pile wall before the cantilever wall is constructed. In areas where deformations have to be controlled (i.e. opposite existing structures) it is important that an efficient load transfer from the rakers supporting the soldier pile wall to the permanent wall is achieved. This could be achieved by compacting backfill up to the level of the rakers before removing them. It may be necessary to construct the stem of the wall in stages.

A wall constructed to restrict movement will have pressures on it somewhat higher than 'active' values and probably influenced by compaction of backfill. Where the factors of

safety against sliding and bearing failure are adequate the wall will probably be restrained from achieving the deformation necessary for the active state. Thus the deformation is likely to be fairly small ( $< 1$  in. subsequent to construction of the soldier pile wall). Where factors are low substantially more movement could develop.

At factors of safety against sliding of less than 1.5 it is likely that significant long term creep would take place in the foundation and this would add considerably to wall deformations.

#### 9. REVIEW OF BRIDGE ABUTMENT FOUNDATION

It is considered that piles driven and founded as recommended in the Trow Group report will be adequate to support the railway bridge.

Consideration has been given to possible lateral loading of the piles caused by straining of the silt and clay layers under embankment loading. This generally is considered to be a problem where low factors of safety against failure of the embankment exist. At factors of safety of about 2.0 the piles are not considered to come under significant lateral loading.

From the information available the bottom of the pile cap is in silt or fine sand. A calculation has given a factor of safety of 1.9 against instability and the piles should not therefore be unduly loaded from the subsoil. Where shallow layers of clay are found to be locally present at the pile cap level it is recommended that the pile cap is extended down to the bottom of the clay.

## 10. CONCLUSIONS AND RECOMMENDATIONS

It is considered that the wall foundations east of panels S-H and N-B are inadequate to provide normal security against bearing failure and liable to unacceptable deformation.

It is recommended that these wall panels are supported on pile foundations to minimize movements subsequent to their construction. Piles are necessary in order to ensure a satisfactory foundation, however the possible effect of pile driving on adjacent buildings must be taken into account during construction. For this reason it is recommended that piles are not driven to rock but are founded about 3 m into the dense silt (reported in the Trow Group report to be at about elevation 142 m). This will result in much less vibration than driving the piles to rock and cause minimum vibration of adjacent building. Vibration of the buildings must be monitored during both the driving of the bridge abutment piles to rock and the retaining wall piles into the silt.

For example, a 324 mm O.D. pipe pile of wall thickness 6.3 mm driven 3 m into dense silt should provide a "Factored Bearing capacity at Ultimate Limit States" of about 550 KN. The pile capacity at "Serviceability Limit States" can be taken as 400 KN. Pile capacity must be confirmed by carrying out a pile load test. Piling should be supervised by a geotechnical engineer.

Where lateral soil resistance is to be utilized in resisting forces on the above pipe pile a coefficient of horizontal subgrade reaction of 4 MPa/m for the firm clays and loose silts can be assumed in the analysis.




Earth pressures on the retaining walls should be calculated from the equation given in section 7 with an equivalent earth pressure coefficient of 0.35 for all panels apart from S-B, S-C, N-K and N-H for which a value of 0.4 should be adopted.

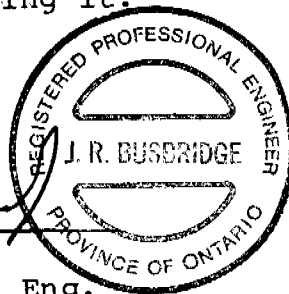
Foundations for panels west of and including panels S-H and N-B are considered to be adequate. It is recommended that excavations are inspected by a geotechnical engineer and that a 100 mm layer of well graded granular backfill is compacted between the clay and the footing to ensure a rough contact.

The use of heavy compaction equipment in the areas behind the retaining wall must be avoided. Light, hand operated vibratory plant such as the Bomag BW 75 is recommended for compaction of the fill behind the wall.

The piezometric level in the aquifer must be checked to ensure it is below the road level at locations where spread footings are to be adopted. At all locations adequate drainage must be provided to ensure that water pressure in the aquifer is maintained below a value equal to the weight of the soil overlying it.

GOLDERS ASSOCIATES

  
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JRB/pds

TABLE 1EQUIVALENT EARTH PRESSURE COEFFICIENTS

Wall Panel	Assumed "As Built" Condition	Design Condition
	Kae	Kae
S-C	0.30	0.43
S-E	0.27	0.33
S-H	0.28	0.36

TABLE 11"DESIGN CONDITION" - CALCULATED FACTORS OF SAFETY

Wall Panel*	Coeff. of Earth Pressure	Short Term Conditions		Long Term Conditions			
		Factors of Safety		Factors of Safety			
		Bearing	Sliding	Bearing		Sliding	
		$c_u = 30 \text{ kPa}$		$\phi'$		$\phi'$	
				22.5	25.0	22.5	25.0
S-E	0.35	1.5	1.3	1.2	1.7	1.3	1.7
S-F	0.35	1.6	1.4	1.3	1.6	1.3	1.5
S-H	0.35	2.0	1.9	2.0	2.4	1.5	1.6
S-C**	0.40	1.7	1.2	1.7		1.2	

\* Calculations have been carried out for the south wall.  
Equivalent north wall panels are considered to be similar.

\*\* Based on the angle of internal friction in silt of  $28^\circ$ .

TABLE 111

ASSUMED 'AS BUILT CONDITION' - CALCULATED FACTORS OF SAFETY

Retaining Wall Panel	Earth Pressure Coefficient Kae*	Method of Analysis**	SLIDING RESISTANCE			BEARING CAPACITY		
			Computed Factor of Safety SF.	Recommended Factor of Safety SF <sub>min</sub>	SF <sub>min</sub>	Computed Factor of Safety SF	Recommended Factor of Safety SF <sub>min</sub>	SF <sub>min</sub>
S-C	0.30	1	1.4	1.5	0.93	2.2	-	-
		2	1.4	1.5	0.93	2.1	2	1.05
		4	1.4	1.5	0.93	2.0	2	1.00
		5	1.4	1.5	0.93	2.2	2	1.10
S-E	0.27	1	1.6	1.5	1.07	1.7	-	-
		2	1.6	1.5	1.07	1.8	3	0.60
		4	1.6	2.0	0.80	2.2	3	0.73
		5	1.6	1.5	1.07	2.0	3	0.67
S-F	0.30	1	1.4	1.5	0.93	1.8	-	-
		2	1.4	1.5	0.93	2.0	3	0.67
		4	1.4	2.0	0.70	2.5	3	0.83
		5	1.4	1.5	0.93	2.3	3	0.77
S-H	0.28	1	1.5	1.5	1.00	2.3	-	-
		2	1.5	1.5	1.00	2.5	3	0.83
		4	1.5	2.0	0.75	3.3	3	1.10
		5	1.5	1.5	1.00	3.1	3	1.03

\* As defined in Section 7.

\*\* Numbers refer to references given in footnotes.

## LIST OF ABBREVIATIONS

The abbreviation 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 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

#### Standard Penetration Resistance, *N*:

The number of blows by a 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

*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) <i>Cohesionless Soils</i>	' <i>N</i> ' Blows/0.30m or Blows/ft.
Relative Density	
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	kPa	' <i>Cu</i> ' psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

### 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 = $-\Delta e / \Delta \log_{10} \sigma'$
$c_c$	coefficient of consolidation
$T_v$	time factor = $c_c t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

### (e) Shear strength

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

in terms of effective stress  
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress  
 $\tau_f = c_u + \sigma \tan \phi_u$

\*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.

RECORD OF BOREHOLE 101																	
LOCATION See Figure 2			BORING DATE NOV. 10, 1981			DATUM GEODETIC											
SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm			PENETRATION TEST HAMMER WEIGHT — DROP —														
BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/				COEFFICIENT OF PERMEABILITY, k, CM./ SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/0.3m	ELEVATION SCALE	SHEAR STRENGTH cu, kPa		NAT. V. + Q. - REM. V. - U. -		WATER CONTENT, PERCENT					
								20	40	60	80	10	20	30			40
POWER AUGER BORING 82.5mm I.D., HOLLOW STEM AUGERS	153.8	GROUND SURFACE					154									<div><div>G.S.</div><div>SURFACE SEAL</div><div>BACKFILL</div><div>BENTONITE SEAL</div><div>GRAVEL FILTER</div><div>MH</div><div>MH</div><div>MH</div><div>PIEZOMETER</div><div>W.L. IN PIEZOMETER AT ELEV. 150.1 ON NOV. 13, 1981</div></div>	
	0.0	VERY STIFF BROWN CLAYEY SILT, SOME SAND AND GRAVEL (FILL)															
	153.0																
	0.8	FIRM TO STIFF BROWN TO GREY CLAYEY SILT TO SILTY CLAY, SOME SAND AND GRAVEL		1	PH	11											
				2	"	4	152										
				3	"	PH											
	150.9			4	"	PH											
	2.9	STIFF TO FIRM GREY SILTY CLAY WITH SAND AND GRAVEL BECOMING CLAYEY TO SANDY SILT WITH DEPTH. HORIZONTAL LAMINATIONS WITH OCCASIONAL SEAMS OF SAND AND SILT		5	"	PH	150										
				6	"	PH	148										
	147.4			7	"	PH											
6.4	LOOSE GREY SANDY SILT TO SILTY FINE SAND		8	"	3	146											
			9	"	8												
144.7			10	"	31												
9.1	DENSE LAYERED FINE SAND AND SILT																
144.2																	
9.6	END OF HOLE						144										

VERTICAL SCALE  
1:50 (METRIC)

Golder Associates

DRAWN MHW  
CHECKED L.R.

## RECORD OF BOREHOLE 102

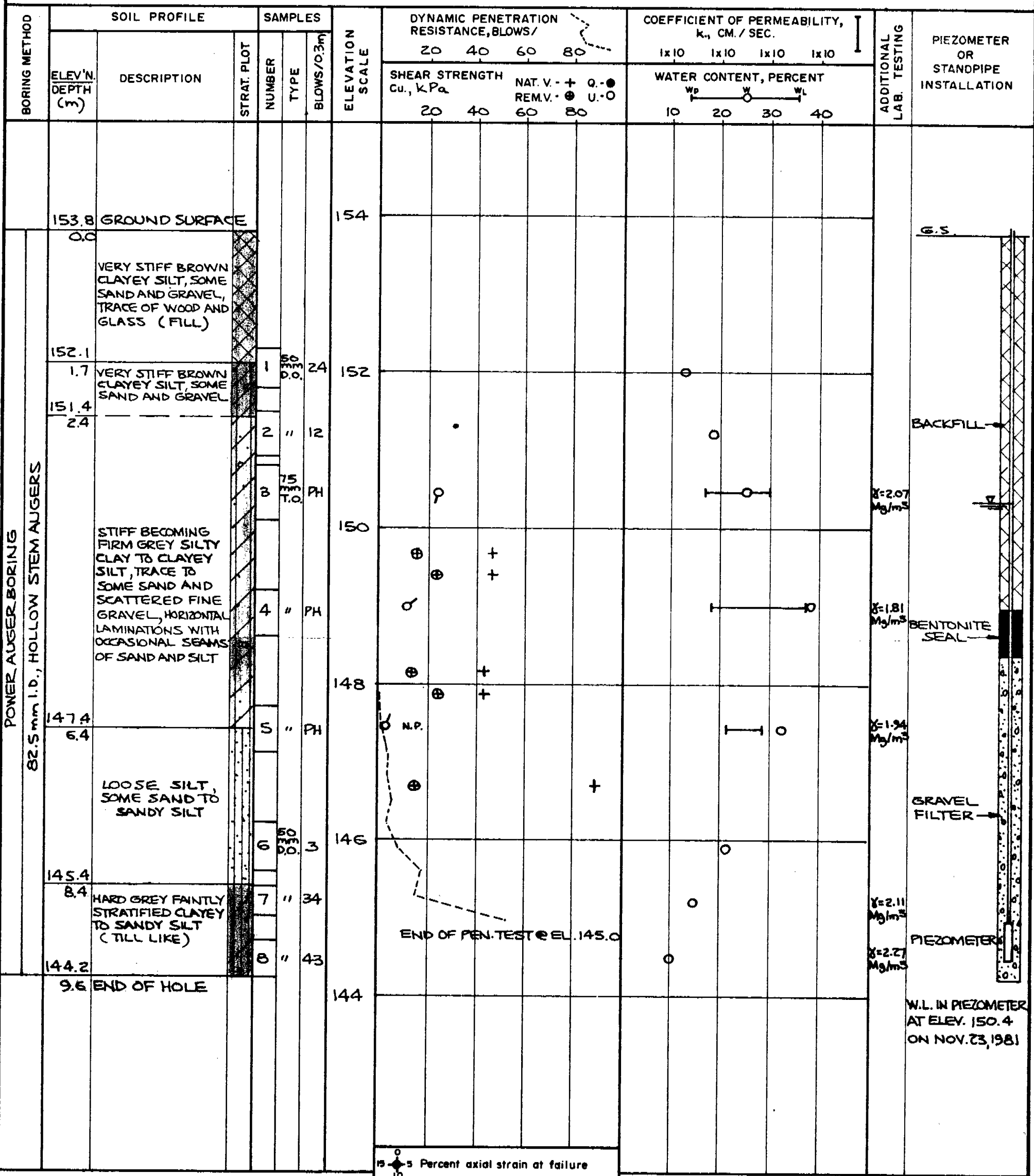
LOCATION See Figure 2

BORING DATE NOV. 10-11, 1981

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm

PENETRATION TEST HAMMER WEIGHT 0.62 kN, DROP 760 mm



# RECORD OF BOREHOLE 103

BORING DATE NOV. 11, 1981

DATUM      GEODETIC

PENETRATION TEST HAMMER WEIGHT 0.62 kN DROP 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS /				COEFFICIENT OF PERMEABILITY, K <sub>v</sub> , CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/0.3 m											
								SHEAR STRENGTH Cu., kPa				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
								NAT. V. - + O.-● REM.V. - ● U.-O				W <sub>p</sub> W <sub>L</sub>					
								20	40	60	80	10	20	30	40		
POWER AUGER BORING 82.5 mm I.D., HOLLOW STEM AUGERS	153.9	GROUND SURFACE					154										G.S.
	0.0	VERY STIFF MOTTLED GREY AND BROWN SILTY CLAY, SOME GRAVEL (FILL)															
	153.1																
	0.8	VERY STIFF MOTTLED GREY AND BROWN SILTY CLAY, SOME SAND AND GRAVEL															
	151.5																
	2.4																
		FIRM GREY SILTY CLAY, HORIZONTAL LAMINATIONS WITH OCCASIONAL SEAMS OF SAND AND SILT															BENTONITE SEAL
	148.2																
	5.7																
	VERY LOOSE TO LOOSE GREY FINE SAND, TRACE TO SOME SILT																GRAVEL FILTER
145.6																	
8.3	HARD GREY CLAYEY TO SANDY SILT, SOME GRAVEL (SILT TILL)																
144.3																	
9.6	END OF HOLE						144										W.L. IN PIEZOMETER AT ELEV. 148.0 ON NOV. 11, 1981 AND AT ELEV. 150.3 ON NOV. 12, 1981
								END OF PEN. TEST @ EL. 144.4									

## Golder Associates

DRAWN MHW  
CHECKED L.R.



ORING DATE NOV. 11-12, 1981

LOCATION See Figure Z

DATUM      GEODETIC

SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm

PENETRATION TEST HAMMER WEIGHT 0.62kN, DROP 760 mm

[illegible]

VERTICAL SCALE  
1:50 (METRIC)

## Golder Associates

DRAWN MHW  
CHECKED L.R.

RECORD OF BOREHOLE 105

LOCATION See Figure 2

BORING DATE NOV. 12, 1981

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm

PENETRATION TEST HAMMER WEIGHT 0.62 kN, DROP 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/				COEFFICIENT OF PERMEABILITY, $k_v$ , CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/0.3m		20 40 60 80				1x10 1x10 1x10 1x10					
								SHEAR STRENGTH $C_u$ , KPa		NAT. V. + REM. V. -		Q. - U. -		WATER CONTENT, PERCENT			
								20 40 60 80						10 20 30 40			
POWER AUGER BORING 82.5 mm I.D., HOLLOW STEM AUGERS		EXCAVATED GROUND SURFACE					152									<div><div>G.S.</div><div>BACKFILL</div><div>BENTONITE SEAL</div><div>GRAVEL FILTER</div><div>PIEZOMETER</div><div>CAVED MATERIAL</div><div>W.L. IN PIEZOMETER AT ELEV. 150.2 ON NOV. 23, 1981</div></div>	
	0.0	VERY STIFF BROWN IRREGULARLY LAYERED CLAYEY SILT TO SANDY SILT WITH SOME GRAVEL TO SILTY CLAY		1	30 D.O.	10											
				2	"	10											
	149.9			3	75 TO 100	PH	150										
	1.7																
		FIRM TO STIFF GREY SILTY CLAY, TRACE OF GRAVEL. HORIZONTAL LAMINATIONS WITH OCCASIONAL SEAMS OF SAND AND SILT		4	"	PH	148										
	146.6			5	"	PH											
	5.0																
		VERY LOOSE TO LOOSE GREY FINE SAND, TRACE TO SOME SILT		6	30 D.O.	4	146										
				7	"	3											
				8	"	4											
	144.1						144	END OF PEN. TEST @ EL. 144.3									
	7.5	DENSE GREY SANDY SILT TO SILTY FINE SAND		9	"	34											
	143.5																
	8.1	END OF HOLE															
							142										

0 5 10

Percent axial strain at failure

15 10 5 Percent axial strain at failure

VERTICAL SCALE  
1:50 (METRIC)

Golder Associates

DRAWN M.H.W.  
CHECKED L.R.

RECORD OF BOREHOLE 106

LOCATION See Figure 2

BORING DATE NOV. 12-13, 1981

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm

PENETRATION TEST HAMMER WEIGHT — DROP —

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/				COEFFICIENT OF PERMEABILITY, $k_v$ , CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/0.3m											
								SHEAR STRENGTH $C_u$ , kPa		NAT. V. - + Q. - ● REM. V. - ● U. - ○		WATER CONTENT, PERCENT					
								20	40	60	80	10	20	30	40		
POWER AUGER BORING 82.5 mm I.D., HOLLOW STEM AUGERS							152										
	151.5	EXCAVATED GROUND SURFACE															
	0.0	STIFF BROWN TO GREY SILTY CLAY, SOME SAND AND GRAVEL		1		30	2										
				2	"	7											
	149.7			3		75	PH	150									
	1.8																
		FIRM GREY SILTY CLAY, HORIZONTAL LAMINATIONS WITH OCCASIONAL SEAMS OF SAND AND SILT		4	"	PH	148										
	147.0																
	4.5	SOFT GREY SILT TO CLAYEY SILT		5		30	2										
	146.1			6	"	5	146										
	5.4	LOOSE GREY FINE SAND, TRACE OF SILT															
	145.2			7	"	18											
	6.3	COMPACT TO DENSE GREY SILTY FINE SAND															
144.9																	
	6.6 END OF HOLE						144										

0  
5  
10  
Percent axial strain at failure

VERTICAL SCALE  
1:50 (METRIC)

Golder Associates

DRAWN MHW  
CHECKED L.R.

PENETRATION TEST HAMMER WEIGHT — DROP —

DRAWN MHW  
CHECKED L.R.

RECORD OF BOREHOLE 108

LOCATION See Figure 2

BORING DATE NOV. 13, 1981

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 0.62 kN, DROP 760 mm

PENETRATION TEST HAMMER WEIGHT — DROP —

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS /				COEFFICIENT OF PERMEABILITY, $k_v$ , CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH $C_u$ , kPa				WATER CONTENT, PERCENT					
								20	40	60	80	10	20	30	40		
POWER AUGER BORING 82.5 mm I.D., HOLLOW STEM AUGERS	155.0	GROUND SURFACE															
	0.0	VERY STIFF DARK BROWN CLAYEY SILT, SOME ORGANIC MATTER (TOP SOIL)															
	154.5																
	0.5																
		HARD BROWN SILTY CLAY, SOME SAND AND GRAVEL		1	SO	27											
	152.3																
	2.7			2	"	8											
				3	"	5											
		FIRM TO STIFF GREY SILTY CLAY, TRACE TO SOME SAND AND GRAVEL, HORIZONTAL LAMINATIONS WITH OCCASIONAL SEAMS OF SAND AND SILT		4	PH												
				5	"	PH											
	148.1						148										
	6.9	FIRM GREY CLAYEY SILT TO SANDY SILT TRACE OF GRAVEL		6	PH	4											
	147.0			7	"	13											
	146.8	COMPACT GREY FINE SAND															
	8.2	END OF HOLE					146										

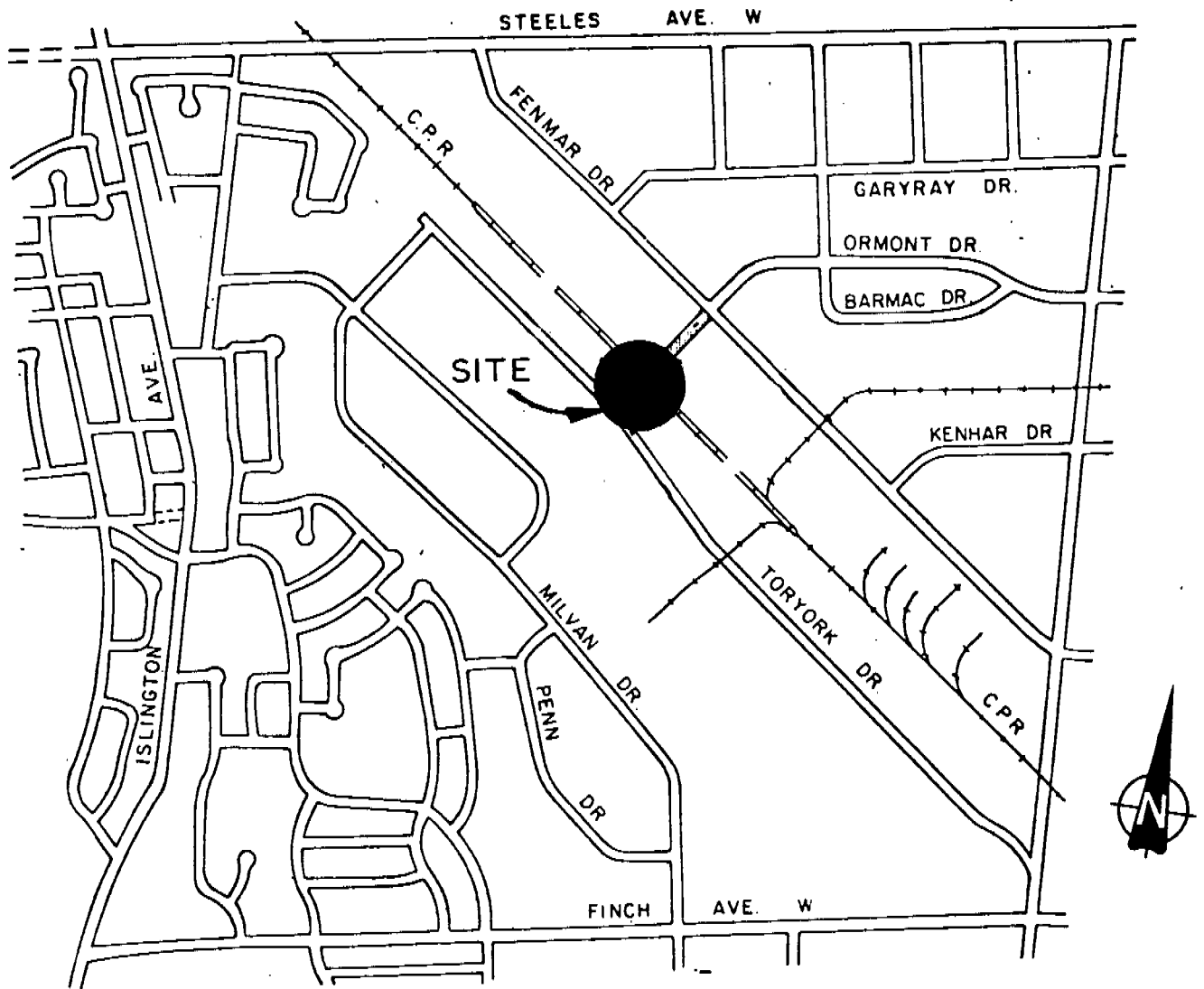
W.L. IN PIEZOMETER AT ELEV. 152.9 ON NOV. 23, 1981

0 5 10 Percent axial strain at failure

15 0 5 Percent axial strain at failure

# SITE LOCATION PLAN

FIGURE 1



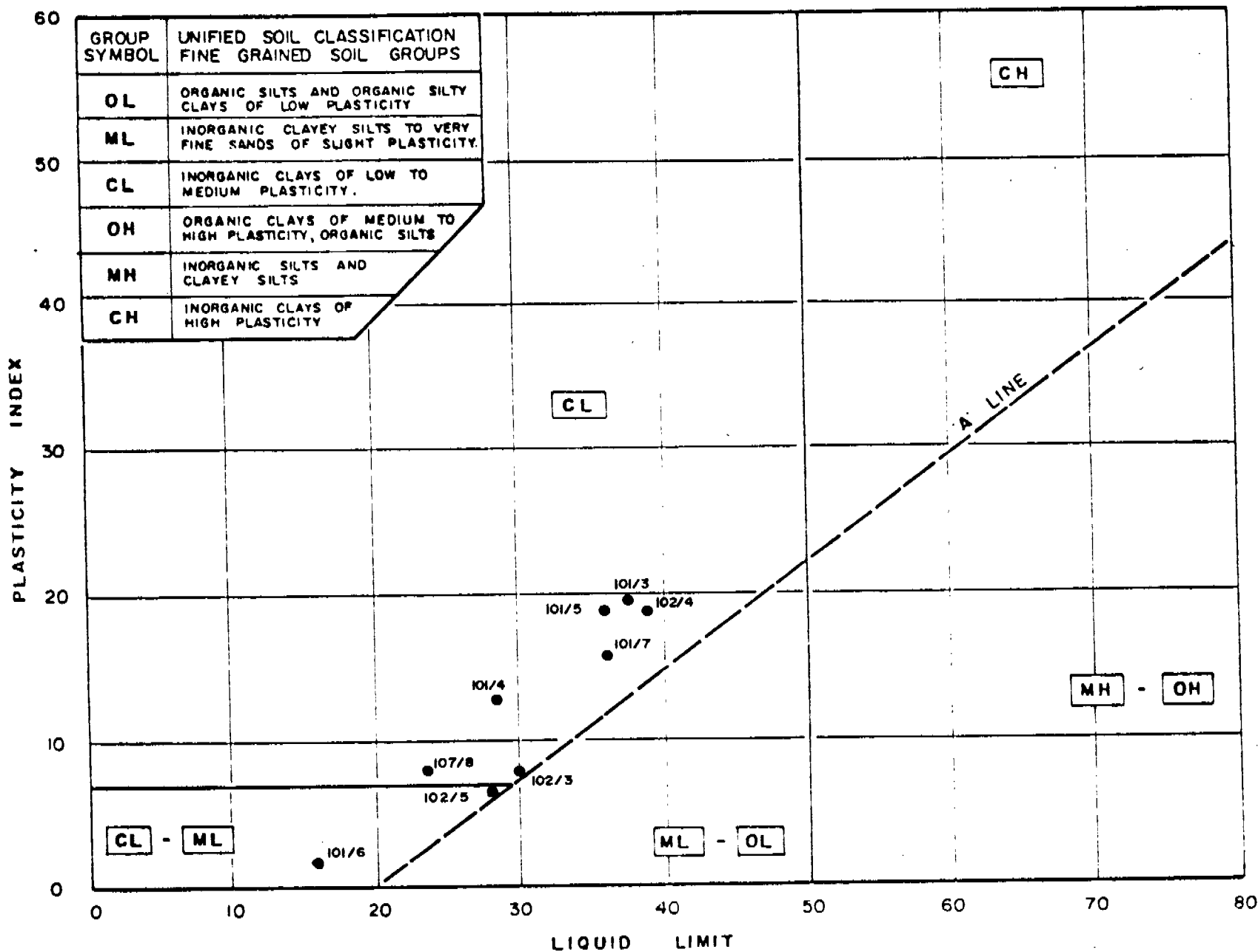
Date NOV 22/81  
Project No. 811-1376

**Golder Associates**

Drawn MHW  
Chkd. J.R.

# PLASTICITY CHART

FIGURE 4



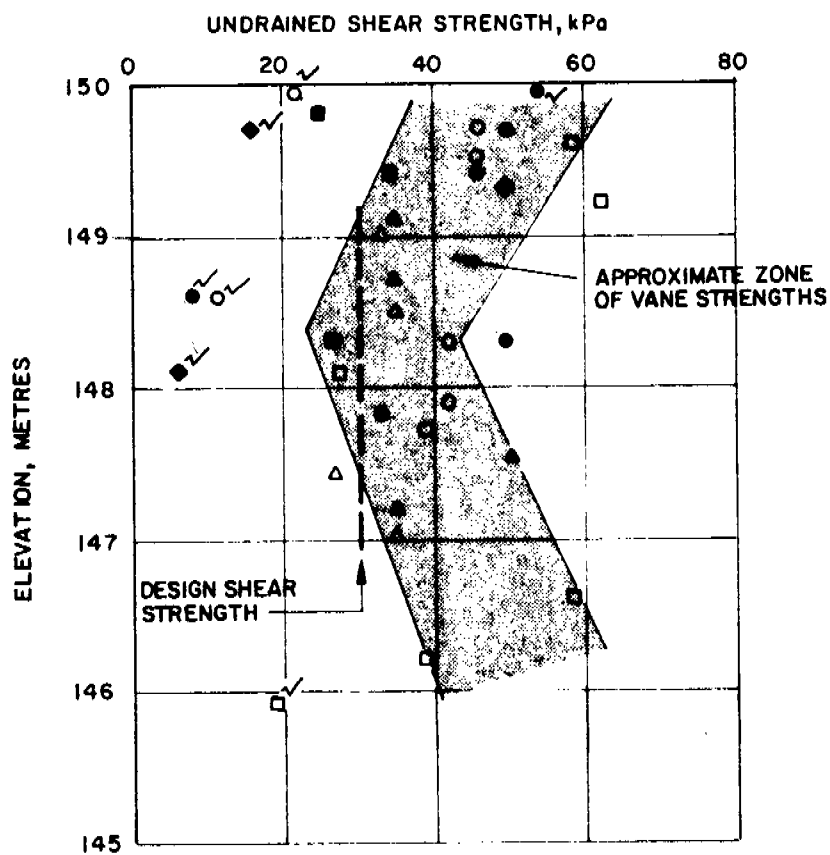
Date DEC 8, 1981  
Project 81-1376

Golden Associates

Drawn MHW  
Ck'd L.R.

# SHEAR STRENGTH PROFILE

FIGURE 5



## LEGEND

●	BOREHOLE	101
○	"	102
■	"	103
□	"	104
▲	"	105
△	"	106
◆	"	107

NOTE: TESTS ARE FROM IN SITU VANES  
OR UNCONFINED LAB. TRIAXIALS  
WHERE MARKED ✓

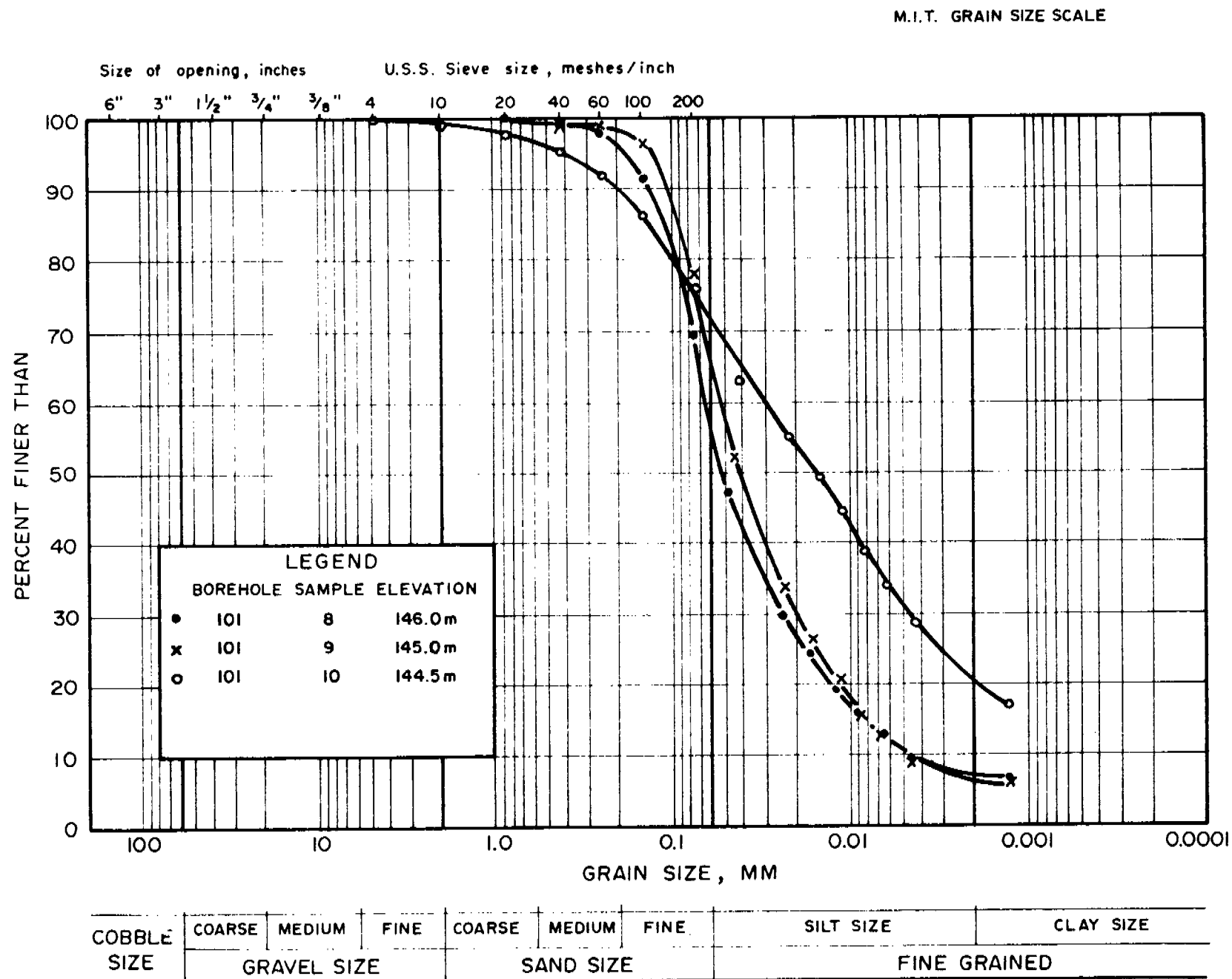
Date DEC. 8, 1981  
Project No. 811-1376

**Golder Associates**

Drawn MHW  
Chkd. J.R.



Golder Associates

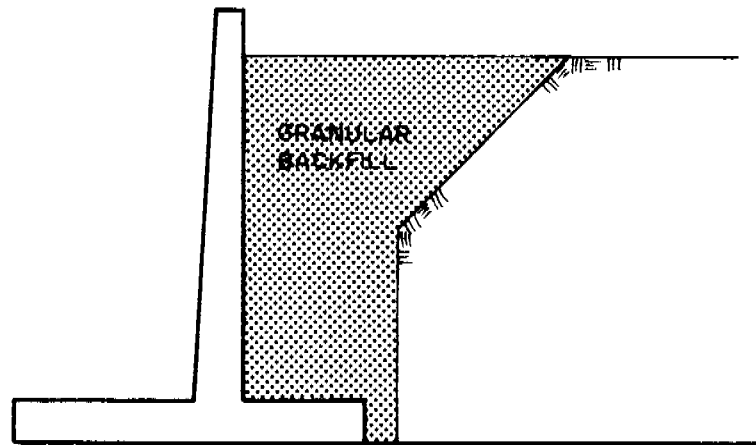


GRAIN SIZE DISTRIBUTION

FIGURE 7

# CONDITIONS FOR EARTH PRESSURE CALCULATIONS

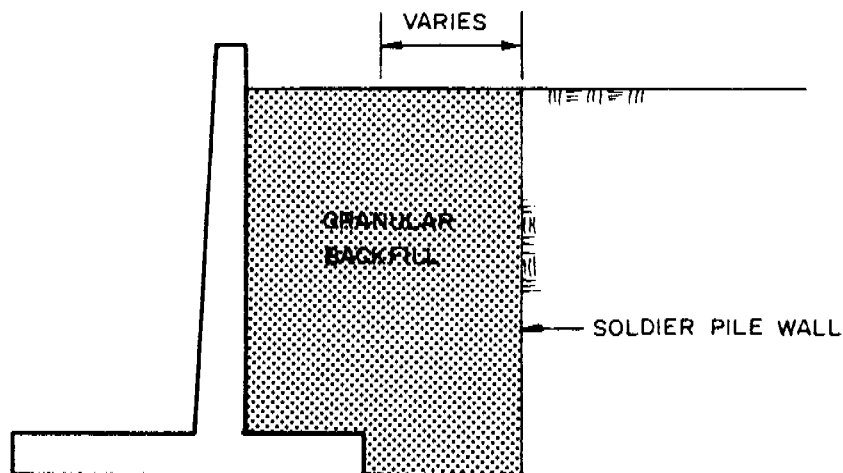
FIGURE 8



DESIGN CONDITION

## REFERENCE

DeLCan DE LEUW CATHER CANADA LTD., ORMONT Dr. CP RAIL  
GRADE SEPERATION, DRAWING No. 24, DATED FEB. 1980



ASSUMED 'AS BUILT' CONDITION

## REFERENCE

S. McNALLY & SONS LTD., ORMONT Dr. CP RAIL GRADE SEPERATION SHORING  
LAYOUT, DRAWING No. C-246-3, DATED OCT. 6, 1981

Date JAN. 6, 1982  
Project No. 811-1376

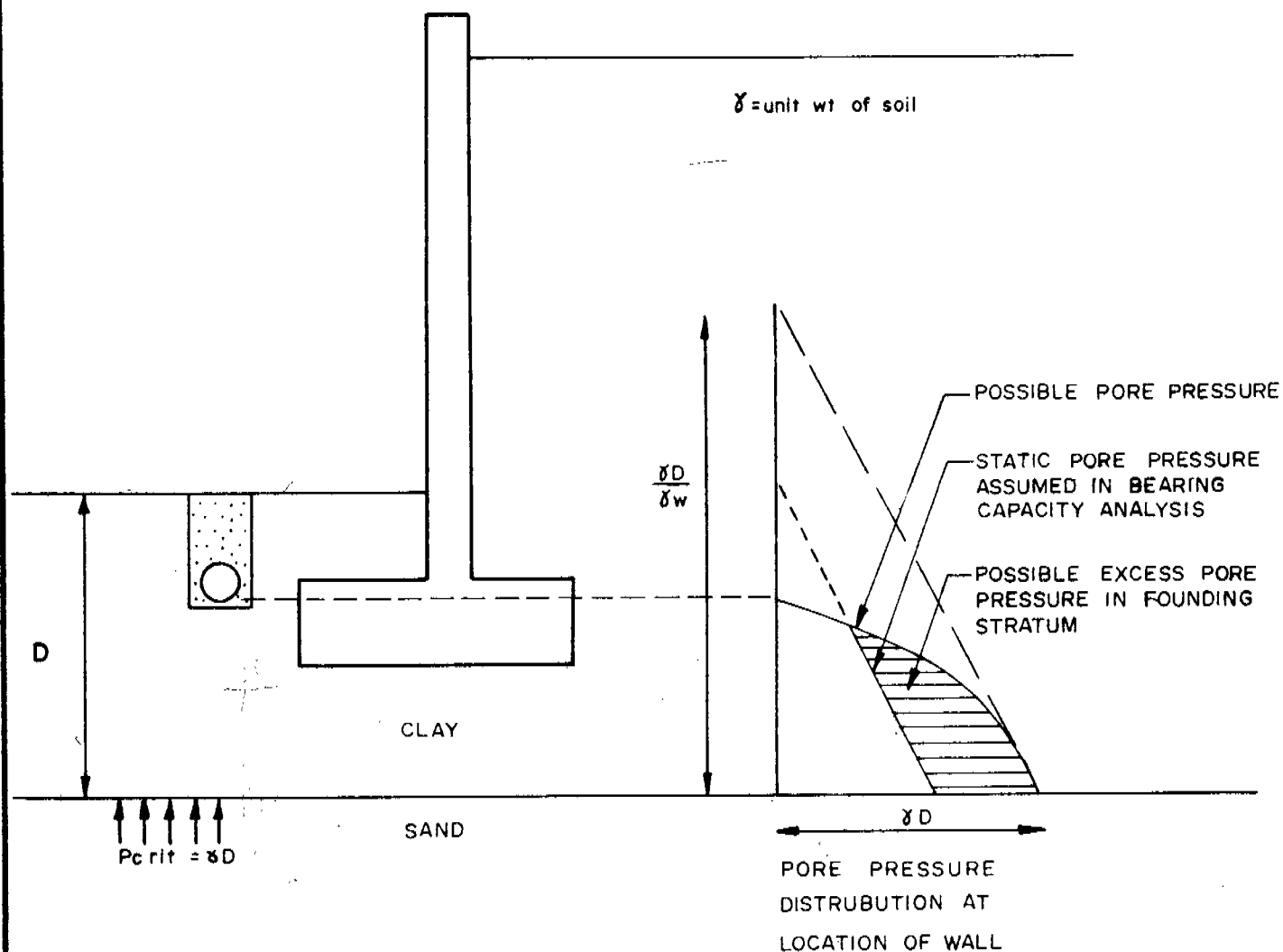
Golder Associates

Drawn SK  
Chkd. *[Signature]*

Form G.A.-D.-4

# LONG TERM PORE PRESSURES IN CASE WHERE DRAINAGE IS DESIGNED TO AVOID HEAVE OF ROAD

FIGURE 9



Date JAN. 6, 1982  
Project No. 811-1376

Golder Associates

Drawn SK  
Chkd. *AKB*

APPENDIX  
STABILITY CALCULATION  
PANEL SE  
ASSUMED AS-BUILT CONDITION

**Golder  
Associates**

SUBJECT **RETAINING WALLS- EARTH PRESSURE**

Job No. **811-1376**

Made by **C. R.**

Date **Dec 29, 1981**

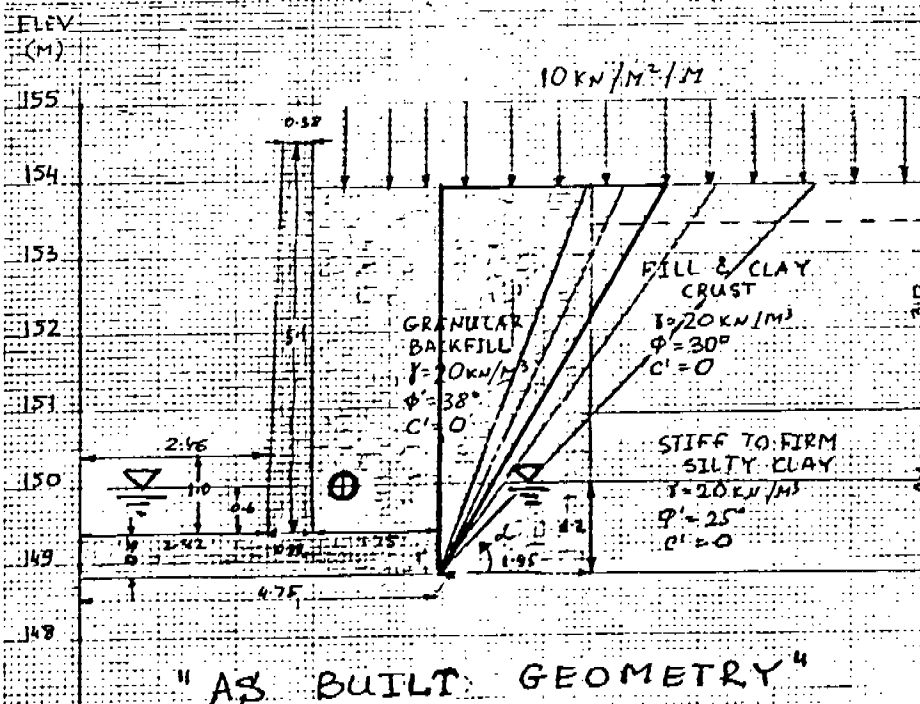
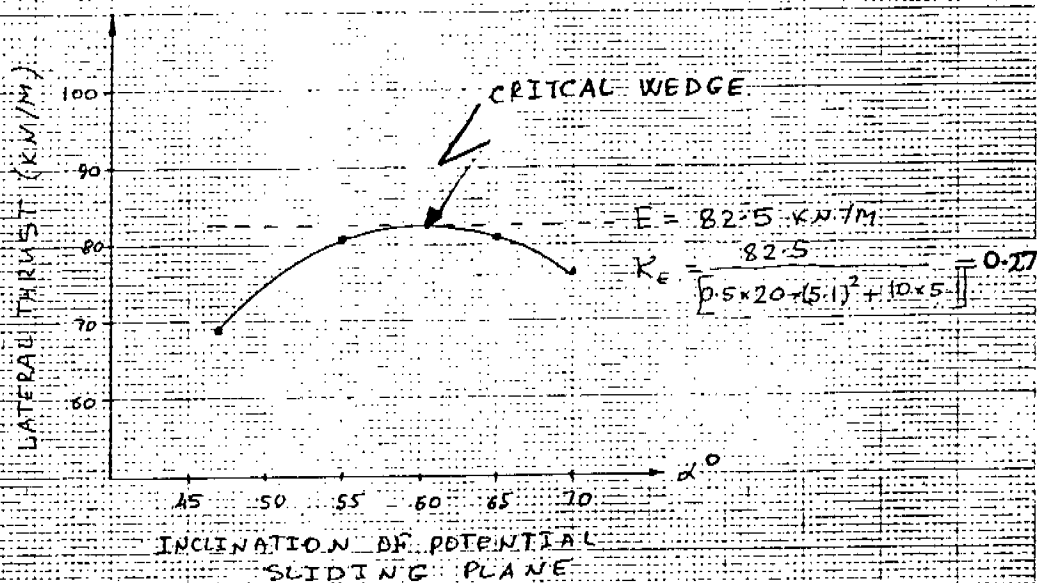
Ref.

Checked **KSM: L**

Sheet **1** of **3**

Reviewed

Sect. S-E



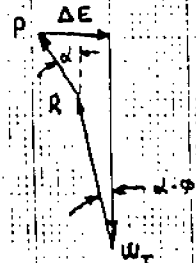
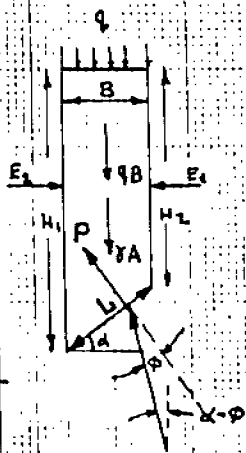
**Golder  
Associates**

SUBJECT **RETAINING WALLS - EARTH PRESSURES**  
 Job No. **811-1376**  
 Rel. \_\_\_\_\_  
 Made by **C.R.**  
 Checked **ESM.L.**  
 Reviewed \_\_\_\_\_  
 Date **Dec 29, 1991**  
 Sheet **2** of **3**

SECTION S-E

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

SLICE No/φ	H1	H2	H	B	A	W	qB	u1	u2	u	cosα	L	P	R <sub>v</sub>	cos(α-φ)	R	SINα	P <sub>h</sub>	SIN(α-φ)	R <sub>h</sub>	ΔE	ΣΔE
1/30°	3.0	0	1.50	2.75	4.12	83.6	27.9	11.5	0	0	0.680	4.10	0	11.5	0.956	11.66	0.733	0	0.291	34.3	34.3	-
2/38°	3.9	3.0	3.45	0.75	2.86	57.2	8.3	6.55	0	0	0.680	1.32	0	6.55	0.987	6.64	0.733	0	0.159	10.6	10.6	-
3/38°	5.1	3.9	4.50	1.42	5.06	100.8	11.2	11.2	11.2	0	0.680	1.65	9.7	10.54	0.987	10.67	0.733	7.1	0.159	17.0	24.1	-
47.12°	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	69.0
1/30°	2.51	0	1.45	1.62	1.86	37.2	16.2	53.4	0	0	0.574	2.82	0	53.4	0.906	58.9	0.819	0	0.423	24.9	24.9	-
2/38°	3.9	2.31	3.10	1.11	3.44	68.8	11.1	79.9	0	0	0.574	1.93	0	79.9	0.956	83.6	0.819	0	0.292	24.4	24.4	-
3/38°	5.1	3.9	4.5	0.84	3.78	75.5	8.4	74.0	11.8	0	0.574	1.46	8.6	74.1	0.956	82.7	0.819	7.0	0.292	24.4	31.1	-
55°	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	80.4
1/30°	0.42	0	0.46	0.43	0.20	4.0	4.3	8.3	0	0	0.423	1.02	0	8.3	0.849	10.1	0.906	0	0.574	5.8	5.8	-
2/38°	3.9	0.92	2.91	1.39	2.45	67.0	13.9	80.9	0	0	0.423	3.29	0	80.9	0.891	90.8	0.906	0	0.454	41.2	41.2	-
3/38°	5.1	3.9	4.5	0.56	2.52	50.4	5.6	56.0	11.8	0	0.423	1.32	7.8	52.7	0.891	59.1	0.906	7.1	0.454	26.8	33.9	-
65°	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	80.9
1/38°	3.9	0	1.45	1.45	2.77	55.4	14.2	69.6	0	0	0.342	4.15	0	69.6	0.848	82.1	0.940	0	0.536	44.5	44.5	-
2/38°	5.1	3.9	4.5	0.49	1.95	34.4	4.4	44.0	11.8	0	0.342	1.27	7.5	41.4	0.848	44.8	0.940	7.1	0.536	25.3	32.0	-
70°	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	76.5



$$\Delta E = P_h + R_h$$

$$W_T = P_v + R_v$$

$$R = \frac{W_T - P \cos \alpha}{\cos(\alpha - \phi)}$$

$$W_T = qA + qB$$

$$P = \bar{u} L$$

**Golder  
Associates**

SUBJECT RETAINING WALLS - EARTH PRESSURE

Job No. 811-1376

Made by C.R.

Date JAN 5, 1991

Ref.

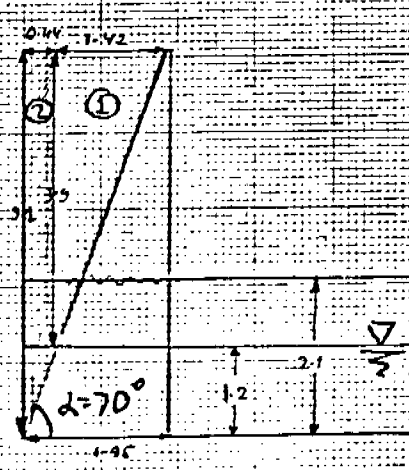
Checked RSMCL

Sheet 2A of 3

Reviewed

COMMENTS TO EARTH PRESSURE CALCULATIONS  
MEANING OF COLUMNS IN CALCULATION TABLES.

- 1 -  $H_1$  - LEFT HEIGHT OF THE SLICE
- 2 -  $H_2$  - RIGHT HEIGHT OF THE SLICE
- 3 -  $H = (H_1 + H_2) / 2$
- 4 -  $B$  - WIDTH OF THE SLICE
- 5 -  $A = BH$
- 6 -  $W = \gamma A$  ( $\gamma = 20 \text{ kN/m}^3$ )
- 7 -  $qB = q \times B$  - WEIGHT OF SURCHARGE ON THE SLICE ( $q = 10 \text{ kN/m}^2$ )
- 8 -  $W_T = W + qB$  - TOTAL WEIGHT OF THE SLICE
- 9 -  $u_1$  - PORE PRESSURE AT THE BASE OF THE SLICE (LEFT)
- 10 -  $u_2$  - PORE PRESSURE AT THE BASE OF THE SLICE (RIGHT)
- 11 -  $\bar{u} = (u_1 + u_2) / 2$
- 12 -  $\cos \alpha$  - COSINE OF SLICE BASE ANGLE
- 13 -  $L = B / \cos \alpha$  - LENGTH OF SLICE BASE
- 14 -  $P = \bar{u} L$  - PORE WATER FORCE AT THE BASE OF THE SLICE
- 15 -  $R_V = W_T \cos \alpha - P \cos \alpha$  - VERT. COMPONENT OF REACTION FORCE
- 16 -  $\cos(\alpha - \phi)$
- 17 -  $R = R_V / \cos(\alpha - \phi)$  - MAGNITUDE OF REACTION
- 18 -  $\sin \alpha$  - SINE OF SLICE BASE ANGLE
- 19 -  $P_H = P \sin \alpha$  - HORIZONTAL COMP. OF WATER FORCE
- 20 -  $\sin(\alpha - \phi)$
- 21 -  $R_H = R \sin(\alpha - \phi)$  - HORIZONTAL COMP. OF REACTION
- 22 -  $\Delta E = P_H + R_H$  - LATERAL THRUST FROM THE SLICE
- 23 -  $\Sigma \Delta E$  - TOTAL LATERAL THRUST





**Golder  
Associates**

**SUBJECT RETAINING WALLS - LOADS**

Job No. 811-1376

Made by L.R.

Date JAN 10, 1982

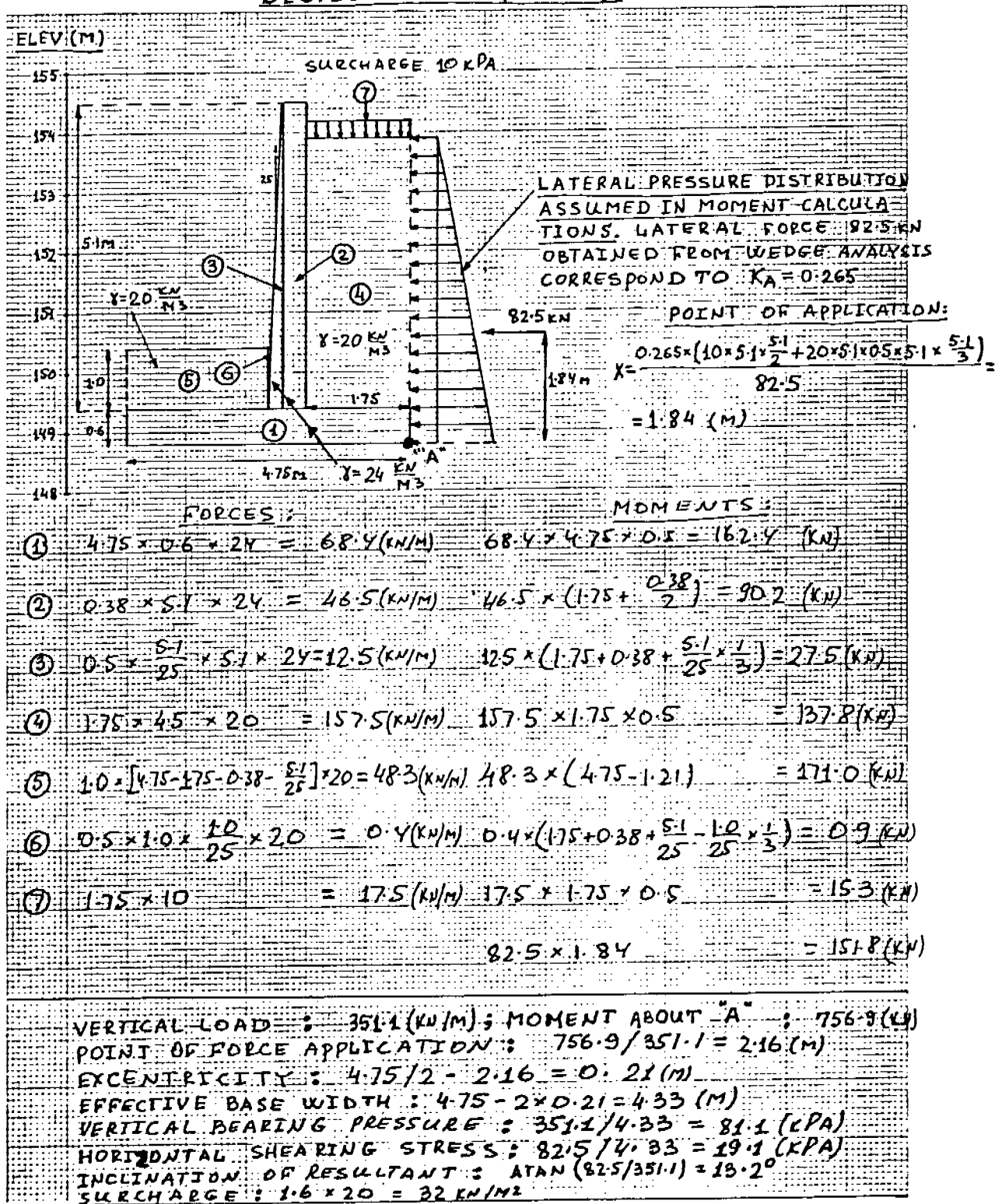
Ref.

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Sheet 1 of 3

Reviewed

**SECTION S-E (LOADS)**



# Golder Associates

## SUBJECT RETAINING WALLS - BEARING CAPACITY

Job No. 811-1376

Made by L. R.

Date JAN 10, 1982

Ref.

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Sheet 2 of 3

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## BEARING CAPACITY FORMULA:

$$q_{ult} = C N_c \cdot \gamma_{c_i} \cdot \gamma_{cd} + q \cdot \gamma_{q_i} \quad (1)$$

C - UNDRAINED SHEAR STRENGTH

 $N_c = 5.14$  - BEARING CAPACITY FACTOR $\gamma_{c_i}$  - INCLINATION FACTOR $\gamma_{cd}$  - OVERBURDEN DEPTH FACTOR

q - SURCHARGE

 $\gamma_{q_i}$  - SURCHARGE INCLINATION FACTOR

## INCLINATION FACTORS:

$$\gamma_{c_i}^{(1)} = \left[ 1 - 2 \times \frac{19.1}{5.14 \times 30} \right] = 0.752 \quad (\text{ref. } (1) \text{ P. 131 equation 3-16a})$$

$$\gamma_{c_i}^{(2)} = 0.5 \left[ 1 - \left( 1 - \frac{19.1}{30.0} \right)^{1/2} \right] = 0.199 \quad (\text{ref. } (2) \text{ P. 119 equation 1, Table 4-3})$$

$$\gamma_{c_i}^{(3)} = \left[ 1 - 1.3 \times \frac{19.1}{81.1} \right] = 0.694 \quad (\text{ref. } (3) \text{ P. 32 equation 3-3})$$

$$\gamma_{c_i}^{(4)} = \left[ 1 - \frac{13.2}{90} \right]^2 = 0.728 \quad (\text{ref. } (4) \text{ Part 2 p. 23})$$

## OVERBURDEN DEPTH FACTORS:

$$\gamma_{cd}^{(1)} = 1 + 0.4 \frac{1.6}{4.75} = 1.135 \quad (\text{ref. } (1) \text{ P. 133 equation 3-27})$$

$$\gamma_{cd}^{(2)} = 0.4 \times \frac{1.6}{4.75} = 0.135 \quad (\text{ref. } (2) \text{ P. 118 equation 1, Table 4-3})$$

$$\gamma_{cd}^{(3)} = 1 + 0.2 \frac{1.6}{4.75} = 1.067 \quad (\text{ref. } (3) \text{ P. 322 equation 3-3})$$

$$\gamma_{cd}^{(4)} = 1.0 \quad (\text{ref } (4) \text{ Part 2 p. 23})$$

## OVERBURDEN INCLINATION FACTORS:

$$\gamma_{q_i}^{(1)} = \gamma_{q_i}^{(2)} = \gamma_{q_i}^{(3)} = 1.0 \quad (\text{ref } (1)-(3))$$

$$\gamma_{q_i}^{(4)} = \gamma_{c_i}^{(4)} = 0.728 \quad (\text{ref } (4) \text{ Part 2 p. 23})$$

**Golder  
Associates**

**SUBJECT RETAINING WALLS - BEARING CAPACITY**

Job No. 811-1376

Made by C.P.

Date JAN 10, 1981

Ref.

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Sheet 3 of 3

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$$q_u^{(1)} = 5.14 \times 30 \times 0.752 \times 1.135 + 32 \times 1.0 = 163.6 \text{ (kPa)} \quad FS = \frac{163.6}{81.1} = 2.02$$

$$q_u^{(2)} = 5.14 \times 30 \times (1 - 0.199 + 0.135) + 32 \times 1.0 = 176.3 \text{ (kPa)} \quad FS = \frac{176.3}{81.1} = 2.17$$

$$q_u^{(3)} = 5.14 \times 30 \times 0.694 \times 1.067 + 32 \times 1.0 = 146.2 \text{ (kPa)} \quad FS = \frac{146.2}{81.1} = 1.80$$

$$q_u^{(4)} = 5.14 \times 30 \times 0.728 \times 1.0 + 32 \times 0.728 = 135.5 \text{ (kPa)} \quad FS = \frac{135.5}{81.1} = 1.67$$

SLIDING RESISTANCE :

UPLIFT PRESSURE

1. AVAILABLE LATERAL RESISTANCE:  $(351.1 - 9.81 \times 16 \times 4.75) \times 4 \times 25^\circ = 129.0$   
 ACTING LATERAL FORCE: 82.5 kN

FACTOR OF SAFETY (LONG TERM)  $129.0 / 82.5 = 1.56$

2. SHORT TERM FACTOR OF SAFETY:  $\frac{4.75 \times 30}{82.5} = 1.73$

REFERENCES:

① FOUNDATION ENGINEERING HANDBOOK,  
 H.F. WINTERKORN & H-Y FANG, EDITORS  
 VAN NOSTRAND REINHOLD COMPANY, 1975

② FOUNDATION ANALYSIS AND DESIGN,  
 J.E. BOWLES  
 MCGRAW-HILL BOOK COMPANY, 1977

③ FOUNDATION DESIGN  
 W.C. TENG  
 PRENTICE-HALL, INC., 1962

④ CANADIAN FOUNDATION ENGINEERING MANUAL  
 CANADIAN GEOTECHNICAL SOCIETY, 1978

\* FOR BEARING CAPACITY FORMULA REF ②, p 120, eq. (4-6a)

GEOTECHNICAL INVESTIGATION  
PROPOSED CP RAIL GRADE SEPARATION  
ORMONT DRIVE  
NORTH YORK, ONTARIO

Prepared for:

DELCAN LIMITED

THE TROW GROUP LIMITED  
Canada, Middle East

Project: T 1625-G  
November 22, 1979

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GEOTECHNICAL INVESTIGATION  
PROPOSED CP RAIL GRADE SEPARATION  
ORMONT DRIVE  
NORTH YORK, ONTARIO

SUMMARY

The report presents the results of a subsurface investigation carried out at the site of a proposed reinforced concrete underpass structure beneath the existing CP tracks at Ormont Drive in North York, Ontario.

One previous boring and nine recent borings advanced to depths of 9.1 to 24.6 m (30 to 80 feet) encountered cohesive fill, very stiff to hard clayey silt till, stiff to firm layered clay, wet stratified silt, wet medium to coarse sand exhibiting sub-artesian water levels, dense silt, very dense sand and shale bedrock.

The wet silt and sand deposits are confined by the overlying firm clay and exhibit a piezometric water level at Elevation 153 m (502 feet) which is above the proposed road grade at Elevation 149.4 m (490 feet) at the underpass.

The proposed bridge abutments and retaining walls supporting the railway embankment should be founded on end-bearing piles driven into the shale bedrock. The retaining walls along the west approach may be supported on conventional spread footings in the firm clay at an allowable bearing pressure of 50 kPa ( 1000 psf), although piles may be needed for the support of the higher sections adjacent to the bridge.

Comments are also given in the report regarding adequate dewatering after partial excavation and prior to full excavation, localized braced excavations for pile caps, earth pressures, permanent drainage requirements and minimum flexible pavement thicknesses.



A supplementary report presenting the results of the pumping test currently underway together with an assessment of the stability of the diverted railway embankment is forthcoming.



## INTRODUCTION

A subsurface investigation has been carried out at the site of a proposed roadway beneath the existing CP tracks at Ormont Drive in North York, Ontario. This investigation was authorized by Mr. J. H. Josselyn of DeLCan Limited, on September 17th, 1979 (DeLCan Project Number 01-234-10).

The purpose of the investigation is to determine the subsurface conditions at the site, and based on these conditions, to provide geotechnical recommendations for the foundation design of the proposed underpass.

## PROJECT DESCRIPTION

It is understood that the existing CP tracks will be detoured along the west side of the right-of-way for a two stage construction of the proposed underpass and an additional rail line. The temporary detour embankment will be sloped at 2 horizontal to 1 vertical, thereby allowing partial removal of the existing embankment and about 1.5 m (5 feet) depth of excavation below the present grade in the area of the proposed bridge abutments.

Adequate dewatering will be carried out prior to excavating to significant depths, in order to prevent basal instability above a confined aquifer underlying the final road grade.

The foundations for the bridge abutments will be carried out in laterally braced excavations, approximately 6.1 m (20 feet) wide by 18.3 m (60 feet) long by 6.7 m (22 feet) deep, comprising a soldier pile and timber lagging or steel sheet piling support system, which will be cut off after the total excavation is complete. The proposed footing base is at about Elevation 148.2 m (486 feet).





After constructing the abutment foundations, the reinforced concrete deck and a short section of high retaining wall supporting the railway embankment will be installed, in a two stage operation such that regular rail service may resume. Thereafter, the existing 1520 mm (60 inch) diameter storm sewer will be diverted northwards behind the proposed north embankment and retaining wall.\*. Excavation will be carried out along the proposed roadway for probable sloped embankments at the east approach and retaining walls at the west approach. The presence of adjacent one-storey industrial buildings along the west approach will require the installation of a temporary shoring system, such as soldier piles and lagging.

The roadway will be a flexible asphalt surface descending to Elevation 149.4 m (490 feet) at the underpass, with deeper trench excavations carried to 1.5 m (5 feet) below the final road level for the installation of a local 300 mm (12 inch) diameter storm sewer.

#### FIELDWORK

The fieldwork for this investigation was carried out between September 11th to 19th, 1979. It consisted of nine boreholes designated by DeLCan Limited, and shown on Drawing 1. The borings were advanced to depths of 9.1 m to 24.6 m (30 to 80 feet).

Disturbed samples were obtained within the subgrade using conventional 35 mm (1 3/8 inch) I.D. split-spoon sampling equipment and standard penetration test methods (A.S.T.M. D1586-C.S.A A 119.1).

Field vane shear strengths were measured in weaker cohesive deposits, and dynamic cone penetration tests were carried out to provide supplementary information on the relative density of granular materials and a qualitative comparison of subsoil stratigraphy.

\* Existing and relocated invert at Elevation 151 m (495.3 feet) at the CP crossing and Elevation 149 m (488.7 feet) at Toryork Road.



Four water samples were taken from the open boreholes for a chemical analysis of the groundwater.

All soil and rock samples were subjected to a visual examination in the field and brought to our laboratory for detailed examination and testing.

The location and ground surface elevation of the boreholes were supplied by DeLCan Limited.

#### SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each boring and the results of laboratory tests carried out on representative samples of the subgrade are given on the borehole logs, Drawings 3 to 11.

The previous subsurface investigation<sup>1</sup> at Borehole 1, and the deep borings (Boreholes 2, 3 and 4) at the underpass encountered varying soil conditions comprising cohesive fill, very stiff to hard clayey silt till at about Elevation 152.7 (501 feet), stiff to firm layered clay at about Elevation 151.8 m (498 feet), wet stratified silt at about Elevation 148.2 m (486 feet), dense to very dense wet sand at Elevation 144.2 m to 148.8 m (473 feet to 488 feet), very dense silt at about Elevation 142.1 m (466 feet), dense sand at about Elevation 136.3 m (447 feet), and shale bedrock at Elevation 133.2 m (437 feet).

Similarly, the shallow borings along the west approach (Boreholes 5 and 6) and the east approach (Boreholes 7, 8, 9 and 10) encountered stiff to hard clayey silt, to silty clay and hard clayey silt till, over wet, fine to coarse grained sand. The sand occurs at about Elevation 145 m (475 feet) at Boreholes 1 and 2, and the surface

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<sup>1</sup>The Trow Group Limited Report Number J 8455, Preliminary Subsurface Investigation, Proposed Grade Separation, Toryork and Fenmar, Weston, Ontario. Prepared for the Borough of North York, April 1975.



risers eastward to about Elevation 152 m (499 feet) at the Boreholes 9 and 10. The sand deposit varies in thickness from 1.5 m (5 feet) at Borehole 1, to 5 m (16 feet) at Boreholes 3 and 4, to at least 3 m (10 feet) or greater at Boreholes 7, 8, 9 and 10.

The hard, clayey silt till and the wet, medium to coarse grained sand are separated by a stiff to firm, stratified clay at the west approach (Boreholes 5 and 6).

Two piezometers installed in the sand (Borehole 4) and slightly above the sand in the wet stratified silt (Borehole 2), and sealed in the overlying clay, exhibited sub-artesian pressures at about Elevation 153 m (502 feet). These levels stabilized after one day at Borehole 2, however the piezometer at Borehole 4 was not read until four days after installation. It is felt that the stabilized water levels occurred within a few hours to one day after drilling, as indicated by wet, flowing conditions in the pervious, medium to coarse grained sand layer. There was significant disturbance and stress relief in the sand, due to groundwater seepage to the open borehole, as indicated by low blow counts of  $N = 0$  to 10 in the standard penetration test, and the need for a sand trap device during sampling in the sand.

The short-term water levels measured in the open boreholes at the completion of drilling are at about Elevation 153 m to 154 m (502 feet to 505 feet). These levels are subject to seasonal fluctuations, consequently higher levels should be anticipated during wet periods of the year.

The results of a chemical analysis of four groundwater samples indicate only traces of sulphate. The pH results vary from 7.6 to 7.8, indicating a slightly alkaline groundwater environment which should be compatible with high strength concrete.



The soil and groundwater conditions vary slightly from those encountered in a previous investigation carried out at the CPR/Steeles Avenue grade separation, approximately 1200 m (3940 feet) to the north-west<sup>2</sup>.

The major differences are that the water-bearing sand deposit was not encountered at Steeles Avenue. The groundwater level was measured at about Elevation 140.9 m (462 feet) in the piezometers installed above bedrock in three borings (Boreholes 24, 28 and 30) and shale bedrock was encountered below Elevation 131.1 m (430 feet).

#### GEOTECHNICAL RECOMMENDATIONS

##### i) Foundations

*Recommendations as per letter Dec 4/79*

It is recommended that the abutments, the adjoining railway fill retaining wall, and the portions of the high west approach retaining wall, that are too heavy to bear on the clay, be founded on end-bearing piles driven to refusal into the shale bedrock. Concrete filled pipe piles, steel H-piles or prestressed concrete piles are suitable, although the H-pile is more likely to reach sound rock.

If driven with sufficient energy to refusal in the rock, the proposed 12 by 53 steel H-pile section will safely carry the design load of 623 kN (70 tons) and probably more than 104 MPa (15 ksi) provided adequate confirming load testing and/or Pile Driving Analyser work is carried out.

As a guide to pile capacity, a 244 mm (9 5/8 inch) O. D. pipe pile with 19 mm (.75 inch) wall was recently driven to bedrock refusal using a rated energy of 35.25 kJ (26,000 ft/lbs) and a final set of 75 blows/inch and was load tested to 2.7 MN (305 tons) with settlements

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<sup>2</sup>The Trow Group Limited Report Number J 5819, Subway and Trunk Sewer, Steeles Avenue West, North York, August 1970



essentially elastic. A lower final set tolerance would have been possible with a heavier hammer and possibly more cushioning. The design stress in the pile was close to 104 MPa (15 ksi). These same capacities should be available for batter piles if driven to an acceptable refusal.

A provision for retapping each pile on the day after installation should be made in addition to the usual checking to ensure that heaving does not occur as a result of driving adjacent piles. A smaller diameter pile with even thicker wall has been found to be an economic solution on other end bearing pile projects.

If there is any doubt that the pile has not reached a positive refusal in the dense sand or in bedrock, its capacity and the input energy of the pile driven should be checked using the Pile Driving Analyser.

Low to intermediate height retaining walls are to be located along the west approach. The height varies from about 4.5 m (14.8 feet) near the underpass to 1.5 m (5.0 feet) at the west end near Boreholes 5 and 6. Except for the heavier sections that must be supported by piles, the lower sections may be supported on conventional spread footings founded at upper levels in the stratified, stiff to firm clay utilizing an allowable bearing pressure of 50 kPa (~1,000 psf). The clay has a thin desicated crust which rapidly decreases to a shear strength of 25 kPa (~500 psf).

All footings exposed to seasonal freezing conditions must have at least 1.2 m (4 feet) of soil cover as frost protection.

Provided that the subgrade beneath the spread footings for the retaining walls is not loosened due to construction activity or surface run-off, the total settlements of the proposed footings, imposing the allowable bearing pressures are expected to be less than 25 mm (1 inch).



Although the differential settlements between the structures supported by piles and by spread footings will be minor, some provision must be made for flexible joints between the structural elements.

Typical footing bases should be inspected by a geotechnical engineer from our office to confirm the bearing capacity of the soil prior to the placement of formwork, steel and concrete.

ii) Excavations

The proposed roadway will descend about 6.1 m (20 feet) below the existing grade to about Elevation 149.4 m (490 feet) at the underpass structure, with slightly deeper excavations for the local storm sewer to drain the subway. The finished grades at the ends of the retaining walls are at about Elevation 151.8 m (498 feet) at the west approach and Elevation 155.5 m (510 feet) at the east approach.

As previously indicated above, the existing storm sewer is to be diverted to the north at the same elevation, i.e. Elevation 151 m (495.3 feet) at the CP crossing and Elevation 149 m (488.7 feet) at Toryork Road.

The excavations for the underpass structure, underground services and the deeper retaining wall sections will penetrate close to or into the sub-artesian pressures in the wet silt and wet sand layers producing basal instability.

Positive dewatering of this sand stratum below the level of any construction is essential before the excavation proceeds below about Elevation 151 m (495 feet). A pump test is underway at the present time to assess the pumping requirements to achieve this state. The results of this testing, together with further dewatering requirements and an assessment of the stability of the temporary CPR relocation, will be given when this work is complete.



A brief study of the available geological information<sup>3</sup> indicates that the sandy beds are possible extensive formations recharged by higher ground to the northeast, therefore significant volumes of water and only localized drawdown may occur in the wet silts and sands confined by the overlying clay and till deposits.

It is proposed that the construction of pile caps will be carried out in braced excavations in order to maximize support to the temporary railway diversion. This is quite acceptable provided that an adequate dewatering system is put into operation as discussed above.

It is recommended that soldier piles and timber lagging, or piling be used for localized braced excavations. The geotechnical requirements associated with the design of this shoring is considered in the appendix.

The area surrounding the pile caps should be backfilled with well-compacted, free draining granular materials which will relieve any groundwater seepage. The approved backfill materials must be adequately filtered against the in-situ materials and positively connected with a free draining granular outlet, such as permanent drainage and pressure relief systems. These will be required beneath roadways when the dewatering system is removed and the sub-artesian pressures return to the observed piezometric levels. The requirements in this regard will be discussed in the pump test report.

In areas where more working space is available, the slopes of temporary excavations must not exceed an angle of 45 degrees to the horizontal in accordance with the Construction Safety Act.

According to the borings, the excavation for the relocation of the existing storm sewers, north of the road, will be entirely in clay

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Watt, A.K., "Pleistocene Geology and Groundwater Resources  
of the Township of North York, York County"  
Ontario Department of Mines, Vol. 64, Part 7, 1955, p. 32



which should be stable during excavation provided that the water-bearing sand below it is depressurized beforehand. Presumably, the new sewer will be laid west of the existing tracks before they are diverted and the remainder completed after the temporary relocation is complete. No excavation problems are envisaged although braced shoring will be needed along the west approach. The geotechnical requirements for shoring are indicated in the appendix.

### iii) Earth Pressures

The retaining walls must be proportioned such that the overturning moment due to the backfill and surcharge is stabilized by the weight of the wall and the weight of soil above the base of the wall. The minimum factor of safety against overturning is:

$$\text{F.S. (overturning)} = 1.5 \text{ for granular backfill}$$

The lateral earth pressure, 'p', at any depth 'h' is given by the following expression:

$$p = k (\gamma h + q)$$

where:  $k = 0.35$  allowing some outward movement of the wall, or  $0.5$  where little or no outward movement is permissible

$\gamma$  = bulk unit weight of free-draining backfill  
assume =  $20.4 \text{ kN/m}^3$

$h$  = depth to point of interest, (m)

$q$  = magnitude of the adjacent surcharge, i.e. tracks, adjacent sloped embankment or private property beside west approaches (kPa). For continuous loading adjacent to the wall, reference should be made to Drawing 17.

The foregoing expression assumes no hydrostatic pressure acting against the walls. A properly installed drainage system consisting of weepholes, backed with pervious filters all the way up the wall, should be provided to prevent the build-up of water pressures during periods of heavy precipitation.





The horizontal component of all lateral earth pressures tends to cause the retaining walls to slide along their base. The common practice requires a minimum factor of safety against sliding as:

$$F.S. \text{ (sliding)} = 1.5$$

Provided that the concrete footing is founded on natural undisturbed soil consisting of stiff to firm stratified clay, the adhesion between the base and soil may be taken as:

$$C_a = 25 \text{ kPa (500 psf)}$$

This expression applies for short to intermediate loading conditions. For long term stability, the clay should be assumed to be a granular material with a sliding resistance equal to:

$$R = N \tan \phi$$

where:  $N$  = normal load acting on the soil at the base  
of the wall and  
 $\phi = 25^\circ$  (estimated)

A factor of safety of 1.3 is suggested for this long term state.

If the factor of safety is difficult to attain, a key may be constructed under the base slab in order to increase the passive resistance. The key is generally located under the stem such that the vertical reinforcing bars may be extended into the key. A passive earth pressure coefficient,  $k_p = 2.5$ , is considered applicable. No allowance for sliding resistance can be made in front of this key. The submerged unit weight  $\gamma = 11 \text{ kN/m}^3$ , should be used below the road surface in front of the key.



iv) Embankment Slopes

The east approach will be constructed with a sloped embankment. The maximum long term slope angle is 2 Horizontal:1 Vertical provided that adequate drainage measures are installed and deep rooted vegetation is maintained. It is important that surface water does not flow in an uncontrolled manner down the embankment slopes.

The west approach will require temporary shoring prior to the installation of the retaining walls. The guidelines for the design of the temporary shoring system are given in the appendix.

v) Roadways

All topsoil, deleterious and loosened materials must be removed to the undisturbed soil and the exposed subgrade thoroughly proof-rolled with a heavy roller in dry weather. Any soft or wet spots revealed by proof-rolling must be excavated and backfilled by compacted granular material.

A permanent and adequate drainage blanket or drain with provision for pressure relief wells will be required along about 122 m (400 feet) of roadway immediately east of the underpass, in order to prevent base heave when the sub-artesian pressures return after construction dewatering. It is felt that permanent relief measures will also be required along the west approach as the wet silt layer may reflect high hydrostatic pressures which will cause uplift of the roadway. This latter relief could be incorporated in the trench bedding of the storm sewer. Suggested relief measures will be discussed in the pump test report.

Paved surface areas must be shaped, crowned and provided with adequate drainage.



Provided this is done, and that all excavations for service trenches are backfilled to a dense state, the following minimum pavement thicknesses are suggested for roadways subjected to heavy truck loads.

TABLE II

Description	Minimum Flexible Pavement Thickness		Material
	mm	inches	
Asphaltic Concrete	110	4½	Hot Mix Asphalt
Basecourse	150	6	MTC Granular 'A'
Sub-base	450	18	MTC Granular 'B'
Note: A system of granular base equivalents may be used along with the experience gained from the performance of existing pavement sections in this area.			

GENERAL COMMENT

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized subsurface conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be greater than has been carried out for design purposes. Therefore, contractors bidding on, or undertaking the works, should decide on their own investigations, as well as their own interpretations of the factual borehole results to draw their own conclusions as to how the subsurface conditions may affect them.

THE TROW GROUP LIMITED

M. G. Sarafinchin, P. Eng.

  
W. A. Trow, P. Eng.

MGS/ph  
Enc.

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Attention: Mr. J. Josselyn



APPENDIX A



TEMPORARY SHORING REQUIREMENTS  
FOR WEST APPROACHES AND SANITARY SEWER DIVERSTION

Where the sides of excavations cannot be sloped at 60 or 45 degrees to the horizontal in natural soil or 45 degrees in fill, support by means of soldier piles and lagging is recommended. Our recommendations are as follows.

EARTH PRESSURE

Depending on the degree of support provided, the estimated earth pressure distribution in stiff to firm clays is illustrated in Drawing 15.

This pressure distribution should be used in the design of soldier piles, rakers and raker footings and sewer trench stuts.

The earth pressure coefficient,  $k$ , may vary within the following limits:

0.2 where small movements are permissible

0.4 where no movement is permissible, i.e. adjacent structures or underground services

The surcharge load,  $q$ , may consist either of adjacent building footings, track loads or construction loads such as a row of concrete trucks.

For the latter surcharge condition, for design, it may be assumed that a row of loaded concrete trucks are parked close to the excavation. This is equivalent to a surcharge of 7.6 kPa (160 psf) acting at a depth of 2.4 m (8 feet) as shown in Drawing 16.

The surcharge load resulting from adjacent footings, and track loads may be estimated from the chart shown in Drawing 18.



The soldier piles act as areas of rigidity which cause arching of the retained soil, consequently from a practical point of view, it is considered that the following lagging board thicknesses are sufficient.\*

50 mm for 2.0 m pile spacing  
75 mm for 2.6 m pile spacing  
100 mm for 3.2 m pile spacing

As an added precaution it is recommended that the board thickness be increased by 25 mm below a depth of 7.6 m, i.e. if 75 mm boards are used to 7.6 m, 100 mm should be used below.

#### SOLDIER PILES

Subsequent to adequate dewatering, soldier piles should preferably be installed in pre-augered holes taken below the base of the deepest excavation. The holes should be backfilled with 20.7 MPa (3000 psi) concrete below the excavation level and with 1 1/2 bag mix above this.

The required depth of penetration,  $d$ , of the soldier pile below the excavation level or any locally deeper footing excavation may be calculated from the expression:

$$R = NB (0.5 \gamma d^2 + 2cd)$$

$R$  = passive resistance required

$N$  = factor to account for the three-dimensional resistance around a buried pile tip, a value of  $B = 2$  is suggested

$B$  = diameter of concrete-filled hole, or width of flange of driven pile

$\gamma$  = unit weight of soil below excavation

$c$  = cohesion of soil below excavation, use 25 kPa (500 psf)

\* See Leonards Foundation Engineering, p. 949



The calculated value of  $d$ , should be increased by 50 per cent to obtain a factor of safety. It is determined by equating active and passive earth pressure moments about the lowest support point.

#### RAKERS

If rakers are used they must be installed while an earth berm still exists in front of the soldier piles up to the level of raker connection. The minimum extent of this berm should be a width of 1.5 m (5 feet) at the top and sloping at 45 degrees from there. Slots may be cut in the berm to allow raker installation. The full design load must be jacked into the rakers before the berm is removed in any locations where no lateral movement can be tolerated, i.e. beside existing perched footings.

#### RAKER FOOTINGS

The permissible bearing pressure,  $q$ , of a raker footing set below excavation level may be computed from:

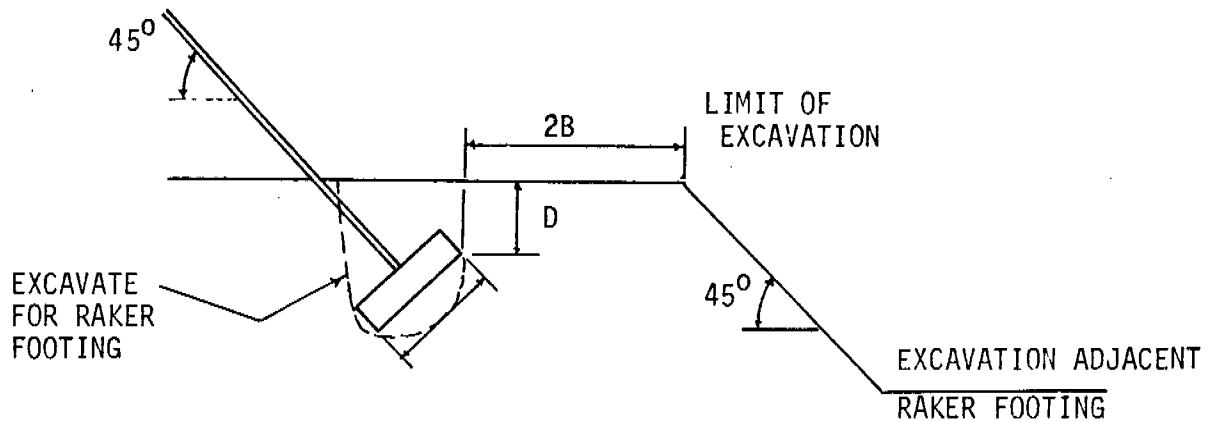
$$q = \frac{NC}{F}$$

C is cohesion of founding soil, use 25 kPa (500 psf)

F is required factor of safety, use 2.0

N is bearing capacity factor depending on inclination of raker and depth of footing cover

i.e., for  $D/B = 1/2$ ,  $N = 4$  with raker at 45 degrees provided footing is kept a distance of  $2 B$  from the edge of any adjacent excavation



The width  $B$  is the largest dimension in the concrete perpendicular to and concentric with the raker within the concrete mass.

#### GENERAL NOTE

The shoring system must be designed for the worst condition that may apply. This is not necessarily at the completion of excavation.

It is essential that lateral movements be monitored. Movements in excess of 12 mm (1/2=inch) indicate the need for more shoring.



## BOREHOLE LOG

JOB No. J 8455

BOREHOLE No. 1

DRAWING No. 2

PROJECT PROPOSED UNDERPASS

LOCATION TORYORK DRIVE

WESTON, ONTARIO

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

2" DIA. CONE

PUSHED

VANE TEST AND SENSITIVITY (S)

LABORATORY PENETROMETER

NATURAL MOISTURE

PLASTIC AND LIQUID LIMIT

UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE

% STRAIN AT FAILURE

HOLE LOCATION AND DATUM SEE DRAWING No. 1

LEG SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT.		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.F.
				20	40	10	20	30	
	FILL-2 feet (possible topsoil) over SILT TILL-compact to dense, brown, some layering, moist	506.5							
		502							130
	CLAY-firm, silty, some sand sizes, grey, moist to wet, some gravel sizes	499	10						138
		490							130
	APPROXIMATE ROAD SURFACE	487	20						133
	SILT-dense to very dense, loose, wet, fine to medium, layered with some sand and clay, running in hole	479	30						108
	SILT TILL-very dense, fine gravel sizes, grey, moist	474							145
	SAND-very dense, silty, wet, dark brown, thin seams of peat	469	40						142
	SILT-very dense, sandy, some thin organic seams, slightly clayey, brown, moist to wet,	454	50						137
	sand seam at 50 feet								
	SAND-medium to coarse, poorly sorted, dirty, very dense, grey, wet (possible bedrock at bottom)	446.5	60						
	END OF BOREHOLE								
	NOTES:								
	1. Borehole advanced uncased by continuous flight auger equipment on April 9th, 1975.								
	2. Piezometer installed at 25 feet; sealed at 3 feet to 5 feet depth.								
	3. Water Level Records:								
	Time W.L. At Hole Open								
	(ft.) To (ft.)								
	On completion 15.5 20.0								
	After 6 days 5.0 piezometer								
	4. Groundwater Chemistry:								
	pH value = 7.8								
	Sulphate Content: (SO <sub>4</sub> )ppm. = traces.								
	5. Dynamic cone test performed 10 feet north of hole on April 18th, 1975.								

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 2

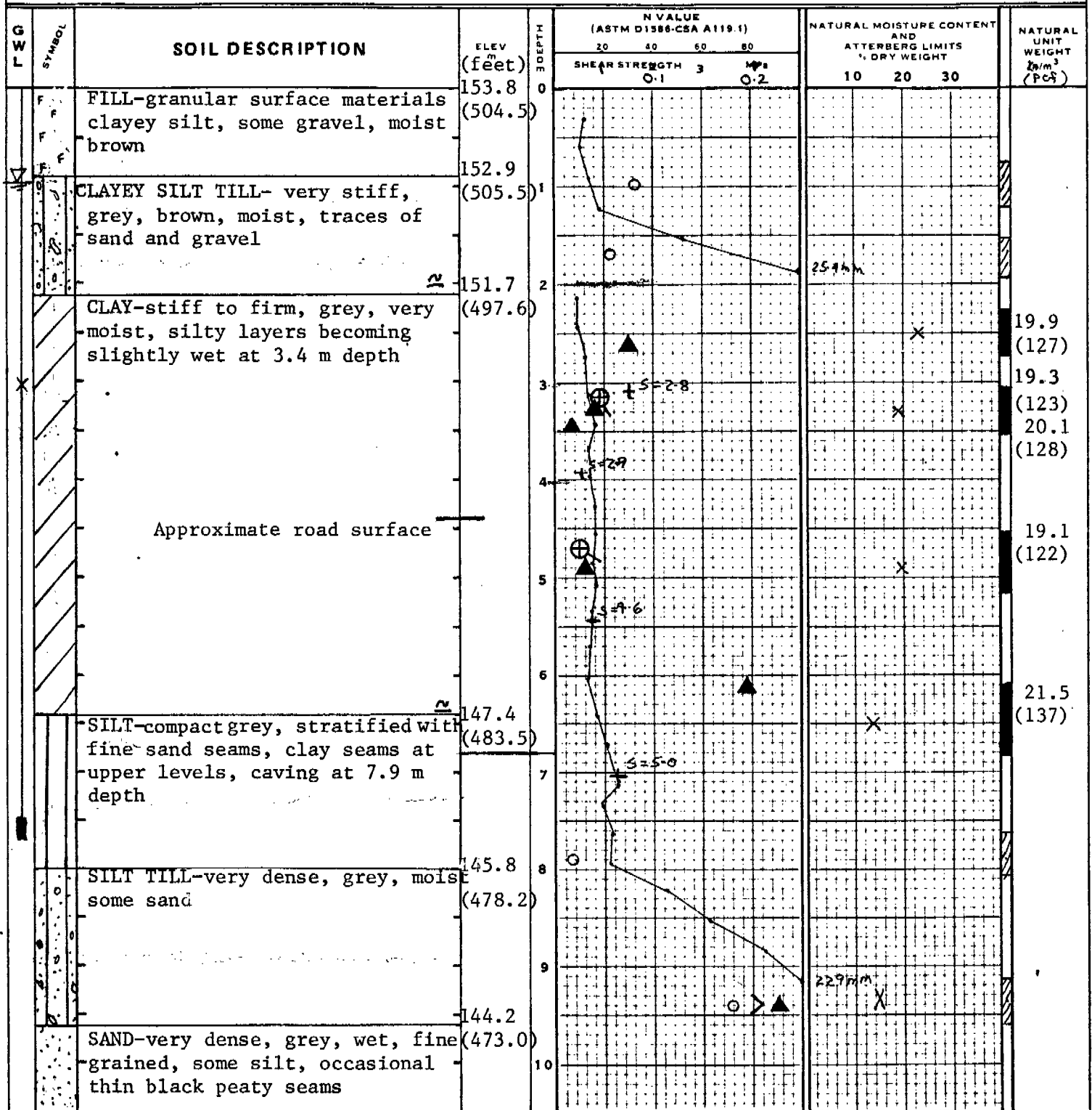
DRAWING No. 3

PROPOSED CP GRADE  
PROJECT SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
SPT (N) VALUE    
DYNAMIC CONE TEST  
SHELBY TUBE    
FIELD VANE TEST  + 5  
LAB VANE TEST  t

NATURAL MOISTURE  x  
PLASTIC AND LIQUID LIMIT  o  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  15 5  
% STRAIN AT FAILURE  10  
PENETROMETER  ▲

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1



# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 2

DRAWING No. 3A

PROJECT PROPOSED CP GRADE SEPARATION

LOCATION ORMONT DRIVE

NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE

SPT (N) VALUE

DYNAMIC CONE TEST

SHELBY TUBE

FIELD VANE TEST

LAB VANE TEST

NATURAL MOISTURE

PLASTIC AND LIQUID LIMIT

UNDRAINED TRIAXIAL AT

OVERBURDEN PRESSURE

% STRAIN AT FAILURE

PENETROMETER

LWG	SYMBOL	SOIL DESCRIPTION	ELEV m (feet)	DEPTH m (feet)	N VALUE (ASTM D1586-CSA A119.1)		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT kN/m <sup>3</sup> (pcf)
					20	40	10	20	30	
					SHEAR STRENGTH					
					MPa					
					0.1		0.2			
		SAND-very dense, grey, wet, fine grained, some silt, occasional thin black peaty seams		10						
				11				X		
			142.2 (466.4)	12				X		
		SILT-very dense, grey, very moist, stratified, with thin fine sand seams, occasional clay seams occasional peaty seams		13						
				14				X		
				15						
				16				X		
				17				X		
		SAND-very dense, grey, wet, medium to coarse grained, some gravel, frequent shale fragments with depth	136.9 (449.0)	18						
				19				X		
		SPOON REFUSAL	135.4 (444.1)	20						



## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to spoon refusal on probable shale bedrock at 18.3 m depth on September 11-12, 1979.
2. Dynamic Cone Penetration test performed 1.8 m west of borehole.
3. Piezometer installed at 7.6 m depth and sealed at 3.0 m depth.
4. Water Level Records:

TIME	WATER LEVEL	HOLE OPEN
on completion	4.3 m	5.2 m
Piezometer	5.9 m	backfilled
1 day	0.9 m	
6 days	0.9 m	
23 days	1.0 m	

5. Chemical analysis of groundwater sample from borehole.

pH - 7.8      Sulphate,  $\text{SO}_4$ , Content = traces

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 3

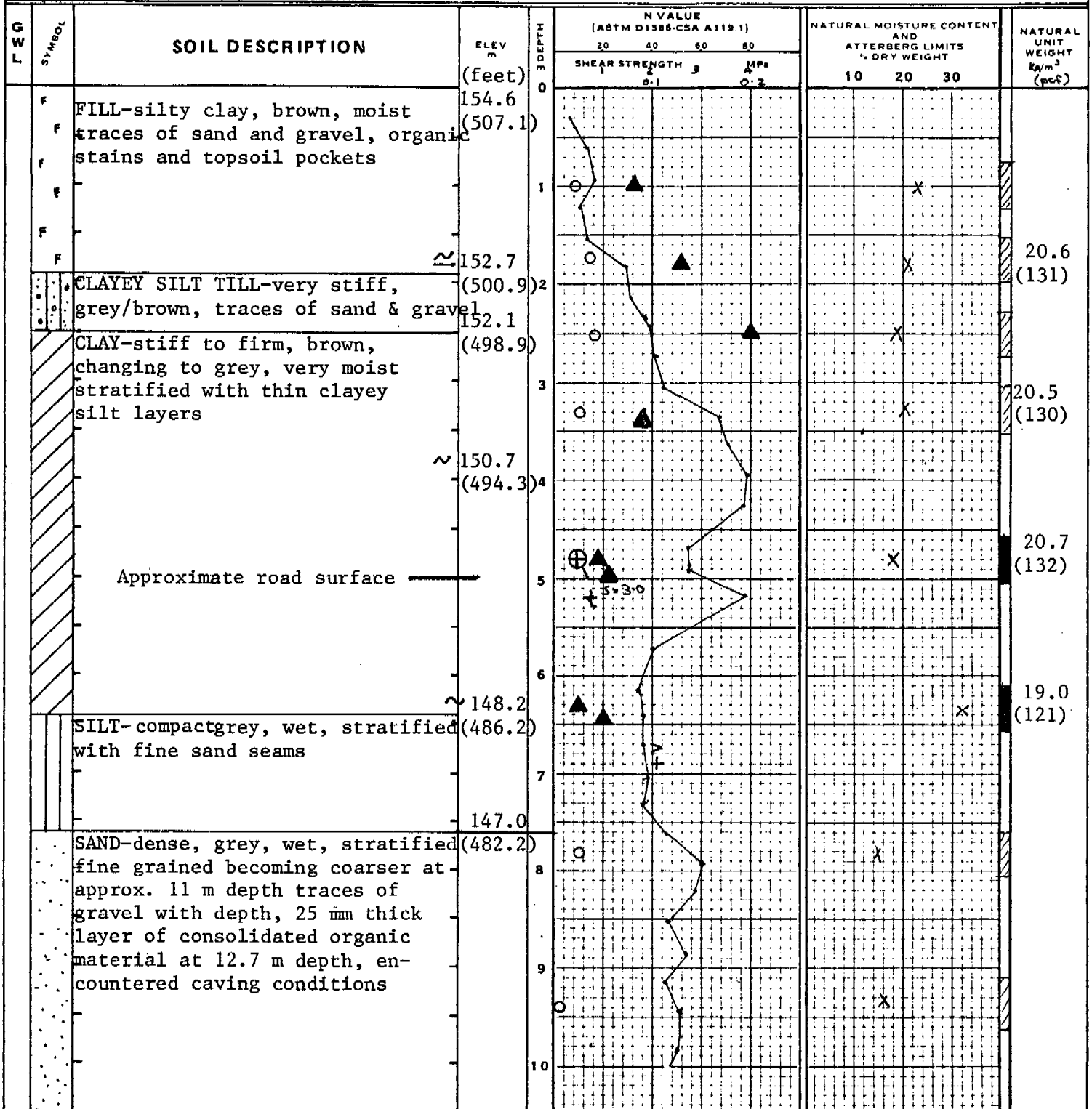
DRAWING No. 4

PROPOSED CP GRADE  
PROJECT SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE  
SPT (N) VALUE    
DYNAMIC CONE TEST  
SHELBY TUBE    
FIELD VANE TEST  + S  
LAB VANE TEST  t

NATURAL MOISTURE  X  
PLASTIC AND LIQUID LIMIT    
UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE  15 0 5  
% STRAIN AT FAILURE  10  
PENETROMETER  ▲



# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 3

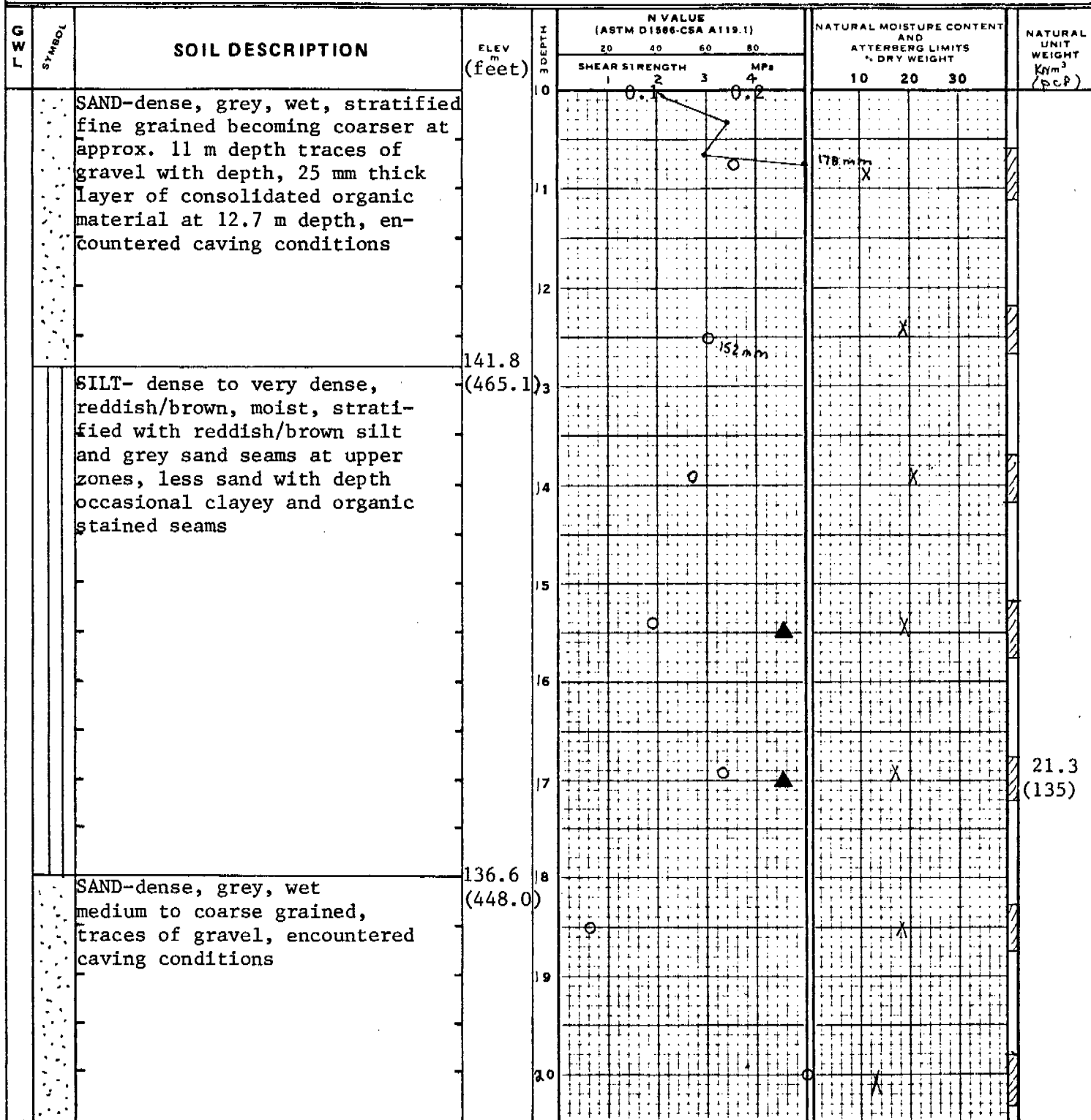
DRAWING No. 4A

PROPOSED CP GRADE  
PROJECT SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE  
SPT (N) VALUE  
DYNAMIC CONE TEST  
SHELBY TUBE  
FIELD VANE TEST  
LAB VANE TEST

NATURAL MOISTURE  
PLASTIC AND LIQUID LIMIT  
UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE  
% STRAIN AT FAILURE  
PENETROMETER



2 ksf 4 ksf



THE TROW GROUP LIMITED

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 3

DRAWING No. 4B

PROPOSED CP GRADE  
PROJECT SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE  
SPT (N) VALUE ☐ ☐  
DYNAMIC CONE TEST ☐  
SHELBY TUBE ☐  
FIELD VANE TEST ☐  
LAB VANE TEST ☐

NATURAL MOISTURE  
PLASTIC AND LIQUID LIMIT ☐  
UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE ☐  
% STRAIN AT FAILURE ☐  
PENETROMETER ☐

LWG	SYMBOL	SOIL DESCRIPTION	ELEV m (feet)	DEPTH m (feet)	N VALUE (ASTM D1586-CSA A119.1)				NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT t/m <sup>3</sup>
					20	40	60	80	10	20	30	
		SAND-dense, grey, wet, stratified fine grained becoming coarser at approx. 11 m depth traces of gravel with depth, 25 mm thick layer of consolidated organic material at 12.7 m depth, encountered caving conditions	133.2 (436.9)	20 21								
		SHALE BEDROCK-highly weathered at surface, interbedded with limestone layers, 12 mm to 125 mm thickness at 21.9 m depth changing to 12 mm to 250 mm and 100 mm average thickness below 23.4 m depth, frequent sub-horizontal bedding joints and occasional sub- vertical joints	130.0 (426.4)	22 23 24								
		END OF BOREHOLE		25								
		NOTES: 1. Borehole advanced uncased by continuous flight auger equipment and NXL double tube core barrel to termi- nation at 24.6 m depth on Sept. 18-19/79. 2. Dynamic Cone Penetration test performed adjacent to borehole. 3. Water Level Records: TIME WATER LEVEL HOLE OPEN on completion 3.9 m 8.1 m 4. Chemical Analysis of groundwater sample from borehole; pH = 7.6 Sulphate, SO <sub>4</sub> , content = traces 5. Details of NXL Core CORE NO. 1. 2. 3. Core Recovery 90 100 100 % ROD % 0 26 42 Average Fracture very 42 58 Spacing (mm) broken Limestone - 12 56 Content %		26 27 28 29 30								





# BOREHOLE LOG

**JOB No.** T 1625-G

**BOREHOLE No.** 4

DRAWING No. 5A

PROPOSED CP GRADE		AUGER SAMPLE	NATURAL MOISTURE					
PROJECT SEPARATION		SPT (N) VALUE	PLASTIC AND LIQUID LIMIT					
LOCATION ORMONT DRIVE		DYNAMIC CONE TEST	UNDRAINED TRIAXIAL AT					
NORTH YORK, ONTARIO		SHELBY TUBE	OVERBURDEN PRESSURE					
HOLE LOCATION AND DATUM SEE		FIELD VANE TEST	% STRAIN AT FAILURE					
DRAWING NO. 1		LAB VANE TEST	PENETROMETER					
FWG	SYMBOL	SOIL DESCRIPTION	ELEV (feet)	DEPTH (feet)	N VALUE (ASTM D1586-CSA A119.1)	SHEAR STRENGTH MPa	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	NATURAL UNIT WEIGHT Kc/m <sup>3</sup> (pcf)
		SAND-grey, wet, fine grained some silt becoming medium to coarse grained with traces of gravel at 12.2 m depth, caving below 8.2 m depth		10	0.1	0.2	10 20 30	
		caving occurred at 10.7 m depth		11				
				12				
			143.2	13				
		SILT-very dense, reddish/brown very moist, stratified with brown silt and grey sand seams, occasional clayey seams, traces of gravel below 18.6 m depth	(469.7)	14				
				15				
				16				
				17				
				18				
				19				
		SAND (see Drawing No. 5B for description)	136.2 (446.7)	20				21.6 (138)

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 4

DRAWING No. 5B

PROJECT PROPOSED CP GRADE SEPARATION

LOCATION ORMONT DRIVE

NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE  
SPT (N) VALUE

DYNAMIC CONE TEST

SHELBY TUBE

FIELD VANE TEST

LAB VANE TEST

NATURAL MOISTURE  
PLASTIC AND LIQUID LIMIT

UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE

% STRAIN AT FAILURE  
PENETROMETER

LWG	SYMBOL	SOIL DESCRIPTION	ELEV m (feet)	DEPTH m (feet)	N VALUE (ASTM D1586-CSA A119.1)	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	NATURAL UNIT WEIGHT t/m <sup>3</sup>
					20 40 60 80 SHEAR STRENGTH MPa	10 20 30	
		SAND-dense, grey, wet, medium to coarse grained, some gravel traces of silt, caving at 20.7 m depth, augers grinding at 22.3 m depth	133.5 (437.9)	20			
				21			
				22			
		PROBABLE SHALE BEDROCK grey, weathered	133.0 (437.9)	23	0.76 2 mm		
		END OF BOREHOLE	133.0 (436.2)	23			
		NOTES: 1. Borehole advanced uncased by continuous flight auger equipment to spoon refusal at 23.0 m depth on September 13, 1979. 2. Dynamic Cone Penetration test performed 2.4 m northeast of borehole. 3. Piezometer installed at 9.2 m depth and sealed at 5.2 m depth. 4. Water Level Records:		24			
		TIME WATER LEVEL HOLE OPEN		25			
		on completion 5.0 m 10.7 m		26			
		Piezometer 8.4 m backfilled		27			
		4 days 2.3 m -		28			
		5 days 2.4 m -		29			
		22 days 2.4 m		30			
		5. Chemical analysis of groundwater sample from borehole. pH = 7.8 Sulphate, SO <sub>4</sub> , Content - traces					

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 5

DRAWING No. 6

PROJECT PROPOSED CP GRADE SEPARATION

LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1

AUGER SAMPLE  
SPT (N) VALUE

DYNAMIC CONE TEST

SHELBY TUBE

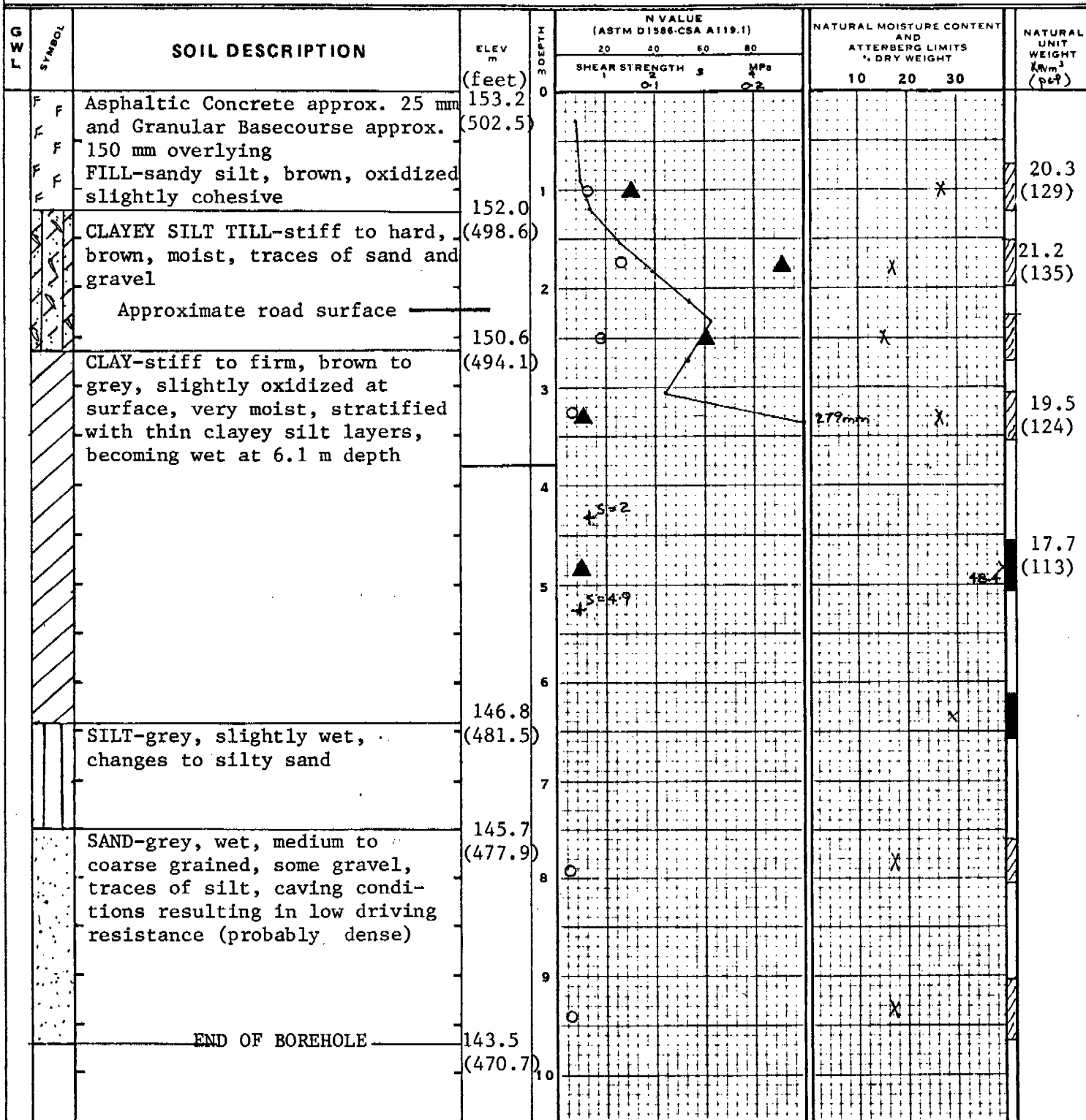
FIELD VANE TEST

LAB VANE TEST

NATURAL MOISTURE  
PLASTIC AND LIQUID LIMIT

UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE

% STRAIN AT FAILURE  
PENETROMETER





## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to termination at 9.6 m depth on September 19, 1979.

2. Dynamic Cone Penetration test performed 3 m north of borehole.

3. Water Level Records

<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	4.9 m	7.5 m

4. Chemical analysis of groundwater sample from borehole.

pH = 7.6      Sulphate,  $\text{SO}_4$ , Content = traces

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 6

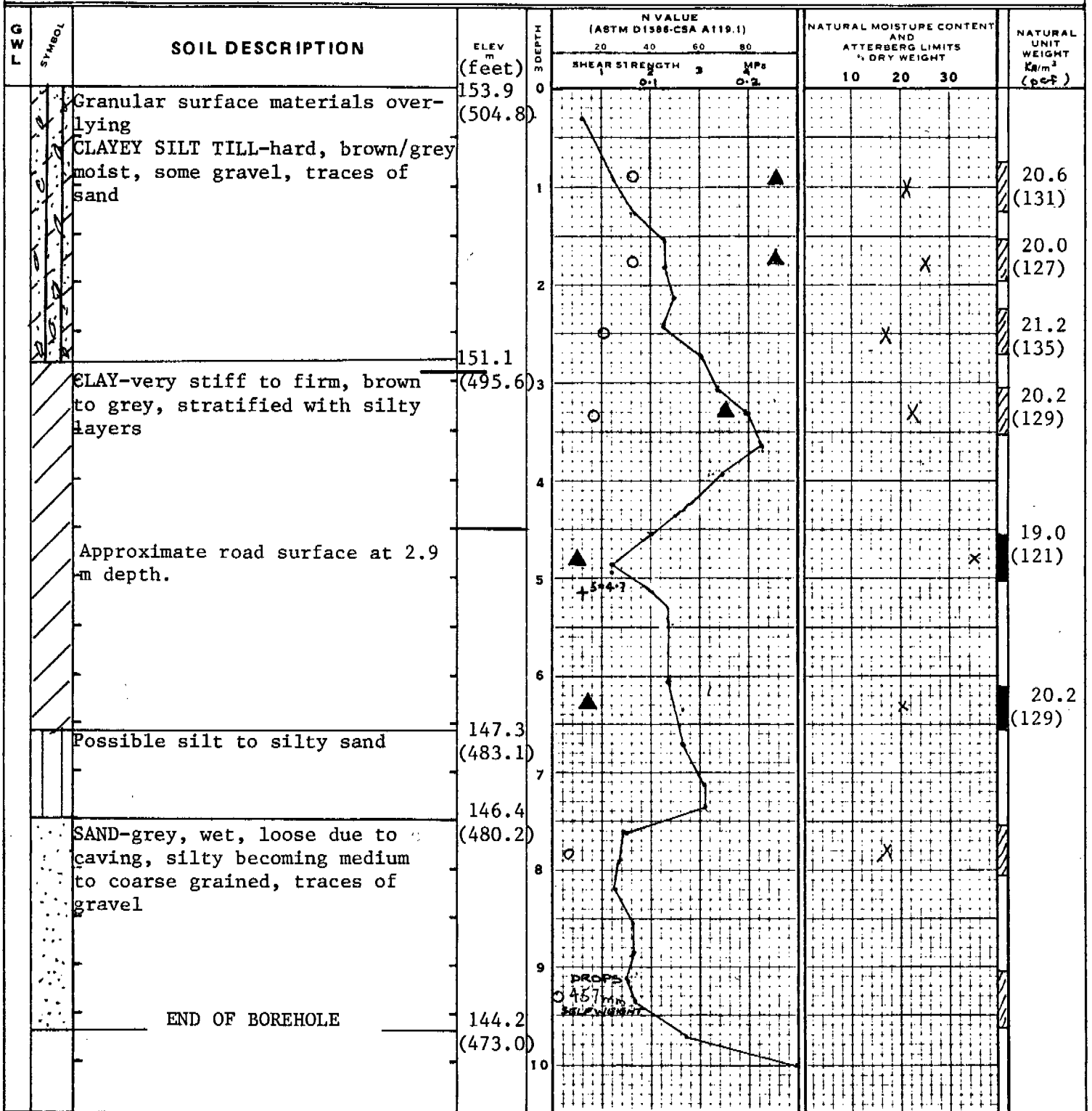
DRAWING No. 7

PROJECT PROPOSED CP GRADE SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
SPT (N) VALUE  
DYNAMIC CONE TEST  
SHELBY TUBE  
FIELD VANE TEST  
LAB VANE TEST

NATURAL MOISTURE PLASTIC AND LIQUID LIMIT  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  
% STRAIN AT FAILURE  
PENETROMETER

HOLE LOCATION AND DATUM SEE DRAWING NO. 1





## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to termination at 9.6 m depth on September 19, 1979.
2. Dynamic Cone Penetration test performed adjacent to borehole.
3. Water Level Records:

<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	4.8 m	7.9 m

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 7

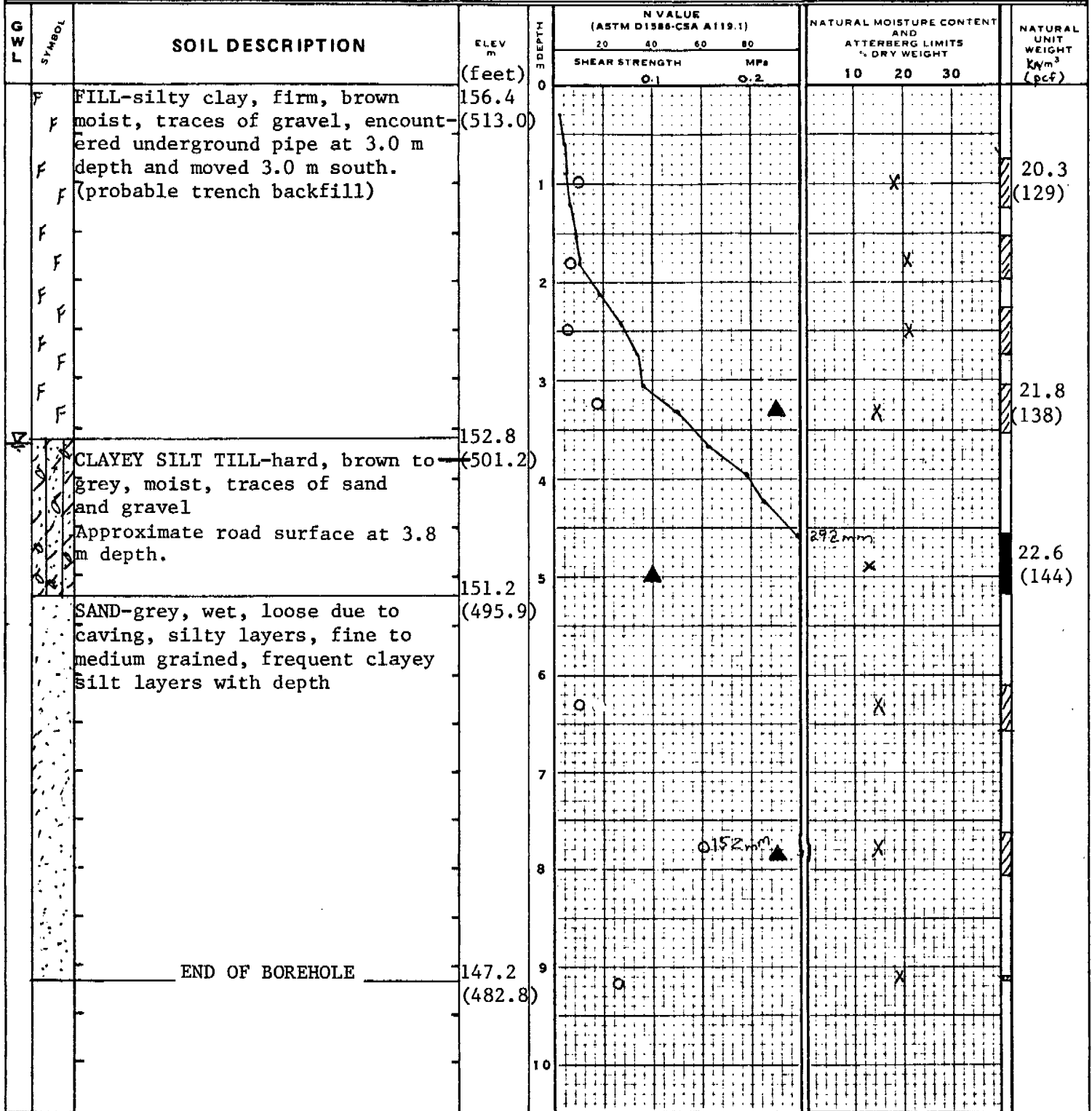
DRAWING No. 8

PROPOSED CP GRADE  
PROJECT SEPARATION  
LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
SPT (N) VALUE    
DYNAMIC CONE TEST  
SHELBY TUBE    
FIELD VANE TEST  + S  
LAB VANE TEST  t

NATURAL MOISTURE  x  
PLASTIC AND LIQUID LIMIT  o  
UNDRAINED TRIAXIAL AT  
OVERBURDEN PRESSURE  15 10 5  
% STRAIN AT FAILURE  15 10 5  
PENETROMETER  ▲

HOLE LOCATION AND DATUM SEE  
DRAWING NO. 1



2 ksf 4 ksf



THE TROW GROUP LIMITED



## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to termination at 9.1 m depth on September 17, 1979.
2. Dynamic Cone Penetration test performed adjacent to borehole.
3. Water Level Records:

<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	4.3 m	5.6 m
1 day	3.5 m	5.4 m
2 days	3.5 m	5.2 m



# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 8

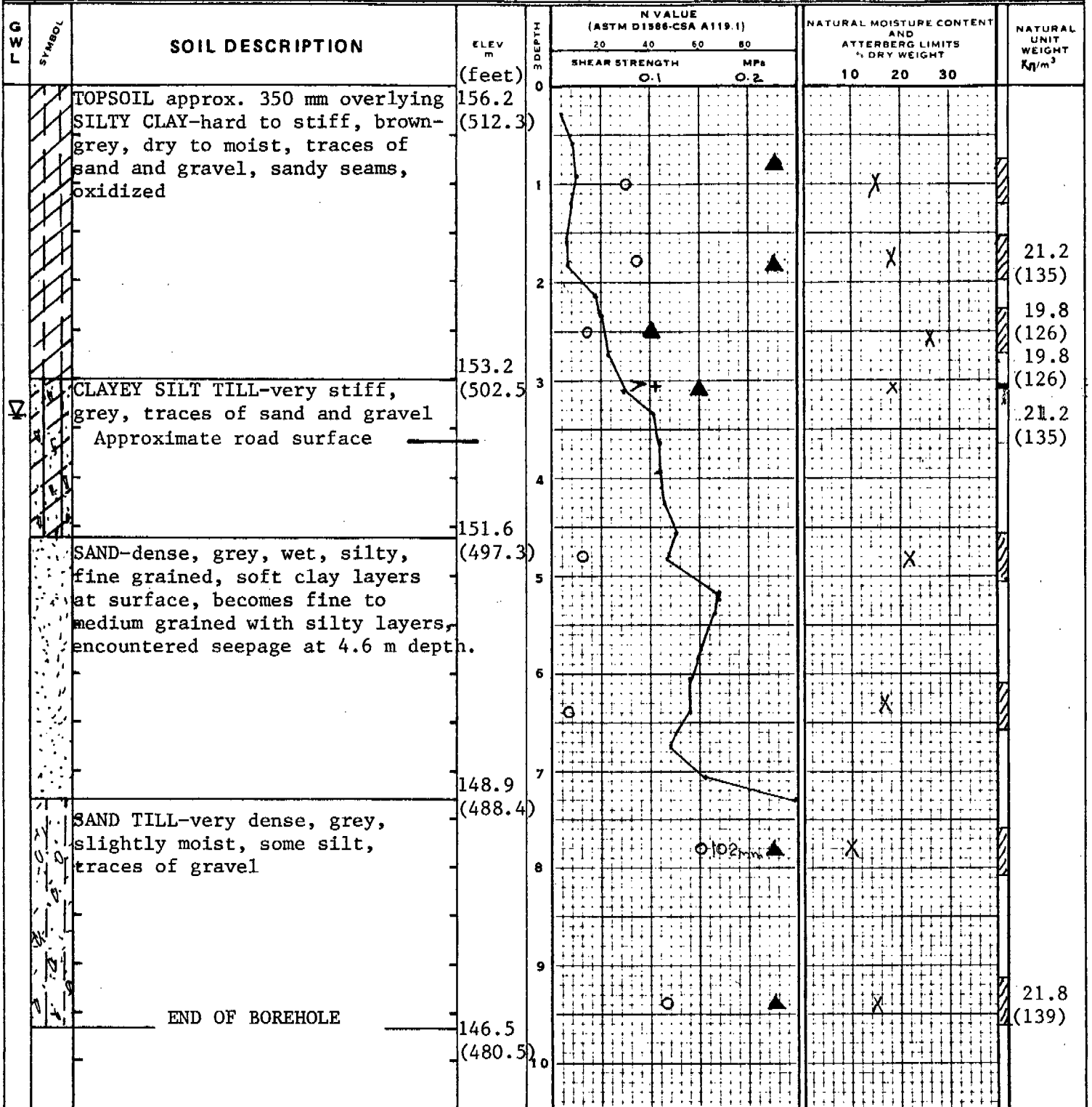
DRAWING No. 9

PROJECT PROPOSED CP GRADE SEPARATION  
 LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
 SPT (N) VALUE    
 DYNAMIC CONE TEST  
 SHELBY TUBE    
 FIELD VANE TEST    
 LAB VANE TEST

NATURAL MOISTURE    
 PLASTIC AND LIQUID LIMIT    
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE    
 % STRAIN AT FAILURE    
 PENETROMETER

HOLE LOCATION AND DATUM SEE  
 DRAWING NO. 1





## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to termination at 9.6 m depth on September 18, 1979.
2. Dynamic Cone Penetration test performed 0.6 m east of borehole.
3. Water Level Records:

<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	8.7 m	8.9 m
end of day	4.1 m	4.9 m
one day	3.3 m	4.4 m

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 9

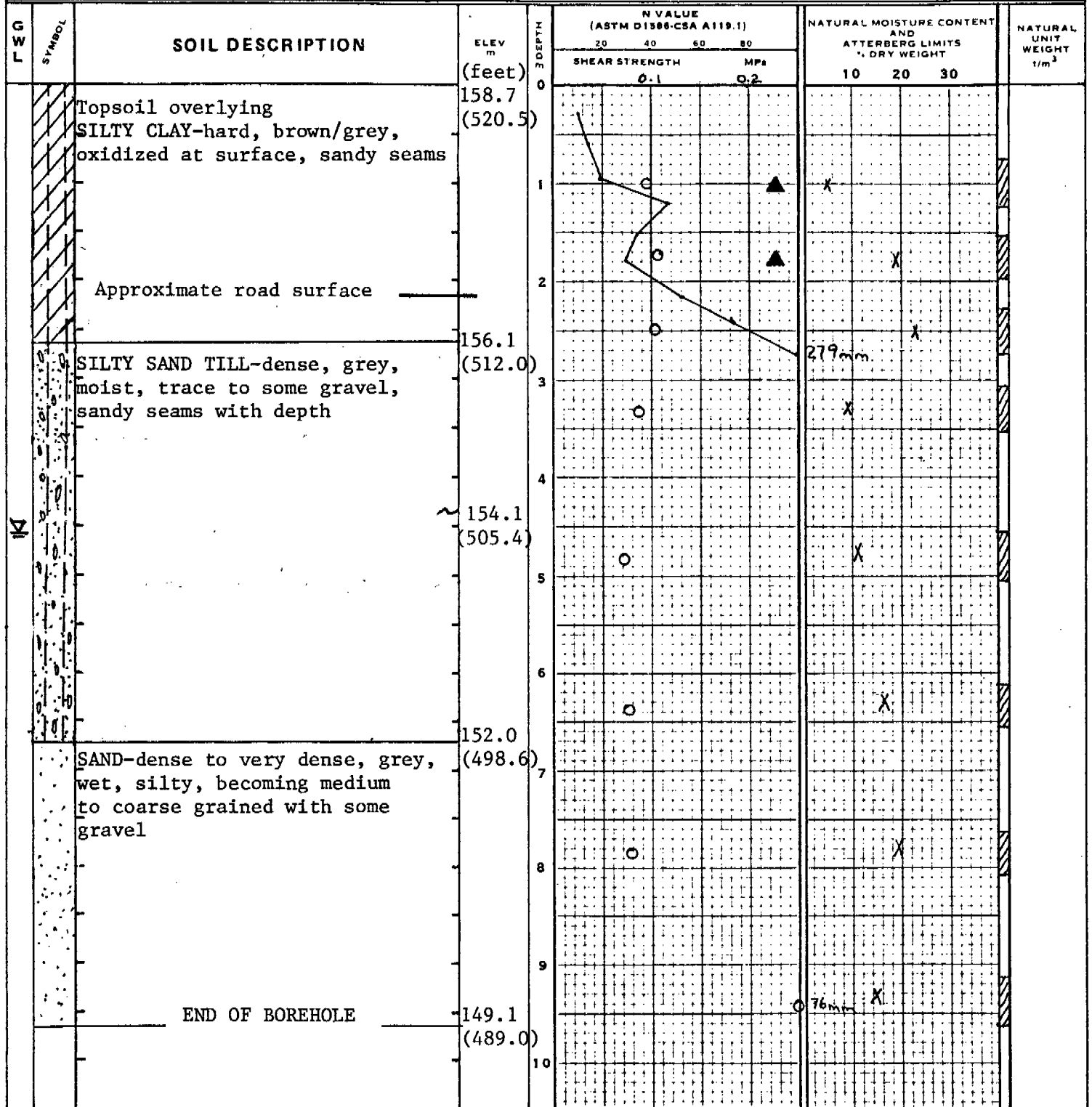
DRAWING No. 10

PROJECT PROPOSED CP GRADE SEPARATION  
 LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
 SPT (N) VALUE    
 DYNAMIC CONE TEST   
 SHELBY TUBE    
 FIELD VANE TEST  + S  
 LAB VANE TEST  ±

NATURAL MOISTURE  X  
 PLASTIC AND LIQUID LIMIT   
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  15 5  
 % STRAIN AT FAILURE  10  
 PENETROMETER

HOLE LOCATION AND DATUM SEE  
 DRAWING NO. 1





## NOTES:

1. Borehole advanced uncased by continuous flight auger equipment to spoon refusal at 9.6 m depth on September 19, 1979.

2. Water Level Records:

<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	5.0 m	5.5 m
1 hour	4.6 m	5.0 m

3. Chemical analysis of groundwater sample from borehole.  
pH = 7.8 Sulphate,  $\text{SO}_4$ , Content = traces

# BOREHOLE LOG

JOB No. T 1625-G

BOREHOLE No. 10

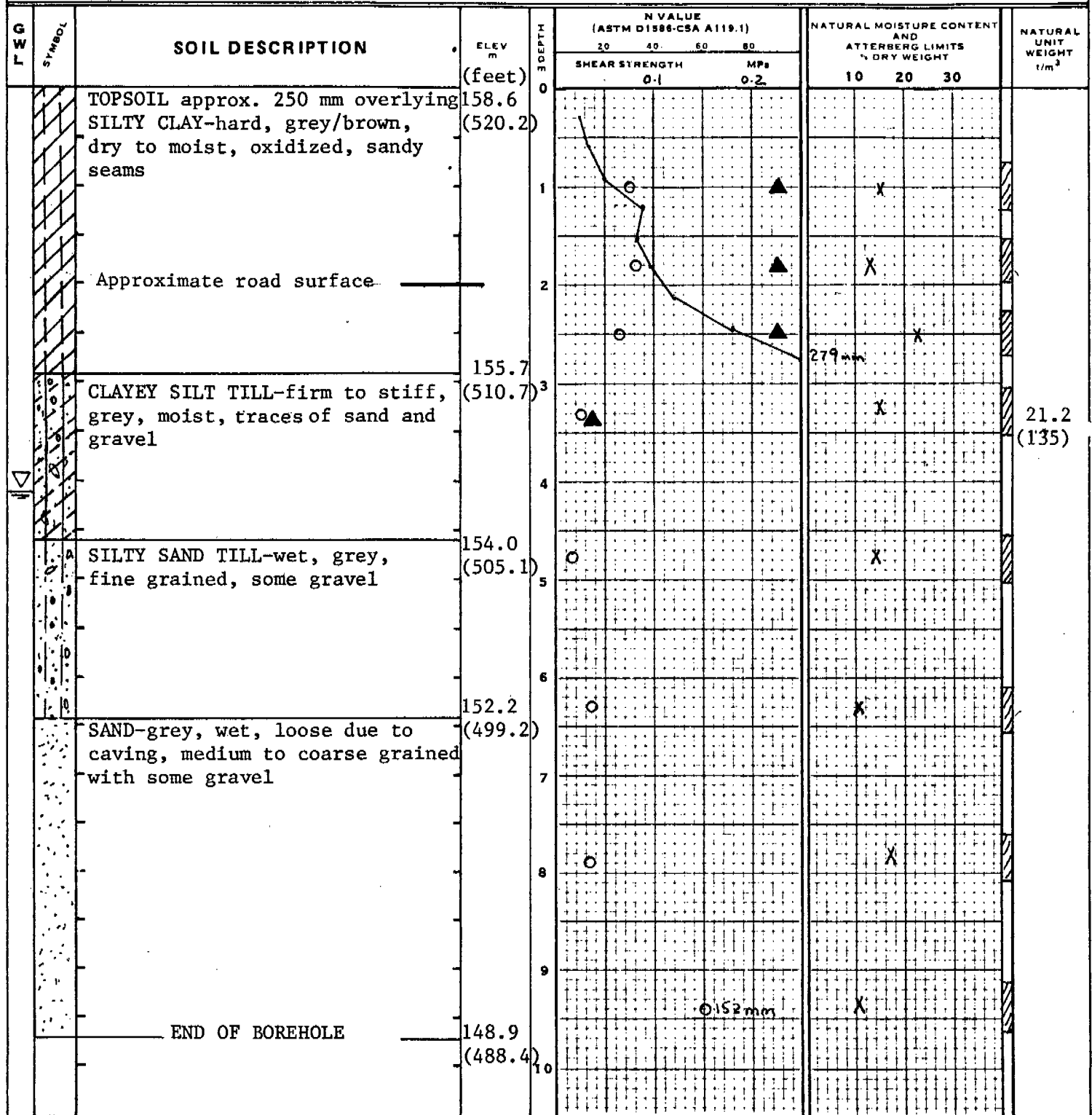
DRAWING No. 11

PROJECT PROPOSED CP GRADE SEPARATION  
 LOCATION ORMONT DRIVE  
NORTH YORK, ONTARIO

AUGER SAMPLE  
 SPT (N) VALUE    
 DYNAMIC CONE TEST  
 SHELBY TUBE    
 FIELD VANE TEST  + 5  
 LAB VANE TEST  ±

NATURAL MOISTURE  x  
 PLASTIC AND LIQUID LIMIT  —  
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  15 5  
 % STRAIN AT FAILURE  10  
 PENETROMETER  ▲

HOLE LOCATION AND DATUM SEE  
 DRAWING NO. 1





## NOTES:

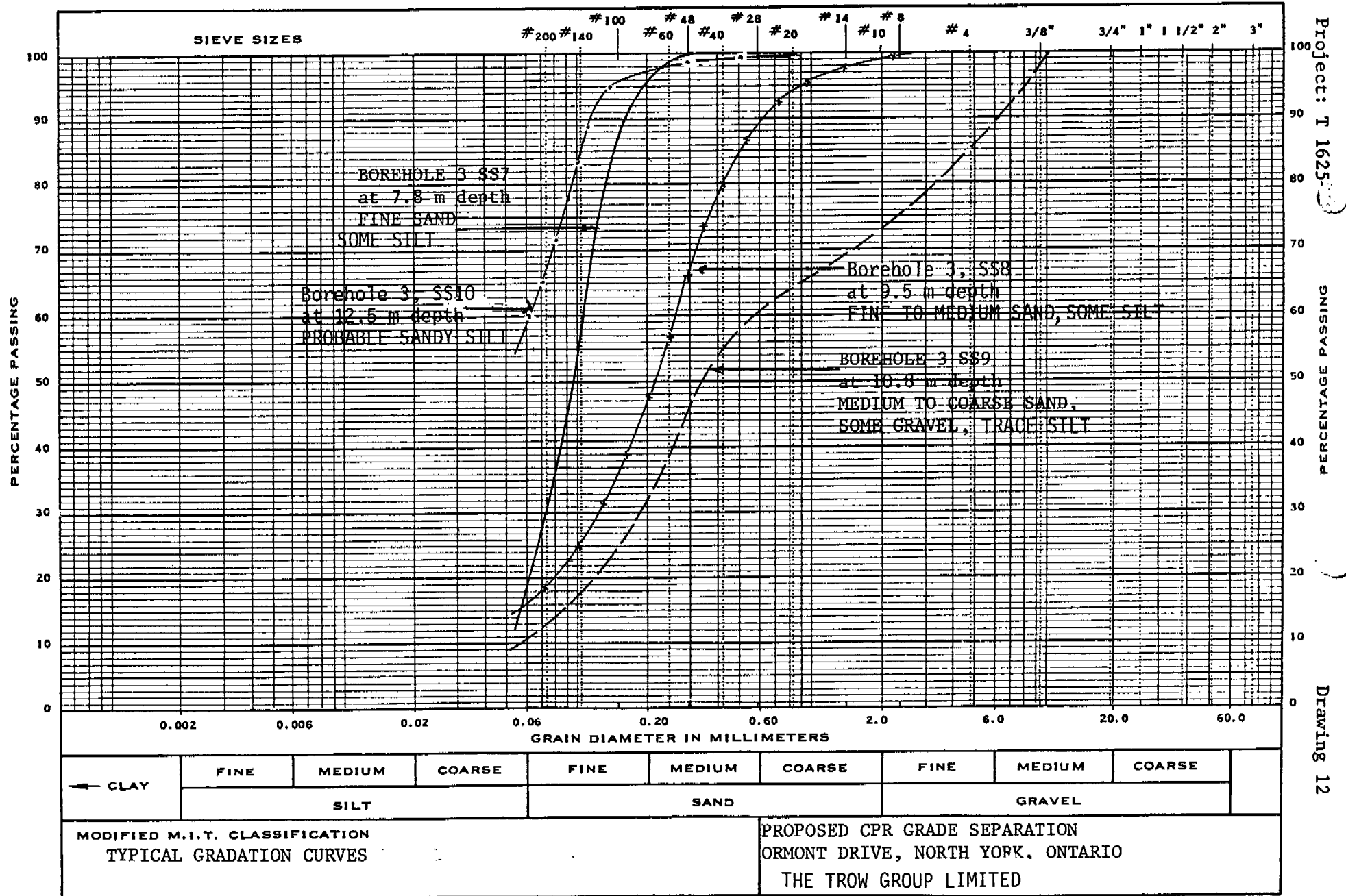
1. Borehole advanced uncased by continuous flight auger equipment to spoon refusal at 9.7 m depth on September 18, 1979.
2. Dynamic Cone Penetration test performed 1.2 m east of borehole.

## 3. Water Level Records:

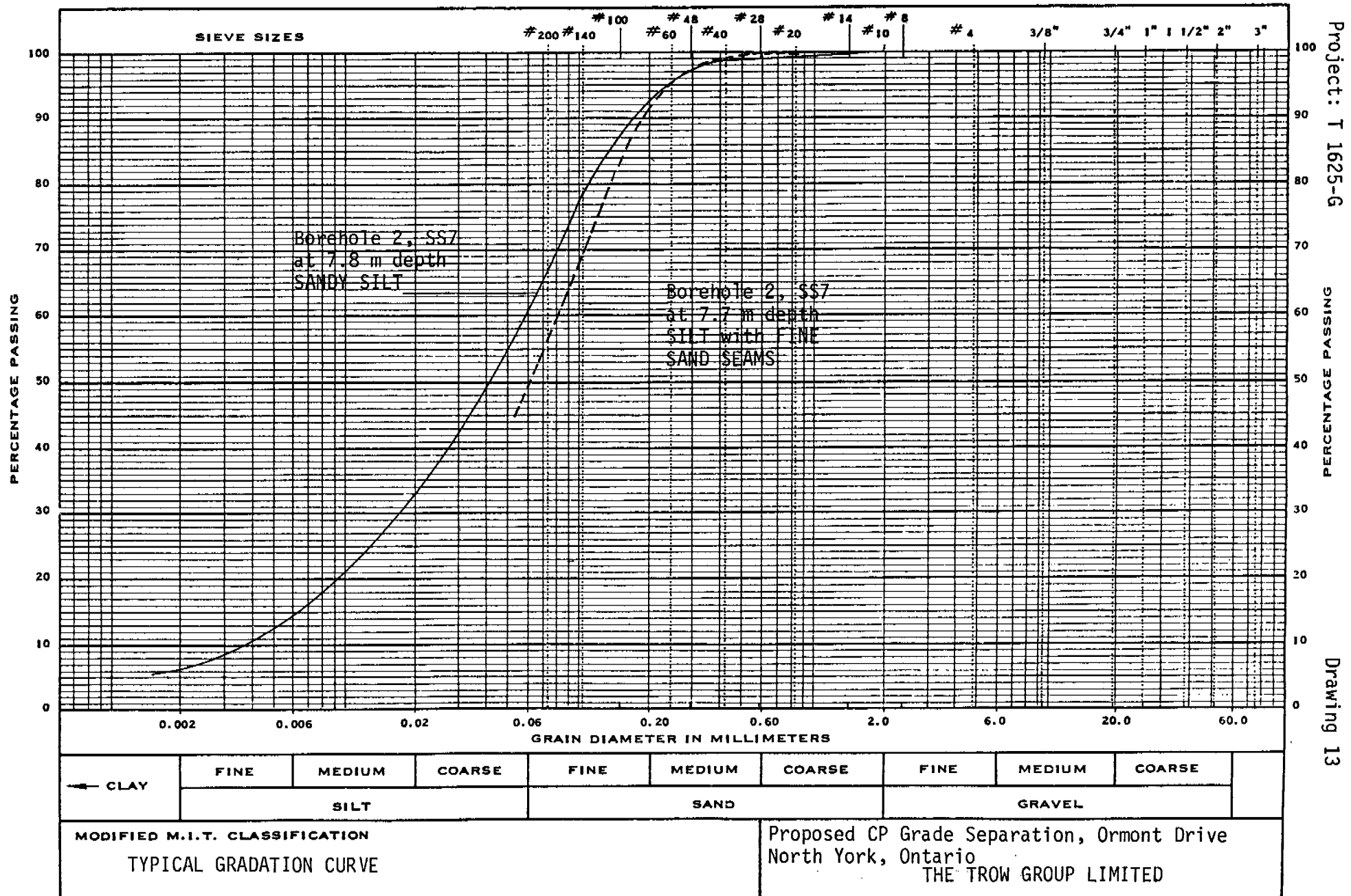
<u>TIME</u>	<u>WATER LEVEL</u>	<u>HOLE OPEN</u>
on completion	4.2 m	4.3 m
one day	4.1 m	4.2 m

4. Chemical analysis of groundwater sample from borehole.  
pH = 7.4      Sulphate,  $\text{SO}_4$ , Content = traces

# MECHANICAL ANALYSIS

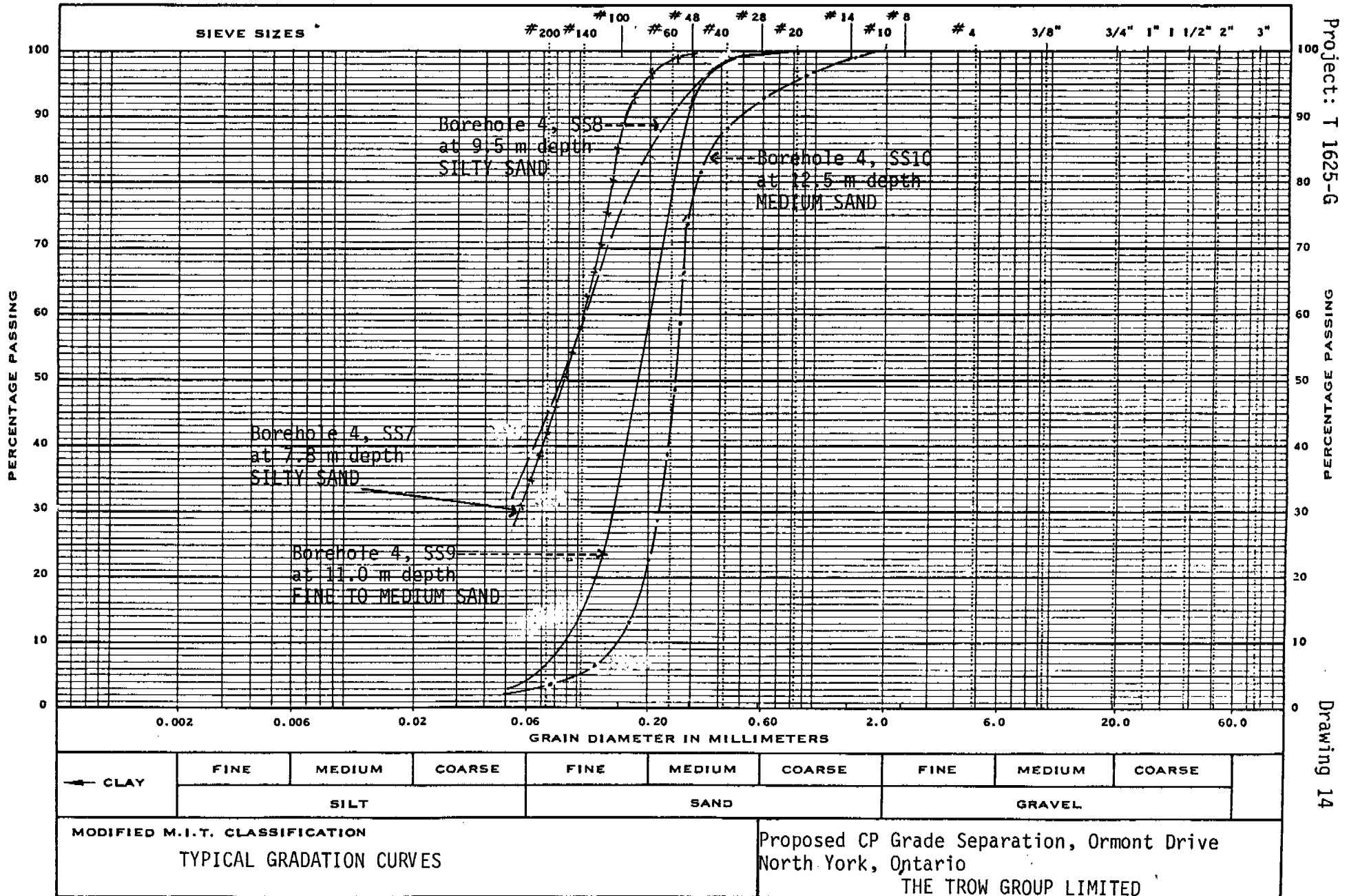


# MECHANICAL ANALYSIS

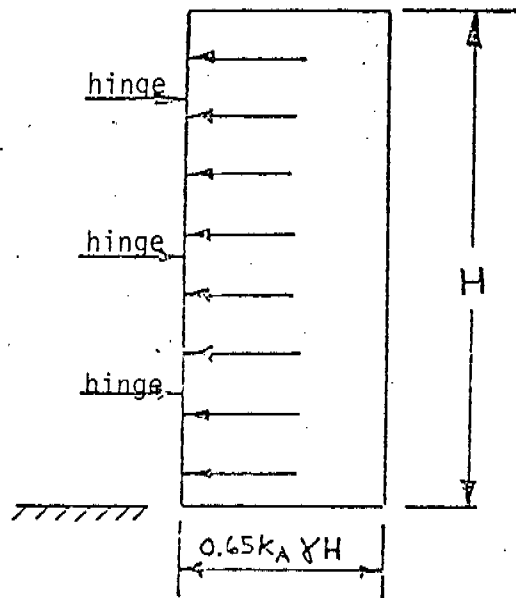




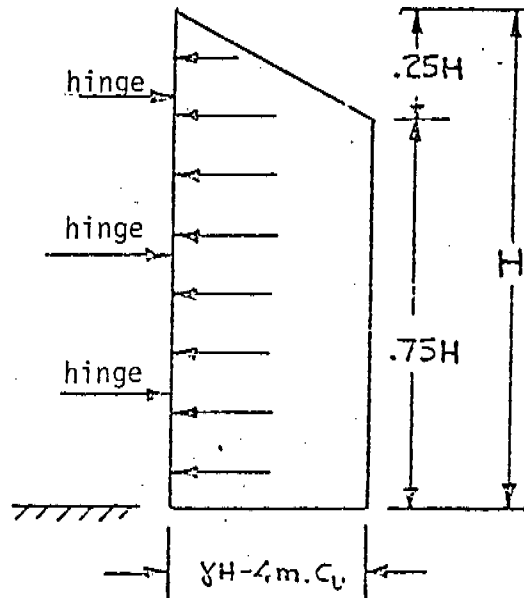
# MECHANICAL ANALYSIS



# a) SANDS

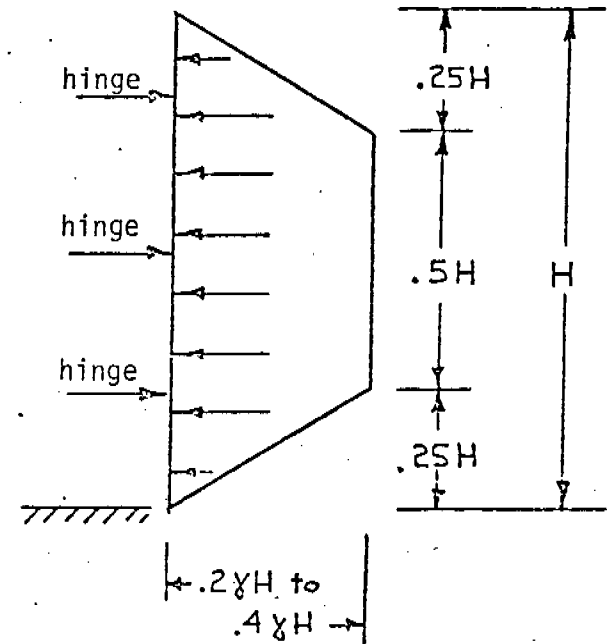


# b) SOFT TO FIRM CLAYS $C < 1000$ psf



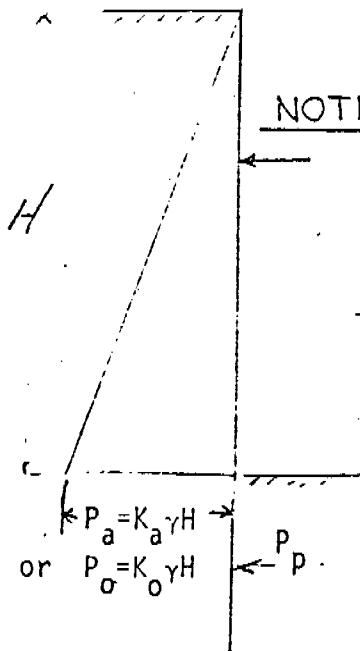
$m = 1$  unless a considerable depth of soft clay just below excavation level. In this case use  $m = 0.4$   
 Do not exceed  $0.3\gamma H$

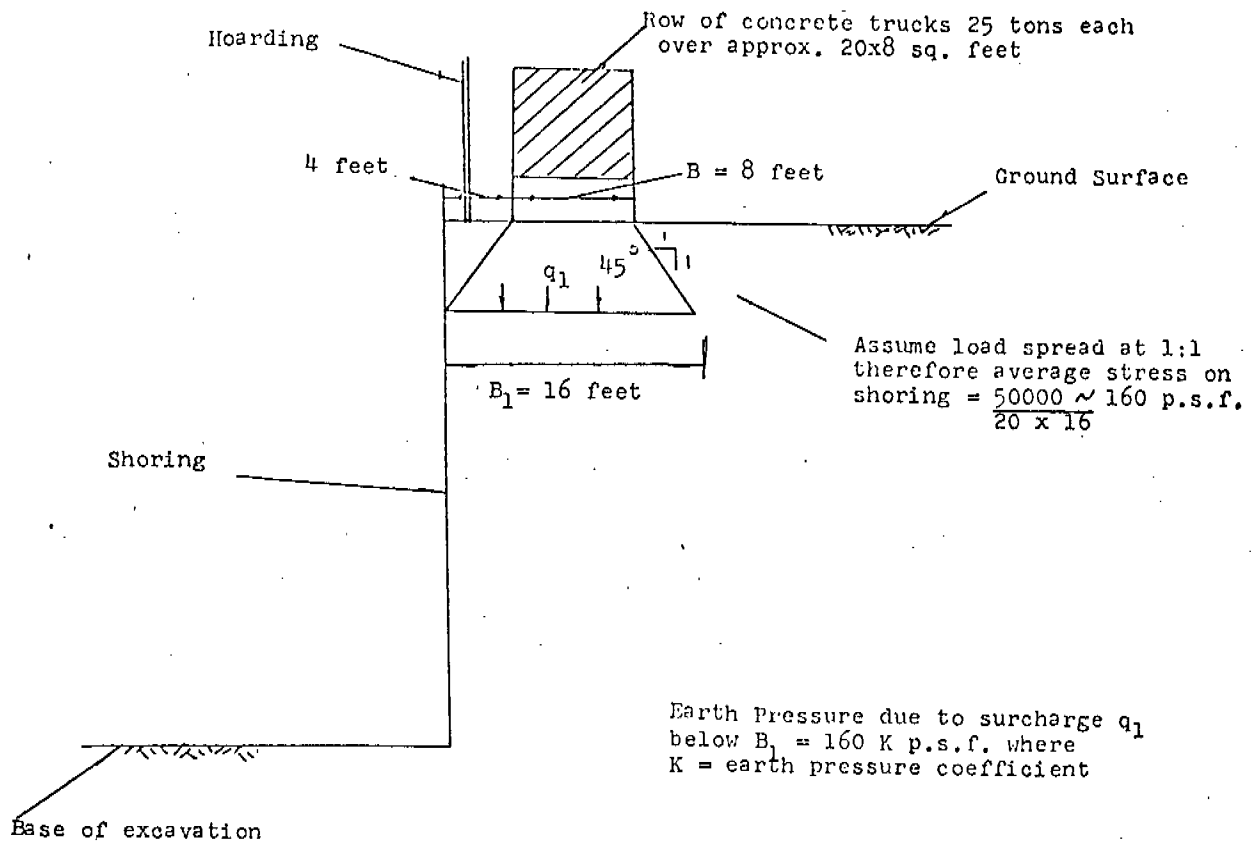
# c) STIFF FISSURED CLAYS



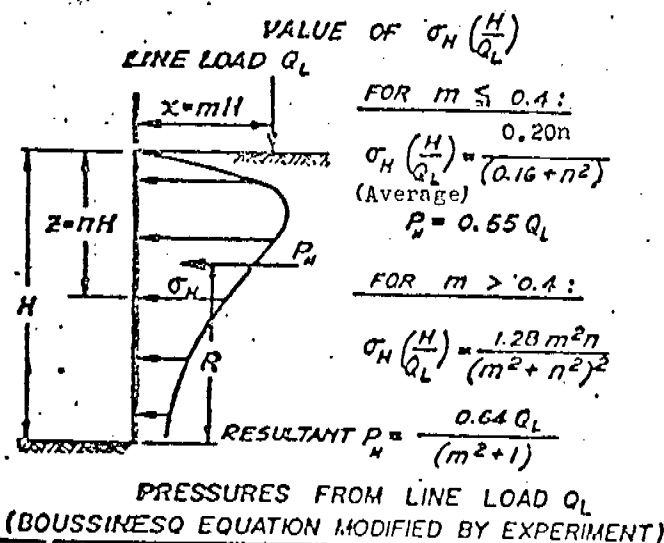
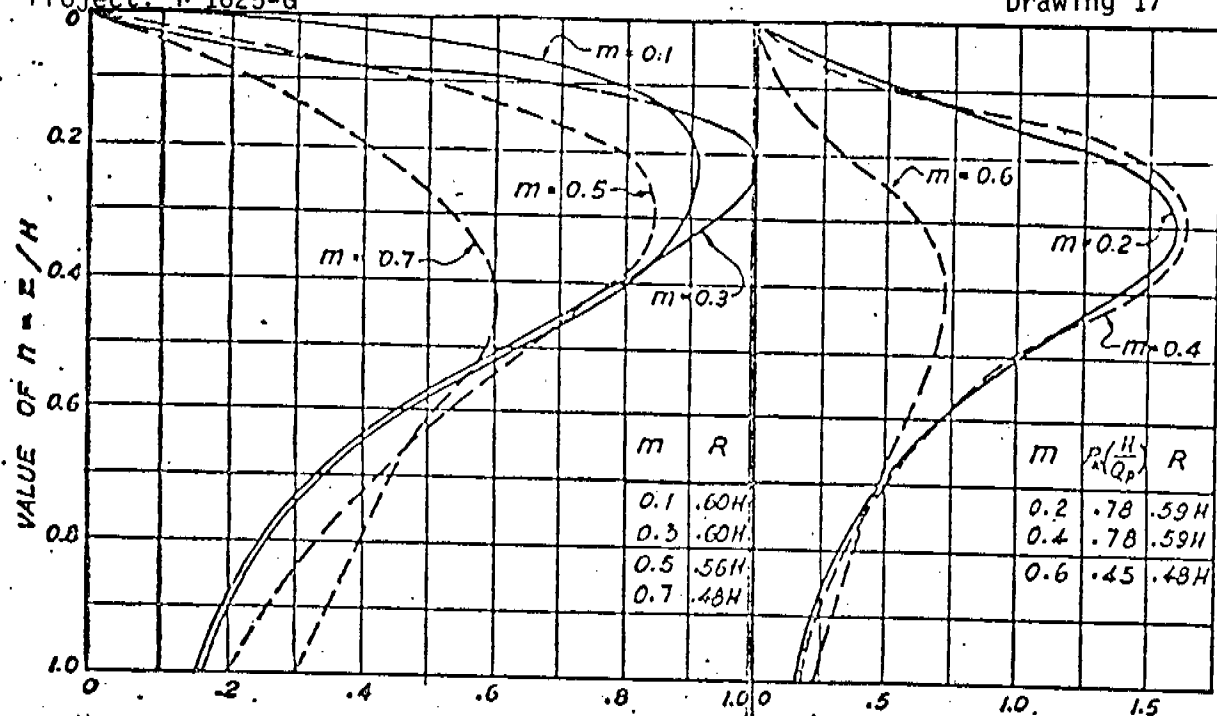
## NOTES:

1. Check all Systems for Partial Excavation Condition.
2. IF FREE WATER LEVEL ABOVE BASE OF EXCAVATION, HYDROSTATIC PRESSURE MUST BE ADDED TO ABOVE PRESSURE DISTRIBUTION IN SANDS.
3. IF SURCHARGE LOADINGS ARE PRESENT AT OR NEAR GROUND SURFACE THESE MUST BE INCLUDED IN THE LATERAL PRESSURE CALCULATION.



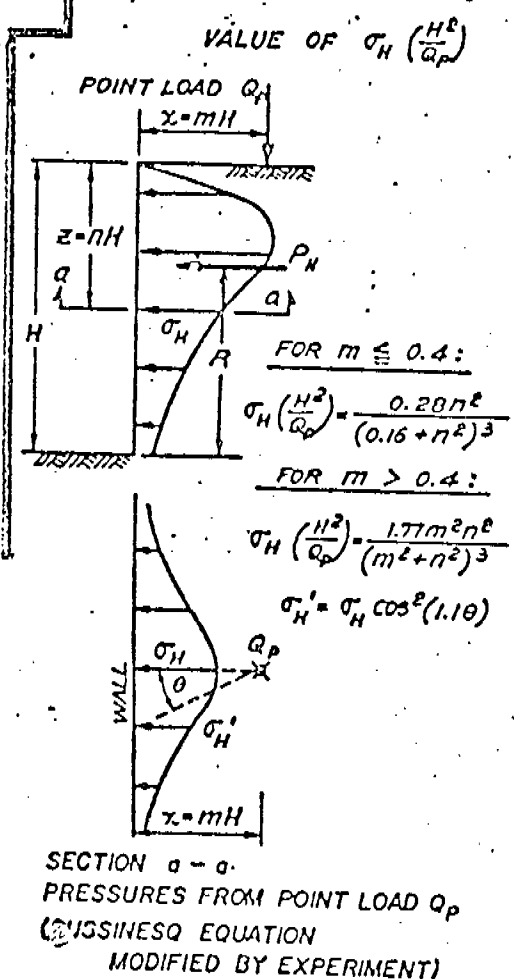


Pressure from concrete truck surcharge

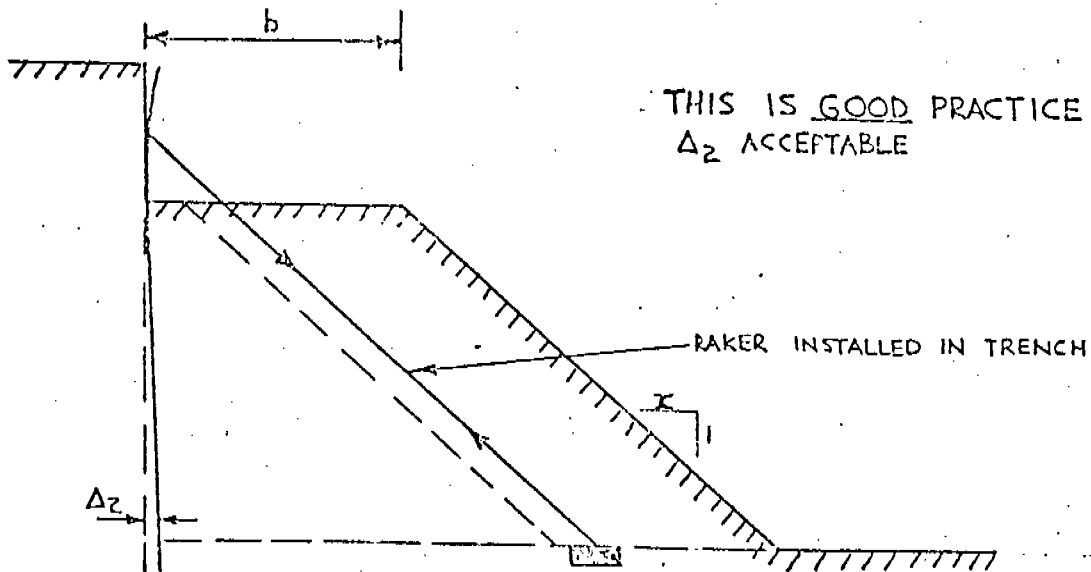
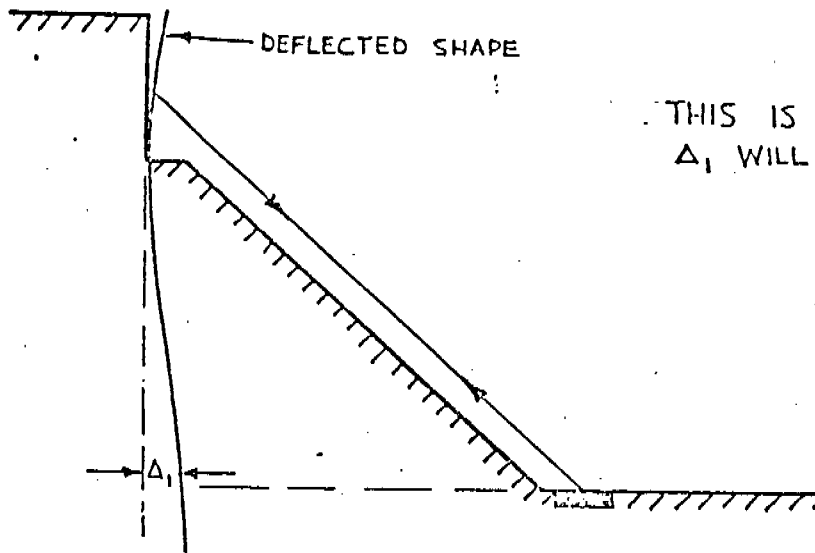


m	R
0.1	.60H
0.3	.60H
0.5	.56H
0.7	.48H

m	$R \left( \frac{H}{Q_p} \right)$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H



HORIZONTAL PRESSURES ON WALL DUE TO SURCHARGE



SOIL TYPE	Recommended $\alpha$	Recommended $b$
SOFT TO FIRM CLAYS	subject to stability analysis	subject to stability analysis
STIFF CLAYS & SANDY CLAYS	1	5
DENSE SANDS	1	5
LOOSE SANDS	1.5	10

The excavation must not go deeper than 1.5 feet below support level before the support is put in.

# Explicit Hand calc's Demonstration of

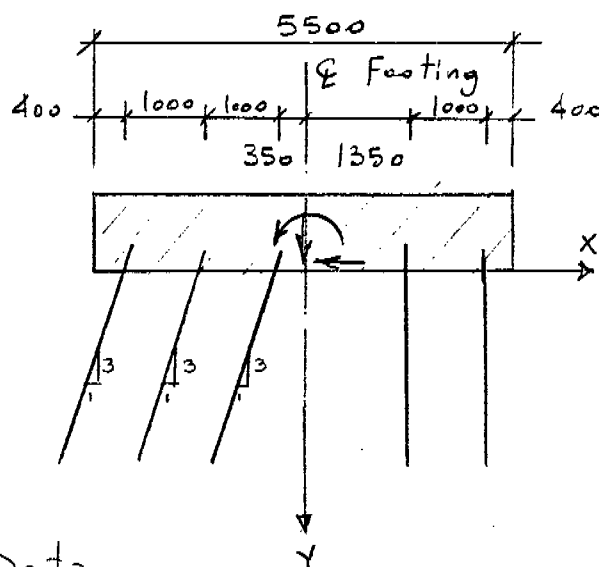
## A. HRENNIKOFF Method.

(Conf. Trans. of ASCE Journal Vol. 115 1950)  
pp 351-381

### Selected Sample:

(Arbitrary Chosen)

(Piles @ 1000 mm spacing)



Applied pile Loads.

$$\begin{aligned} V_q &= 589 \text{ kN/m} \downarrow \\ H_q &= 265 \text{ kN/m} \leftarrow \\ M_q &= 190 \text{ kN-m/m} \curvearrowright \end{aligned}$$

### Pile Data:

$$\begin{aligned} A &= 6330 \text{ mm}^2 & (6330 \times 10^{-6} \text{ m}^2) \\ I &= 79.9 \times 10^6 \text{ mm}^4 & (79.9 \times 10^6 \text{ mm}^4) \end{aligned}$$

$$E = 200 \text{ GPa}$$

$$k = 8.0 \text{ MPa} \quad (\text{Soil coefficient})$$

$$L_{\text{pile}} = 6325 \text{ mm. (For slopped piles)}$$

$$L_{\text{pile}} = 6000 \text{ mm. (For vertical )}$$

ACTION  
BY

Assume Soil is very weak:

ie) The pile is assumed to be driven to a firm stratum. All axial reaction comes from the foot reaction and no skin friction exists. (See pp. 366)

$$(\delta\alpha)_f = \frac{P_a \cdot L}{AE} \therefore n = \frac{AE}{L}$$

L can be assumed a constant value for all piles (say  $L = 6325 \text{ mm.}$ )

$$n = \frac{(6330 \times 10^{-6}) \times 200 \times 10^6}{6.325} = 200158 \text{ kN/m.}$$

$$k = 8 \text{ MPa}$$

$$\beta = \sqrt[4]{\frac{8.0}{4 \times 200 \times 10^3 \times 79.9 \times 10^{-3}}} = 0.595 \text{ 1/m} \quad \beta = \left(\frac{k}{4EI}\right)^{1/4}$$

$$\delta = \frac{k}{\beta} = \frac{8 \times 10^3}{0.595} = 13445 \text{ kN/m.}$$

$$M_f = \frac{k}{2\beta^2} = \frac{8 \times 10^3}{2(0.595)^2} = 11298 \text{ kN-m/m.}$$

$$m_\alpha = \frac{k}{2\beta^3} = \frac{8 \times 10^3}{2(0.595)^3} = 18989 \text{ kN-m/rad.}$$

$$\phi_1 = 108.43^\circ$$

$$\phi_2 = 90^\circ$$

$$\begin{aligned} \sin \phi &: 0.9487 & (0.900) \\ \cos \phi &: -0.3161 & (0.100) \\ \sin 2\phi &: -0.6000 \end{aligned}$$

$$\begin{aligned} \sin \phi &: 1.0 & (1.00) \\ \cos \phi &: 0. & (0.0) \\ \sin 2\phi &: 0. \end{aligned}$$

ACTION  
BY

$$X_x = - \sum [N (n \cdot \cos^2 \phi + t_s \cdot \sin^2 \phi)]$$

$$X_x = - \left[ (200158 \times (0.100) + 13445 \times (0.900)) \times 3 + 0 + 13445 \times (1.00) \times 2 \right] = -123239 \text{ kN/m}$$

$$X_y = Y_x = -\frac{1}{2} (n - t_s) \sum (N \cdot \sin 2\phi)$$

$$X_y = Y_x = -\frac{1}{2} (200158 - 13445) [3 \times (-0.600)] = +168042 \text{ kN/m}$$

$$X_d = M_x = -\frac{1}{2} (n - t_s) \sum (N \bar{x} \cdot \sin 2\phi) + m_s \sum (N \sin \phi)$$

$$X_d = -\frac{1}{2} (200158 - 13445) \times (-0.600) \left( -2.35^* - 1.35^* - 0.35^* \right)$$

4.05

$$+ 11298 \times \left( 3 \times 0.9487 + \frac{2 \times 1.000}{4.8461} \right)$$

$$X_d = M_x = -226856 + 54751 = -172105 \text{ kN-m/m}$$

\* Arbitrary chosen Coord. origin is the Footing &

$$Y_y = - \sum N (n \cdot \sin^2 \phi + t_s \cdot \cos^2 \phi)$$

$$Y_y = - \left[ 200158 \times (0.900) \times 3 + 200158 \times (1.00 \times 2) + 13445 \times (0.100) \times 3 + 0 \right] = -944776 \text{ kN/m}$$

$$Y_d = M_y = - \sum (n \cdot \sin^2 \phi + t_s \cos^2 \phi) N \bar{x} - m_s \sum (N \cos \phi)$$

$$M_y = - \left[ \begin{array}{l} -735021 \\ (200158 \times (0.900) + 13445 \times (0.100)) (-2.35 - 1.35 - 0.35) + \\ + \{ 200158 \times (1.000) (1.35 + 2.35) \} - \\ - 11298 \times [(-0.3161) \times 3] \end{array} \right] = +5150 \text{ kN-m/m}$$

3.7

+740585

-10714

ACTION  
BY



$$M_{\alpha} = - \sum (n \cdot \sin^2 \phi + t_g \cos^2 \phi) \sum \bar{z}^2 - 2m \sum (\bar{x} \cdot \cos \phi) - m_{\alpha} \sum (N)$$

$$\begin{aligned}
 M_{\alpha} = & - \left[ [200158 \times (0.900) + 13445 (0.100)] \times (0.35^2 + 1.35^2 + 2.35^2) \right. \\
 & \left. + [200158 \times (1.00) + 0] \times (1.35^2 + 2.35^2) \right] - 2825412 \\
 & - 2 \times 11298 \left[ -0.3161 (-0.35 - 1.35 - 2.35) \right] - 28928 \\
 & - 5 \times 18989 = -2949285 \text{ kN-m/m.} - 94945
 \end{aligned}$$

Equilibrium Equation :

$$X_x \Delta x + X_y \Delta y + X_{\alpha} \alpha + X = 0$$

$$X_y \Delta x + Y_y \Delta y + Y_{\alpha} \alpha + Y = 0$$

$$X_{\alpha} \Delta x + Y_{\alpha} \Delta y + M_{\alpha} \alpha + M = 0$$

$$-123239 \Delta x + 168042 \Delta y - 172105 \alpha = 265$$

$$168042 \Delta x - 944776 \Delta y + 5150 \alpha = -589$$

$$-172105 \Delta x + 5150 \Delta y - 2949285 \alpha = 190$$

By simplifying :

$$(1) \quad -\Delta x + 1.3635 \Delta y - 1.3865 \alpha = 0.00215$$

$$(2) \quad \Delta x - 5.6223 \Delta y + 0.0306 \alpha = -0.00351$$

$$(3) \quad -\Delta x + 0.0299 \Delta y - 17.1365 \alpha = 0.00110$$

ACTION  
BY

(1)	-1	+ 1.3635	- 1.3965	= 0.00215
(2)	<del>✓</del>	- 5.6223	+ 0.0306	= -0.00351
(2')		+ 4.2588	+ 1.3659	= + 0.00136

(1)	<del>✓</del>	- 5.6223	+ 0.0306	= -0.00351
(3)	-1	+ 0.0299	- 17.1365	= 0.00110
(3')		+ 5.5924	+ 17.1059	= + 0.00241

$$(2') \quad 4.2588 + 1.3659 = 0.00136$$

$$(3') \quad 5.5924 + 17.1059 = 0.00241$$

$$(2') \quad -23.817 + 7.6387 = -0.00761$$

$$(3') \quad 23.817 + 72.8506 = 0.01026$$

$$65.2119 \alpha = 0.00265$$

$$\alpha = 4.06 \times 10^{-5} \text{ radian} \quad \longleftrightarrow \text{vs.} \quad 3.5473 \times 10^{-5} \text{ rad}$$

$$\Delta y = \frac{0.01026 - 72.8506 \times (4.06 \times 10^{-5})}{23.817}$$

$$\Delta y = 0.307 \text{ mm.} \quad \longleftrightarrow \text{vs.} \quad 0.3062 \text{ mm.}$$

$$-\Delta x = 0.00215 + 1.3965 \times (4.06 \times 10^{-5}) - 1.3635 \times \left( \frac{0.307}{1000} \right)$$

$$\Delta x = -1.79 \text{ mm.} \quad \longleftrightarrow \text{vs.} \quad -1.7836 \text{ mm.}$$

ACTION  
BY

SUPPORTING RUN: TO PREVIOUS  
HAND CALCULATION.

(20)

XEQ "PILE 01"

BATTER PILES

ELASTIC  
ANALYSIS  
5 ROWS MAX  
VER. 03-01.12  
\*\*\*\*\*

SIGN CONVENTION  
\*\*\*\*\*

H + RIGHT  
V + DOWN  
M + C/W

SUBGRADE  
\*\*\*\*\*

KSUB=KN/M3?  
-8.000.000 RUN

PILE DATA  
\*\*\*\*\*

A/PILE=MM2?  
6.330.000 RUN  
I/PILE=MM4?  
79.9+06 RUN  
L/PILE=MM?  
6.325.000 RUN  
E=MPa?  
200.000.000 RUN  
NRWS=?  
5.000 RUN  
TOTAL PILES=?  
5.000 RUN

APPLIED  
LOADS  
\*\*\*\*\*

V=KN/Slice?  
589.000 RUN  
H=KN/Slice?  
-265.000 RUN  
M=KN-M/Slice?  
-190.000 RUN

ROW NO. 1.

NO. PILES?  
1.0000 RUN  
X=MM?  
-2.350.0000 RUN  
BATTER?  
100.4000 RUN

ROW NO. 2.

NO. PILES?  
1.0000 RUN  
X=MM?  
-1.350.0000 RUN  
BATTER?  
100.4000 RUN

ROW NO. 3.

NO. PILES?  
1.0000 RUN  
X=MM?  
-350.0000 RUN  
BATTER?  
100.4000 RUN

ROW NO. 4.

NO. PILES?  
1.0000 RUN  
X=MM?  
1.350.0000 RUN  
BATTER?  
90.0000 RUN

ROW NO. 5.

NO. PILES?  
1.0000 RUN  
X=MM?  
2.350.0000 RUN  
BATTER?  
90.0000 RUN

FOOTING  
DISPLACEMENTS  
MM, RADIAN  
\*\*\*\*\*

{ dx= -1.7836  
dY= 0.3062  
ALPHA= 3.5473F-5 }

(21)

PILE DISPLACEMENTS  
PILE FORCES  
K MM  
\*\*\*\*\*

ROW NO. 1.

dL= 0.7744  
dT= -1.6221

PL= 155.1725  
PT= 22.2329  
PM= -19.0301

ROW NO. 2.

dL= 0.0001  
dT= -1.6109

PL= 161.9171  
PT= 22.0822  
PM= -18.9034

ROW NO. 3.

dL= 0.8417  
dT= -1.5997

PL= 168.6616  
PT= 21.9315  
PM= -18.7767

ROW NO. 4.

dL= 0.3540  
dT= -1.7836

PL= 70.9425  
PT= 24.4066  
PM= -20.8577

ROW NO. 5.

dL= 0.3895  
dT= -1.7836

PL= 78.0504  
PT= 24.4066  
PM= -20.8577

GOODBYE

RCL 27  
0.0151 \*\*\*  
RCL 29  
2,542.8176 \*\*\*  
RCL 30  
168,338.6897 \*\*\*  
RCL 34  
1,143.7214 \*\*\*  
RCL 35  
9.8115 \*\*\*  
RCL 36  
191.9605 \*\*\*  
RCL 37  
-703.0010 \*\*\*  
RCL 38  
958.6483 \*\*\*  
RCL 39  
59.5506 \*\*\*  
RCL 40  
-5,399.7070 \*\*\*  
RCL 41  
-132.3596 \*\*\*  
RCL 41  
-132.3596 \*\*\*  
RCL 42  
1,660.1163 \*\*\*

168x 1.5

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

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Paper No. 2401

ANALYSIS OF PILE FOUNDATIONS WITH  
BATTER PILES

BY A. HRENNIKOFF,<sup>1</sup> ASSOC. M. ASCE

WITH DISCUSSION BY MESSRS. JACOB KAROL, J. OWEN LAKE,  
GEORGE S. MURPHY, AND A. HRENNIKOFF.

SYNOPSIS

This paper presents a method of analysis of pile loads in foundations involving nonparallel piles, when both longitudinal and lateral resistances of piles are taken into account. Although the basic ideas underlying this method are substantially the same as in the other similar methods, the manner of applying these ideas is different in so far as a clear separation is made of the aspects of the problem that are reasonably definite and generally accepted from those that are uncertain. The element of uncertainty is present in the form of three coefficients, characterizing deformability of the piles, whose values can only be roughly estimated. By way of approximation, the two less important of these coefficients may be ignored and the third may be expressed in terms of data possessed by the designer, which lead to workable formulas suitable for practical use. A method of estimating the pile coefficients is also given, and by varying the values of these coefficients over a wide range, it is shown by an example that the error introduced by ignoring the minor coefficients is only moderate.

PILE FOUNDATIONS

Batter piles are not used in foundations of structures carrying horizontal loads as often as their advantages would justify. Perhaps one of the reasons for this anomaly is the absence of a workable method of analysis.

In this connection one cannot fail to notice a conspicuous inconsistency in the usual design practice. On the one hand, many foundations carrying horizontal loads are provided with only vertical piles, so that all the horizontal restraint is caused by the lateral pile resistances alone. On the other hand, when the arrangement is such that two or more groups of nonparallel piles are present, the lateral pile resistances are completely ignored, as if they were of

NOTE.—Published in February, 1949, *Proceedings*. Positions and titles given are those in effect when the paper or discussion was received for publication.

<sup>1</sup> Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.

no consequence. This approach is out of line with the one adopted in the case of parallel piles.

It may be pointed out that pile foundations very seldom, if ever, fail because the piles break, but almost invariably fail because the movement of the foundation becomes excessive. This statement is in full agreement with the recognized method of determining the allowable loads on piles both in longitudinal and in transverse directions by loading tests. Thus, most building codes specify that the safe axial load on a pile must be such that the settlement produced does not exceed a definite small value, such as  $\frac{1}{8}$  in. With regard to the allowable lateral loads most of the codes are silent, but here again resistances specified by some authorities are based on definite small displacements of the pile heads.<sup>2</sup>

In the foregoing it was shown that, from the viewpoint of design, the pile load is considered merely as a symbol for deflection. If a specification limits the pile load to 20 tons, it is not because a 25-ton load is believed to break it, or at least bring it too close to failure, but because the settlement corresponding to it is expected to be excessive.

This symbolic use of the allowable load is not limited to the piles alone, but is common in other cases as well. For example, the allowable load on soil is governed largely by the expected settlement of the structure.

The close relation existing between the allowable load on a pile and the resultant displacement has an important bearing on the validity of the generally

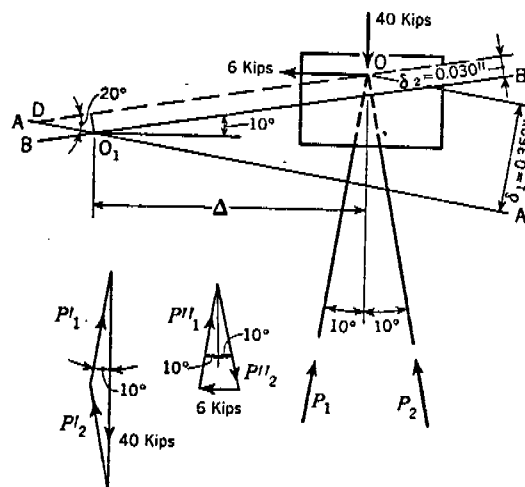


Fig. 1

accepted method of pile analysis, previously referred to, which consists of disregarding the lateral pile resistances. One might think that, once the lateral pile resistance is omitted, the total pile resistance is underestimated, and the design is conservative. This conclusion, however, generally is not correct, as is shown by the following.

Fig. 1 represents in a simplified form a foundation supported by two piles inclined in opposite directions at 10° with the vertical. The foundation carries a vertical load of 40 kips and a horizontal load of 6 kips, applied as shown. The lateral pile resistance is disregarded. It is assumed that the allowable axial load on each pile is 40 kips, and that the axial displacement of the pile head produced by this load is  $\frac{1}{8}$  in. The latter statement implies that displacement of the foundation in excess of  $\frac{1}{8}$  in. is objectionable.

<sup>2</sup>"Lateral Pile-Loading Tests," by L. B. Feagin, *Transactions, ASCE*, Vol. 102, 1937, p. 236.

By constructing two separate force triangles, one for horizontal loads and one for vertical loads, and then by combining the forces, the two pile loads are

$$P_1 = \frac{1}{2} \times 40 \times \sec 10^\circ + \frac{1}{2} \times 6 \times \sec 80^\circ = 37.6 \text{ kips}$$

and

$$P_2 = \frac{1}{2} \times 40 \times \sec 10^\circ - \frac{1}{2} \times 6 \times \sec 80^\circ = 3.2 \text{ kips}$$

Inasmuch as both  $P_1$  and  $P_2$  are compression loads and are less than 40 kips, the pile loads are satisfactory.

Consider now the horizontal displacement of the foundation. Assuming that the deformation is proportional to the load, the axial displacements of the pile heads are found to be, for pile 1,  $\delta_1 = \frac{3}{8} \times \frac{37.6}{40} = 0.352$  in., and, for pile 2,  $\delta_2 = \frac{3}{8} \times \frac{3.2}{40} = 0.030$  in.

The deflected position  $O_1$  of the point of intersection of the piles (Fig. 1) is found at the intersection of the line  $AA_1$ , normal to pile 1 and located at 0.352 in. below  $O$ , and the line  $BB_1$ , normal to pile 2 and situated at 0.030 in. below  $O$ .

Evidently,  $OD = 0.352 \operatorname{cosec} 20^\circ$  and  $DO_1 = 0.030 \operatorname{cosec} 20^\circ$ , so that the horizontal displacement of the foundation is  $\Delta = (0.352 - 0.030) \times \operatorname{cosec} 20^\circ \times \cos 10^\circ = 0.928$  in., which is much greater than  $\frac{1}{4}$  in. and therefore is inadmissible.

The large value of this displacement is, of course, the direct result of the small batter of the piles; it would decrease with an increase in the batter. However, practical considerations of pile driving preclude too great a batter, and the situation presented herein appears typical. Thus, the fact that the axial pile loads are moderate is no guarantee that the displacement of the foundation is small and the design adequate, unless the lateral movement is small also, or unless the lateral pile loads are moderate in comparison with the allowable values, based on small displacements.

#### EXISTING METHODS OF ANALYSIS

Before presenting the method proposed in this paper it is desirable to outline briefly predecessors of the method.

In the method developed by Swedish engineers and presented in the United States by C. P. Vetter,<sup>3</sup> M. ASCE, the lateral and rotational pile resistances  $R$  and  $M$ , acting on the ends of piles (Fig. 2), are determined by assuming the pile to act as a free beam fixed at some unknown depth  $h$ . This assumption has two weaknesses: (1) No method is suggested for determining the unknown depth  $h$ ; and (2) the pile is actually loaded along its entire length by the side pressure of earth, and it does not resemble a free beam of span  $h$ . Also, no section of the pile is fixed both in position and direction, as is assumed with regard to the section at  $A$ . The effect of these erroneous assumptions is that no single value of  $h$  can result in true values of both end resistances  $M$  and  $R$  corresponding to different displacements of the pile head. No estimate of the error resulting from these incorrect assumptions seems to be possible.

<sup>3</sup> "Design of Pile Foundations," by C. P. Vetter, *Transactions, ASCE*, Vol. 104, 1939, p. 758.

The method of pile design<sup>4</sup> of Carl Culmann is based on the resolution of  $P_{R0}$ , the resultant force acting on the foundation, into three component forces (Fig. 3), acting in the directions of the piles and passing through the centers of the respective pile groups. This method possesses the following weaknesses:

a. By assumption, the foundation must be supported by three nonparallel groups of piles, with one or several parallel piles in each group. From a practical point of view the requirement of the three nonparallel groups of piles may be an unnecessary complication.

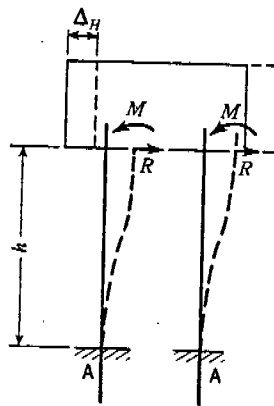


FIG. 2

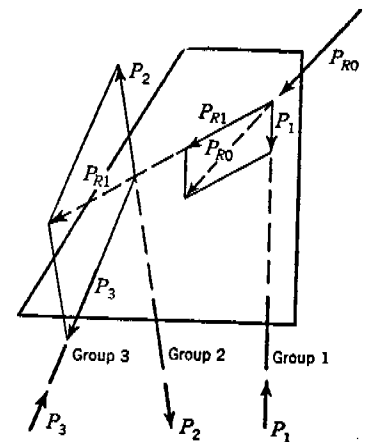


FIG. 3

b. The piles are assumed to develop axial forces only, and since no analysis of lateral deformations is made, the method is subject to the weakness previously discussed in connection with Fig. 1.

c. Each pile group is replaced in analysis by a resultant pile located at the center of gravity of the pile group. When several piles are present in each group, this assumption ignores the rotational resistances of the pile groups, developed by virtue of spreading the piles over an area, and thereby underestimates the strength of the foundation.

The method of H. M. Westergaard,<sup>5</sup> M. ASCE, also ignores the lateral pile resistances, and is thus subject to the same criticism.

#### ASSUMPTIONS OF PROPOSED METHOD

The method presented in this paper is based on the following assumptions:

1. The load carried by each pile is proportional to the displacement of the pile head. In the most general case this displacement consists of three components: The axial (in relation to the pile) displacement,  $\delta_i$ ; the transverse displacement,  $\delta_t$ ; and the rotational displacement,  $\alpha$ . Although the displace-

<sup>4</sup>"Theoretical Soil Mechanics," by K. Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 363.

<sup>5</sup>"The Resistance of a Group of Piles," by H. M. Westergaard, *Journal, Western Soc. of Engrs.*, Vol. 22, 1917, p. 704.



- ments thus defined are proportional to the forces producing them, they need not necessarily be considered as elastic.
- 2. All piles behave alike with regard to the load-deformation relation.
- 3. The footing, in which the pile heads are embedded, is absolutely rigid.
- 4. The problem is two-dimensional—that is, the piles, as well as the external forces, are arranged in planes transverse to the length of the foundation, and they are symmetrical with regard to the transverse middle plane. Under these conditions all the pile movements take place in transverse planes.
- 5. The footing movements are small.

It is of course fully realized that none of these assumptions is exactly true, but it is believed that in most cases they do not misrepresent the actual behavior of the piles greatly, and, at the same time, no reasonably workable analysis can be developed without their assistance. When physical conditions in the foundation manifestly disagree with any of these assumptions—for example, when the footing is not rigid—the theory, of course, fails. The same assumptions have been used by Mr. Vetter and Professor Westergaard.

#### THE PILE CONSTANTS

The pile constants are defined as the forces with which the pile acts on the foundation when the pile head is given a unit displacement. There are three sets of these constants, corresponding to three different kinds of displacements.

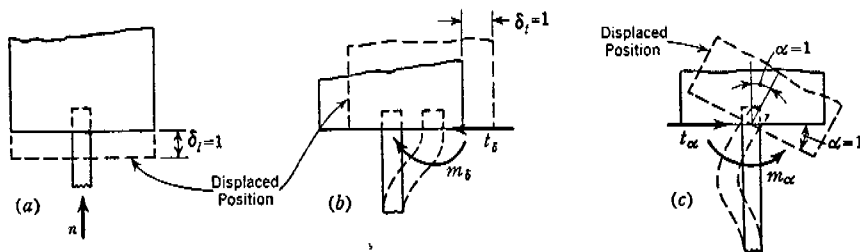


FIG. 4

Unit longitudinal displacement,  $\delta_l = 1$  in. (Fig. 4(a)), brings into play a single constant  $n$  which is measured in pounds per inch and acts in the axial direction.

A unit transverse movement,  $\delta_t = 1$  in. (Fig. 4(b)), produces a transverse resistance  $t_t$  and a rotational resistance  $m_t$ , measured in pounds per inch and pound-inches per inch, respectively.

A unit rotation,  $\alpha = 1$  radian (Fig. 4(c)), is accompanied by a rotational resistance  $m_\alpha$  and a transverse resistance  $t_\alpha$ , measured in pound-inches per radian and pounds per radian, respectively. It may be observed that no axial resistances accompany either transverse or rotational displacements, and no moments or transverse forces are brought into play by the axial displacement.

It should be emphasized that the pile constants as defined herein represent the effect of the pile on the footing and not vice versa. Therefore, the directions of these constants must be carefully noted. As may be expected, the

three primary constants ( $n$ ,  $t_s$ , and  $m_a$ ), are directed opposite to the pile head displacements creating them, whereas the two secondary constants ( $m_s$  and  $t_a$ ), which are only incidental to the displacements produced, stand in such a relation to their primary partners (that is, to  $t_s$  and  $m_a$ , respectively) that, if the  $t$  constant is directed to the right, the  $m$  constant is directed counterclockwise, and vice versa.

By the Betti theorem,<sup>6</sup>  $t_a = m_s$ , so that in the most general case—that is, when the pile head is deeply embedded in the footing—there are only four constants characterizing the load-deformation relation of a pile— $n$ ,  $t_s$ ,  $m_s = t_a$ , and  $m_a$ .

When the footing barely engages the piles, their heads may be considered as hinged instead of fixed, and  $m_s = 0$ . Likewise, the two  $\alpha$  constants need not be considered in this case, and only  $n$  and  $t_s$  remain.

#### THE FOUNDATION CONSTANTS

Fig. 5 represents the foundation of a structure supported by piles and referred to rectangular axes. For convenience, the  $x$ -axis is made coincident with the footing base; the positive

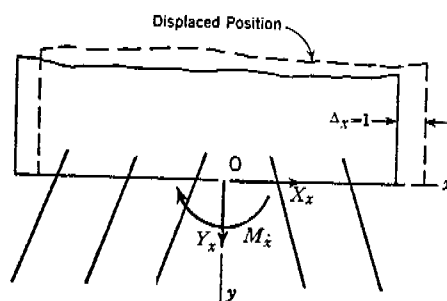


FIG. 5

directions of the axes are taken to the right and downward, and the origin is chosen arbitrarily. The foundation constants are defined as the resultant forces with which all piles act together on the footing, when the footing is given a unit translational displacement in the positive direction of one of the axes or a unit rotation about the origin in the clockwise direction.

Each unit displacement in general brings into play a resisting force, which may be represented by its two components acting along the coordinate axes and the moment about the origin. Thus, unit displacement along the  $x$ -axis gives rise to the constants  $X_x$ ,  $Y_x$ , and  $M_x$ , in Fig. 5. A unit displacement along the  $y$ -axis creates the constants  $X_y$ ,  $Y_y$ , and  $M_y$ , whereas a unit rotation about the origin produces the constants  $X_a$ ,  $Y_a$ , and  $M_a$ . By the Betti law,<sup>6</sup> three pairs of these constants are equal, thus leaving only six independent values:  $X_x$ ,  $Y_x = X_y$ ,  $M_x = X_a$ ,  $Y_y = Y_a$ , and  $M_y = M_a$ . The positive signs of these functions correspond to the positive directions of the axes and to the clockwise direction for the moments.

These foundation constants can be evaluated in terms of the pile constants and the geometry of the foundation. In order to avoid errors it is necessary to adhere strictly to the sign convention previously explained, keeping in mind that the foundation constants represent the action of the piles on the foundation, and not the opposite effect.

<sup>6</sup>"Theory of Statically Indeterminate Structures," by W. M. Fife and J. B. Wilbur, McGraw-Hill Book Co., Inc., New York, N. Y., 1937, p. 39.

## ANALYSIS OF PILE FOUNDATION

In order to determine the pile loads produced by the external forces acting on the structure it is necessary to find the three component displacements of the foundation: The displacement  $\Delta_x$  along the  $x$ -axis; the displacement  $\Delta_y$  along the  $y$ -axis; and the angle of rotation,  $\alpha$ . These displacements are found by setting up three simultaneous equations expressing the equilibrium of the foundation under the action of the external forces and the pile forces induced by these displacements.

Let the components of the external forces, acting in the positive directions of the axes, and their moment about the origin, be  $X$ ,  $Y$ , and  $M$ . The forces induced by the displacement  $\Delta_x$  are then  $X_x \Delta_x$  and  $Y_x \Delta_x$  along the  $x$ -axis and the  $y$ -axis, respectively; and  $M_x \Delta_x$ , a moment about the origin.

The forces produced by the displacements  $\Delta_y$  and  $\alpha$  are expressed in a similar manner. The equations of equilibrium of the footing are then

$$X_x \Delta_x + X_y \Delta_y + X_\alpha \alpha + X = 0 \dots \dots \dots (1a)$$

$$Y_x \Delta_x + Y_y \Delta_y + Y_\alpha \alpha + Y = 0 \dots \dots \dots (1b)$$

and

$$X_\alpha \Delta_x + Y_\alpha \Delta_y + M_\alpha \alpha + M = 0 \dots \dots \dots (1c)$$

from which the component displacements of the footing— $\Delta_x$ ,  $\Delta_y$ , and  $\alpha$ —are determined. Once these component displacements are known, the displacements of the individual pile heads are found by geometry, and from them the axial, lateral, and rotational pile loads are easily computed.

## PRINCIPAL AXES

In a possible modification of the procedure the coordinate axes, together with their origin, may be moved to a position outside the footing base, so located that a displacement of the footing along one of the axes would produce a force on the footing acting exactly along that axis, and the rotation about the origin would produce a moment without any force. The similarity of this approach to the analysis of fixed-ended arches by means of the elastic center may be easily recognized. In this case, the three displacements of the foundation may be found directly without solving the simultaneous equations. Although apparently simple in principle, this approach has been found quite laborious in practice and for this reason is not pursued herein.

## EXPRESSIONS FOR THE FOUNDATION CONSTANTS

Fig. 6 represents a foundation supported by several nonparallel groups of piles. Group 1 includes  $N_1$  piles making an angle  $\phi_1$  with the foundation base; group 2 includes  $N_2$  piles at an angle  $\phi_2$ , and so on—the total number of pile groups being  $g$ . The angle  $\phi$  is measured from the positive direction of the  $x$ -axis clockwise to the given pile.

The footing is displaced a unit distance parallel to itself in the positive direction of the  $x$ -axis. In view of the rigid embedment, the piles retain their original inclinations at the ends and the head of a typical pile, A, has moved to

$A_1$ , a distance  $\cos \phi_1$  along the pile and  $\sin \phi_1$  across the pile away from point A. These displacements induce pile A to act on the foundation with the following forces, applied at the point  $A_1$  or A, and shown in their proper directions in Fig. 6: A force,  $n \cos \phi_1$ , along the pile; a force,  $t_s \sin \phi_1$ , across the pile; and a

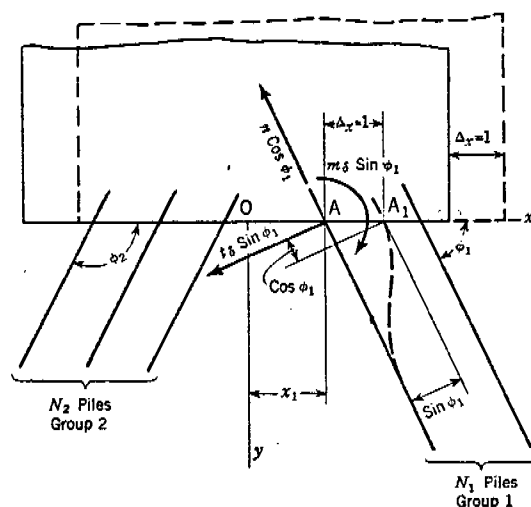


FIG. 6

moment,  $m_s \sin \phi_1$ . Similar forces are evidently applied at the heads of the other piles, and the only difference in their algebraic expressions is in the subscript of the angle  $\phi$ .

The sums of the components of all the induced pile forces in the directions of the axes represent the foundation constants  $X_z$  and  $Y_z$ . The part of  $X_z$  contributed by the two resistances of pile A is, with its proper sign,  $-(n \cos^2 \phi_1 + t_s \sin^2 \phi_1)$ . For all  $N_1$  piles of the same group this contribution is  $N_1$  times

greater. Summing up all  $g$  groups, the corresponding foundation constant is

$$X_z = - \sum_g [N (n \cos^2 \phi + t_s \sin^2 \phi)] \dots \dots \dots (2a)$$

in which the symbol  $\sum_g$  covers  $g$  terms of the form expressed by the brackets.

The contribution of pile A to  $Y_z$  is  $t_s \sin \phi_1 \cos \phi_1 - n \cos \phi_1 \sin \phi_1$ , or  $-\frac{1}{2} (n - t_s) \sin 2 \phi_1$ . Thus,

$$Y_z = - \frac{1}{2} (n - t_s) \sum_g (N \sin 2 \phi) \dots \dots \dots (2b)$$

The moment produced by the resistances of pile A about the origin is  $Y_z x_1 + m_s \sin \phi_1$ , or  $-\frac{1}{2} (n - t_s) \sin 2 \phi_1 (x_1) + m_s \sin \phi_1$ . The moment of all the piles of group 1 about the origin is  $-\frac{1}{2} (n - t_s) \sin 2 \phi_1 (x_1 + x_2 + \dots + x_{N_1}) + N_1 m_s \sin \phi_1$ , or  $-\frac{1}{2} (n - t_s) N_1 \bar{x}_1 \sin 2 \phi_1 + N_1 m_s \sin \phi_1$ , in which  $\bar{x}_1$  is the abscissa of the center of gravity of all the pile heads in group 1. Summing up these moments for all  $g$  groups,

$$M_z = - \frac{1}{2} (n - t_s) \sum_g (N \bar{x} \sin 2 \phi) + m_s \sum_g (N \sin \phi) \dots \dots \dots (2c)$$

in which  $\bar{x}$  is the coordinate of the center of the group of  $N$  piles having the angle  $\phi$ .

Fig. 7 presents the diagram needed for determining the  $Y$  constants of the foundation. The footing is moved a unit distance parallel to itself in the positive direction of the  $y$ -axis, so that the head of a typical pile of group 1 is moved from  $A$  to  $A_1$ , a distance  $\sin \phi_1$  along the pile and  $\cos \phi_1$  across the pile. The forces induced in the pile by these displacements are shown in their proper directions in Fig. 7.

Summation of the  $Y$  components of all the pile forces, and of their moments about the origin, gives

$$Y_y = - \sum_p [N (n \sin^2 \phi + t_s \cos^2 \phi)] \dots (3a)$$

and

$$M_y = - \sum_p (n \sin^2 \phi + t_s \cos^2 \phi) N x - m_s \sum_p (N \cos \phi) \dots (3b)$$

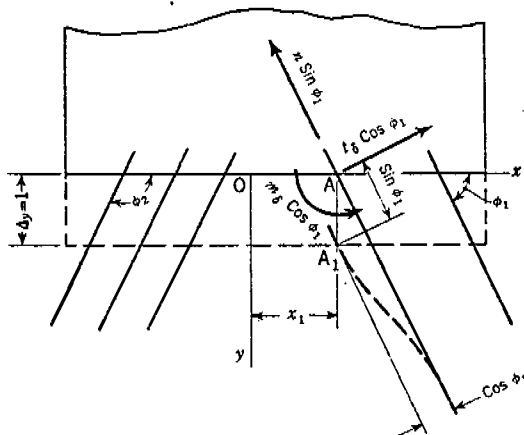


FIG. 7

In Fig. 8 the footing is rotated through a unit angle in the clockwise direction about the origin. The head of a typical pile  $A$ , a distance  $x_1$  from the origin, moves vertically downward through a distance  $x_1$ . (The correct statement would be that a rotation  $d\alpha$  results in the displacement  $x_1 d\alpha$ .) This displacement is resolved into the components  $x_1 \sin \phi_1$  along the pile and  $x_1 \cos \phi_1$  across the pile. Because of these displacements the pile exerts forces on the foundation— $n x_1 \sin \phi_1$  along the pile, and  $t_s x_1 \cos \phi_1$  across the pile—in addition to the moment,  $m_s x_1 \cos \phi_1$ , about the origin. The pile head also undergoes a unit rotation, which brings into play the transverse force  $t_\alpha$  and the moment  $m_\alpha$  in the directions shown in Fig. 8.

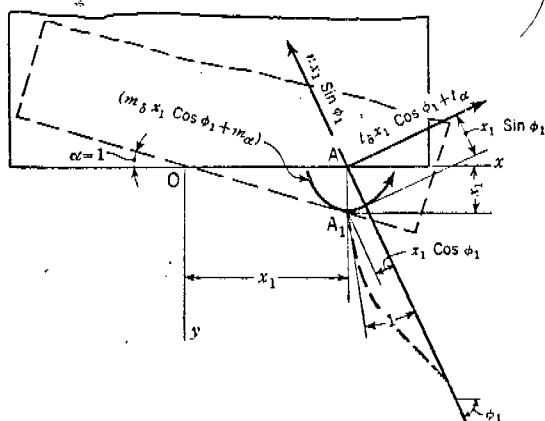


FIG. 8

of the contributions of all piles to the moment about the origin, and keeping in mind that  $m_s = t_\alpha$ , the remaining foundation constant is found to be

$$M_\alpha = - \sum_p [(n \sin^2 \phi + t_s \cos^2 \phi) \sum_N (x^2)] - 2 m_s \sum_p (N x \cos \phi) - m_\alpha \sum_p (N) \dots (4)$$

The foundation constants determined by Eqs. 2, 3, and 4, with their proper signs, represent forces and moments exerted by the piles on the footing and not by the footing on the piles. Positive signs of these functions indicate forces acting in the positive directions of the coordinate axes or, in the case of moments, in the clockwise direction. The  $x$  coordinates and the trigonometric functions in their expressions must be taken with proper algebraic signs.

Eqs. 2, 3, and 4 presuppose full embedment of the piles in the footing, which makes them behave as if they had fixed ends. When the embedment is small, a pin-ended condition ensues, and the corresponding formulas for the pile constants may be found by substituting  $m_s = m_a = 0$  in Eqs. 2, 3, and 4.

#### PILE FORCES

Substitution of the values of the foundation constants and of the external forces  $X$ ,  $Y$ , and  $M$  acting on the structure in Eqs. 1 results in determination of the foundation displacements  $\Delta_x$  along the  $x$ -axis,  $\Delta_y$  along the  $y$ -axis, and the angle of rotation  $\alpha$ . Positive values of  $\Delta_x$ ,  $\Delta_y$ , and  $\alpha$  correspond to positive directions of the axes and clockwise rotation.

Prior to determining the pile forces it is necessary to obtain the expressions for the pile head displacements. This is done in relation to an arbitrary pile with a coordinate  $x_1$  (see Figs. 6, 7, and 8) as follows: The longitudinal displacement downward,

$$\delta_l = \Delta_x \cos \phi_1 + \Delta_y \sin \phi_1 + \alpha x_1 \sin \phi_1 \dots \dots \dots (5a)$$

the transverse displacement to the right,

$$\delta_t = \Delta_x \sin \phi_1 - \Delta_y \cos \phi_1 - \alpha x_1 \cos \phi_1 \dots \dots \dots (5b)$$

and the rotation clockwise,  $\alpha$ .

The expressions for the pile forces are as follows: The longitudinal force (compression),

$$P = n \delta_l \dots \dots \dots (6a)$$

the transverse force, acting on the foundation to the right,

$$Q = -t_s \delta_t + m_s \alpha \dots \dots \dots (6b)$$

and the moment, acting on the foundation clockwise,

$$S = m_s \delta_t - m_a \alpha \dots \dots \dots (7)$$

#### SOLUTION BY RATIOS

Although the expressions for the pile loads (Eqs. 6 and 7) are functions of the pile constants, it is important to realize that it is not the constants themselves but their ratios that determine the pile loads, as may be made clear from the following. Suppose that all four pile constants decrease in the same ratio  $q$ . Then, as may be seen from Eqs. 2, 3, and 4, all the foundation constants are reduced  $q$  times. The result is that the foundation displacement, determined

from Eqs. 1, and the pile head displacements  $\delta_i$ ,  $\delta_s$ , and  $\alpha$  (Eqs. 5), all become  $q$  times greater. Turning now to the pile forces, it may be seen that Eqs. 6 and 7 are made up of the products of the pile constants and the pile displacements, and because the former functions decrease  $q$  times and the latter increase  $q$  times, the pile forces remain unchanged.

Thus, in the general case of fixed-ended piles, their loads are fully determined by the three ratios,

$$r_1 = \frac{t_d}{n} \dots \dots \dots (8a)$$

$$r_2 = \frac{m_\delta}{n} \dots \dots \dots (8b)$$

and

$$r_3 = \frac{m_\alpha}{n} \dots \dots \dots (8c)$$

When the piles behave as having pinned ends, only one ratio,  $r_1$ , remains,  $r_2$  and  $r_3$  both becoming zero.

It is preferable to approach the problem by the method of ratios rather than that of pile constants, not only because of the smaller number of coefficients involved, but also because  $r_1$ , the most important of the ratios, is more easily ascertainable than the pile coefficients, as made clear later. For the purpose of explaining the ratio approach new quan-

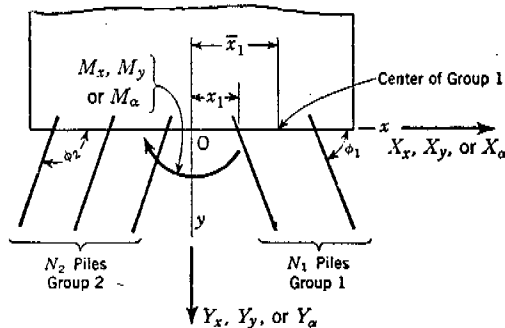


FIG. 9

ties, referred to as the reduced foundation constants, are introduced. Their expressions are as follows (see Fig. 9):

$$X'_x = \frac{X_x}{n} = - \sum_a [N (\cos^2 \phi + r_1 \sin^2 \phi)] \dots \dots \dots (9a)$$

$$Y'_z = \frac{X_y}{n} = \frac{Y_z}{n} = -\frac{1}{2} (1 - r_1) \sum_a (N \sin 2\phi) \dots \dots \dots (9b)$$

$$M'_x = \frac{M_x}{n} = \frac{Y_\alpha}{n} = -\frac{1}{2}(1-r_1) \sum_a (N \hat{x} \sin 2\phi) + r_2 \sum_a (N \sin \phi) \dots (9c)$$

$$Y'_y = \frac{Y_y}{n} = - \sum_a [N (\sin^2 \phi + r_1 \cos^2 \phi)] \dots \dots \dots (9d)$$

$$M'_v = \frac{M_v}{n} = \frac{Y_\alpha}{n} = - \sum_{\alpha} [(\sin^2 \phi + r_1 \cos^2 \phi) N \bar{x}] - r_2 \sum_i (N \cos \phi) \dots (9e)$$

and

$$M'_a = \frac{M_a}{n} = - \sum_{\sigma} [(\sin^2 \phi + r_1 \cos^2 \phi) \sum_N (x^2)] - 2 r_2 \sum_{\sigma} (N \pm \cos \phi) - r_3 \sum_{\sigma} (N), \quad (9f)$$

Although the foundation constants proper are unknown, the reduced constants can be easily determined, because they are expressed in terms of the pile ratios and not in terms of the pile constants. By rearranging to the form:

$$\frac{X_z}{n} (n \Delta_z) + \frac{Y_z}{n} (n \Delta_y) + \frac{M_z}{n} (n \alpha) + X = 0 \dots (10)$$

and substituting

$$\Delta'_z = n \Delta_z \dots (11a)$$

$$\Delta'_y = n \Delta_y \dots (11b)$$

and

$$\alpha' = n \alpha \dots (11c)$$

for the expressions in the brackets, named the reduced foundation movements, Eqs. 1 become

$$X'_z \Delta'_z + Y'_z \Delta'_y + M'_z \alpha' + X = 0 \dots (12a)$$

$$Y'_z \Delta'_z + Y'_y \Delta'_y + M'_y \alpha' + Y = 0 \dots (12b)$$

and

$$M'_z \Delta'_z + M'_y \Delta'_y + M'_x \alpha' + M = 0 \dots (12c)$$

—from which  $\Delta'_z$ ,  $\Delta'_y$ , and  $\alpha'$  can be determined. The actual foundation movements and the pile head displacements cannot be found by this approach, but the pile loads are easily determined as follows:

From Eqs. 5, the reduced pile head displacements are

$$\delta'_1 = n \delta_1 = \Delta'_z \cos \phi_1 + \Delta'_y \sin \phi_1 + \alpha' x_1 \sin \phi_1 \dots (13a)$$

$$\delta'_2 = n \delta_2 = \Delta'_z \sin \phi_1 - \Delta'_y \cos \phi_1 - \alpha' x_1 \cos \phi_1 \dots (13b)$$

and  $\alpha'$ .

From Eqs. 6 and 7 the pile loads (see Fig. 10) are given by

$$P = \delta'_1 \dots (14a)$$

and

$$Q = -r_1 \delta'_1 + r_2 \alpha' \dots (14b)$$

$$S = r_2 \delta'_1 - r_3 \alpha' \dots (15)$$

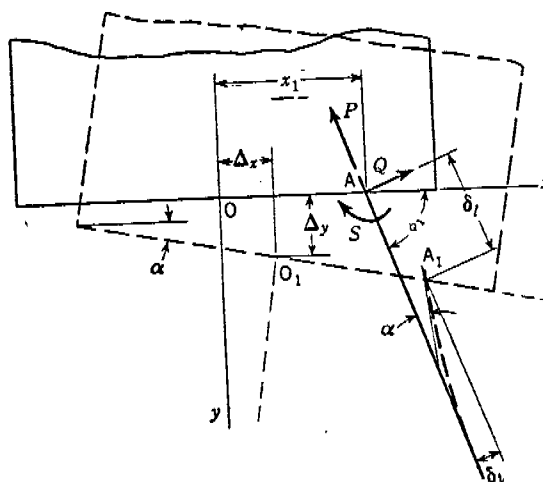


FIG. 10

#### APPROXIMATE THEORY

Determination of the numerical values of the pile ratios, as well as of the pile constants, is at best a rather uncertain procedure. Fortunately, however, the difficulty arising because of this uncertainty can be overcome by means of a moderate approximation, which at the same time adds to the practicability



of the method. Comparative calculations have shown that it is possible, without serious error, to ignore completely the ratios  $r_2$  and  $r_3$ , and to express the remaining ratio,  $r_1$ , in terms of the data possessed by the designer.

It is interesting to note that the ratios  $r_2$  and  $r_3$  (or, which is just the same, the constants  $m_s$  and  $m_a$ ) are disregarded in the conventional method of analysis of vertical piles (Fig. 11). According to this method, the axial load on the extreme pile (at a distance  $x_1$  from the origin to the pile center) is

$$P = \frac{Y}{N} + \frac{M x_1}{\sum x^2} \dots \dots \dots (16)$$

which clearly omits the effects of  $m_s$  and  $m_a$ . Actually, the influence of  $m_s$  is manifested in the end moments on the piles, due to the horizontal displacement of the foundation produced by the horizontal force  $X$ , thus augmenting the external moment  $M$  in its effect on the values of the longitudinal pile loads. At the same time the rotation of the footing caused by inequality of the longitudinal pile loads results in formation of the end moments, which are dependent on  $m_a$ , and somewhat reduce the vertical pile loads. Thus, the usual design practice, when applied to foundations supported by the vertical piles alone, definitely ignores the effect of  $r_2$  and  $r_3$  without explicitly stating so.

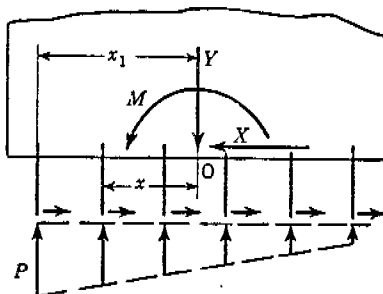


FIG. 11

Analysis of pile foundations by means of a single ratio  $r_1$  involves omitting from the equations concerned terms containing the ratios  $r_2$  and  $r_3$ . The moment on the end of the pile (Eq. 15) is, of course, eliminated. The complete set of equations required for the pile load analysis by the approximate theory includes, in addition to Eqs. 9a, 9b, 9d, 11, 12, and 14a.

$$M'_x = X'_a = -\frac{1}{2} (1 - r_1) \sum_0 (N x \sin 2\phi) \dots \dots \dots (17)$$

$$M'_y = Y'_a = - \sum_0 [(\sin^2 \phi + r_1 \cos^2 \phi) N x] \dots \dots \dots (18)$$

$$M'_a = - \sum_0 [(\sin^2 \phi + r_1 \cos^2 \phi) \sum_N (x^2)] \dots \dots \dots (19)$$

and

$$Q = -r_1 \delta'_t \dots \dots \dots (20)$$

Eqs. 17, 18, 19, and 20, as well as Eqs. 9a, 9b, 9d, 11, 12, 13, and 14a, involve no approximation when the piles behave as hinged at the top ends, but the values of  $r_1$  for the pin-ended condition and the fixed-ended condition are quite different.

If the allowable loads on the pile in the axial and transverse directions are  $P_a$  and  $Q_a$ , respectively, and the pile head displacements caused by these loads are  $(\delta_a)_t$  and  $(\delta_a)_t$ ,

$$\frac{P_a}{Q_a} = n = \frac{P_a}{(\delta_a)_t} \dots \dots \dots (21a)$$

and

$$l_b = \frac{Q_a}{(\delta_a)_t} \dots \dots \dots (21b)$$

Substituting Eqs. 21 in Eq. 8a and assuming that for properly established allowable loads  $(\delta_a)_t = (\delta_a)_t$ , results in

$$r_1 = \frac{Q_a}{P_a} \dots \dots \dots (22)$$

Inasmuch as the lateral pile resistances, as well as the axial resistances, are constantly relied upon in the usual design, the values of both  $P_a$  and  $Q_a$  must be known to the designer, whether by code, by test, or by guessing. Therefore, the ratio  $r_1$  must also be known to him. This ratio can be wrong only if unreasonable or inconsistent values have been chosen for  $Q_a$  and  $P_a$ , in which case the resultant error in all fairness cannot be charged to the method, just as the theory of reinforced concrete cannot properly be charged with the discrepancy arising in the case of a reinforced concrete structure when the design is based on the assumption of 3,000-lb concrete and the actual strength of concrete in the structure is only 1,500 lb per sq in.

The comparison with reinforced concrete is instructive also from another viewpoint. It is well known that the relation between stress and strain in concrete is neither strictly linear, even at the beginning of the stress-strain curve, nor fully elastic. Furthermore, the so-called secant modulus, even in concrete of the same strength characteristics, is a quantity quite variable for different specimens and much different from the generally accepted value of  $E = 1,000 f'_c$ , yet the conventional design of reinforced concrete is seldom questioned with regard to the bending stresses in concrete and steel. It seems reasonable to adopt a similar attitude in the design of pile foundations. Then  $r_1$  may be looked upon as a relative measure of pile resistances, even when the dependence of the allowable pile loads on the displacements becomes obscure.

It appears that a moderate variation in the assumed value of  $r_1$  does not alter the values of the pile loads a great deal. If for some special reasons the assumed allowable pile loads  $Q_a$  and  $P_a$  are based on different values of the displacements  $\delta_{at}$  and  $\delta_{at}$ , a simple adjustment can be made in Eq. 22.

The actual values of the pile loads  $P$  and  $Q$ , found by Eqs. 14a and 20, should not exceed the allowable values so that

$$P \leq P_a \dots \dots \dots (23a)$$

and

$$Q \leq Q_a \dots \dots \dots (23b)$$

Substituting in Eq. 23b the expression for  $Q$  from Eq. 20, and eliminating the minus sign as immaterial, it is found that

$$r_1 \delta'_t = \frac{Q_a}{P_a} \delta'_t \leq Q_a \dots \dots \dots (24a)$$

or

$$\delta'_t \leq P_a \dots \dots \dots (24b)$$

Compliance with Eq. 24b, which is equivalent to Eq. 23b, insures that the lateral pile load and the lateral displacement of the pile head are not excessive.

When stated in the form:

$$\delta_t \leq \frac{P_a}{n} \dots \dots \dots (24c)$$

Eq. 24b obviously means that the lateral pile head movement must not exceed the axial movement caused by the allowable longitudinal load. Evidently, this condition must be satisfied by piles in all cases, whether they are analyzed by a single ratio,  $r_1$ , or by all three ratios.

When the lateral pile resistance is negligible,  $r_1$  may be assumed to be zero in the expressions for the foundation constants and for the pile loads. However, the condition expressed by Eq. 24b still holds in this case and warns against an excessive lateral pile movement, similar to that discussed under the heading "Pile Foundations."

In most cases the piles are nearly vertical, with the result that the value of the horizontal movement of the foundation  $\Delta'_x$  is fairly close to the value of the lateral pile movement  $\delta_t$ ; for this reason the condition:

$$\Delta'_x \leq P_a \dots \dots \dots (25)$$

may be used as approximately equivalent to Eq. 24b. Thus, Eqs. 23a and 25, or Eqs. 23a and 24b, may be used in all cases as the criteria of adequacy of the design, in lieu of Eqs. 23.

#### APPROXIMATE FORMULAS FOR THE PILE CONSTANTS

Determination of the pile constants  $n$ ,  $t_s$ ,  $m_s$ , and  $m_a$  or of the three ratios  $r_1$ ,  $r_2$ , and  $r_3$  need not be considered as a necessary part of the present investigation. However, in order to check the agreement of the approximate and the more exact analyses of the pile foundation, it has been found desirable to make a rough computation of these constants. The method used is admittedly crude, being based on a number of rather arbitrary assumptions. This circumstance, however, should not detract from the usefulness of the numerical investigation founded on it, because its main purpose lies not in determining the true values of the coefficients, but rather in establishing their order for different foundation conditions, with the view of demonstrating the closeness of the approximate and the more exact pile analyses under virtually all possible combinations of piles and soils. The range of the primary physical coefficients determining the values of the pile constants will be chosen so wide that the true values of the latter are not likely to go beyond the limits assumed in the investigation.

#### VALUES OF $n$

It follows from Eq. 21a that small values of  $n$  occur with lightly loaded piles which settle a great deal, whereas large values of  $n$  occur when the piles are loaded heavily but their settlements are small. Allowance must be made for both the deformation of the soil under the foot of the pile and the elastic shortening of the pile. A  $\frac{1}{2}$ -in. axial displacement of the head under the allowable load will be considered a large settlement; the small settlement will be

taken equal to the elastic shortening of the pile alone. In the latter case a distinction will be made between a very weak soil (so soft that it offers no axial support to the pile), in which the pile is assumed to be driven to a firm stratum below so that all its axial support comes from the foot reaction, and other soils, in which the pile is assumed to be supported by skin friction alone. The pile shortenings in these two types of soils are found by the formulas:

For very weak soil,

$$(\delta_a)_t = \frac{P_a L}{A E} \dots \dots \dots (26a)$$

and for other soils,

$$(\delta_a)_t = \frac{P_a L}{2 A E} \dots \dots \dots (26b)$$

Two rather extreme classes of piles are used in the following analyses—a 30-ft timber pile, 9 in. in average diameter, and a 100-ft reinforced concrete pile, 20 in. square. The data pertaining to these piles, together with the computed values of  $n$ , are given in Table 1.

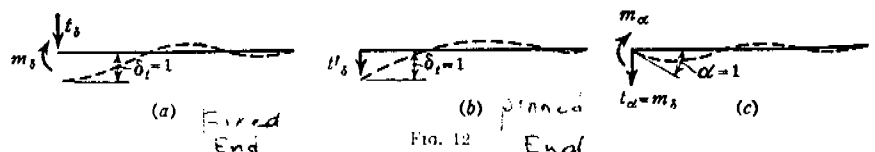
TABLE 1.—DATA AND COMPUTED VALUES OF  $n$  FOR SPECIMEN PILES

Pile	Section <sup>a</sup> (in.)	Area <sup>a</sup> (sq in.)	Modulus of elas- ticity, $E$ (kips per sq in.)	Load, $P_a$ (kips)	$(\delta_a)_t$ (In.)			$n$ (KIPS PER IN.)		
					Maxi- mum	Minimum		Mini- mum	Maximum	
						Very weak soil	Other soil		Very weak soil	Other soil
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
30-ft timber.....	9 <sup>b</sup>	63.5	1,500	30	0.5	0.1134	0.0567	60	265	530
100-ft reinforced concrete	20 <sup>c</sup>	400	3,000	140	0.5	0.14	0.07	230	1,000	2,000

<sup>a</sup> Average. <sup>b</sup> Round. <sup>c</sup> Square.

#### CONSTANTS $t_s$ , $m_s$ , AND $m_a$

Approximate values of the constants  $t_s$ ,  $m_s$ , and  $m_a$  can be determined by considering the pile as a beam on an elastic foundation of infinite length, loaded at the free end.<sup>7</sup> The end conditions necessary for determining the constants



$t_s$  and  $m_s$  are shown in Fig. 12(a), in which the transverse force is  $t_s$  and the end moment  $m_s$ , whereas the deflection at the end is unity, and the angle change is zero. For determination of  $m_a$ , the end conditions (Fig. 12(c)) are: A transverse force  $t_a = m_s$ , a moment  $m_a$ , zero end deflection, and unity

<sup>7</sup> "Strength of Materials," by S. Timoshenko, D. Van Nostrand Co., Inc., New York, N. Y., Vol. 2, 1941.

angle change. Translating the notation of S. Timoshenko into that of Fig. 12,

$$t_s = \frac{k}{\beta} \dots \dots \dots (27a)$$

$$m_s = \frac{k}{2\beta^2} = \frac{t_s}{2\beta} \dots \dots \dots (27b)$$

and

$$m_a = \frac{k}{2\beta^3} = \frac{m_s}{\beta} \dots \dots \dots (27c)$$

in which  $k$  is the elastic constant of the soil, defined as the ratio of the load per unit length of pile and the corresponding deflection of the pile at the same point, that is,

$$k = \frac{p}{\delta_t} \dots \dots \dots (28)$$

Also, the coefficient

$$\beta = \sqrt[4]{\frac{k}{4EI}} \dots \dots \dots (29)$$

in which  $I$  and  $E$  are, respectively, the moment of inertia of the cross section and the modulus of elasticity of the pile. When the pile behaves as having pinned ends (Fig. 12(b)),

$$t'_s = \frac{k}{2\beta} = \frac{1}{2} t_s \dots \dots \dots (30)$$

This method involves the approximations and assumptions that: (a) The intensity of the earth reaction at each point of the pile is proportional to the deflection of the pile at the same point; (b) the cross section of the pile is constant throughout its full length; and (c) the pile is infinitely long. Under the last condition the greatest error occurs when the pile is short and rigid and the soil is weak. Correction for the finite pile length is possible but laborious, and calculations indicate that this error is moderate even in the extreme cases.

#### ESTIMATION OF THE SOIL PRESSURE CONSTANT $k$

The value of the soil pressure constant depends on the stiffness of the soil, the size of the pile, and its cross-sectional shape. Three widely different classes of soil are considered: (1) Very weak, compressing 1 in. under a pressure of 100 lb per sq ft; (2) weak, compressing 1 in. under a pressure of 1 kip per sq ft; and (3) medium, compressing 1 in. under a pressure of 10 kips per sq ft. Generally speaking, soil firmer than medium is not likely to require the presence of piles, and therefore is not considered in connection with the pile constants.

A round pile, by virtue of its shape, is believed to produce a sort of wedge action on the soil. This condition is allowed for by assuming that for a round pile the deformation is double that of a square pile of the same diameter.

The computation of  $k$  may be illustrated by the following examples, both of which assume very weak soil: 20-in. square pile,  $k = \frac{100 \times 20}{12 \times 12} = 13.88$  lb per sq in.; and 9-in.-diameter round pile,  $k = \frac{100 \times 9}{2 \times 12 \times 12} = 3.12$  lb per sq in.

The values of the pile constants  $t_s$ ,  $m_s$ , and  $m_a$ , together with values of  $k$  and  $\beta$ , corresponding to different piles and soils are given in Table 2.

It is interesting to present at this point the available result of an independent determination of the pile constants. In a rather elaborate theoretical in-

TABLE 2.—VALUES OF  $t_s$ ,  $m_s$ ,  $m_a$ ,  $k$ ,  $\beta$ ,  $n$ ,  $r_1$ ,  $r_2$ , AND

Pile	Section <sup>a</sup> (in.)	$I$ (in. <sup>4</sup> )	$E$ (kips per sq in.)	Type of soil	$k$ (lb per sq in.)	1,000 $\beta$ (in. <sup>-1</sup> )	$t_s$ (kips per in.)	$m_s$ (in-kip per in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
30-ft timber	9-in. round	322	1,500	Very weak	3.12	6.34	0.493	38.9
				Weak	31.2	11.28	2.77	123
				Medium	312	20.03	15.59	389
100-ft reinforced con- crete	20-in. square	13,333	3,000	Very weak	13.88	3.05	4.55	745
				Weak	138.8	5.43	25.6	2,300
				Medium	1,388	9.65	143.7	7,450

vestigation, based on the data of lateral pile loading tests performed by L. B. Feagin, M. ASCE, A. E. Cummings,<sup>a</sup> M. ASCE, arrives at the following values (in this paper called the pile constants) for a round timber pile of 12-in. average diameter, with  $E = 1.878$  kips per sq in., in the soil described as medium sand— $t_s = 32$  kips per in. and  $m_s = 1,200$  in-kip per in. Reduction of the constants of the 9-in.-diameter timber piles in Table 2 to comparable sizes and stiffnesses gives resultant values of  $t_s = 27.3$  kips per in. and  $m_s = 895$  in-kip per in. With all the uncertainties of the underlying theories and the indefiniteness of the soil designation, the agreement of these two sets of constants must be considered as close.

The values of the three  $r$  ratios are also given in Table 2, two sets of ratios being given for each class of soil and each type of pile. The maximum values correspond to the least values of  $n$  in Table 1; the minimum values, to the greatest values of  $n$ .

#### NUMERICAL EXAMPLE

The theory presented herein is applied to the analysis of the pile foundation under the retaining wall shown in Fig. 13. The wall is supported by transverse rows of piles at 3-ft centers along the wall, five piles per row, the two back piles being vertical and the three front ones inclined at a batter of 3 on 1. The earth pressure is found by Rankine's theory, using the following data: The unit weight of concrete is 150 lb per cu ft;  $w$ , the unit weight of soil, is 100 lb per cu ft; and  $\theta$ , the angle of friction of the soil, is  $30^\circ$ . The earth pressure at the back is represented by the active pressure:

$$E_a = \frac{1}{2} w h^2 \frac{1 - \sin \theta}{1 + \sin \theta} \dots \dots \dots (31a)$$

<sup>a</sup> Discussion by A. E. Cummings of "Lateral Pile-Loading Tests," by L. B. Feagin, *Transactions, ASCE*, Vol. 102, 1937, p. 255.

At the front it is represented by one half of the passive pressure or

$$\frac{1}{2} E_p = \frac{1}{2} \left( \frac{1}{2} \right) w h^2 \frac{1 + \sin \theta}{1 - \sin \theta} \dots \dots \dots (31b)$$

Both these pressures are horizontal and, to avoid complicating the problem by

$r_3$  FOR SPECIMEN PILES IN VARIOUS TYPES OF SOILS

$m_\alpha$ (in-kip per radian)	$n$ (KIPS PER IN.)		$r_1$		$r_2$ (IN.)		$r_3$ (IN. <sup>2</sup> )	
	Maximum	Minimum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
6,140	265	60	0.00186	0.00821	0.1468	0.648	23.16	102.3
10,900	530	60	0.00523	0.0462	0.232	2.05	20.57	181.9
19,400	530	60	0.0294	0.260	0.734	6.48	36.6	323.3
244,000	1,000	280	0.00455	0.01625	0.745	2.66	244	872]
435,000	2,000	280	0.0128	0.0913	1.180	8.43	217.5	1,555
772,000	2,000	280	0.0719	0.513	3.725	26.6	386	2,760

\* Average.

further uncertainties of soil mechanics, are assumed to have the same values in all cases of pile-soil combination, irrespective of the displacements of the foundation and structure. The origin of the coordinates is placed at the central pile and the external forces acting on the 3-ft section of the wall, referred to the origin, are shown in Fig. 13.

The problem is solved repeatedly using all twelve sets of the pile ratios and considering first that all the ratios  $r_1$ ,  $r_2$ , and  $r_3$  are effective (cases 2 to 13, Table 3). Then the ratio  $r_1$  alone is considered to be effective (cases 2a, to 13a). In addition, the problem is solved by disregarding all pile resistances other than axial (case 1).

Employment of large and closely spaced reinforced concrete piles in a structure of this size may be criticized as incongruous. Nevertheless, although such piles would admittedly be too large for the conditions indicated, the analysis based on the ratios peculiar to them may be looked upon as the one

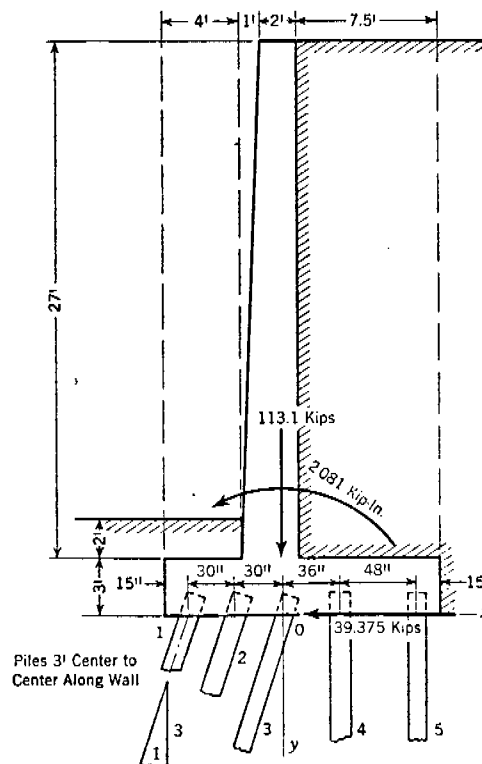


FIG. 13

presenting the picture of mechanical behavior of these piles in larger and more heavily loaded structures. As far as the structure in question is concerned, the piles endowed with the ratios corresponding to the large reinforced concrete piles will be considered as having the same moderate allowable load as the timber piles. This load in all cases is taken as  $P_a = 40$  kips.

TABLE 3.—PILE LOADS FOR WALL OF FIG. 13

Pile (1)	Type of soil (2)	Case No. (3)	PILE RATIOS			REDUCED FOUNDATION MOVEMENTS			PILE No. 1		
			$r_1$ (4)	$r_2$ (in.) (5)	$r_3$ (in.) (6)	$\Delta^1_x$ (kips) (7)	$\Delta^1_y$ (kips) (8)	$\alpha^1$ (kips per in.) (9)	$P$ (kips) (10)	$Q$ (kips) (11)	$S$ (in-kip) (12)
Timber, 9 in. round 30 ft long	Very weak	1	0	0	0	-311.1	-40.8	+0.638	+23.5	....	....
		2	0.00186	0.1468	23.16	-212.6	-19.32	+0.3384	+29.7	+0.45	-39
		2a	0.00186	....	....	-233.0	-23.84	+0.413	+27.5	+0.44	....
	Weak	3	0.00821	0.648	102.3	-113.6	+2.136	+0.0405	+35.6	+0.91	-74
		3a	0.00821	....	....	-129.9	-1.603	+0.1176	+32.9	+1.03	....
		4	0.00523	0.232	20.57	-149.6	-5.74	+0.1590	+32.8	+0.80	-37
	Medium	4a	0.00523	....	....	-162.3	-8.58	+0.2101	+31.2	+0.84	....
		5	0.0462	2.05	181.9	-42.4	+17.44	-0.1525	+38.6	+1.16	-37
		5a	0.0462	....	....	-39.0	+18.05	-0.1429	+37.6	+1.32	....
		6	0.0294	0.734	36.6	-54.8	+14.75	-0.1108	+37.6	+1.25	-29
		6a	0.0294	....	....	-54.9	+14.64	-0.0973	+36.8	+1.34	....
		7	0.260	6.48	323.3	-17.50	+22.55	-0.2043	+38.5	+0.13	+30
		7a	0.260	....	....	-12.65	+23.57	-0.2158	+38.6	+0.12	....
Reinforced concrete, 20 in. square 100 ft long	Very weak	8	0.00455	0.745	244	-135.5	-2.448	+0.0854	+35.7	+0.65	-118
		8a	0.00455	....	....	-173.2	-10.92	+0.2417	+30.7	+0.78	....
		9	0.01625	2.66	872	-78.0	+10.02	-0.0805	+38.7	+0.91	-114
	Weak	9a	0.01625	....	....	-83.3	+8.52	-0.0162	+35.4	+1.24	....
		10	0.0128	1.18	217.5	-87.4	+7.89	-0.0407	+37.4	+0.97	-85
		10a	0.0128	....	....	-97.8	+5.39	+0.0254	+34.6	+1.17	....
		11	0.0913	8.43	1,555	-36.58	+18.57	-0.1561	+38.0	+1.04	+23
		11a	0.0913	....	....	-23.98	+21.24	-0.1852	+38.3	+1.14	....
	Medium	12	0.0719	3.725	386	-34.65	+19.08	-0.1720	+33.8	+1.05	-21
		12a	0.0719	....	....	-28.12	+20.39	-0.1737	+38.1	+1.22	....
		13	0.513	26.6	2,760	-17.70	+21.92	-0.1582	+35.5	-0.68	+253
		13a	0.513	....	....	-9.69	+23.91	-0.2196	+38.3	-1.35	....

The method of analysis of the pile loads is illustrated by the following example (case 2, Table 3), in which the minimum pile ratios are assumed for the timber pile in very weak ground. From Table 2,  $r_1 = 0.00186$ ;  $r_2 = 0.1468$  in.; and  $r_3 = 23.16$  in. For the first group of piles:  $N_1 = 3$ ;  $\phi_1 = 108^\circ 26'$ ;  $\sin \phi_1 = \frac{3}{\sqrt{10}} = 0.94869$ ;  $\cos \phi_1 = -\frac{1}{\sqrt{10}} = -0.31623$ ; and  $\sin 2\phi_1 = -0.6$ . For the second group of piles:  $N_2 = 2$ ;  $\sin \phi_2 = 1$ ;  $\cos \phi_2 = 0$ ;  $\sin 2\phi_2 = 0$ ; and  $\phi_2 = 90^\circ$ .



Thus,

$$X'_x = - [3 (0.1 + 0.00186 \times 0.9) + 2 \times 0.00186] = - 0.30874$$

$$X'_y = Y'_x = - \frac{1}{2} (1 - 0.00186) 3 (- 0.6) = 0.89833$$

$$M'_x = X'_x = - 0.89833 (- 30) + 0.1468 (3 \times 0.94869 + 2) = - 26.239 \text{ in.}$$

$$Y'_y = - [3 (0.9 + 0.00186 \times 0.1) + 2 \times 1] = - 2.7006 - 2 = - 4.7006$$

# UNDER VARIOUS COMBINATIONS OF PILE RATIOS

PILE LOADS											
PILE No. 2			PILE No. 3			PILE No. 4			PILE No. 5		
P (kips)	Q (kips)	S (in-kip)	P (kips)	Q (kips)	S (in-kip)	P (kips)	Q (kips)	S (in-kip)	P (kips)	Q (kips)	S (in-kip)
(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)
+41.6	....	....	+59.8	....	....	-17.8	....	....	+12.7	....	....
+39.4	+0.44	-39	+49.0	+0.44	-38	-7.1	+0.45	-39	+9.1	+0.45	-39
+39.3	+0.43	....	+51.0	+0.43	....	-9.0	+0.43	....	+10.9	+0.43	....
+36.7	+0.91	-74	+37.9	+0.90	-73	+3.6	+0.96	-78	+5.5	+0.96	-78
+36.3	+1.02	....	+39.6	+1.02	....	+2.6	+1.07	....	+8.3	+1.07	....
+37.4	+0.80	-37	+41.9	+0.79	-37	0.0	+0.82	-38	+7.6	+0.82	-38
+37.2	+0.83	....	+43.2	+0.82	....	-1.0	+0.85	....	+9.1	+0.85	....
+34.3	+1.23	-40	+30.0	+1.29	-43	+12.0	+1.65	-59	+4.6	+1.65	-59
+33.5	+1.39	....	+29.5	+1.45	....	+12.9	+1.74	....	+6.1	+1.74	....
+34.5	+1.28	-30	+31.3	+1.31	-31	+10.8	+1.53	-36	+5.5	+1.53	-36
+34.0	+1.37	....	+31.2	+1.39	....	+11.1	+1.61	....	+6.5	+1.61	....
+32.7	+0.63	+17	+26.9	+1.13	+5	+15.2	+3.22	-47	+5.4	+3.22	-47
+32.5	+0.65	....	+26.4	+1.18	....	+15.5	+3.29	....	+5.4	+3.29	....
+33.1	+0.65	-118	+40.6	+0.64	-117	+0.6	+0.67	-122	+4.7	+0.67	-122
+37.5	+0.77	....	+44.4	+0.76	....	-2.2	+0.79	....	+9.4	+0.79	....
+36.4	+0.92	-116	+34.2	+0.94	-118	+7.1	+1.05	-137	+3.3	+1.05	-137
+34.9	+1.24	....	+34.4	+1.24	....	+7.9	+1.35	....	+7.2	+1.35	....
+36.3	+0.98	-86	+35.1	+0.98	-86	+6.4	+1.07	-94	+4.5	+1.07	-94
+35.3	+1.17	....	+36.0	+1.15	....	+6.3	+1.25	....	+7.5	+1.25	....
+33.6	+1.18	+12	+29.2	+1.31	-1	+12.9	+2.02	-66	+5.5	+2.02	-66
+33.1	+1.30	....	+27.8	+1.46	....	+14.6	+2.19	....	+5.7	+2.19	....
+33.9	+1.17	-27	+29.0	+1.29	-34	+12.9	+1.85	-63	+4.6	+1.85	-63
+33.2	+1.34	....	+28.2	+1.45	....	+14.1	+2.02	....	+5.8	+2.02	....
+30.9	+0.09	+213	+26.5	+0.85	+174	+16.3	+4.87	-34	+8.7	+4.87	-34
+32.0	-0.28	....	+25.8	+0.79	....	+16.1	+4.90	....	+5.5	+4.90	....

$$M'_y = Y'_x = - 2.7006 (- 30) - 2 \times 60 - 0.1468 \times 3 (- 0.31623) = - 38.843 \text{ in.}$$

$$M'_x = - \{ (0.9 + 0.00186 \times 0.1) [(- 30)^2 + (- 60)^2] + 36^2 + 84^2 \} - 2 \times 0.1468 \times 3 (- 30) (- 0.31623) - 23.16 (3 + 2) = - 12,527 \text{ in.}^2$$

The simultaneous equations are, therefore,

$$\begin{aligned} - 0.30874 \Delta'_x + 0.89833 \Delta'_y - 26.239 \alpha' - 39.375 &= 0 \\ 0.89833 \Delta'_x - 4.7006 \Delta'_y - 38.843 \alpha' + 113.1 &= 0 \\ - 26.239 \Delta'_x - 38.843 \Delta'_y - 12,527 \alpha' - 2,081 &= 0 \end{aligned}$$

from which the reduced foundation displacements are found to be

$$\alpha' = +0.3384 \text{ kip per in.}, \Delta'_y = -19.32 \text{ kips, and } \Delta'_z = -212.6 \text{ kips.}$$

The reduced displacements of the head of pile 1 are  $\delta'_1 = -212.6 (-0.31623) - 19.32 \times 0.94869 + 0.3384 (-60) 0.94869 = +29.72$  kips;  $\delta'_2 = -212.6 \times 0.94869 + 19.32 (-0.31623) - 0.3384 (-60) (-0.31623) = -214.43$  kips; and  $\alpha' = +0.3384$  kips per in. The loads on this pile are  $P = 29.72$  kips;  $Q = -0.00186 (-214.43) + 0.1468 \times 0.3384 = 0.45$  kips; and  $S = 0.1468 \times (-214.43) - 23.16 \times 0.3384 = -39.3$  in-kip. Assuming the allowable longitudinal load on the pile to be 40 kips, the criteria of adequacy of the foundation design (Eqs. 23a and 25) show that the value of  $P$  is satisfactory, but that the value of  $\Delta'_z$  is excessive.

All the data pertaining to the foundation movements and the pile loads, obtained by repeated solutions of the problem by different sets of the pile ratios, are presented in Table 3. The results have been verified by evaluating all the pile loads and by checking them by means of the three equations of statics.

#### DISCUSSION OF THE SOLUTION RESULTS

It may be observed at the outset that the range of relative stiffnesses of piles in axial and lateral directions, implied in the values of various sets of pile ratios in Table 3, is very wide, extending all the way from zero value of lateral resistance in case 1 to a lateral resistance value more than half as great as the axial resistance in case 13 ( $r_1 = 0.513$ ). Some interesting and unexpected results become apparent from Table 3.

a. The most striking feature is the excessive amount of horizontal displacement of the foundation, as indicated by the value of  $\Delta'_z$ , which quantity greatly exceeds  $P_a = 40$  kips in case of very weak and partly weak soils. Therefore, according to the lateral displacement criterion, Eq. 25, the foundation of Fig. 13 must be deemed inadequate, except in medium soil. This result may appear unexpected to many engineers.

b. Increase in the lateral stiffness of piles always leads to an increase in the lateral pile loads  $Q$ , as may well be expected. These loads are quite small, however, usually of the order of 0.5 kip to 1.5 kips per pile, so that their sum amounts to from only 2.5 kips to 7.5 kips per bent, against the external lateral load of 39.375 kips. Although the effect of the lateral pile resistances on the carrying capacity of the bent is small, its effect on the lateral rigidity of the foundation is tremendous. Thus, when the piles have no lateral resistance (case 1, Table 3)  $\Delta'_z = -311.1$  kips, but when (case 6a)  $r_1 = 0.0294$  and  $r_2 = r_3 = 0$  (that is, when the lateral pile resistance is less than 3% of the axial resistance),  $\Delta'_z = -54.90$  kips, or the lateral displacement of the foundation decreases six times. Thus, a completely misleading and much more alarming picture of the lateral foundation displacement is obtained if the lateral pile resistance is ignored, even when it is quite small.

c. Although the horizontal displacements of the foundation,  $\Delta'_z$ , are always negative (that is, are always directed to the left), the vertical displacements

$\Delta'$ , and the rotational movements  $\alpha$  vary in direction, depending on the pile-soil conditions. The combination of these displacements sometimes results in most baffling values of the pile loads.

Thus, one would naturally expect that the greatest axial load is carried by pile 1. This is so in the medium and weak soils, but in the very weak soil, especially with the less rigid timber piles, pile 3 becomes the one which is loaded most. This condition is particularly pronounced in case of piles completely devoid of lateral resistance (case 1). Then the axial load on pile 3 is  $P = 59.8$  kips, whereas on pile 1 it is only 23.5 kips. This striking result is accentuated by the negative axial load on the adjacent pile 4 and the positive load of 12.7 kips on pile 5.

In spite of this apparent irregularity of the pile loads in different groups the loads in the same group vary linearly. Thus, the loads on pile 2 are the mean between the load values on piles 1 and 3.

d. The greatest axial load when piles are endowed with lateral resistances in medium, weak, and even, in some cases, of very weak ground varies between 35 kips and 39 kips, and it mostly acts on pile 1. This means that, when the lateral pile resistances are ignored (case 1), the greatest axial pile load is exaggerated by approximately 60%, and often a wrong pile is indicated as carrying the maximum load.

e. The constancy of the maximum axial load for nearly all pile-soil combinations is also surprising. Apart from cases 1, 2, and 4, involving very weak soil and flexible piles, the greatest variation of this load amounts to only 10%.

In comparing the more exact solutions based on all three pile ratios with the approximate ones involving the same values of  $r_1$  and omitting the other ratios, the results on the whole are quite favorable to the approximate method. Both with respect to the deformations and to the pile loads, the two corresponding solutions in most cases are related in a definite manner, which may be predicted beforehand. In a few special cases this regularity is broken, evidently because of the peculiarities of the combinations of the pile ratios. These deviations are not serious, however, and the difference in the results by the two methods is nearly always quite small.

f. When the pile ratios  $r_2$  and  $r_3$  are ignored, the horizontal displacement of the foundation is usually increased by an amount close to 10%, making the approximate method that much on the safe side. In one case of very weak soil (case 8) the approximate method is about 28% on the safe side. On the other hand, in the medium soils, and partly in the weak soils, the approximate method somewhat underestimates the horizontal displacements, which under these conditions are all comparatively small.

g. The values of the lateral pile loads, in nearly all cases, are found by the approximate method to be slightly higher than the ones found by the more exact method. In the two cases in which this relation is reversed, the difference between the two methods is very small, and may possibly be attributed to the errors of computation. The amount by which the lateral pile load found by

the approximate method exceeds the value found by the exact method is usually close to 10%, although in cases 8, 9, and 10 this excess varies between 15% and 30%. Except for cases 7 and 13, involving rigid piles in medium ground, all lateral pile loads are quite small.

*h.* The greatest axial loads are usually slightly greater when found by the approximate method, the difference varying between 0% and 10%. The reverse is true in five cases, in four of which the approximate method is on the unsafe side by from 2% to 4%, and in one case (case 9), by 10%.

From these remarks it may be seen that, on the whole, the approximate method compares favorably with the exact method within the limits of the example considered—that is, for the assumed arrangement of a five-pile bent, carrying the specified forces, under virtually all possible conditions of deformability of soil and piles. Even the extreme and rather rare discrepancies of 10% on the unsafe side and of 28% on the safe side should not be considered as excessive in a subject as uncertain as that of foundation problems. It is desirable, however, to verify this conclusion on the examples of some other pile arrangements and some other combinations of the external forces  $X$ ,  $Y$ , and  $M$ .

#### CONCLUSIONS

The approximate method of analysis of pile loads presented in this paper has the following advantages:

1. It is reasonably simple and not too laborious.
2. It is free from the two common weaknesses of other current methods of analysis: Failure to investigate the horizontal foundation displacement, which is as important as the vertical displacement, and failure to make allowance for the lateral pile resistance, which leads to an enormous exaggeration of the lateral displacement accompanied sometimes by a drastic rearrangement of the pile loads, thus leading to an entirely erroneous picture of the mechanical behavior of the foundation.
3. The only physical property of piles and soil involved in the method is the ratio  $r_1$ , which can be determined simply as the ratio of the allowable lateral and axial pile loads, which data must be known to the designer, no matter what method of analysis he is prepared to use.
4. The error involved in disregarding the two minor pile ratios,  $r_2$  and  $r_3$ , present in the more exact method, appears to be not excessive.

#### ACKNOWLEDGMENT

The writer wishes to express thanks to his two former students, V. L. Mosher and J. D. Anderson, for some of the preliminary calculations involved in this research.

## DISCUSSION

JACOB KAROL,<sup>3</sup> M. ASCE.—In his desire to make the method of analysis simpler to use, Professor Hrennikoff has introduced reduced foundation constants into the basic equations of equilibrium from which the pile loads are more easily determined, and has developed a criterion,  $\Delta'_x \leq P_a$  (Eq. 25), which presumably indicates the adequacy of the design. Considering the cases in Table 3 for which  $r_2$  and  $r_3$  are zero (pin-ended piles), the criterion indicates that the even-numbered cases, 2a to 12a, are weaker than their respective odd-numbered counterparts, 3a to 13a. It will subsequently be shown that, considering displacements only, the opposite is true. Cases 2a and 8a must be deemed inadequate because the allowable axial pile load of 40 kips has been exceeded.

The actual displacements of the foundation are an inverse function of  $n$ . Thus, if the piles were infinitely rigid axially (a case approximated by very short piles driven to rock), the lateral displacement of the foundation even in case 1 would be zero, since, by Eq. 11a,  $\Delta_x = \Delta'_x/n$ . Furthermore, considering the example in Fig. 1, if it is assumed that the axial displacement of the pile is  $-3/80$  in., or one tenth of the original value, it follows that the lateral displacement of the foundation will be reduced to one tenth of 0.928 in., or 0.093 in. Most designers would probably decide that the foundation was adequate for both load and lateral displacement, even though the latter was 2.6 times as great as the vertical displacement. The examples illustrate the important fact mentioned by the author, but insufficiently emphasized, that the pile loads are functions of the "relative" values of the pile constants (pile ratios), whereas the foundation displacements are functions of the "absolute" values of these constants. Since the adequacy of a foundation is determined by both the pile loads and the foundation displacements, the designer must establish the absolute values as accurately as possible for any particular design.

Dividing through the reduced foundation movements in Table 3 by the values of  $n$  from Table 2, the actual foundation movements are obtained and are shown in Table 4. As a matter of interest, the movement of the top of the retaining wall at the inner face is also given, assuming that the wall rotates as a rigid body. The elastic deformation of the stem would have to be added to the lateral movement shown to obtain the final displacement of the top of the wall.

If a solution of the pile loads is obtained by the conventional method for vertical pile groups using Eq. 16, and a part of the transverse shear is assumed to be carried by the horizontal components of the axial loads in the batter piles, the resulting axial loads are practically the same as those shown for cases 7a and 13a. The unbalanced transverse shear, as indicated by these cases, could be assumed to be carried by the vertical piles only. Therefore, the conventional method of analysis, with this slight modification, may be considered satisfactory for "medium" soils. In addition, the lateral movement of the

<sup>3</sup> Design Engr., Howard-Needles-Tammen and Bergendoff, Kansas City, Mo.

foundation is given by the expression:

$$\Delta_x = \frac{Q}{t_s} \dots \dots \dots (32)$$

in which  $Q$  is the unbalanced transverse shear carried by one vertical pile.

The error in the interpretation of the results of Table 3 is due to the faulty definition of the value of  $r_1$  as the ratio of the allowable lateral load to the allowable axial load on the pile, and to the use of this value in developing the

TABLE 4.—MOVEMENTS OF WALL IN FIG. 13 UNDER  
VARIOUS COMBINATIONS OF PILE RATIOS

Case No.	n	FOUNDATION MOVEMENTS			MOVEMENTS AT TOP OF WALL	
		$\Delta_x$ (in.)	$\Delta_y$ (in.)	$\alpha$ (radians)	$\Delta_x$ (in.)	$\Delta_y$ (in.)
2a	265	-0.870	-0.090	+0.00156	-0.308	-0.076
3a	60	-2.164	-0.027	+0.00196	-1.458	-0.009
4a	530	-0.307	-0.016	+0.00040	-0.164	-0.012
5a	60	-0.650	+0.301	+0.00238	+0.207	+0.322
6a	530	-0.104	+0.028	-0.00018	-0.170	+0.026
7a	60	-0.211	+0.393	-0.00360	-1.507	+0.361
8a	1,000	-0.173	-0.011	+0.00024	-0.086	-0.009
9a	280	-0.298	+0.030	+0.00006	-0.277	+0.030
10a	2,000	+0.049	+0.003	+0.00001	-0.044	+0.003
11a	280	-0.086	+0.076	-0.00066	-0.324	+0.070
12a	2,000	-0.014	+0.010	-0.00009	-0.045	+0.009
13a	280	-0.034	+0.085	-0.00078	-0.317	+0.078

criterion. This definition arbitrarily assumes that the allowable transverse and vertical displacements of the pile head are the same. For example, for timber piles in medium soil, this results in a value of  $Q_a$  for case 6a of 1.18 kips and for case 7a of 10.4 kips. There is no reason why the value of  $Q_a$  should not be the same in the two cases, since the pile and the soil are the same. The discrepancy indicates that in general the allowable lateral displacement of the pile will not be equal to the allowable vertical displacement. Hence the value of  $r_1$  as originally defined in Eq. 8a, should be used in the analysis, and the criteria in Eqs. 23 should be used to determine the adequacy of the foundation. Despite these criticisms, Professor Hrennikoff has made a notable contribution to the analysis of pile foundations.

J. OWEN LAKE,<sup>10</sup> Assoc. M. ASCE.—Because of the variable physical and mechanical properties of soils the actual forces induced in piled foundations cannot be predicted with any considerable degree of accuracy no matter how mathematically refined the analysis; yet, until a rigorous solution—even though based inevitably on certain simplifying assumptions—has been made, the degree of inaccuracy inherent in empirical expressions or formulas, based on assump-

<sup>10</sup> Engr., The British Steel Piling Co., Ltd., London, England.

tions which are obviously not realized in practice, can only be surmised. Therefore, Mr. Hrennikoff's analysis of the effect of fixity at the heads of piles embedded in footings supported on nonparallel piles is of considerable value. However, the particular example presented by the author exaggerates the importance of neglecting the lateral resistance of the piles, for not only are the piles spaced at exceptionally close centers but the passive resistance of the soil in front of the retaining wall is neglected.

Under the heading "Discussion of the Solution Results," paragraph *b*, the author states that the effect of the lateral pile resistances on the carrying capacity of the bent is small but that their effect on the lateral rigidity of the foundation is tremendous; for the particular example presented in the paper the lateral displacement is shown to be reduced six times by a lateral pile resistance which is less than 3% of the axial resistance. In view of this condition it must be observed that most foundations will have a deep capping to the piles capable of providing a resistance somewhat greater than the aforementioned 3%.

It should be observed that under certain conditions it is inadmissible to allow for any lateral resistance of the piles—for instance, where piles supporting relieving platforms to sheet pile bulkheads are located within the zone of expanding soil immediately behind the wall.

Under the heading, "Existing Methods of Analysis," the author states that the method developed by Swedish engineers,<sup>3</sup> for vertical piles loaded laterally, does not give a method for determining the unknown depth to the point of fixity. Another paper, however, by Donald Hamish Little,<sup>11</sup> describes an approximate graphical method, and an analytical solution of the problem is presented by the writer<sup>12</sup> in the written discussion of Mr. Little's paper.

GEORGE S. MURPHY,<sup>13</sup> Assoc. M. ASCE.—A retaining wall supported wholly on vertical piles is subjected to translation and forward rotation about the toe. With batter piles present, however, the wall is subject to translation and backward rotation about the heel. This latter condition is brought about by the upward push of the heads of the batter piles, resulting from their rotation about an instantaneous center as the piles bend.

With the usual 30° batter, a forward translation of the pile head will produce an upward movement 0.577 times the horizontal movement. Since the height of the wall is usually about 2.2 times the base, the wall will tilt backward an amount equal to 2.2 times the upward movement of the toe. It follows that, with a forward translation,  $x$  (Fig. 14), the backward tilt of the top of the wall will be  $2.2 \times 0.577 x = 1.27 x$ , or about 25% more than the forward movement of the base.

The problem of relating these relative displacements to the pressures developed may be viewed in three ways: First, if the entire earth wedge behind the wall is considered to move forward with the displacement of the base of

<sup>11</sup> "Some Dolphin Designs," by Donald Hamish Little, *Journal, Inst. of Civ. Engrs.*, November, 1946, p. 18.

<sup>12</sup> Discussion by J. Owen Lake of "Some Dolphin Designs," by Donald Hamish Little, in Supplement, *Ibid.*, October, 1947, p. 453.

<sup>13</sup> Toledo, Ohio.

the wall, but compressed by the backward tilt, then the relative backward movement of the top of the wall against the earth is  $5x/4$  rather than  $x/4$ .

The other extreme would be to assume that the plastic flow of the earth mass occurs only where the wall permits, so that only an active pressure is exerted on the wall except for the small section at the top where the absolute movement of the wall is backward, with a maximum of  $x/4$  at the apex.

A third action is conceivable by assuming a cylindrical surface of shear failure from the heel of the wall to the surface of the fill, with the axis of rotation at the wall top (see Fig. 15). This would impose no more than active pressure on the vertical wall, but such an assumption has doubtful validity, especially since the axis of rotation also has a vertical translation.

It seems likely either that the true condition is intermediate between the first two, or that it is a mean of all three, with the several components having variable weights for different types of soil, but that, in any case, some passive earth pressure is developed by the wall when batter piles are used.

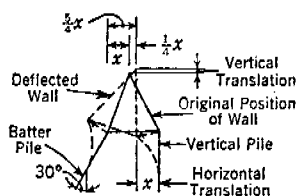


Fig. 14

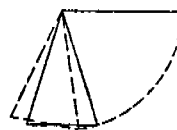


Fig. 15

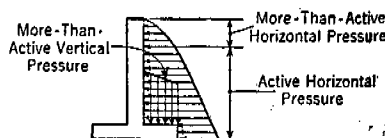


Fig. 16

When the horizontal translation is sufficiently great, uplift is developed in the most forward vertical pile. In this case, even if the pressure, in excess of all the active horizontal pressure that can be developed, is neglected (as the author has done), some passive pressure must be developed on the upper surface of the base.

Taking into account the passive pressure developed against the back of the vertical wall, all the vertical piles may be hung from the base, as a result of which some greater-than-active pressure (Fig. 16) would develop along the entire top surface at the base. The only relief from this condition is that derived from the axial compression of the batter piles; but, in the case here considered, the depression so caused is less than the rise caused by the forward translation of the batter pile head.

In summary, it would appear that the author has not considered the development of horizontal and vertical passive earth pressures in the case of walls supported partly by batter piles, in so far as such pressures affect the distribution of pile loads and the stresses in the wall.

Such pressures are generated, at any rate, in the period during which the piles are developing deflections corresponding to the maximum earth pressures. Possibly, after the lapse of sufficient time, some of the passive resistance may leach away through the continued operation of plastic adjustments of the



internal structure of the soil (such as by permanent consolidation), with the result that intensities equivalent to active pressures are approached at all points; nevertheless, it is reasonable to assume that at least some of the pressure intensities in excess of the active pressure will remain as elastic compression of the soil.

A. HRENNIKOFF,<sup>14</sup> Assoc. M. ASCE.—The misunderstanding that led to the criticism voiced by Mr. Karol is regrettable. The writer's sole reason for computing the approximate numerical values of the pile stiffness factors  $n$ ,  $t_s$ ,  $t_a$ , and  $m_a$ , for different piles and soils, was to establish the range of the pile ratios for the purpose of demonstrating that the error involved in the writer's approximate theory (based on a single pile ratio  $r_1$ ) is small when compared to the more exact theory, under virtually all possible pile-soil conditions (see under the heading, "Approximate Formulas for the Pile Constants"). Once the pile ratios are established there is no need to go back to the stiffness factors, since it is the ratios and not the stiffness factors which determine the distribution of loads among the piles of a given bent.

Accordingly, in his example, the writer makes use of the computed pile ratios, but not of the stiffness factors. In the paragraph immediately preceding Table 3 it is stated that the allowable axial pile load in all the cases listed in Table 3 is assumed to be the same—namely, 40 kips. From the viewpoint expounded by the writer, this assumption denotes that, in all cases, the piles are equally deformable axially—that is, they possess equal axial stiffness factors  $n$ . Their principal pile ratios  $r_1 = \frac{t_s}{n}$ , therefore, are directly

proportional to the lateral stiffness factors  $t_s$ ; consequently, case 2a, which has a smaller  $r_1$  than case 3a, involves a lower lateral stiffness and is more dangerous to the stability of the foundation than is case 3a, as is indicated by Table 3.

In comparing cases 2a and 3a, Mr. Karol assumes that they both involve the same lateral stiffness factors  $t_s$ , and that a smaller value of  $r_1$  in case 2a is caused by a greater axial stiffness factor  $n$ . By these assumptions he concludes correctly that case 2a is the safest; but, under the conditions prescribed by Mr. Karol, it would be incorrect to assign the same values of the allowable axial load to both cases, as is done and explicitly stated by the writer. Table 4 and its subsequent interpretation are affected by the same misunderstanding.

In connection with Mr. Lake's discussion, the writer wishes to point out that no progress can be made, in the analysis of complex engineering problems, by considering all the factors in their entirety. The proper approach is to separate the individual factors and to account for them one by one. The writer's problem was limited solely to the analysis of pile behavior in a foundation under a known set of loads, and it did not extend to a consideration of different factors affecting these loads.

Mr. Lake is quite correct in stating that the variable passive pressure in front of the pile cap would contribute substantially to the safety of the piles. The writer admittedly did not consider this factor. Moreover, even if he had known how to allow for this variation of the passive pressure, he would have

<sup>14</sup> Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.

The analysis assumes that the groundwater level in any layer below the cut will not rise above the base of the cut. Shear strength parameters assumed for the different soil layers, together with the piezometric surface assumed for all the layers, are given in Figure 3. The analysis resulted in a minimum factor of safety of 1.3 for the arc shown in Figure 3. This is considered to be adequate for the long term stability of an open highway cut.

The short term condition, allowing an additional 1 m depth of cut for drains and pavement construction, has been checked using Taylor's stability charts (Taylor, 1948). For clay with a shear strength of 30 kPa, the minimum factor of safety is 1.5.

The temporary stability of cuts in which retaining structures, including bridge abutments are to be constructed, are dependent on the bracing of the cuts and the stability against basal heave. We have not considered the former but our comments on basal heave are given above.

The long term overall stability of a cut, whose sides are supported by retaining structures on piled foundations, is complicated by the contribution of the retaining wall in resisting the destabilizing forces. A properly designed retaining wall will adequately support the earth pressures above the level of its base. A slip-circle type analysis is not considered to be realistic in this instance. We have investigated the potential for deep seated failure by considering the balance of active and passive pressures and horizontal shear surfaces in a weak layer immediately under the pile cap elevation. The situation analyzed is illustrated in Figure 5. For Panel S E with the base of the pile cap in clay, the calculated factor of safety is 2.2. The corresponding value for the bridge abutment (with the base of the pile cap in silt and fine sand) is 1.9.

We consider that the use of subgrade reaction in designing the pile group is justifiable since it results in a more realistic distribution of stresses in the piles. However, resistance to the piles is coming from the same soil which is providing the stabilizing forces  $P_p$  and  $S$  in Figure 4. In order that the factor of safety does not fall below 1.5, we recommend that the contribution of the clay and loose silt to lateral resistance of the pile group be limited to 70 kN and 52 kN per metre length of the retaining wall for panels with the pile caps on the clay and silt respectively.

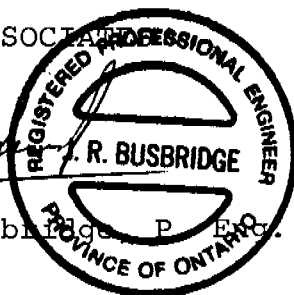
All stability analyses carried out are extremely sensitive to the assumed groundwater pressures. The potential for long term excess pore pressures has been discussed in our report 811-1376 (refer to Figure 9 of that report). It is recommended that the permanent road drainage is reviewed in the light of the pumping test carried out by Trow Ltd. to ensure that drawdown will be to at least the finished road level. In the temporary condition, the pressure in the underlying soil must be maintained below the base of the excavation at all times, without any potential for upward seepage.

We trust that this letter answers the various points raised in your letter. If further clarification is required, we will be pleased to hear from you.

Yours truly,

GOLDER ASSOCIATES

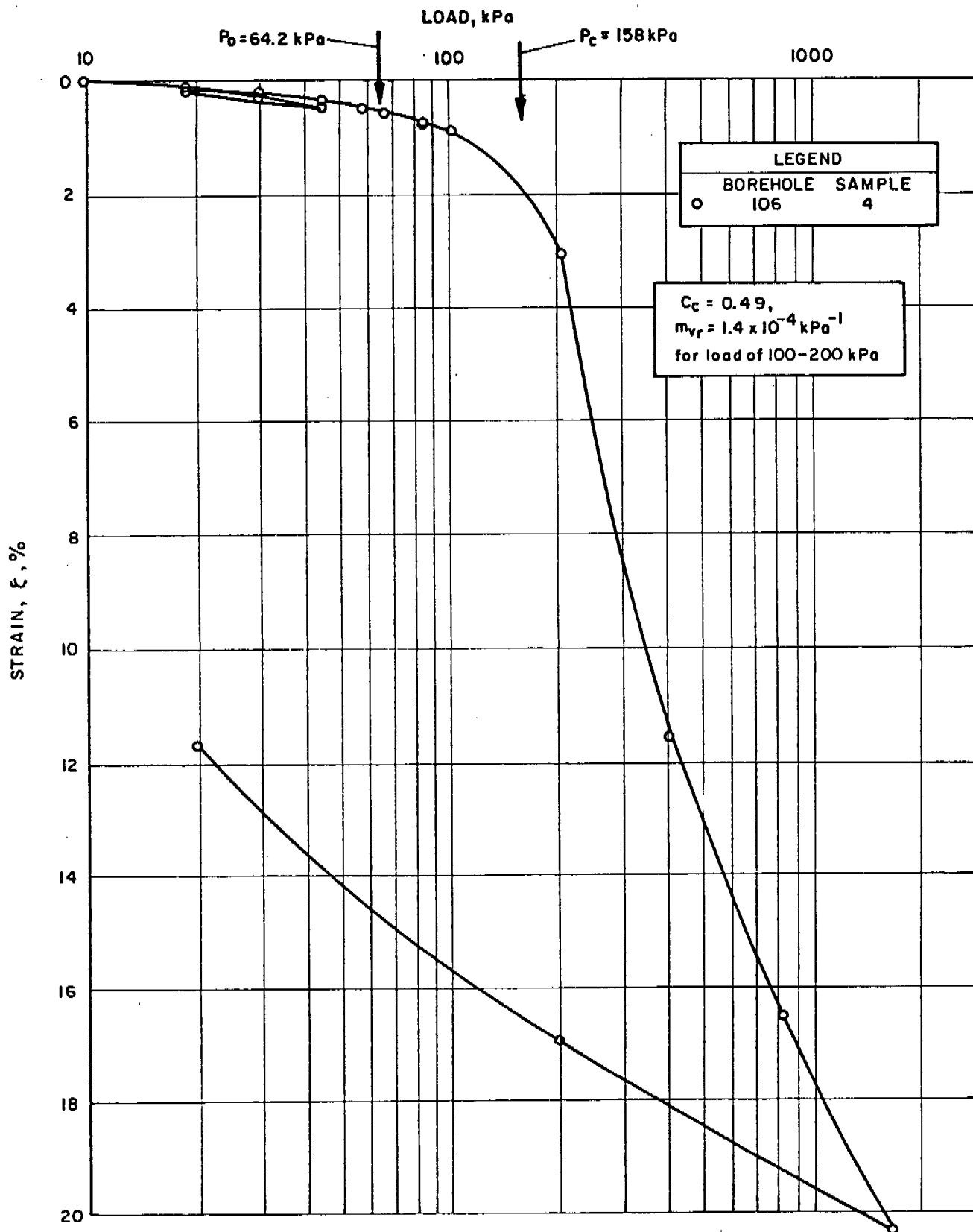
*J. R. Busbridge*  
J. R. Busbridge



JRB/cg

# CONSOLIDATION TEST RESULTS

FIGURE 7



Date MARCH 22, 1982  
 Project No 811-1376

Golder Associates

Drawn MHW  
 Chkd. BSL

# DeLCan

DE LEUW CATHER, CANADA LTD.

CONSULTING ENGINEERS AND PLANNERS

March 24, 1983

Our Ref: 01-1234

Mr. K. Kleinsteiber  
Structural Office  
Ministry of Transportation & Communications  
3501 Dufferin Street  
Downsview, Ontario  
M3K 1N6

Dear Sir:

Re: Ormont Drive Grade Separation  
City of North York

You will recall that I reported in January of this year that we had neither seen nor heard from Nyal E. Wilson & Associates Ltd. for many months.

Recently Mr. Wilson did appear on site (March 11, 1983) and reported to our Resident Engineer. Subsequently we received a copy of a letter from Mr. Wilson dated March 17, 1983 under cover of the Contractor's letter of March 18, 1983.

We enclose two copies of these letters for your information. A review of these letters has been requested of Trow Ltd. and Golder Associates.

Please call if you require further information.

Yours very truly,



W. V. Anderson, P.Eng.

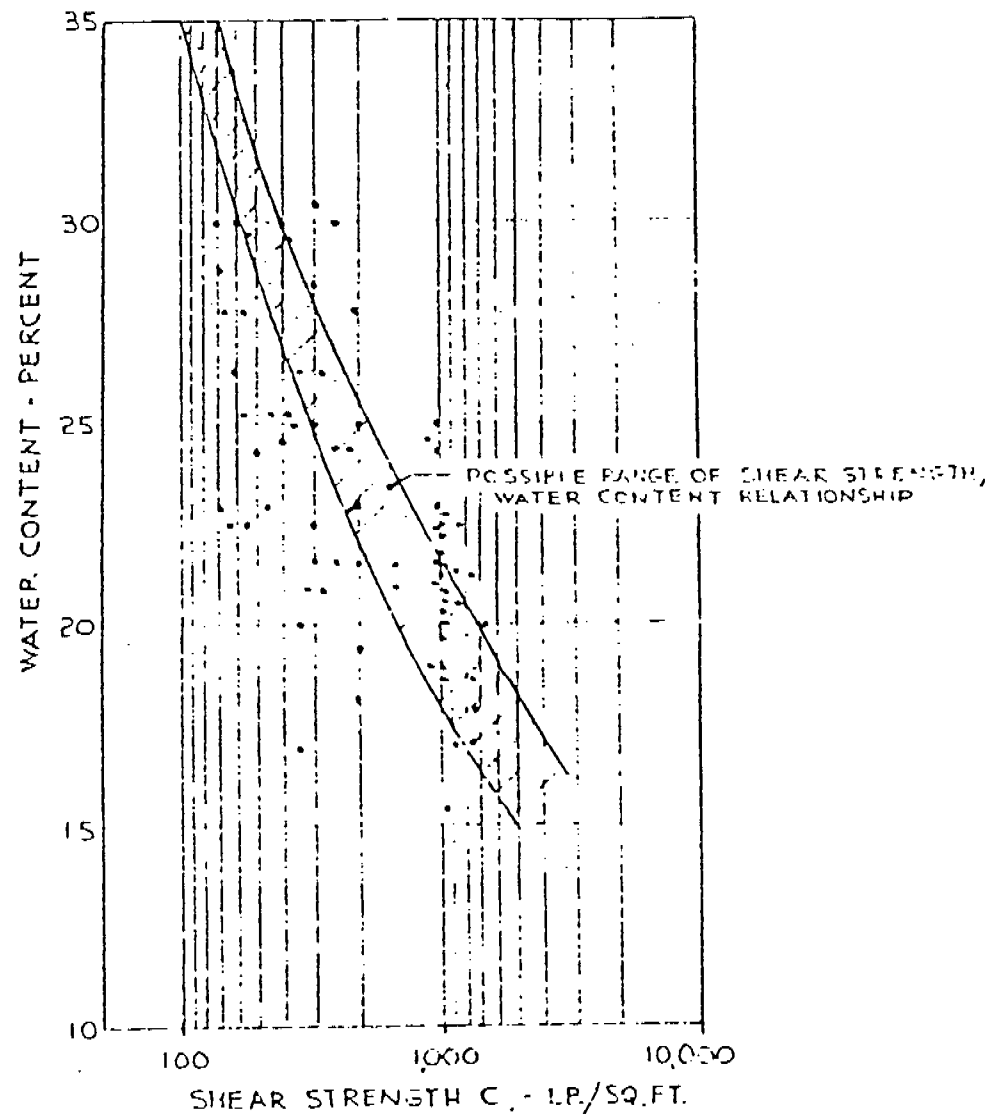
WVA:do  
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133 WYNFORD DRIVE, DON MILLS, ONTARIO M3C 1K1  
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REGINA, SASKATOON, EDMONTON, CALGARY, VICTORIA, NEW WESTMINSTER, CHILLIWACK, NANAIMO,  
KAMLOOPS, CAMPBELL RIVER, QUESNEL, PRINCE GEORGE, TERRACE, FORT ST. JOHN.

UNDRAINED SHEAR STRENGTH  
VERSUS WATER CONTENT  
SILTY CLAY - HAMILTON BAY AREA

FIGURE 1



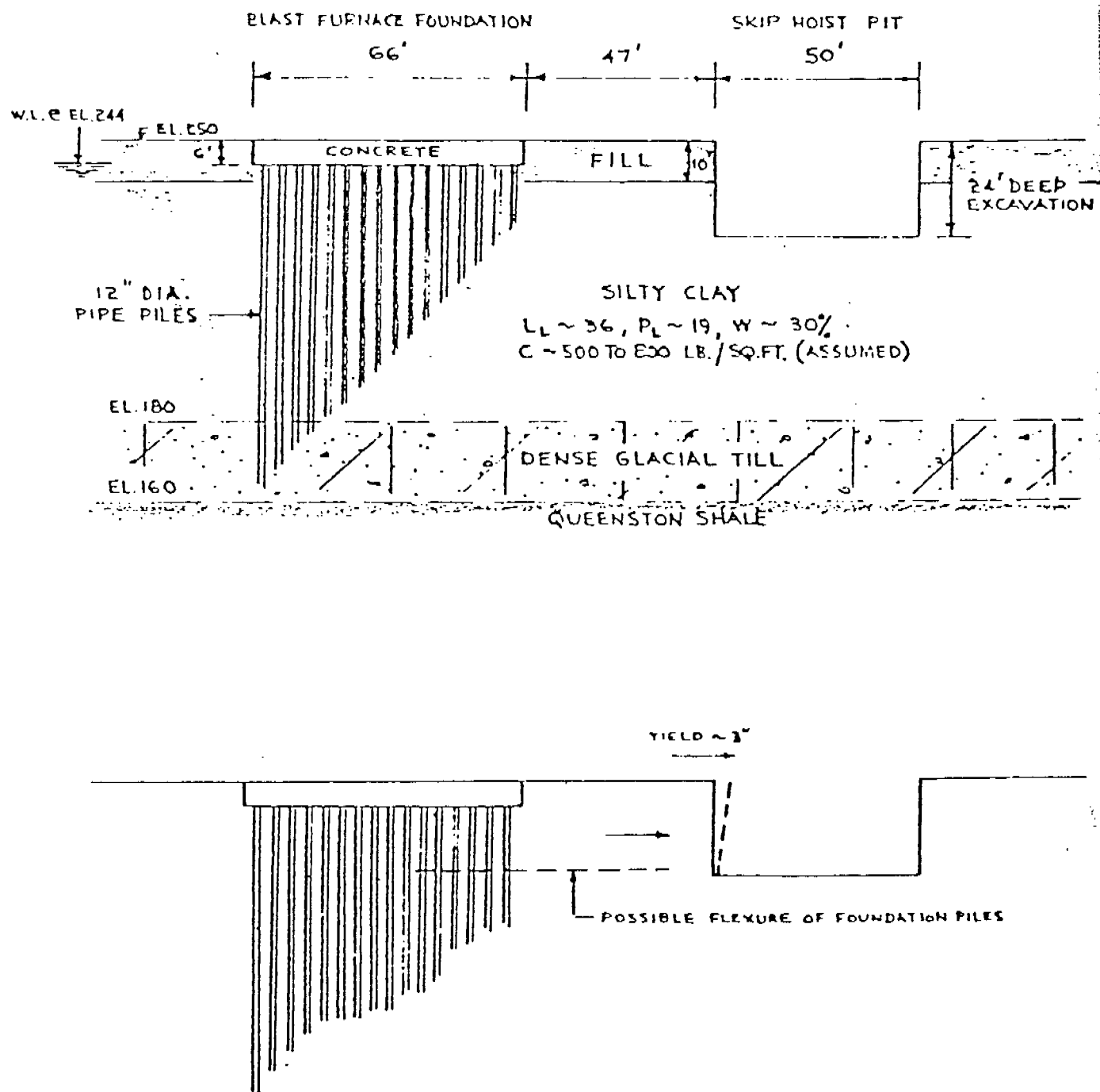
TYPICAL UNDRAINED SHEAR STRENGTH,  
WATER CONTENTS FOR SILTY CLAY,  
HAMILTON BAY AREA.

$L_L$  = 35 TO 40

$P_L$  = 18 TO 22

# SKETCH OF BLAST FURNACE FOUNDATION AREA

FIGURE 2



We therefore consider that the excavation is safe against base heave, but recommend that surcharge loading behind the sheared wall be prohibited.

### (3) Piling Type

If piles are to be used to support the retaining walls, we recommend the use of closed-end pipe piles, since they should terminate at a higher level in the dense sand.

We understand that the Hiley Formula is to be used as a criterion for finalized and pile capacity. We have no confidence in any pile driving formula as a criterion. We would recommend that the efficiency of the pile driving, including the pile driver, be checked by the Pile Driving Analyser on widely-spaced selected piles at the start of the project. This should also be done on the piles proposed for load test. With Analyser control it may be possible to eliminate one of the two proposed load tests.

For preliminary planning, we suggest that the 50-ton design piles be driven to a final set of 10 blows per inch for the last 3 inches and with the blows increasing. A driver with a rated energy of at least 20 ft. kip, with an efficiency of at least 40 per cent, is required.

### (4) Dewatering

The dewatering effect achieved to date at this site does not conform to the recommendations given in our original soil report. Our comments on this were given in Hydrology Consultants' letter dated December 2nd, 1981 to your Mr. Washington. A copy of this letter is attached.

### (5) Piles Versus Caissons

Although the use of caissons is not impossible, they would require very positive prior dewatering to below desired founding level in the dense sand. There would be a risk of faulty installation even with an experienced caisson contractor. We do not recommend their use here.



(6) Disturbance to Clay by Pile Driving

The driving of piles will cause some remoulding and temporary weakening of the clay adjacent to each pile. The extent of the weakening will depend upon the number and proximity of the piles. The weakening will reduce the factor of safety against base heave to some extent. We would not expect it to be a problem at this site, but close inspection is required to detect whether movement is taking place. With footing support this problem is avoided.

(7) Horizontal Subgrade Reaction

In view of the concerns about the retaining walls expressed in the last few months and the generally sensitive nature of the contract, we would prefer to see all horizontal loading taken in end bearing by batter piles. We do not have confidence in the use of horizontal subgrade reaction coefficients as a simultaneous supplement to end bearing resistance.

If you have any questions regarding this letter, please do not hesitate to contact this office.

Yours very truly,  
TROW LTD.

*W.A. Trow*  
W.A. Trow, P. Eng.

WAT/cb

Encls.

Dist.: DeLCan

Attention: Mr. W.V. Anderson, P. Eng. (1)

City of North York

Attention: Mr. R.D.W. Band, P. Eng. (1)

January 15, 1982

Trow

MEMO

TO: Bill Trow, Rexdale Branch

FROM: John Emery  
Mike MacKay Hamilton Branch

Re: Stability Calculations  
Ormont Drive Retaining Walls  
North York

---

Please find enclosed our calculation sheets for the factors of safety for the above noted project. The results are presented in tabular form along with Golder's safety factor values in parentheses.

We have assumed the following in our calculations:

1. Water table in granular backfill drained to weeping tiles installed at heel of wall, appropriate weeping holes, and drainage to storm drain in road, resulting in GWT at base of footing (conservative assumption) and in non-granular backfill, at Trow report elevation (Borehole 2, conservative assumption);
2. Surcharge influence added when calculating bearing safety factor;
3. Surcharge influence subtracted when determining eccentricity (inclination; conservative assumption);
4. 1/B limits not considered (will tend to reduce N<sub>y</sub> portion slightly but increase in N<sub>c</sub> and N<sub>q</sub> terms of bearing capacity formula greater; conservative assumption);
5. Clay  $c' = 0$ ,  $\phi' = 27.5^\circ$  from Golder shear strength laboratory results. Silt  $\phi' = 30^\circ$  assumed since Trow report indicates it is compact to dense, i.e.,  $\phi' = 35^\circ$  (conservative assumption).
6. Factor of 1.1 applied to silt  $\phi'$  in bearing capacity only.
7. Bona Foods building does not influence earth pressures on wall.
8. Wall sections S-C, S-E and S-F representative, and S-F probably represents worst condition since least amount of granular (granular only taken to face of lagging; conservative);

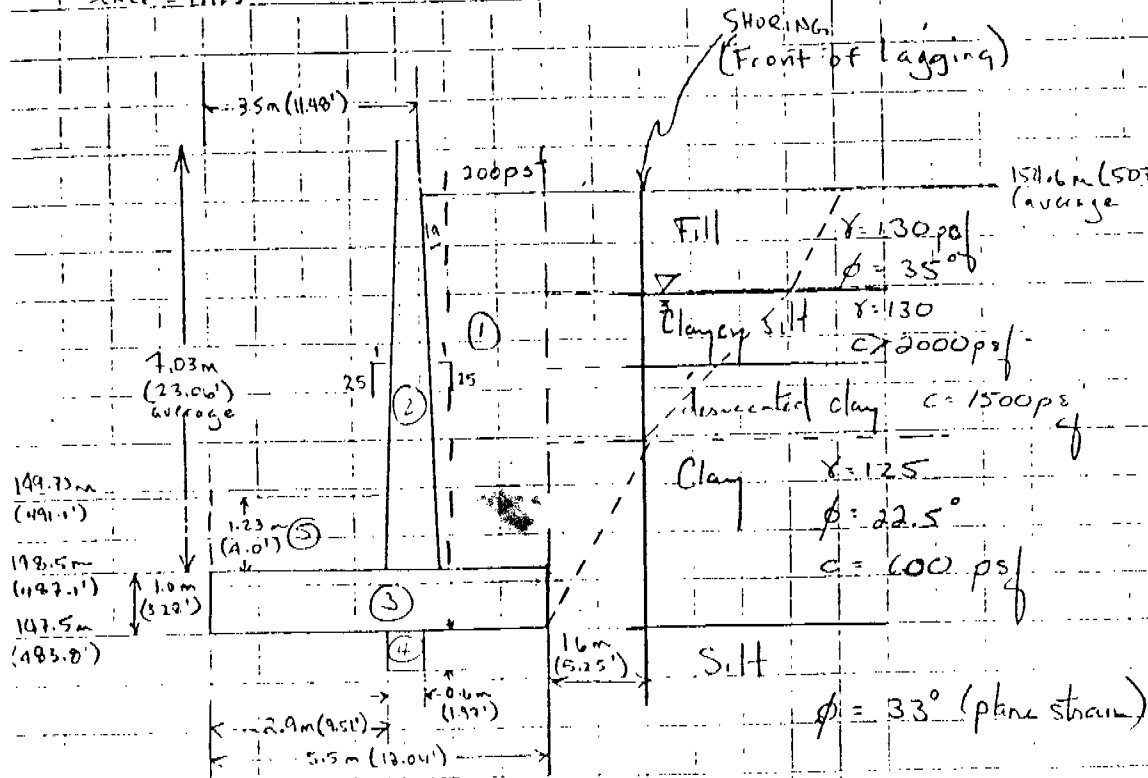
9. Passive pressure and key influences not included (conservatives).
10. It should be noted that the three major differences between our calculations and Golder's are:
  1. We have considered the positive influence of the granular backfill in reducing active pressures;
  2. We have considered a realistic degree of drainage in the backfill and at the toe.
  3. We have used somewhat higher  $\phi'$  values based on Golder's results ( $\phi' = 27.5^\circ$  for clay) and our experience ( $\phi' = 30.0^\circ$  for silt, conservative)

Our safety factors against sliding and bearing are all adequate based on generally conservative assumptions, laboratory and field data. A safety factor of  $>1.5$  in sliding (no consideration of passive or key) and  $>2.0$  in bearing (granular backfill) are typically required.

The Ontario Bridge Code suggests working stress approach for walls with non-uniform backfill. More importantly if its design requirements concerning backfill, i.e., Section 6.9.6, were followed, the safety factors would be even greater.

We conclude that a wall at this site, designed in accordance with the Ontario Bridge Code, would not require piling. Care of course must be taken to avoid foundation base disturbance, to provide adequate lowering of GWT, and to inspect footing bases during construction.

SCALE = 1:125



## SECTION S-C

### Area + Volume

$$① 5.64 \times 19.9 = 112.24$$

$$112.24 \times 7.76 = 871.18$$

$$A_1 = 120.208 \text{ ft}^2$$

$$W_1 = 120.2 \times 127 = 15.3 \text{ Kips}$$

$$② 1.25 \times 23.06 + 2 \left( \frac{1}{2} \times 0.92 \times 23.06 \right) = 50.09 \text{ ft}^2$$

$$W_2 = 50.09 \times 153 = 7.7 \text{ Kips}$$

$$③ 18.04 \times 3.28 = 59.17 \text{ ft}^2$$

$$W_3 = 59.17 \times 153 = 9.05 \text{ Kips}$$

$$④ 2 \times 2 = 4 \text{ ft}^2$$

$$W_4 = 4 \times 153 = 0.61 \text{ Kips}$$

$$⑤ 4 \times 9.51 = 38.04 \text{ ft}^2$$

$$W_5 = 38.04 \times 127 = 4.83 \text{ Kips}$$

$$\Sigma W = 37.49 \text{ Kips}$$

$$+ \text{surcharge} = \frac{1.13}{38.42}$$

$$69.3$$

Length of Panel = 10m (32.8')

In short-term:

Strength of clayey silt + dissected clay is sufficient that no pressure transferred to sand backfill  
 so only sand contributes to driving force along with surcharge

$$\begin{aligned} \text{so } P_{as} &= \frac{1}{2} K_a \gamma H^2 + K_a H q \\ &= \frac{1}{2} (0.27)(127)(10.8)^2 + 0.27(2.77)(10.2) \\ &= 1.78 + 7.62 = 9.43 \text{ Kips} \end{aligned}$$

$$I_s = \frac{H}{V} = \frac{9.43}{37.49} = 0.251 = 14.1^\circ \quad \therefore \alpha = 14.1^\circ$$

Short-term factors of safety

Bearing:

$$q_{ult} = \frac{38.62}{18.04} = 2.14 \text{ Kips / sq ft.}$$

$$q_{ult} = c N_c i_c + \gamma D N_q i_q + \frac{1}{2} \gamma B N_\gamma i_\gamma$$

$$i_c = i_q = \left(1 - \frac{\alpha}{90}\right)^2 = \left(1 - \frac{14.1}{90}\right)^2 = 0.711$$

$$i_\gamma = \left(1 - \frac{\alpha}{\phi}\right)^2 = \left(1 - \frac{14.1}{33}\right)^2 = 0.33$$

For  $\phi = 33^\circ$  (short-term)

$$N_c = 38.8$$

$$N_q = 26.3$$

$$N_\gamma = 24.8$$

$$\phi = 28^\circ \quad N_c = 25.8$$

$$N_q = 14.7$$

$$N_\gamma = 10.6$$

$$\therefore q_{ult} = c N_c i_c + \gamma D N_q i_q + \frac{1}{2} \gamma B N_\gamma i_\gamma$$

$$= 0.127 (4) (26.3) (0.711) + \frac{1}{2} (0.062) (18.04) (24.8) (0.33)$$

$$\phi = 33^\circ \quad | \quad = 9.5 + 4.6 = \underline{15.8} \quad \phi = 28^\circ \quad | \quad 5.3 + 1.5 = 6.8$$

$$F.S._{\phi=33} = \frac{14.1}{2.14} = \underline{6.6}$$

$$F.S._{\phi=28} = \frac{6.8}{2.14} = \underline{3.2}$$

Sliding:

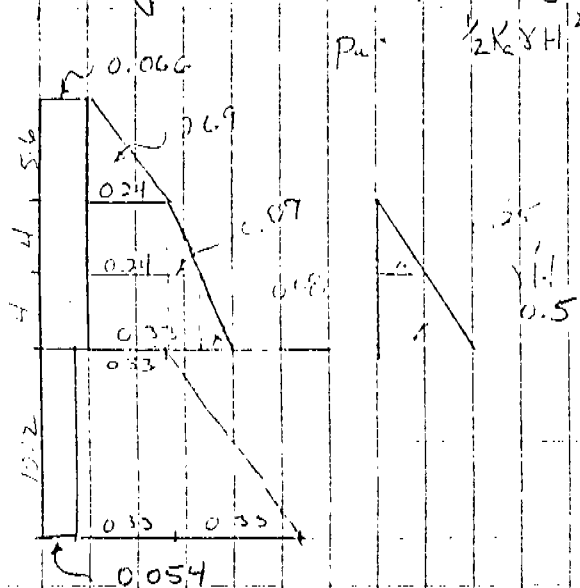
F.S.<sub>ST</sub>

$$= \frac{\Sigma \text{Resisting}}{\Sigma \text{Driving}}$$

$$= \frac{37.49 \tan 30^\circ}{7.86}$$

$$= \underline{2.75}$$

In long-term - clay strength overcome as  $C \rightarrow 0$



$$p_u = \frac{1}{2} K_a \gamma H^2 + K_a H q$$

$$p_{u1} = 0.672 + 0.369 = 1.04$$

$$p_{u2} = (0.96 + 0.18) + 0.125 = 1.26 + 0.125 = 1.52$$

$$p_{u3} = (1.32 + 0.16) + (1 + 0.5) = 2.98 + 0.5 = 3.24$$

$$p_{u4} = 3.37 + 1.78 + 0.55 = 5.7$$

$$P_u = 11.5$$

$$\alpha = \frac{11.5}{37.49} = 0.303 = 17.0^\circ$$

long-term factors of safety

$$q_{all} = 2.14 \text{ (Kips) / sq ft}$$

$$c_c = c_q = \left(1 - \frac{\alpha}{90}\right)^2 = \left(1 - \frac{17.0}{90}\right)^2 = 0.66$$

$$c_s = \left(1 - \frac{\alpha}{\phi}\right)^2 = \left(1 - \frac{17.0}{33}\right)^2 = 0.24$$

$$\phi = 33$$

$$N_c = 38.8$$

$$N_q = 26.3$$

$$N_\gamma = 24.8$$

$$\phi = 28$$

$$N_c = 25.8$$

$$N_q = 14.7$$

$$N_\gamma = 10.6$$

$$c_c = 0.66$$

$$c_q = 0.66$$

$$c_s = 0.24$$

$$q_{ult} = c N_c c_c + \gamma D N_q c_q + \frac{1}{2} \gamma B N_\gamma c_s$$

$$= 0.127 (4) (26.3) 0.66 + \frac{1}{2} (0.062) (18.04) (24.8) (0.24)$$

$$\phi = 33 \quad = 8.82 + 3.33$$

$$= 12.15$$

$$F.S._{L/S} = \frac{12.15}{2.14} = 5.68$$

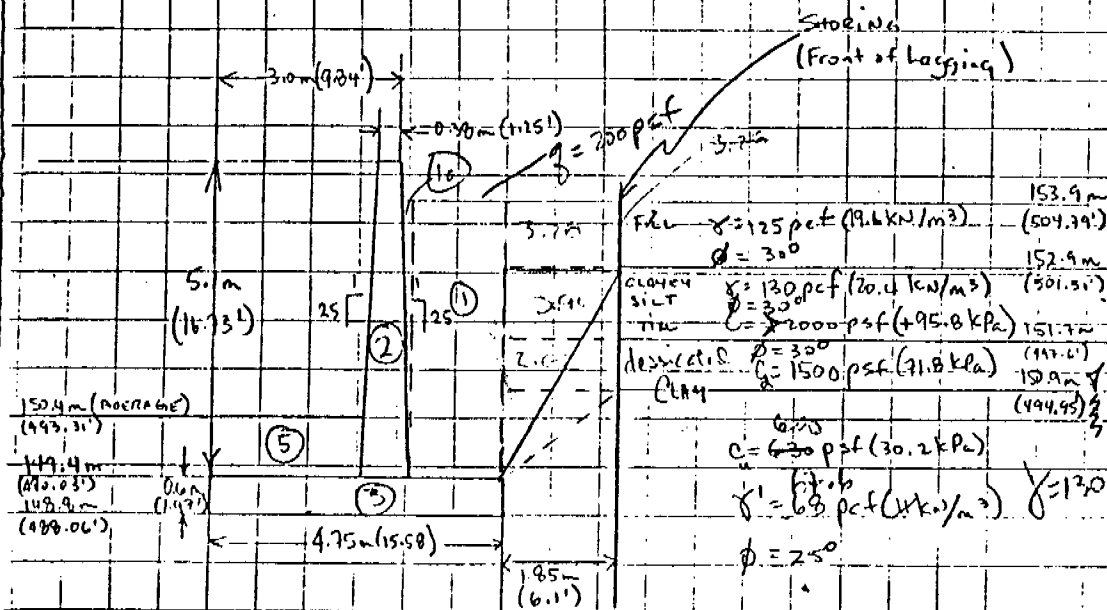
$$\phi = 28 \quad = 4.93 + 0.75$$

$$= 5.68$$

$$F.S._{\phi=28} = \frac{2.75}{0.48} = 5.73$$

$$F.S._{SI} = \frac{37.49 \tan 30}{11.5} = 1.89$$

SCALE = 1:25



LENGTH OF EACH PANEL = 10m (32.8')

Phyllanthus

Through a change - and a designated change - sufficient that no pressure is transferred to the local bank.  
As only small contributors to business funds + surcharge.

$$\begin{aligned} P_{a_2} &= \frac{1}{2} K_a \delta_1^2 + K_a H_1 \\ &= \frac{1}{2} (0.22)(1.12^2) + (11.32)(1.12) = 0.27 \text{ kip} \\ &= 2.2 + 1.6 = 3.8 \text{ kips} \end{aligned}$$

$$\textcircled{1} 14.76 \times 4.92 = 72.62$$

$$\textcircled{1a} \quad 1 - (.66) \times 14.76 = 7.94$$

77.96 ft<sup>2</sup>

$$W_1 = 77.46 \times .127 = 9.84 \text{ kips}$$

$$\textcircled{2} (14.73 \times 1.25) + 2(12 \times 0.67 \times 16.73) = 32.12 \text{ ft}$$

$$W_2 = 32.12 \times 0.153 = 4.92 \text{ kg}$$

$$(5) 15.58 \times 1.97 = 30.69 \text{ ft}^2$$

$$W_3 = 30.69 \times 0.153 = 4.7 \text{ kips}$$

④  $13.28 \times 7.92 = 25.98 \text{ ft}^2$

$$W_1 = 25.98 \times .127 = 3.30 \text{ Kips}$$

$$\Sigma W = 22.76 \text{ kN}$$

$$+ \text{Surcharge} = 4.92 \times 0.2$$

$$= .984$$

$$\Sigma W_1 = 23.74 \text{ K}$$

$$I_s = \frac{H}{V} = \frac{3.8}{22.74} = .199 = 9.5^\circ \quad \therefore \alpha = 9.5^\circ$$

Short Term Factors of Safety:

$$\text{BEARING } q_{\text{all}} = \frac{\Sigma W}{B} = \frac{23.74}{15.58} = 1.52 \text{ kips/ft}^2$$

$$q_{\text{ult}} = C N_c i_c + \gamma D N_q i_q + \frac{1}{2} \gamma B N_\gamma i_\gamma$$

$$i_c = i_q = \left(1 - \frac{\alpha}{90^\circ}\right)^2 = \left(1 - \frac{11.26}{90}\right)^2 = 0.80$$

$$i_\gamma = 1$$

For  $\phi = 0$  (short term)

$$N_c = 5.14$$

$$N_q = 1$$

$$N_\gamma = 0$$

$$q_{\text{ult}} = C N_c i_c + \gamma D N_q i_q + \cancel{\frac{1}{2} \gamma B N_\gamma i_\gamma} \rightarrow 0$$

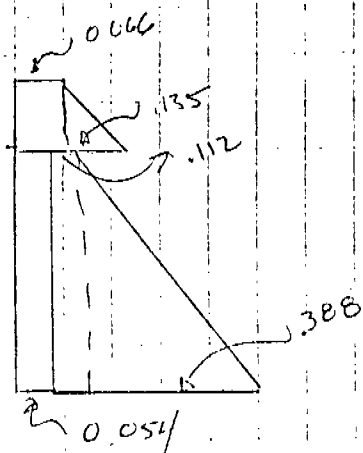
$$= (0.6)(5.14)(0.80) + (6.127)(5.75)(1)(0.80)$$

$$= 3.0$$

$$F.S. = \frac{3.0}{1.52} = \underline{\underline{2.0}}$$

$$F.S. \text{ sliding} = \frac{\Sigma \text{Resisting}}{\Sigma \text{Driving}} = \frac{.6 (15.58)}{3.8} = \underline{\underline{2.5}}$$





$$p_a = 0.066 (3.28) + (13.5)(1.5)(3.28)$$

$$= .437$$

$$p_{az} = 0.054 (11.32) + .112 (11.32) + 1.5 (3.88)$$

$$= .611 + 1.27 + 2.19$$

$$= 4.07 \text{ Kips}$$

$$P_H = 4.51 \text{ Kips.}$$

$$\alpha = \frac{4.51}{22.76} = 11.21^\circ$$

long-term factors

$$q_{all} = 1.52 \text{ Kips / ft.}$$

$$c_c = c_g = (1 - \alpha/90)^2 = .766$$

$$c_s = (1 - \alpha/27.5)^2 = .351$$

$$\text{For } \phi = 27.5^\circ$$

$$N_g = 14.0$$

$$N_s = 9.85$$

Beam

$$q_{all} = .127 (3.28) (14.0) (.766) + \frac{1}{2} (0.062) (15.58) (9.85) (.351)$$

$$= 4.47 + 1.67 = 6.14$$

$$F.S. = \frac{6.14}{1.52} = \underline{\underline{4.0}}$$

Sliding

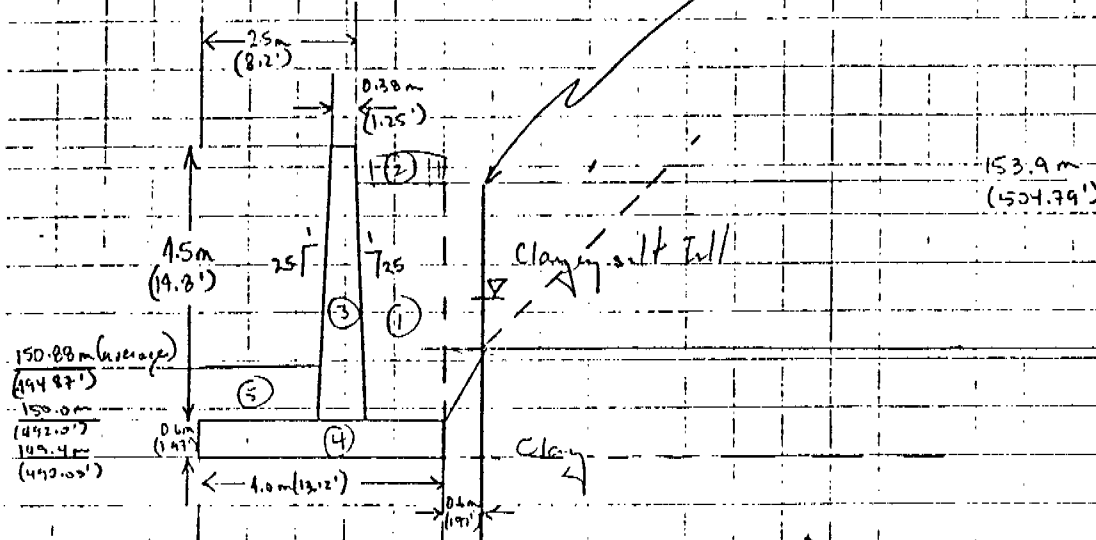
F.S.

$$\frac{\Sigma \text{Resisting}}{\Sigma \text{Driving}} = \frac{22.76 \tan 27.5}{4.51} = \underline{\underline{2.6}}$$

# SECTION S-F

SCALE = 1:125

SHIELDING  
(Front of Lagging)



LENGTH OF PANEL = 10m (32.8')

Area + Volume

$$① \quad 12.79 \times 4.58 \times 127 = 7.44 \text{ Kips}$$

$$② \quad 4.83 \times 0.2 = 0.97 \text{ Kips}$$

$$③ \quad 1.75 \times 14.8 \times .153 = 3.96 \text{ Kips}$$

$$④ \quad 1.97 \times 13.12 \times .153 = 3.95 \text{ Kips}$$

$$⑤ \quad 6.36 \times 2.87 \times .127 = 2.32 \text{ Kips}$$

$$V_g = 18.64 \text{ Kips}$$

$$V_e = 17.67 \text{ Kips}$$

In short term: clay + clayey silt are self supporting  
oo sand only

$$p_a = \frac{1}{2} K_a \gamma H^2 + K_a H \gamma_z$$

$$= \frac{1}{2} (0.27)(1.27)(5.9)^2 + 0.27(5.9)(1.3)$$

$$= .6 + 2.07$$

$$= 2.67 \text{ Kips}$$

$$\alpha = \frac{2.67}{17.67} = 8.6^\circ$$

# Short-term factors of safety

$$q_{all} = \frac{18.64}{13.12} = 1.42 \text{ Kips / ft.}$$

$$c_e = i_q = (1 - \frac{2}{40})^2 = 0.82$$

$$c_s = 1$$

For  $\phi = 0$  (short-term)

$$N_c = 5.14$$

$$N_q = 1.0$$

$$N_\gamma = 0$$

Resulting:

$$q_{ult} = q N_c c_e + \gamma D N_q c_q + \frac{1}{2} \gamma B N_\gamma c_\gamma$$

$$= .5 (5.14)(.82) + (.127)(2.87)(1)(.82)$$

$$= 2.1 + .30 = 2.40$$

$$F.S.B = \frac{q_{ult}}{q_{all}} = \frac{2.40}{1.42} = \underline{\underline{1.69}}$$

Shaking:

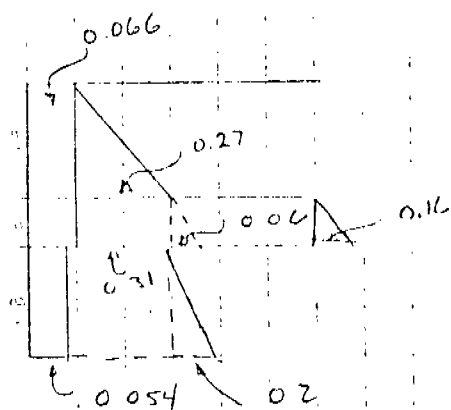
F.S.

$\frac{\text{Resistance}}{\text{Demand}}$

$$= \frac{(.5)(13.12)}{2.67}$$

$$= \frac{6.56}{2.67} = \underline{\underline{2.4}}$$

long-term factors of safety



$$p_{a1} = 0.84 + .41 = 1.25 \text{ Kips}$$

$$p_{a2} = 0.71 + 0.08 + 0.17 + .21 = 1.17 \text{ Kips}$$

$$p_c = 1.83 + 0.59 + 0.32 = 2.74 \text{ Kips}$$

---


$$5.16 \text{ Kips}$$

$$\alpha = \frac{5.16}{17.67} = 16.3^\circ$$

long-term factors

$$q_{c11} = 1.42 \text{ Kips / ft}$$

$$c_c = c_g = (1 - \frac{2}{90})^2 = 0.67$$

$$\phi = 27.5$$

$$c_x = (1 - \frac{2}{\phi})^2 = 0.12$$

$$\text{For } \phi = 27.5$$

$$\begin{aligned} N_c &= 0 \\ N_g &= 14.0 \\ N_x &= 9.85 \end{aligned}$$

Bearing:

$$\begin{aligned} q_{ult} &= c N_c c_c + \gamma D N_g c_g + \frac{1}{2} \gamma B N_x c_x \\ &= .127 (2.87) (14.0) (0.67) + \frac{1}{2} (.062) (13.12) (9.85) (.12) \\ &= 2.42 + .5 = 3.90 \end{aligned}$$

$$F.S.B = \frac{3.90}{1.42} = \underline{\underline{2.75}}$$

Sliding:

$$F.S. = \frac{\Sigma \text{Resisting}}{\Sigma \text{Driving}} = \frac{17.67 \tan 27.5}{5.16} = \underline{\underline{1.8}}$$

# memorandum



To: K.L. Kleinsteinber,  
Head, Approvals Section,  
Structural Office,  
West Building, Downsview.

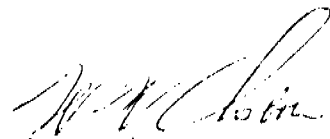
Date: 82 01 14

Telephone: 224-7456

Re: Ormont Drive C.P.R. Grade Separation  
Contract No. 811-025  
Geotechnical Investigations

Attached are consultant reports from Deleuw Cather and  
Trow Ltd. submitted through this office, but intended for  
your review and recommendations.

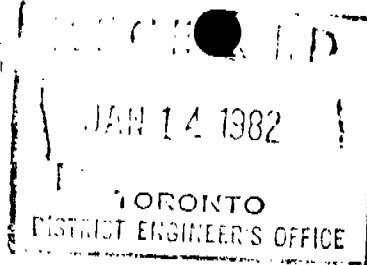
WVO/tm  
Attach.

  
W.W. Osborn, P. Eng.,  
District Municipal Engineer  
District No. 6



CITY OF NORTH YORK

5100 YONGE STREET  
NORTH YORK  
WILLOWDALE, ONTARIO  
M2N 5V7



only attached

Telephone (416) 6243

File No

PUBLIC WORKS DEPARTMENT

January 14, 1982

Ministry of Transportation and Communications,  
5000 Yonge Street,  
Willowdale, Ontario.  
M2N 6E9

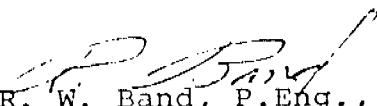
Attention: Mr. W. W. Osborn, P.Eng.

Dear Sirs:

Re: Ormont Drive C.P. Rail Grade Separation  
Contract 811-025

With reference to my conversation with yourself  
and Mr. Kleinsteinber of the Bridge Office, I attach hereto  
all the pertinent reports with reference to the problem,  
including our consultants' recommendations.

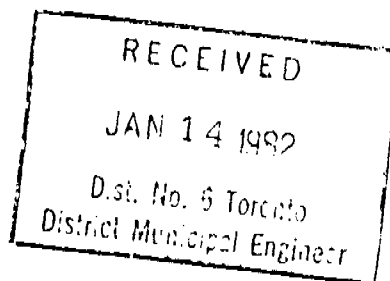
Yours very truly,

  
R. W. Band, P.Eng.,  
DIRECTOR OF

ENGINEERING CONSTRUCTION.

RWB/pm

Att:8



# DeLCan

DE LEUW CATHER, CANADA LTD.

CONSULTING ENGINEERS AND ARCHITECTS

January 13, 1982

Our Ref: 01-1234-A-80

City of North York  
5100 Yonge Street  
Willowdale, Ontario  
M2N 5V7

Attention: Mr. R.D.W. Band, P.Eng.

Dear Sir:

Re: City of North York  
Ormont Drive/CP Rail Grade Separation  
Contract No. 811-025

As you are aware, the contractor on this project (S. McNally & Sons Limited) has raised a serious concern with regard to the suitability of the retaining wall foundations which form a part of the Ormont Drive/CP Rail grade separation design. We have carried out a detailed and careful investigation of the contractor's concerns and submit herein the results of our investigations and the recommendations which derive from it. In general, this report comprises a brief history of the project, the origin and nature of the contractor's concerns, the investigation of these concerns, and DeLCan's conclusions and recommendations.

## History of the Project

DeLCan (De Leuw Cather, Canada Ltd.) carried out the design of this project and the preparation of drawings, specifications and contract documents for the City of North York. The design was carried out in conjunction with geotechnical investigations, analyses and recommendations produced by The Trow Group Ltd. (now Trow Ltd.), specialist foundation advice from Brian Isherwood & Associates, and specialist groundwater control advice from Hydrology Consultants Limited. The geotechnical report provided by The Trow Group Ltd. was No. T1625-G dated November 22, 1979. This report was modified by letters from The Trow Group Ltd. which provided additional information and design recommendations and which were dated November 22, December 4 and 13, 1979 and January 28, 1980. Copies of these documents are enclosed for your information. In accordance with the recommendations contained in these documents, the retaining walls for the project were designed to be supported on spread footing foundations.

# DeLCan

City of North York  
January 13, 1982  
Page Two

The project went to tender on this basis. In the fall of 1981 a contract for the construction of this project was awarded to S. McNally & Sons Limited and construction commenced. To date the contractor on the project has neither submitted to us nor has he developed a satisfactory dewatering scheme for the site, nor has he developed a satisfactory temporary shoring scheme to permit construction of the permanent foundations, bridge abutments and retaining walls on the project. Whatever limited construction of such dewatering and temporary works as has been carried out on the site by the contractor to date has been done on the contractor's sole initiative and at his own risk.

## Origin and Nature of Concern

On October 28, 1981 the contractor met with DeLCan to express concern, based on the advice that he had received from his specialist geotechnical engineers with regard to the properties of the subsoil and the design of the retaining walls on spread footings, that the spread footings would not prove to be satisfactory.

This was clearly a serious allegation and, as such, it was immediately discussed in detail in joint meetings held among Trow Ltd., the contractor and DeLCan. Based upon an apparent disagreement between Trow Ltd. and the contractor with regard to the adequacy of the retaining walls on spread footings, we advised the City that a second opinion on the geotechnical conditions and parameters affecting the design of this project should be obtained. We also discussed this proposal with Trow Ltd. It was agreed that Golder Associates should be retained to provide this second opinion. We also suggested that Trow Ltd. review the project themselves and provide us with any comments that they felt were appropriate.

While these investigations were ongoing, the contractor carried on with definition and implementation of his dewatering scheme, development of his temporary works (such as shoring) required for construction of the permanent foundations, abutments and retaining structures on the project, and with aspects of the construction not directly related to the design of the permanent retaining wall.

## Investigation by Trow Ltd.

Trow Ltd. submitted their letter of November 16, 1981, a copy of which is attached hereto.

In summary, Trow Ltd. concluded that the retaining walls can be supported on the spread footings generally as proposed in the contract drawings, and suggested that the key be extended locally and that the deeper sections of wall be subexcavated to ensure bearing on silt rather than on clay.



# DeLCan

City of North York  
January 13, 1982  
Page Three

In subsequent discussion among Trow Ltd., Golder Associates and DeLCan, Trow Ltd. stated that the walls were satisfactory as-designed, and discussed other methods of possible modification such as improvements to the drainage of the site. Trow Ltd. consider that pile foundations are not required for any of the retaining walls.

## Investigation by Golder Associates

In accordance with the terms of their retention and following numerous detailed discussions and meetings among Trow Ltd., Golder Associates and DeLCan, Golder Associates submitted their final report on the matter (No.811-1376) on January 11, 1982. Copies of this report are attached.

In summary, Golder Associates concluded that all wall panels east of panel S-H and N-B (that is, all but the four lowest wall panels) must be supported on pile foundations.

## Conclusions and Recommendations

While there is obviously a disagreement between two acknowledged experts in the geotechnical field, Trow Ltd. and Golder Associates, regarding the type of foundation which is appropriate for the proposed retaining walls, there has nonetheless been clear agreement that the soils at the site will be difficult to work with and sensitive to disturbance by the contractor's operations; that the soils are extremely sensitive to changes in water level; and that construction will require a good deal of care as a result.

At the same time, we have concluded that given the overall context of the project, including the fact that the contractor has been engaged and has commenced construction, and that the City of North York has made commitments to the nearby property owners, the serious delays which would be incurred by the adoption of a solution which would involve the acquisition of property for the purpose of minimizing or even eliminating altogether the requirement for retaining walls on the project would not be in the best interest of the City. Given that if the project is to proceed expeditiously and in a cost-effective manner, the walls are required; and that this is in the best interest of the City; then we would be remiss were we not to recommend adoption of a structural scheme which satisfied the expressed concerns of both Trow Ltd. and Golder Associates as to the safety of the completed project.

# DeLCan

City of North York  
January 13, 1982  
Page Four

This condition will be met if the foundations for the retaining walls are modified to include the piles recommended by Golder Associates, using amalgam of the more conservative of the design parameters recommended by Golder Associates and Trow Ltd. We recommend that this program of foundation modification be implemented.

Some guidance with regard to the costs implicit in this matter is given by the following table of preliminary cost estimates.

Phase 1 - Investigation	\$ 75,000
Phase 2 - Design Modification	\$ 40,000
Phase 3 - Construction	
- Additional Construction Cost	\$250,000
- Potential Claim for Delay (see McNally's letter of December 17, 1981)	No amount specified

We estimate that the completion of the design modifications will take no more than six weeks.

Although it did not enter into the development of our recommendation, as a matter of interest we wish to confirm that the contractor verbally has indicated that he would not proceed to construct the retaining walls on spread footings. While we do not consider that he has the right under the contract to refuse to carry out this work, nonetheless we feel that his position should be brought to your attention. This, when taken in addition to the contractor's letter of December 17, 1981, makes the resolution of this matter an urgent affair.

We are therefore prepared to proceed with the inclusion of the proposed modifications to the retaining wall foundations immediately upon receipt of your instructions to this effect.

Yours very truly,



W. V. Anderson, P.Eng.

WVA:do  
Enclosures

c.c. W. Trow - Trow Ltd.  
J. Busbridge - Golder Associates  
B. D. Henderson  
N. N. Aylon  
J. D. Dekker  
J. H. Josselyn



Consulting  
Engineers

43 Baywood Road  
Rexdale, Ontario, Canada  
M9V 3Y8  
Telephone (416) 749-1290  
Telex 065-27280

Project: T 3080-K

November 16, 1981

Delcan Limited  
133 Wynford Drive  
Don Mills, Ontario  
M3C 1K1

*Received  
Nov 17/81  
W. Anderson  
De Z. L. N.*

Attention: Mr. V. Anderson

Ormont Road  
Earth Retaining Structure

Dear Sirs,

This letter is in answer to your letter dated November 10th, 1981. It also confirms comments relayed to you by telephone during the week of November 8th, 1981.

Temporary Shoring

You have sent us plans showing the contractor's proposed shoring arrangement for the south approaches to the CPR underpass. The east and west sides of the road are essentially identical. We shall limit our comments to the east side since this is the more critical one in our opinion, in view of the close proximity of Bona Foods. We wish to remark at this point that the shoring drawings are incomplete in our opinion, as regards the explanation of excavation and bracing sequence and other details.

The drawings show the shoring close to the back of the proposed retaining wall for the approximate south half of the Bona Foods building, where the depth of cut is less. It is indicated to be about 3.4 m west of the Bona Foods building. Farther north the temporary shoring wall has been indicated to come close to the Bona Foods building, i.e. within about 1.6 m in an area where the depth of cut for the retaining wall footing will extend to approximate Elevation 148.7 m or about 3.8 m below the footings at the north end of Bona Foods.

We consider this to be extremely hazardous to Bona Foods and therefore stress that more than the usual caution is needed to prevent settlement of that building. A shoring location farther from Bona Foods would be much more desirable.

In the review of the contractor's Drawing C 246-3 we note that Section B-B for the support of the deeper sections, i.e. near and beyond the north end of Bona Foods, has been marked hold.

However in both sections the inferred shoring philosophy is to support the earth pressure temporarily on inclined struts braced against a wall of natural soil in the area of the roadway just beyond the retaining wall toe. After the footing for the retaining wall is poured, an inclined raker brace will be placed between the heel of the footing and the shoring and then the struts will be taken out.

Our comments are as follows:

- (1) In view of the close proximity of the shoring to the rear of Bona Foods, we consider that a higher empirical earth pressure criteria should be assumed in the shoring design. We recommend the pressure distributions indicated on the attached sketch.
- (2) We have found that most of the earth pressure is taken by the two temporary struts since the contribution of the soldier pile toe in the soft clay is minimal.
- (3) Assuming the earth pressures in the attached drawing, we find that the capacity of the natural soil on the road side end of the struts is marginal for the support of the Section B-B situation. We consider that the natural soil must be present up at least to Elevation 152 m. The situation for Section A-A is satisfactory although the toe of the retaining wall should be paved flush with the natural soil and the earth face should extend vertically upward from there.

- (4) Because of the assumed high earth pressure and the eccentricity of loading of the eventual raker, there is a danger of overloading the weak clay at the rear of the footing. In addition, since the horizontal component of this raker load must be taken through the footing to the road side temporary wall, and since the clay is weak at this lower level, there is a slight tendency for rotation of this wall once the temporary struts are removed. Because of this we recommend that an additional temporary raker be placed on the toe side to act as a brace against tipping and rotation as suggested in the attached sketch. We also suggest that the inclined raker be moved out as far as possible to reduce the eccentricity on the footing.
- (5) Where the wall is 3.4 m from Bona Foods, a less severe earth pressure assumption is considered applicable.
- (6) Where the overall depth of cut to the bottom of the footing is more than  $11\frac{1}{2}$  feet, the temporary berm on the road side should be taken at least to Elevation 152 m.

#### Permanent Stability

In the area where the temporary shoring comes to within 1.6 m of Bona Foods, the eventual space behind the retaining wall will have to be backfilled with granular material. It can be shown that the earth pressure exerted by the gravel backfill will be the governing pressure as regards long-term stability. The wall in all locations is so far removed from Bona Foods, i.e. about 5.2 m that the severe assumptions for the temporary shoring no longer apply, i.e. the gravel when compacted to a density at least equal to 100 per cent standard Proctor density will prevent movement at Bona Foods.

If one takes the severe earth pressure assumption given in our report, wherein the earth pressure coefficient is assumed equal to 0.5, it can be shown that the sliding resistance at all locations beside Bona Foods incorporates a

factor of safety in excess of 1.5 provided that the footing key is continued under all retaining wall footings back to approximate Station 38.79 m. The height of granular backfill at this latter point is about 3.6 m assuming the ground surface to be Elevation 154 m and the footing is reduced to 4 m width. Farther to the south along Bona Foods and on the west side of the road a lower coefficient of 0.35 is considered permissible. The key on the north side may terminate at its present location, i.e. about Station 68.79 m where the height of backfill is about 4.6 m or 15 feet assuming the final ground surface to be Elevation 154 m.

Consequently, as stated recently by telephone and as indicated in correspondence following our November 22, 1979 report, piles are not considered necessary for support of any section of the retaining walls. It is noted that in the deeper sections the base of the retaining wall is almost through the soft clay into the stronger silt below. In this case it may be prudent to dig deeper to ensure complete bearing on the silt.

It is noted that the key should be into the silt in these higher sections of wall according to Borings 2 and 6.

It is essential that the groundwater table be depressed before digging in these deep areas.

It would be prudent to relocate the drain presently shown on the design drawings in front of the toe of the retaining walls. Alternatively, any backfill for the drain must be well compacted.

#### Additional Borings

In view of the differences of opinion that have arisen in the construction to date, it is considered that a closer check on the clay strength and variability is desirable.

You have retained Golder Associates to make borings for you. Since we respect the work of this firm, we see no reason to introduce additional costs to this project by duplicating their work. However we certainly should be given copies of the factual data they have obtained.

It may be, after their data has been reviewed, that further work is needed. It is impossible to comment further on this requirement at this time.

Yours very truly,

TROW LTD.



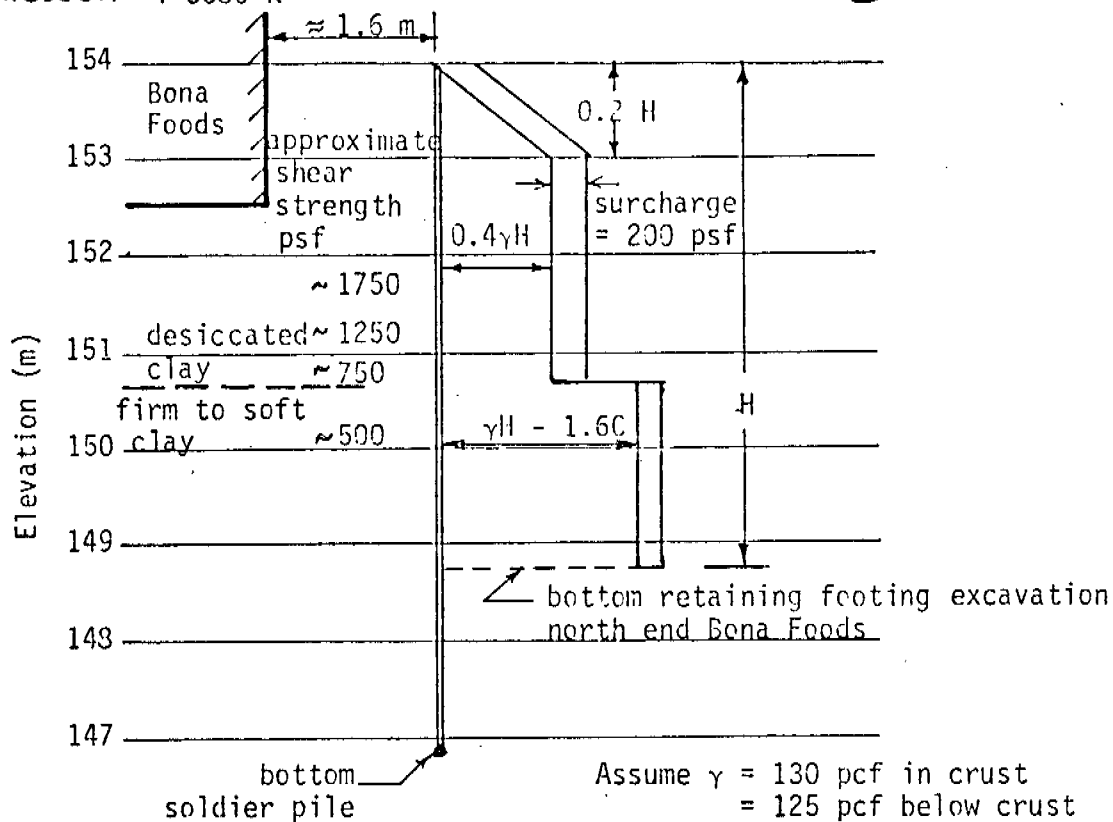
W. A. Trow, P. Eng.

WAT/da

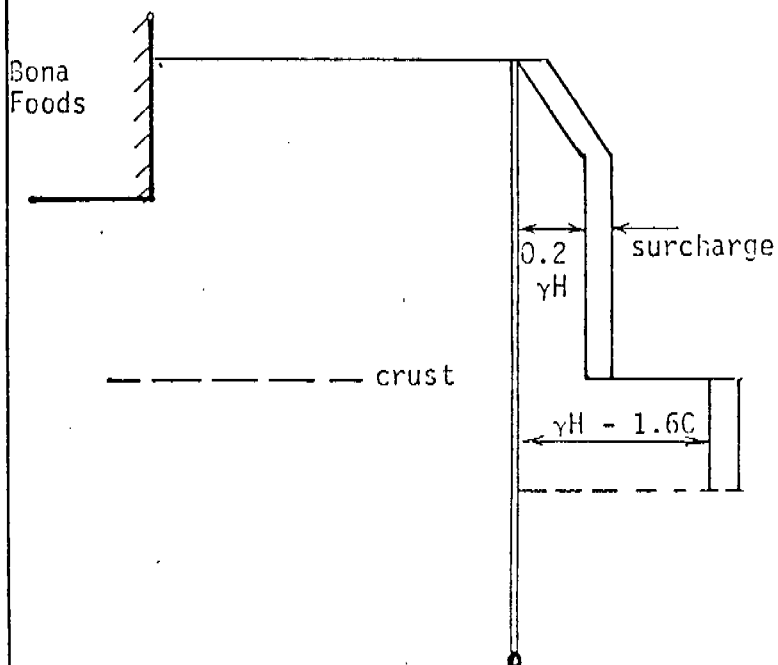
Encl.

Dist.: Delcan Limited (2)

City of North York (1)

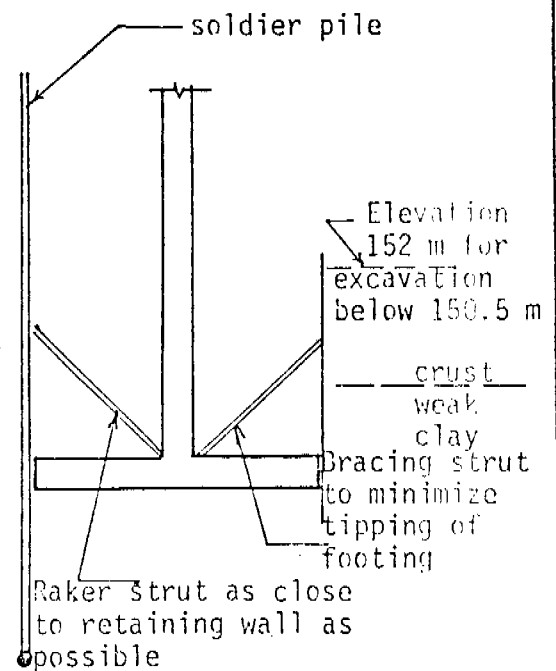


Recommended Shoring Design Assumption  
 Where Shoring Comes Close to Bona Foods



Recommended Shoring Design Assumption  
 Where Shoring is Immediately behind  
 Retaining Wall  
 About 3.8 m from Bona Foods

≠ Trow Ltd.



Suggested Final Bracing  
 Prior to Construction of  
 Vertical Member of  
 Retaining Wall





# The Trow Group Limited

CONSULTING  
ENGINEERS

Project: T 1625-G

43 BAYWOOD ROAD, REXDALE, ONTARIO  
(416) 749-1290 MOV 3Y8  
Telex No. 065 - 27280

January 28, 1980

City of North York  
c/o Delcan  
Deleuw Cather, Canada Limited  
133 Wynford Drive  
Don Mills, Ontario  
M3C 1K1

Attention: Mr. J. H. Josselyn

Additional Comments Regarding  
Dewatering as Related to Construction Schedule  
Proposed CP Rail Grade Separation  
Ormont Drive  
North York, Ontario

Dear Sirs:

On Thursday afternoon, January 17, I attended a meeting in your office at your request to review your proposals for the anticipated order of construction and to discuss dewatering requirements.

John Noonan, Hydrology Consultants, was also present and his preliminary report on the results of the recent pumping test was submitted to you. Messrs Aylon and Dekker of your office were the other members of the meeting.

At the conclusion of the meeting you asked me to prepare a final letter dealing primarily with construction and permanent dewatering requirements for the proposed underpass. Accordingly, I propose to present my comments and recommendations in the order that I would envisage the work to proceed, - which conforms more or less to the order discussed at the meeting.

## Relocating of Storm and Sanitary Sewers

These sewers are to be relocated beyond the north abutment of the railway bridge. The lower invert levels adjacent to Toryork Drive will be



Elevation 149.4 m and the level rises to the east, being at Elevation 151.5 m at its uphill end about 30 meters east of the finally completed railway. At all locations, the inverts will be in clay or clayey till which is expected to remain stable during the excavation and pipe laying work. Although the trench is far enough away from the International Auto Collision footings, shoring of the trench will be needed in this property to permit safe access for cars and the like.

According to the drawdown records indicated in the Hydrology Consultants report, the well, installed last December, - if put into operation in advance of construction, - should draw the groundwater down below invert level at the upstream end of the sewer line, and therefore the excavation should not be endangered by uplift problems.

It is assumed that the sewer line will be installed west of the railway right up to the existing embankment and that, after the trench is properly backfilled, the temporary relocated railway embankment will be installed. After this is satisfactorily completed, the sewer excavation work to the east of the track will be undertaken and completed. If the sewer is eventually to connect to existing facilities on Toryork Drive and if that sewer lies close to the sand aquifer noted below Elevation 146 m approximately, (holes 5 and 6), prior dewatering will be required as discussed below for the road storm drain.

#### Abutment Excavations

It is proposed to install piles and to erect abutment footings and walls in a braced and sheeted excavation. The pile caps will be at approximate Elevation 147.3 m which is well into the water-bearing sand on the east side of the railway. Pumping from the December well, will not lower the water table sufficiently to provide a stable base for pile cap installation. Consequently, the groundwater table must be lowered during this work.

It is recommended that one row of vacuum-assisted well points be installed adjacent to the abutment excavation for this purpose. The actual location could be adjusted to suit contractor convenience. However, we

suggest that one row be placed on the north side of the north abutment enclosure, - south of the relocated sewer line, and the other placed on the south side of the south abutment enclosure. In order to achieve effective drawdown, we suggest that the tips extend close to the bottom of the sand stratum or to Elevation 143 m approximately. Since the practical depth for pumping is about six meters, it is recommended that the pipe manifold and inlet point of the vacuum pump be set in a trench dug adjacent to the sheet pile wall to Elevation 150.5 m. The trench should not be deeper at the north end since it may endanger the relocated sewers. The well points must be installed by a contractor specializing in this work and the points and connecting pipe up to the top of the sand must be surrounded by uniform granular material that will not be plugged by the fines from the sand. The groundwater must be confirmed to be below proposed pile cap excavation level before digging to that level. In order to assist pile driving operations, the trench over the manifold could be temporarily backfilled when necessary. It should be backfilled adjacent to the railway such that the slope into the trench from the railway does not exceed two horizontal to one vertical.

The well points must be left in operation not only during the period of pipe driving, pile cap installation and the like, but it must also remain operative until permanent pressure relief measures are provided for the road as discussed later.

#### Storm Drain for Ormont Drive

This drain is needed to remove storm runoff water collecting in the lower levels of the railway underpass. The invert of the sewer is indicated to be at Elevation 148 m under the railway draining down to Elevation 147.6 m near Toryork Drive. The excavation will begin in sand at the upstream end, which is expected to be in a dewatered, stable state provided that the well points are kept in operation.

Moving to the west they will pass through the silt stratum which contains some thin seams of sand. These seams also are expected to be drained or depressurized so that excavation work can proceed. The base may tend to be

somewhat liverish in this area but judicious use of two inch stone with fine sand worked into it to fill all voids should provide a firm working base. Alternatively, 3/4 inch stone laid inside an envelope of approved porous fibre cloth could be used.

As the excavation proceeds toward Toryork Drive, it will become deeper and therefore braced shoring will be needed to support the soft to firm clay. This is required not only for safety of workmen but also to ensure that the retaining walls are not undermined. Close to Toryork it is noted that the water-bearing sand layer comes close to invert level, i.e. about Elevation 146 m. This appears to be close to the anticipated drawdown limit of pumping at the railway and therefore additional well points will be required in this area to ensure that base heave does not occur during pipe installation and particularly when connecting to the existing main on Toryork Drive.

#### Permanent Groundwater Control

After the construction is complete, the water table will tend to return to the levels existing prior to pumping unless measures are taken to control the water at a lower level. As a consequence, instability of the sections of the east road approach to the bridge would develop.

Although the existing well and well points, - if connected in a positive manner to the storm sewer under the road - will provide some relief, it is recommended that the storm sewer trench be deepened as necessary to ensure permanent contact with the sand and that drainage extended in an easterly direction to provide the required relief. Basically, the required relief involves the permanent depression of the water table such that at all points under the road the weight of soil, including road material acting down on the top of the sand layer exceeds the hydrostatic pressure acting up against the sand - overlying clay contact.

This situation will be achieved west of the track by placing the sewer on a bed of 3/4 inch crushed stone above 12 inches thick, with this stone laid in turn on an acceptable porous fibre cloth which is brought up



the sides of the trench to provide a completed wrap around the stone. Connections should be provided for drainage into catchbasins along the sewer route.

This same arrangement can be used where the sewer enters the sand under and to the east of the bridge. In order to protect the east approach, however, it must be continued in the sand under the centre line of the road at least to station 0 + 170. It is recommended that the stone envelope be deepened in these easterly areas to ensure that 1/2 meter depth of the stone envelope penetrates into the sand and that the envelope rises at least 1/2 meter above the sand-clay contact. It is recommended that a three inch porous plastic pipe be installed in the centre of this stone envelope to increase the drainage efficiency. This drainage system should be directly connected to the storm drainage catch basin under the bridge. It is not necessary to extend the storm drain up to station 0 + 170.

#### Support of South Retaining Wall by Bonus Foods

In our report dated November 27, 1979, it was assumed that the excavation adjacent to Bonus Foods would be supported by a soldier pile - raker shore system with the rakers installed through a berm to minimize the possibility of construction movements.

An alternative, involving soil anchors, has been suggested. Soil anchors are a desirable solution for support of many excavations since they do not obstruct construction work. However, they are not considered to be practical here for the following reasons.

- 1) The anchors would need to extend below the Bonus Foods footings with the possibility of undermining unless extreme care is taken. The permission of Bonus Foods would need to be obtained, and they would have to be warned that the anchors would remain in place after all work is complete.
- 2) The clay is not very strong and therefore a long anchorage length would be needed.



As agreed with John Noonan today, we are sending a copy of this letter to Hydrology Consultants who will comment separately, if necessary.

We hope these comments are suitable for your purposes. Should you have any queries, please do not hesitate to contact this office.

Yours very truly,  
THE TROW GROUP LIMITED

W. A. Trow, P. Eng.

WAT/lg  
cc: M. G. Sarafinchin  
Dist: City of North York (2)  
c/o Delcan  
Hydrology Consultants (1)



# The Trow Group Limited

43 BAYWOOD ROAD, REXDALE, ONTARIO  
(416) 749-1290 M9V 3Y8  
Telex No. 065 - 27280

Project: T 1625-G

December 13, 1979

City of North York  
c/o DelCan Limited  
133 Wynford Drive  
Don Mills, Ontario  
M3C 1K1

Attention: Mr. J.H. Josselyn

Supplementary Comments Regarding Excavation Stability  
Proposed CP Rail Grade Separation  
Ormont Drive near Toryork Drive  
North York, Ontario

Dear Sirs,

Further to our report dated November 22nd, 1979 we wish to make the following additional comments covering excavation stability during and after construction. This letter confirms a telephone conversation with Mr. N. Aylon of DelCan on about December 4, 1979.

## West Approach

During construction, -because of property limitations on each side. -the excavation will need to be shored, presumably with a braced soldier pile and lagging system. The excavation along this site will be in firm clay with an undrained shear strength as low as 25 kPa (~500 psf) approximately, but generally higher than this. After construction is complete the sides of the excavation will be supported by retaining walls, bearing within the clay, on its upper stronger crust or in the overlying till closer to Toryork Drive, or in the stronger silt and silt till found below the clay at the railway bridge:

The basal stability of the excavation, both during construction and in the eventual finished state can be appraised from the following expression:

$$F(\gamma H + q) = N.C *$$

where:

- F is the required safety factor
- $\gamma$  is the unit weight of the soil above base level  
= 20 kN/m<sup>3</sup> (128 pcf) approximately
- H is the depth of the cut during excavation or finally to the road pavement
- q is an allowance for surcharge adjacent to the excavation
- C is the undrained shear strength, assume an average = 30 kPa
- N is a stability factor estimated to be 6 for this problem

Assuming  $q = 10$  kPa (200 psf) and solving, in this expression, the maximum theoretically possible depth of cut is about 8.5 m ( $\approx$ 28 feet). The maximum depth of general cut below present ground level southwest of the railway is about 4.5 m ( $\approx$ 15 feet). It will be slightly deeper for the localized excavations for the retaining walls but, for these localized conditions a higher value for N is permissible. It is therefore considered that the excavation southwest of the railway will be safe, both during construction and afterward, particularly considering that the soldier piles, with concreted bases into the silt or till below excavation level, will remain in place and that the base of the retaining wall footing will be close to the clay.

### Railway Crossing

The greatest depth of cut will be at the railway bridge although this cut will not be deepened to final level until after the bridge is constructed. The foundation will be built within a shored enclosure, which will be left in place below road level, after the bridge is complete. The bridge structure will be on piles. The wing walls adjacent to, but not rigidly attached to the bridge, will be on footings bearing in the dewatered sand on the north side and in the silt or silt till on the south side. There will be a thin layer of clay below the base of the pile cap and the top of the silt at the south side but it will be too thin, and too restrained by these boundaries to be squeezed out even if the shoring for the footing was not present. The wing walls, which will

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\*Canadian Foundation Engineering Manual, Part 4, Fig. 3.14 page 44.



support the edges of the railway fill, will be founded below the clay, therefore no failure surface can occur under the footings. Therefore this area is also considered to be stable.

#### Temporary Excavation for Bridge Footings

After the railway is temporarily diverted to the southwest the foundation construction will begin within two rectangular shored enclosures. The existing railway fill and natural ground will be sloped down to the north at 2 horizontal to 1 vertical. The shored footing excavations will be at least as resistant, -and probably more so than the surrounding soil, -to instability under temporary fill and railway loading.

The maximum difference in elevation between the top of the embankment fill and the base of the construction cut is about 4.25 m (14 feet). The cut will be entirely in very stiff to hard clayey silt till or possibly into the desiccated upper levels of the clay. The stability appraisal should allow for an extreme loading condition of two trains parked over the area during the construction stage. The loading of each train was given as 151 kN/m (10.4 klf) or a total of 302 kN/m (20.8 klf) for two trains. This load would be spread through the ties and underlying ballast and then spread through the compacted railway fill in a distribution of about 2 vertical to 1 horizontal. The estimated spread out pressure at Elevation 151 m (495 feet), the estimated top of the weaker clay, is  $26 \text{ kN/m}^2$  or 26 kPa ( $\sim 540 \text{ psf}$ ). The surcharge weight of embankment fill, and soil above Elevation 151 m (495 feet) is about 105 kPa ( $\sim 2200 \text{ psf}$ ) giving a total pressure of 131 kPa (2740 psf) at Elevation 151 m (495 feet). The ultimate resistance of the clay for the 2 to 1 slope configuration proposed is equal approximately to  $NC = 6.1 \times 30 = 183 \text{ kPa}$  (3820 psf) giving a safety factor of approximately  $\frac{183}{131} = 1.4$  which is considered to be adequate, particularly considering the rigidity provided by the shored and braced footing excavations.

If you have any queries concerning the foregoing stability appraisal, please call us.

Yours very truly,  
THE TROW GROUP LIMITED

*W.A. Trow*  
W.A. Trow, P. Eng.

WAT/gp

Dist: City of North York  
c/o Delcan Limited (2)  
Attention: Mr. J.H. Josselyn



CONSULTING  
ENGINEERS

# The Trow Group Limited

43 BAYWOOD ROAD, REXDALE, ONTARIO  
(416) 749-1290 M9V 3Y8  
Telex No. 065 - 27280

Project: (T 1625-G

December 4th, 1979.

City of North York,  
c/o DelCan Limited,  
133 Wynford Drive,  
Don Mills, Ontario.  
M3C 1K1

Attention: Mr. J.H.Josselyn.

Allowable Bearing Pressures for Retaining Walls  
C.P. Rail Grade Separation  
Ormont Drive - North York, Ontario

Dear Sir:

We are providing our supplementary comments regarding the maximum allowable bearing pressures to be used for the design of the retaining walls at the above site as discussed with Mr. N. Aylon of DelCan, on December 3rd and 4th, 1979.

It is understood that the proposed founding level will vary from about Elevation 147.5 m (484 feet) near the abutments, stepping up to about Elevation 150 m (492 feet) at the west end of the retaining wall. Subsequent to adequate dewatering prior to/and during the construction period, the footing bases will be installed at about 2 to 6 m (7 to 20 feet) depth below the existing grade in the dense to very dense sand (N cone  $\geq 80$  at BH-4 and N cone  $\geq 50$  at BH-3); the dense to very dense silt and silt till (N cone  $\geq 50$  at BH-1, N cone =25 to 80+ at BH-2 and N cone =35 at BH-3); and the firm clay (Cu =500 to 970 psf).

As conventional spread footings are being considered as an alternative to a piled foundation system for the retaining walls near the proposed bridge abutments, then a net soil bearing pressure incorporating surcharge depths, may be used. This will result in slightly higher bearing pressures than the previously recommended value of 50 kPa (1000 psf) given for general design purposes.

The net bearing pressure is the net addition of stress which may be added to the soil in excess of the minimum surcharge weight adjacent to the footing member. The minimum surcharge depth is 1.2 m (4 feet) at the toe of the wall, but increases to 7.0 m (23 feet) behind or above the heel of the wall. This minimum weight will comprise compacted backfill, roadway materials, sidewalks, etc. above the toe; and free-draining granular backfill materials above the heel. The submerged weight of soil is assumed above the toe and the total weight of granular backfill may be assumed behind the wall. In the calculation of the net soil bearing pressure, all dead load including the footing weight less soil displaced by it, plus all live load acting at least once a year, should be considered.

Based on these considerations and using the general Terzaghi equation for ultimate bearing capacity with a factor of safety of 2.5\*. The maximum allowable soil bearing pressures to be used in the design of the retaining walls founded on conventional spread footings are as follows:-

\* Since a considerable weight of soil will be permanently removed, a factor of safety = 2.5 is considered to be satisfactory.

Location	Allowable Bearing Pressure	Comments
Toe	59 kPa (1240 psf)	in the firm clay with a minimum surcharge depth of 1.2 m (4 feet) and submerged unit weight as at the toe of the wall
	62 kPa (1300 psf)	in the firm clay with a minimum surcharge depth of 1.5 m (5 feet) and submerged unit weight as at the toe of the wall
Heel	105 kPa (2200 psf)	in the firm clay with a minimum surcharge depth of 3.0 m (10 feet) and total unit weight as behind the wall
	163 kPa (3400 psf)	in the firm clay with a minimum surcharge depth of 6.1 m (20 feet) and total unit weight as behind the wall.
	192 kPa (4000 psf)	in the dense silt
	287 kPa (6000 psf)	in the dense sand
These allowable bearing pressures are subject to adequate dewatering measures prior to and during construction, and an inspection of the footing bases by one of our geotechnical engineers prior to concreting to confirm the subgrade conditions.		


For the present design, it is understood that a maximum applied stress of 192 kPa (4000 psf), including about a 7.6 m (25 feet) depth of backfill, occurs at the heel of the wall near the abutments. As the present overburden stress is about 144 kPa (3000 psf) at this depth, the net addition of stress applied by the footings to the firm clay will be about (1000 psf). Therefore, the settlements resulting from the application of these bearing stresses is expected to be less than 25 mm (1 inch).

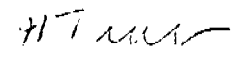
Our additional foundation work related to the overall stability for both the temporary braced cofferdam at the proposed bridge abutments, and the temporary shoring system and permanent retaining walls along the west approach, is forthcoming.

Should you have any questions, please do not hesitate to contact this office.

Yours very truly,

THE TROW GROUP LIMITED

  
M.G. Sarafinchin, P.Eng.

  
W.A. Trow, P.Eng.

MGS/bc

Dist: City of North York  
c/o DelCan Limited

(2)



# The Trow Group Limited

CONSULTING  
ENGINEERS

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Project: T 1625-G/T 1625A-G

November 22, 1979

City of North York  
c/o DeLCan Limited  
133 Wynford Drive  
Don Mills, Ontario  
M3C 1K1

Attention: Mr. J. H. Josselyn

Pump Test for Construction and Permanent  
Dewatering Requirements  
Proposed CP Rail Grade Separation  
Ormont Drive near Toryork Drive  
North York, Ontario

Dear Sir,

This letter confirms our discussions in your offices on November 1st and 15th, 1979 and our proposal for the pumping test at the above site.

Our recent subsurface investigation\* encountered a confined aquifer (clay and clayey silt overlying wet sand) with high sub-artesian pressures measured in two sealed piezometers. Consequently, a pump test is being carried out to accurately determine the drawdown characteristics, construction dewatering requirements, volumes of water to be pumped during and after construction, and the extent and type of permanent drainage requirements to prevent base heave along the paved roadway.

The estimated cost to complete this work is as follows:

a) Initial Office Meetings

Discussion of pump test requirements based  
on borehole and piezometer information

\$ 900.00 approx.

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\* Trow Report T 1625-G, Geotechnical Investigation, November 1979

b) Fieldwork

Based on verbal quotations, well-drilling contractor to install adequately screened and filtered pumped well adjacent to Borehole 3, 3 hours step test, 5 hours well development, 24 hour pump test, and provision to use as temporary construction and permanent relief well (Snider) \$ 5,250.00 approx.

Installation of five (5) observation wells in confined sand aquifer (sealed piezometers) using hollow stem auger equipment under full-time supervision. 2,800.00 approx.

Consultation, full-time supervision and data collection for pump well installation and pump test, analysis and factual report (Hydrology Consultants) 2,000.00 approx.

Monitoring piezometric water levels before, during and after pump test 2,000.00 approx.

c) Engineering

Review of pump test results, interpretation, and construction and permanent dewatering recommendations in final report 2,500.00 approx.

d) Contingency of 5 per cent of above 750.00 approx.

Total Estimated Cost \$16,200.00 approx.


Based on your verbal authorization the installation of five observation wells and one screened and filtered pump well, commenced on November 20, 1979 and will be completed in a few days. The pump test can begin on November 26th and should be complete in 1 day with about 2 or 3 days required to monitor the stabilized groundwater levels.


Our final report including the pump test results and geotechnical recommendations regarding construction dewatering and permanent drainage measures will be submitted 2 or 3 weeks thereafter.

Should you have any questions, please do not hesitate to contact our office.

Yours very truly,

THE TROW GROUP LIMITED

  
M. G. Sarafinchin, P. Eng.

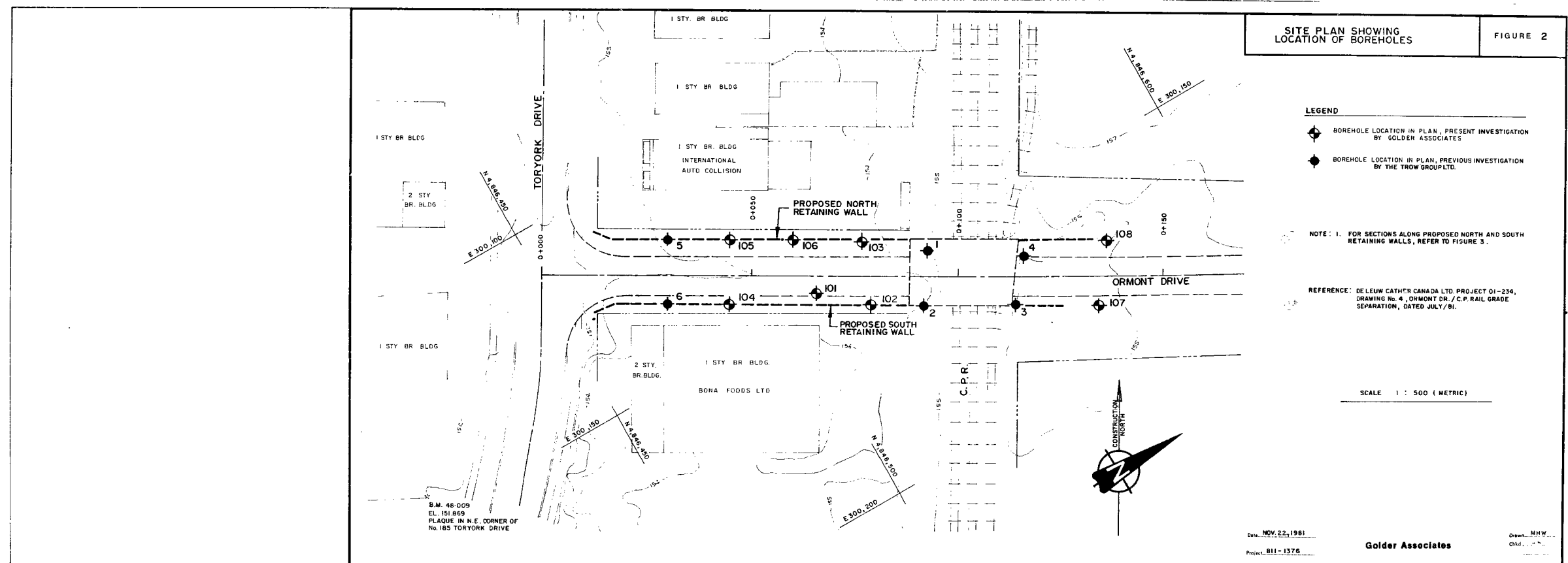
  
W. A. Trow, P. Eng.

MGS/ph

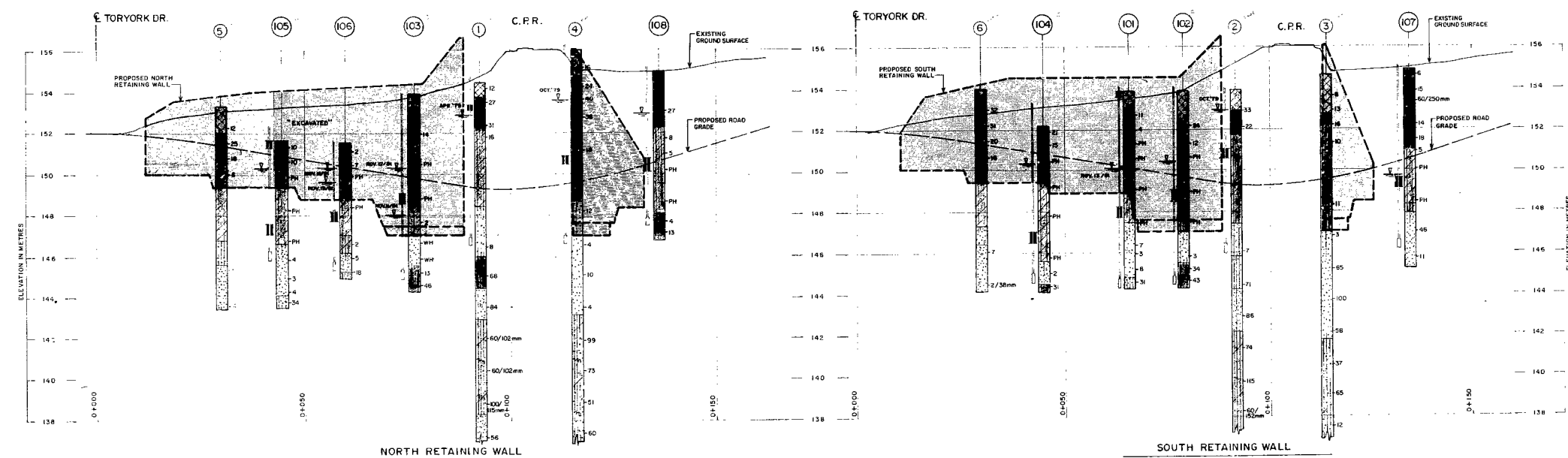
Dist: City of North York (2)  
c/o DeLCan Limited



817-1570



SECTIONS ALONG NORTH AND SOUTH RETAINING WALL ALIGNMENTS FIGURE 3



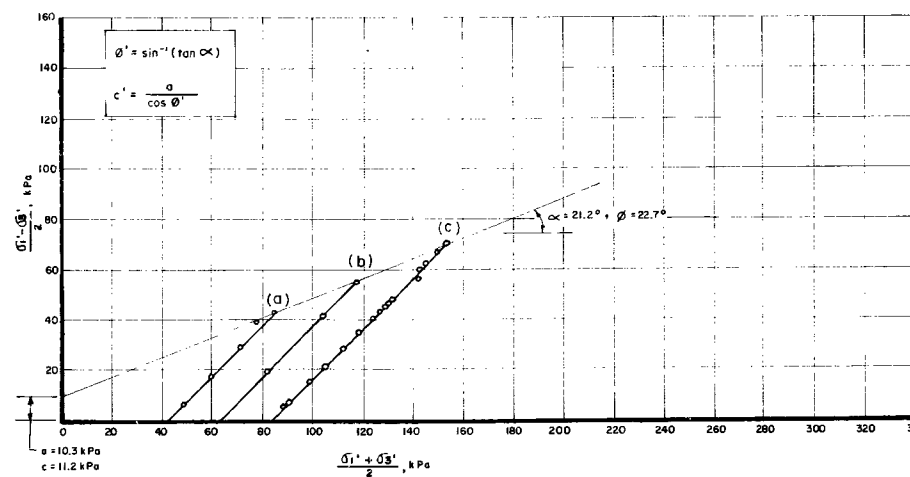
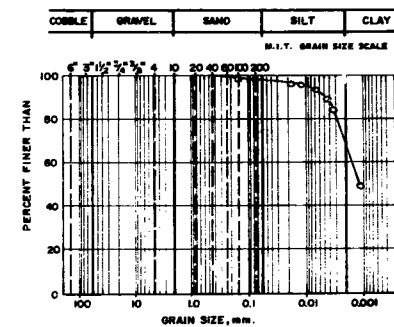
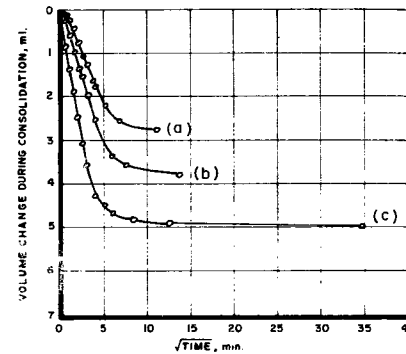
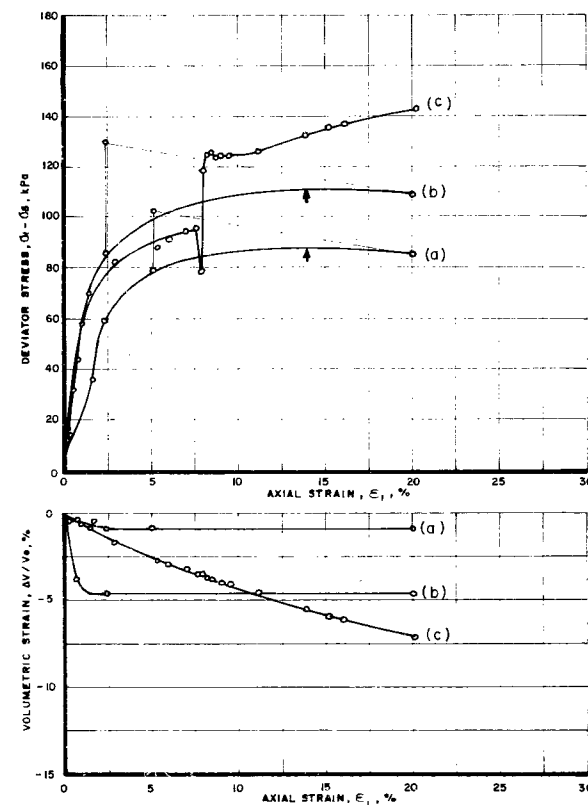
- LEGEND**
- (101) BOREHOLE IN ELEVATION, PRESENT INVESTIGATION BY GOLDER ASSOCIATES
  - STRATA PLUT, SEE DESCRIPTIONS BELOW
  - N' VALUES, BLOWS/0.3 m
  - WATER LEVEL, NOV 23, 1991 (EXCEPT WHERE NOTED)
  - BENTONITE SEAL
  - PIEZOMETER
  - (2) BOREHOLE IN ELEVATION, PREVIOUS INVESTIGATION BY THE TROW GROUP
- SIMPLIFIED STRATIGRAPHY (FOR DETAILS, REFER TO RECORDS OF BOREHOLES)**
- VERY STIFF DARK BROWN CLAYEY SILT, WITH ORGANIC MATTER (TOPSOIL)
  - VERY STIFF BROWN CLAYEY SILT TO SILTY CLAY, SOME FILL AND GRAVEL (FILL)
  - VERY STIFF TO HARD (PINK), BROWN DOTTLED GREY-OLIVE CLAYEY SILT, SANDY SILT TO SILTY CLAY, SOME SOME ABL GRAVEL
  - STIFF TO FIRM GREY SILTY CLAY TO CLAYEY SILT, SCATTERED GRAVEL, COCCONICAL SANDY ZONE
  - LOOSE TO COMPACT GREY SILT, SOME SAND, SANDY SILT TO SILTY FINE SAND
  - VERY LOOSE TO DENSE GREY FINE SAND, TRACE TO SOME SILT
  - HARD GREY CLAYEY TO SANDY SILT, TRACE TO SOME GRAVEL
  - VERY DENSE BROWN AND GREY SANDY SILT, TRACE CLAY, SOME THIN LENS OF SAND
- REFERENCE: DE LUW CATHIER CANADA LTD. PROJECT 01-234, DRAWING Nos 10, 20, 21, 25 AND 26, DRUMONT DR / C.P. RAIL GRADE SEPARATION, DATED JULY / 81

SCALES: HORIZONTAL 1 : 500 (METRIC)  
VERTICAL 1 : 100

Date NOV 22, 1991  
Project 01-234

Golder Associates

Drawn: MHW  
Checked: JH



# S TESTS CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS

FIGURE 6

	a	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	104	104	104		(a)
SAMPLE NUMBER	4	4	4		(b)
SAMPLE DEPTH, m	3.3	3.3	3.3		(c)
SPECIMEN DIAMETER, mm	50.5	50.3	50.5		(d)
SPECIMEN HEIGHT, mm	102.1	101.1	101.6		

WATER CONTENT, BEFORE CONSOLIDATION, %	47	46	45
CELL PRESSURE, $\sigma_3$ , kPa	379.2	331.0	282.7
BACK PRESSURE, kPa	337.9	268.9	200.0
PORE PRESSURE PARAMETER 'B'	0.96	0.96	0.97
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	41.4	62.1	82.7
VOLUME CHANGE DURING CONSOLIDATION, $\Delta V_c/V_0$ , %	6.4	3.4	5.0
WATER CONTENT, AFTER CONSOLIDATION, %	44	44	43
AVERAGE RATE OF STRAIN, %/hr.	—	—	0.25
AVERAGE LOAD INCREMENT, kPa	20.9	41.4	—
AVERAGE LOAD DURATION, hr.	24	24	—
TIME TO FAILURE, days	4	3	4
WATER CONTENT, AFTER TEST, %	43	41	37

MAX. DEVIATOR STRESS ( $\sigma_1 - \sigma_3$ ) max, kPa	87.6	110.3	143.4
AXIAL STRAIN AT ( $\sigma_1 - \sigma_3$ ) max, %	14.0	14.0	20.1
MAX. EFFECTIVE PRINCIPAL STRESS RATIO ( $\sigma_1' / \sigma_3'$ ) max, kPa	21.5	19.2	18.8
AXIAL STRAIN AT ( $\sigma_1' / \sigma_3'$ ) max, %	14.0	14.0	20.1

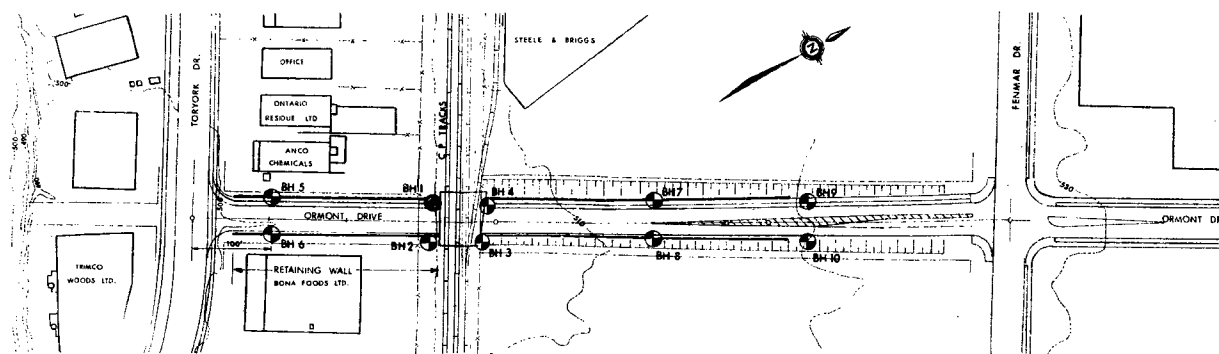
NATURAL WATER CONTENT, $w_r$ , %	45	42	44
LIQUID LIMIT, $w_p$			
PLASTIC LIMIT, $w_p$			
UNIT WEIGHT, $\gamma_t$ , Mg/m <sup>3</sup>	1.74	1.83	1.79

REMARKS  
FILTER DRAINS USED

Date DEC. 8, 1981  
Project BH-1376

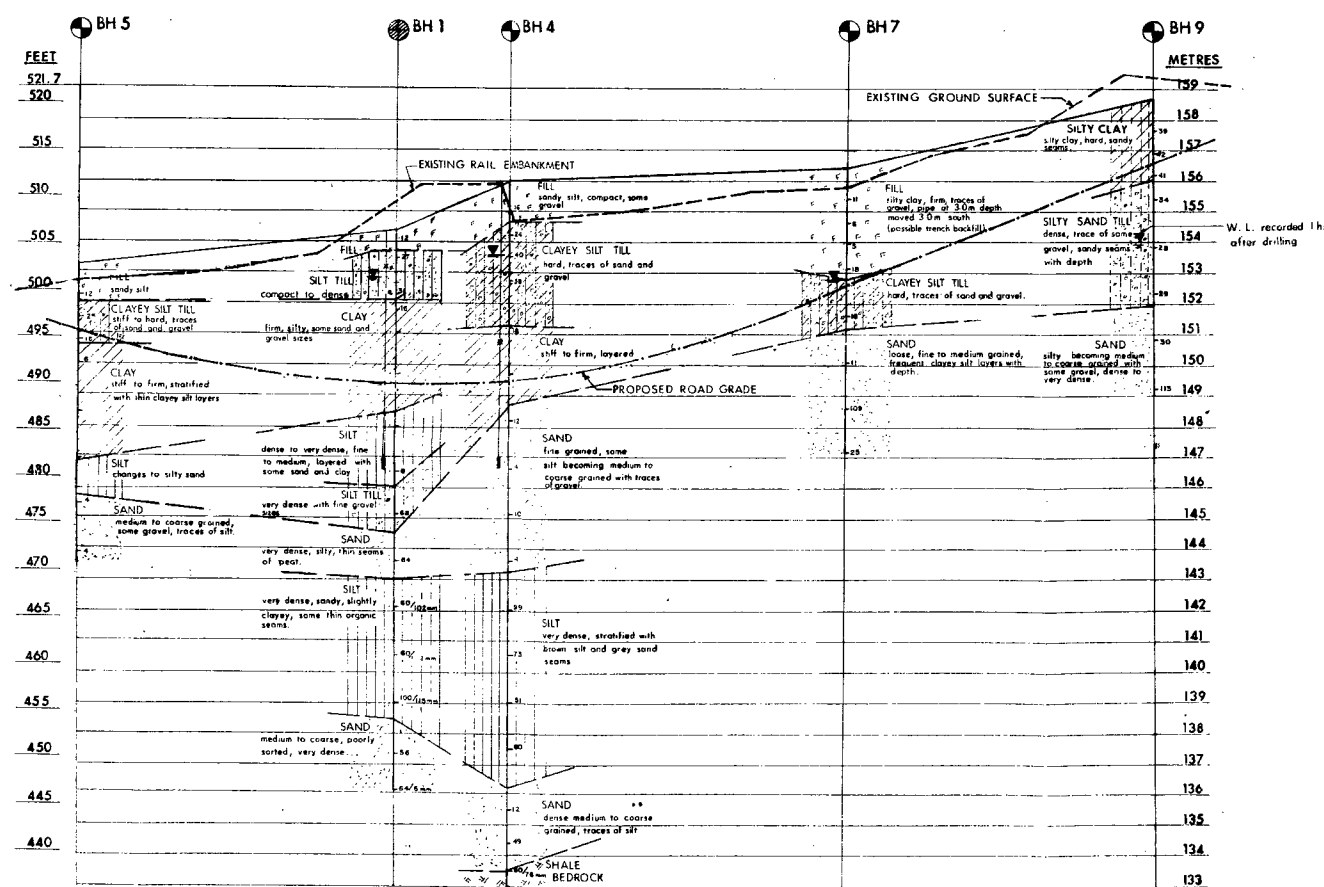
Golder Associates

Drawn MHW  
Checked JCB



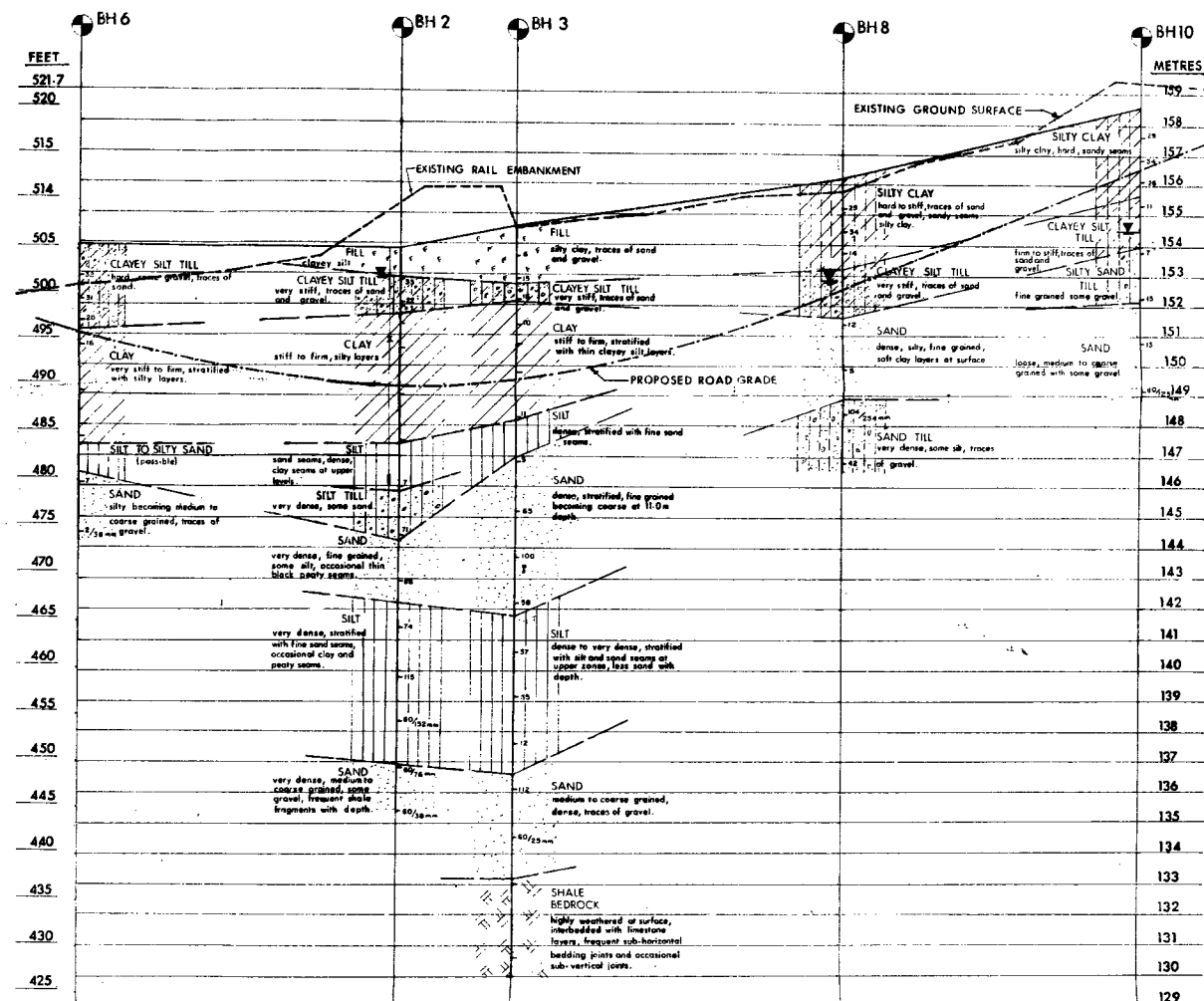
**SITE PLAN**

SCALE 1" = 100'-0"



**INTERPRETED SUBSOIL STRATIGRAPHY**

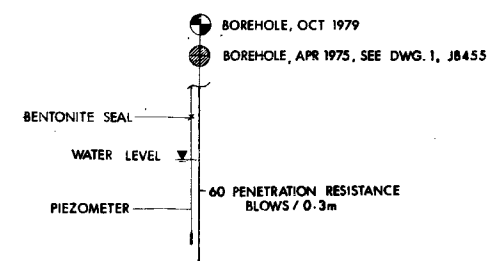
SCALE VERT 1:100  
HORIZ 1:400



**INTERPRETED SUBSOIL STRATIGRAPHY**

SCALE VERT 1:100  
HORIZ 1:400

**LEGEND**



**NOTE**

- 1) The boundaries and soil types have been established only at Bore Hole locations. Between Bore Holes they are assumed and may be subject to considerable error.
- 2) Soil samples will be retained in storage for 3 months and then destroyed unless client advises that an extended time period is required.
- 3) Topsoil quantities should not be established from the information provided at the borehole locations.

**The Trow Group Limited**  
FOUNDATION INVESTIGATION

**PROPOSED C.P. RAIL GRADE SEPARATION**

ORMONT DRIVE NEAR TORYORK DR.

NORTH YORK

ONTARIO

PROJ. T1625-G DATE OCT 1979

DWG. NR 1