HAMILTON SUBURBAN ROADS COMMISSION

COURT HOUSE

HAMILTON - ONTARIO

Mr. R. C. Fraser, P. Eng.

Commission Engineer

64-F-273 C

PROPOSED RECONSTRUCTION OF SYDENHAM ROAD DUNDAS, ONTARIO

REPORT ON THE GEOTECHNICAL ANALYSIS

Submitted by

Dominion Soil Investigation Limited

Reference Number: 4 - 4 - 6

July - 1964

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Hamilton Suburban Roads Commission,
Wentworth County Court House,
Hamilton, Ontario.

Attention: Mr. R. C. Fraser, P. Eng.,
County Engineer

RE: Soil Conditions and Foundations - Proposed re-construction of County Road No. 5 (Sydenham Road) Wentworth County, Ontario.

Dear Sirs:

This letter accompanies our detailed report on the investigation carried out at the above site.

Detailed results of the investigation are given in the report which includes , also a discussion of the various construction features of the proposed road and our recommendations for design.

The investigation has been concerned mainly with the problem of constructing the high fills in the vicinity of the existing over-pass, although information has been obtained elsewhere along the route, mainly for construction purposes.

We find that from the point of view of soil stability, the proposed embankment scheme is feasible although precautions should be taken as described.

We trust that our report contains all the information you require, however, should you have any questions, or if we can be of service in any other way, please do not hesitate to call on us.

CIVIL
K. H. KING
PROVINCE OF ONTARIO

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

K.H. King

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KHK/eb
Encl.

C O N T E N T S

A P P E N D I C E S

E N C L O S U R E S

INTRODUCTION

Authorization was obtained on April 3rd, 1964, from the Hamilton Suburban Roads Commission to perform a geotechnical analysis in connection with the proposed reconstruction of the Sydenham Road in Dundas, Ontario. The purpose of the work was to reveal the subsurface stratigraphy along the new route, to investigate and advise on the stability of the slopes and to recommend design and construction features which will result that the reconstructed road performs satisfactorily and with a minimum of maintenance.

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1. Summarized Subsoil Conditions

The south slope of the Niagara Escarpment is covered with rock talus mixed with clayey glacial tills and recent, loose and unconsolidated talus above it. The composition of the material is quite varied owing to its origin. The substrata are generally well-draining because of the presence of granular pockets and layers.

The water table is generally deep below the surface, except around section No. 8 where it is close to the ground level.

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2. Discussion

The various constructional features of the proposed road reconstruction such as cuts; fills; foundations and drainage design, are discussed in the light of the subsurface conditions revealed by this investigation. Recommendations for design are given on the basis of soil mechanics principles, other considerations being beyond the scope of this report. For convenience, the road is divided into sections in the following discussion.

(1) From Station 0 + 00 to Station 7 + 40

After an about 100-foot long portion of fill not exceeding a couple of feet in depth, the road grade is planned to be below the existing ground level. The maximum depth of the excavation will be approximately 12½ feet.

Based on the stratigraphy revealed in Boreholes Nos. 1 and 2, the materials to be removed in the cuts will be clayey tills and rock fragments to rock flour. No evidence of past major sliding was discovered and the glacial tills should comprise a sound foundation for the new road. Provided adequate drainage measures are undertaken, the side slopes of the cuts should be stable at 1 vertical in 2 horizontal.

The ground water level is located at about elevation 446 in Boreholes Nos. 1 and 2. Several heavy rains occurred during the course of the field work and these gave a good opportunity to observe the drainage conditions. No evidence of poor drainage of the substrata was discovered; however, water seepage was noticed out of the present embankment near to the north shoulder of the existing road in line with Borehole No. 1. It is presumable that this is a steady flow of ground water providing natural drainage from the escarpment and the rate of water seepage is higher after extended rainy periods or at the time of the spring thaw. Therefore, a ditch of sufficient depth should be established along the toe to intercept and remove both the run-off from the

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slope and the seeping ground water. In addition, drains should be provided up the slope to collect any free water which might be encountered.

(2) From Station 7 + 40 to Station 13 + 50

In this section, the geometrical requirements of the layout are such that the new road will be located over the existing slopes at heights above ground level up to about 35 feet. This is illustrated on sections 1, 2 and 3 attached. Two alternative schemes for constructing the road in this section are contemplated, namely, an embankment or a bridge system several spans in length. It is understood that the reconstruction of the whole road will result in a surplus of cut material which could be used for the embankment and therefore this scheme represents a considerable saving in cost over a bridge. For this reason and because of the situation of the proposed embankments, this matter has been given prime consideration in the investigation.

Embankment - Slope Stability

From the soil mechanics point of view, the governing consideration in the embankment proposal is the stability of the slopes. This has been investigated using the circular arc method of analysis. The analyses are illustrated and results quoted on Enclosures Nos. 4 and 7 to 11 inclusive.

In order to define initially the limits of the problem, several analyses were made on the assumption that the factor of safety was equal to unity (See Enclosure No. 4). From this, it was possible to compute the soil strength parameters necessary for stability in terms of

(i) the shear strength required if the angle of shearing resistance of the soil is zero. This is the cohesion of the soil in terms of total stresses determined in undrained tests and represents the conditions existing immediately at the end of construction.

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(ii) The angle of shearing resistance required if the cohesion is equal to zero. In this case, the shear strength is expressed in terms of effective stresses and the analyses represent the long-term stability of the slope taking into account the pore water pressures. Assuming that the final equilibrium ground water conditions will be similar to those that exist at present under the unloaded embankment slope, the analysis may be used to predict the long-term stability in the loaded condition. This method of analysis could also be used to determine the stability at any time if the pore water pressure changes in the slope could be predicted.

The results of trial circles carried out using these methods are illustrated in Enclosure No. 4 which shows that the undrained shear strength required at full stability is at least 1,615 pounds per square foot. The steepest natural slope in this area is represented by stratigraphical section No. 1 and this was analyzed by the circular arc method using the coefficients published in Reference (8). In the present instance, this method gives results which are not exact but are regarded as sufficiently close for practical purposes. Using this approach, it was estimated that for a factor of safety of one, the undrained shear strength required is 1,800 pounds per square foot. The lowest shear strength value measured in this investigation was 2,575 pounds per square foot as determined from an unconfined compression test performed on a sample from Borehole No. 23. It is estimated on the basis of the standard penetration test results and the measured liquidity indices of the samples, that undrained shear strengths of this order of magnitude probably represent the minimum encountered in the slope. On this basis, the factor of safety under undrained conditions is estimated to be about 1.4. Although this value is deemed to be on the low side for the construction of an embankment on an existing slope, it is acceptable in the present case since there will be some consolidation and hence increase in shear strength of the underlying soil during construction.

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Computations using the effective stress method of analysis show that the required angle of shearing resistance is about 26 degrees. This approaches the probable lower limit of the actual value to be expected for this type of clay soil of low plasticity. Therefore, the factor of safety was concluded to be greater than unity in this case also.

In view of the considerable economies to be realized by the construction of an embankment and the fact that the preliminary analysis showed the factor of safety to be critical, it was decided to perform more detailed analyses. For this purpose, it was necessary to determine more closely the effective shear strength parameters of the clay. A shallow test pit was dug in the vicinity of Borehole No. 23 and undisturbed samples of the clay were recovered by driving in a thin-walled tube sampler. These were subjected to a series of consolidated drained triaxial compression tests, the results of which are reported on Enclosure No. 31. The effective angle of shearing resistance is 35^{o} and the effective cohesion intercept is 2 pounds per square inch. Again, these values are based on comparatively few tests and comparison was made with published data (Refs. (7) and (11). This showed that the value of 35^{o} is reasonable and probably represents the upper limiting value to be expected from the clay. Furthermore, it was shown that on the basis of the liquid and plastic limits measured in this investigation, the clay has a lower limiting value of the angle of shearing resistance of about 30^{o}. The stability analyses were performed using both of these values to represent the most probable range of conditions. The cohesion value of 2 pounds per square inch may be ignored because of the generally fissured nature of the clay. This is probably on the conservative side since the fissures are likely to be closed under

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the weight of the embankment fill and there is no ready supply of free water to enable softening to occur over a long period of time.

The slip circles were analyzed with the method given by Bishop in Ref. (6). For convenience, the analyses were carried out using an IBM 7044 electronic computer, the results of one assumed circle being checked manually. The following cases were considered:

a) $\emptyset' = 35^{\circ}$; $c' = 0$; pore pressure = 0
Cross sections Nos. 1, 2 and 3 were analyzed and the results are illustrated on Enclosures Nos. 7, 10 and 11. The most critical values were obtained on cross-section No. 1 which gave a minimum computed factor of safety of 1.5 for an assumed angle of shearing resistance of 35°. In order to assess the effects of a possible rise in the ground water level, it was assumed arbitrarily that a rise of 10 feet might occur. Under this condition, the factor of safety for the most critical slip circle is reduced to about 1.4. This decrease is not considered to be significant.

b) $\emptyset' = 30^{\circ}$; $c' = 0$; pore pressure = 0
Cross-section No. 1 was selected to perform the analyses using the minimum probable angle of shearing resistance of 30°. The results which are presented on Enclosure No. 8 show that the factor of safety is then about 1.1.

c) $\emptyset' = 30^{\circ}$; $c' = 200$ psf; pore pressure = 0
This case was investigated to assess the effect of cohesion on the stability of the slope. As shown on enclosure No. 9, the factor of safety was increased from a value of 1.1 to about 1.2.

In considering the results of the stability analyses, it should be noted that the factor of safety generally decreases as the centre of the assumed slip circles moves away from the slope. This is because a greater part of the slip circles is then cutting through the embankment fill itself and its stability must be close to unity since the assumed angle of shearing resistance of the embankment material is 35°, and the side slopes are about 34° - i.e. 1 vertical in 1½ horizontal.

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The stability analyses have shown that the factor of safety with respect to sliding is greater than unity, this being the value required for equilibrium. Although the margin is small, it is considered to be adequate in the present case for the following reasons:

(i) The end effects have been ignored. In this connection, it should be mentioned that the most critical cross-section No. 1 is very localized.

(ii) Any cohesion in the clay stratum has been neglected.

(iii) Assumed angle shearing resistance of the embankment fill material is probably on the conservative side.

Thus, it is concluded that it is feasible to construct an embankment scheme. However, because the factors of safety are low, careful control during construction should be exercised:

(i) Before the fill is placed, any existing vegetation, organic matter and weathered and loosened soil should be removed from the slope. Furthermore, it is recommended that the existing slope be benched in order to eliminate any possible weak contact plane. The lower part of the embankment fill should consist of free-draining material, either obtained from a borrow source or using the coarser material from the highway cuts. This will accelerate the consolidation of the fill material and will minimize the danger of saturation at any time in the future.

(ii) Observations for any signs of incipient failure should be made and the water levels in the piezometers should be checked periodically to determine if there is any undue rise in the piezometric water levels. By these means, it should be possible to check any possible failure and to take corrective measures before it occurs.

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Bridge - Foundation Conditions

In this case, the road would be carried by a series of bridge spans. The tentative positions of the piers are indicated on enclosure 2, however, only scant data about the subsoil stratigraphy at those locations are available. Therefore, if the extended bridge scheme is selected, further borings at the proposed pier locations are required before the foundation conditions at these points can be analyzed.

Two boreholes (No. 5 and 7) were located at the proposed bridge abutments, which will be built in any event. The abutments can be supported on spread footings placed at shallow depths below the existing ground level; the allowable bearing pressure is 3000 psf between 5 and 15 feet depths and 6000 psf at 15 feet depth or lower.

Settlements will be immediate and the total settlement will not exceed one inch. However, statically determinate structure should be designed because differential settlements can occur.

(3) From Station 13 + 50 to the End of Reconstructed Road

The realigned road will go mostly in a shallow cut, hence the stability of the existing slopes will not seriously be affected. There is, however, one problem all along the new route (and this is the only problem at present also) - drainage. There are signs of erosion everywhere along the south side of the road. Retaining walls (one of them is about 6 feet high near Borehole No. 9) built of large stones have performed satisfactorily and they all served basically one purpose; to protect the earth against the forces of erosion. There are two locations where the poor drainage caused noticable damage; one is between sections 5 and 6A, the other between Nos. 7 and 10.

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Cross Section No. 6

Section No. 6 together with the two intact cross sections at both ends is presented on Enclosure No. 12. It is obvious that the end sections are convex when viewed from above whereas the central section located near the middle of the slide area is concave and lies at a lower level. It was observed that the water seeping out from the nearby vertical limestone bedrock disappeared beneath the topsoil. Presumably after rainy seasons, the amount of seepage from the cracks and joints of the bedrock is so high that it softens the shallow overburden. A test pit was dug in the centre of the slide area and it was found that the substrata consisted of about 20 inches of red clay, changing to greyish near the bottom above the presumably shale bedrock. Judging from the ages of adjacent trees, it was estimated that about 30 to 40 years ago, a major slide must have occurred at this location following which the excess earth was removed and the remainder supported with a retaining wall built of large stone slabs. The latter performed satisfactorily, however, it could not prevent the slow but steady creep of the overburden above the bedrock as indicated by the young trees and bushes which tilt almost 45 degrees from the vertical. (The rate of movement might be higher in the wet seasons and probably the soil does not move at all when the weather is dry.) Similar protective measures should be made for the new road - a solid retaining wall founded on the shale bedrock (allowable bearing pressure 10000 psf plus) should be designed here to prevent the slippage zone from spreading further east and/or west. The good drainage of the fill behind the wall is imperative.

Cross Section No. 8

This is the portion of the road which has needed much maintenance and repair work in the past. From the stratigraphical section shown on

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enclosure No. 6, it appears that there is water circulating in the cracks and joints of the rocks forming the high plateau of the Niagara Escarpment. The seepage of this water into the subsoil causes a build-up of pore pressure and a reduction in shear strength of the clay. The damage, until now, was repaired by dumping boulders to replace the earth that disappeared and this only aggravated the situation because the loads were increased. There are signs of progressive failure and, indeed, the slide area extends to a considerable distance down from the road.

The situation can be remedied by eliminating the cause of the trouble by collecting and draining the water. The soft and loose material should be scraped off the bedrock and a new, stable embankment be erected. The toe of the embankment should be secured with rock fill resting on the hard, natural ground surface. A permanent drain along the north side of the road should intercept, collect and remove the water. Furthermore, two drain lines should be installed under and across the road which would intercept the water flowing along the road and connect it to other drain system. As the water was effectively removed, no further damage is anticipated.

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3. Recommendations

The first portion of the road (between stations 0 + 00 and 7 + 40) will be in a cut, whose walls should be no steeper than 1 on 2. The water from a minor spring should be intercepted and removed by a trench.

The second portion of the road (between stations 7 + 40 and 13 + 50) should be carried by an earth and rockfill embankment, whose grade is according to the profile which was supplied to us. The factor of safety against sliding varies between values of 1.2 and 1.5 which is deemed to be sufficient. Only one span of bridge should be designed immediately above the railway tracks and the foundations for the abutments can be supported on spread footings.

Good drainage and erosion control are the main problems along the last portion of the road (from section 13 + 50 to the end of the reconstructed route).

L. S. ROLKO

DOMINION SOIL INVESTIGATION LIMITED

L. S. Rolko

L. S. Rolko, P. Eng.; A.M., A.S.C.E.

A P P E N D I C E S

PROCEDURE

Field Work

The boreholes were put down during the period between April 9th and May 22nd, 1964. Nineteen borings were performed with a total footage of 739 feet of which 286 feet was diamond core-drilled. The holes were set out with the aid of a drawing (Sydenham Grade Separation, 5-621, Sheet No. 2) provided to us by the Engineering and Roads Department of Wentworth County. All elevations given in this report were referred to a bench mark established on the north abutment of the existing road overpass. This has a given elevation of 472.43 above Geodetic Datum.

The borings were made with a skid-mounted diamond drilling rig equipped for soil sampling and testing. The diameter of the holes was that of the Bx size casing (= 2 7/8") except at the depth where no lining was required to preserve the stability of the holes. Here the size of the boreholes was reduced to 2 inches in diameter. The casing was advanced either by driving or drilling. In the former case, the hole was cleaned out by withdrawing the casing and extruding the earth from it (= dry boring) or by washing. Where the presence of boulders necessitated the advancement of casing by drilling, the hole was cleaned out by diamond drilling with a Bxt size core barrel. A total length of 142 feet of boring was made by this method.

Representative soil samples were taken at close intervals of depth with a 2-inch outside diameter split-spoon sampler driven into the subsoil with a constant energy (140 lb. hammer dropping 30 inches). By counting the blows to advance the sampler one foot, the standard penetration resistances (= N values) were obtained, thereby indicating the density of the subsurface material. In some instances, when the subsoil was so hard that excessive hammering involved the danger of breaking the split spoon or when a boulder or cobble

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was encountered, the "N" values were derived from a shorter penetration. These are recorded on the data sheets as the "extrapolated" values together with the details from which they were calculated. Generally, the overburden was too hard and too stoney to permit the recovery of undisturbed thin-walled tube samples or the determination of shear strength in-situ using a vane tester.

In some instances, disturbed samples were also taken from the withdrawn casing. Two sizes of core were recovered from the bedrock: 27.5 feet of Axt (1 1/8 in. diameter) and 258.5 feet of Bxt (1 5/8 in.diameter).

The stratification of the subsoil in terms of geodeti elevation and depth below the ground level; the position and type of sample together with the standard penetration resistances and the percentages of core recovery are recorded on the geotechnical data sheets comprising Enclosures Nos. 13-29. The methods of advancing the casing and the return or loss of the wash water are also indicated on the above sheets.

Considering that the determination of the accurate position of the water table and its variation with time is of primary importance, piezometers or plastic tubes were installed in the boreholes so that observations may be taken over a period of time. The piezometers were of the double-lined type which permitted flushing with water to prevent air being entrapped. The boreholes above and below the piezometers were sealed carefully with bentonite, hence the measurement of the head of water at a particular elevation is possible. On the other hand, where the strata were permeable throughout the entire length of the borehole, an open plastic tube was put down to the bottom. When more than one installation was made in the same hole, the tubes were differentiated by colours. After installation, a short steel pipe provided with a cap was driven into the subsoil to protect the top of the tubes.

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The positions of the piezometers are presented on the appropriate Geotechnical Data Sheets and for the sake of completeness, these are summarized below:

> Two lines with tips set at different elevations were installed in Boreholes Nos. 5, 6, 7, 8*, 12 & 23.
>
> * One line has been destroyed subsequently.
>
> One line was installed in each of Boreholes Nos. 1, 2, 3, 4, 9, 10, 11, 13, 20, 22, 24, 25 & 26.

During the course of the borings, a shallow test pit was made at the slide area defined by Section 6, the purpose of which was to determine the depth to bedrock and the nature of the overburden. Another test pit was made adjacent to Borehole No. 23 from which undisturbed soil samples were taken with thin-walled Shelby tubes. The depth of this pit was about 6 feet.

Laboratory Tests

Identification and classificat. n tests were performed on representative disturbed samples, the purpose of which was to define the index properties of the subsurface material. Five grain-size analyses were made to determine the gradation curve of the subsoil (see Encl. No. 30) and the liquid and plastic limits of 17 samples were tested to obtain the range of plasticity of the substrata.

The water contents of thirty specimens were measured which, when correlated with the plasticity index, indicate the consistency of the subsoil. The water contents together with the liquid and plastic limits are recorded on the Geotechnical Data Sheets.

Very few samples retained their geometric:1 shape, hence the unit weight of only five specimens was measured. Unconfined compression tests

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were performed on three split-spoon samples which gave the undrained shear strength of the specimens. For the results, see the logs of the boreholes.

The effective shear stress parameters of the clay stratum were determined from consolidated-drained ("slow") triaxial compression tests. These were performed on the samples taken with the thin-walled tube from the test pit near Borehole No. 23. The results obtained were $c' = 288$ psf and $\phi' = 35^\circ$, and for the detailed test data, see Encl. No. 31. The grain-size distribution and the Atterberg limits of the material were also determined and recorded on the above enclosure.

The following explanatory notes need to be added in connection with the test results:

(i) There is a discrepancy between the water contents as measured in the burette and by weight. This is not uncommon; and it is believed that the water content obtained from drying out the specimens is more correct.

(ii) The samples consolidated under 5 and 15 psi absorbed some water during the test which indicates that these pressures were still below the preconsolidation pressure. However, some water was expelled from the last sample which was subjected to 30 psi hydrostatic pressure; consequently the preconsolidation pressure was probably around the latter value.

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APPENDIX "B"

DESCRIPTION OF SITE & GEOLOGY

The section of Sydenham Road (Wentworth County Road No. 5) involved in this investigation is where it scales the Niagara Escarpment immediately north of the Town of Dundas. In this section, the road rises from about elevation 350 to about elevation 660 in a distance of about 4,000 feet. The overall height of the escarpment in this locality is about 300 feet and the average slope normal to the face is about 1 vertical in 3 horizontal. Approximately the upper 60 feet, where rock outcrops, is nearly vertical whilst the lower 240 feet of the escarpment has an average slope of about 1 in 3-1/4. The maximum shape angle of the overburden is about 1 in 1½.

The face of the slope is traversed by numerous shallow drainage gullies which carry the surface water during periods of high run-off. Water seepage is visible in places from joints in the bedrock outcrops. Except for one isolated instance near an existing slide area, free water is not generally visible at the surface of the overburden. The topography of the overburden suggests that landslides have occurred in the past. However, the present vegetation and other surface features indicate that these are now inactive except for the two instances mentioned below.

Where the existing road approaches the rock outcrop near to the top of the escarpment, there are two areas which show signs of instability. Any movement of the slope and of the road appears to be gradual and is possibly due to local water conditions. On the slope above the railway at the west end of the site there is some active surface sloughing. The overburden here appears to be clayey and wet and, furthermore, the vegetation in this area is sparse and consists mainly of small bushes.

Available information indicates the overburden on the escarpment is variable owing to the many geological factors which contributed to its formation. This is illustrated by the brief description given below of the

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geological history of the site.

The exposed rock of the Niagara escarpment was subjected to atmospheric, physical and chemical forces which caused its destruction. The spalled material fallen from the rock outcrops is generally referred to as talus and it consists of boulders, cobbles, gravel, sand and silt; that is, of rock fragments and rock flour of all sizes. The talus material collected near the toe of the slope and remained there; hence, it is a residual soil. It could also be said that the undisturbed talus is a "normally consolidated" material which was not subjected to loads greater than the weight of material at present overlying it.

This talus piled up during thousands of years and it is very conceivable that rock debris of significant thickness covered the slope of the excarpment when the region became glaciated. The glaciers advanced and retreated at least four times during the Pleistocene Epoch. The latest cold period lasted about one hundred thousand years and ended probably seven thousand years ago. The moving glaciers picked up the loose and unconsolidated material and abraded the bedrock wherever encountered. The material transported and deposited by the glaciers is the "till" and it is generally unstratified, unsorted and densely packed.

The ice moved westward in the Dundas Valley (Ref. No. 4) and deposited clay or silt tills (i.e. tills whose main components were either clay or silt) of purple colour which turns into reddish-brown upon weathering. The glaciers over-rode the talus, mixed it with the transported clayey tills and further abraded and broke up the bedrock. The result is a glacial till again, which is much more heterogeneous than the transported "primary" till.

Finally, after the retreat of the ice, the formation of talus continued and it can be expected that the uppermost strata consist of loose rock fragments and rock flour.

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APPENDIX "C"

BEDROCK GEOLOGY

(i) Introduction

From 15 of the boreholes, diamond drill cores of the bedrock were recovered. The following report deals with the geology of the area as revealed by these cores and data from other sources.

(ii) Regional Geology

The bedrock geology of this area has been described most recently and completely by J. F. Caley*.

The entire area is underlain by Paleozoic sedimentary strata belonging to the Ordovician and Silurian systems. In the Dundas area, the Ordovician system is represented by the youngest formation of the system, the Queenston red shale which forms the base of the Niagara escarpment. The lower Silurian rocks consist of sandstone and shales; the middle Silurian is composed essentially of dolomite. These overlie the Queenston unconformably and constitute the face of the Niagara escarpment.

The general attitude of these strata is a gentle dip to the southwest away from the Canadian Shield at about 30 feet to the mile with local variations.

(iii) Local Bedrock Geology from Boreholes

Lithology

Boreholes Nos. 3 to 8 inclusive, 22 and 24 at the southwest end of the drilled area, encountered Queenston red shale beneath the alluvium.

At this locality, the Queenston is a dense, fine-grained, brick-red argillite rather sandy in places and relatively massive. The rock cores well

* Caley, J. F. "Paleozoic Geology of the Toronto-Hamilton area, Ontario" - Geol.Surv.Can. Mem.224, 1940.

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and, rather than showing shaly parting, tends to fracture conchoidally. Occasional light-green bands are due to leaching of iron by surface water containing organic acids. The green material is physically identical with the normal red rock.

The Queenston shale rapidly breaks down on exposure to weathering and forms a fine, reddish clay soil but in its fresh state, as revealed in the borings, it should form a very good, impervious medium for the foundations of any proposed construction.

Boreholes No. 10, 540 feet northeast of No. 8, and 11 and 12, 280 feet east of No. 10, exhibited dark-grey soft fissile shale with 2-inch beds of grey sandstone and dolomite probably representing the Cabot Head member of the Clinton formation.

Borehole No. 26, 25 feet northwest of No. 11 encountered fine-grained dark-red sandstone and red sandy shale probably representing the Grimsby member of the upper Medina formation (Red Medina) while No. 25, 20 feet to the northwest of No. 26 cut light-grey, fine-grained quartzose sandstone apparently the Thorold member of the upper Medina.

Boreholes No. 13 and 20 respectively 600 and 850 feet northeast of No. 25 are located on the top of the escarpment and, of course, penetrate the fine-grained dense hard Lockport dolomite which crowns the escarpment.

<u>Succession</u>

As each borehole was drilled only a short distance into the bedrock (maximum 37 feet) and we are here dealing with an uncomformable contact between the Ordovician and Silurian strata, the following table for the succession near the middle of the area must be considered somewhat idealized and approximate. (See at the end of this Appendix).

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Structure

The Paleozoic rocks have been little deformed and within the area in question may be regarded as virtually flat-lying. Caley reports that evidence of faulting in the Paleozoic sediments is exrremely rare and no such evidence is reported from the Dundas area. No evidence of deformation was observed in drill cores or outcrops within the area.

(iv) Conclusions

A study of the bedrock geology indicates quite conclusively that the sedimentary succession in the project area is normal for the Dundas area and no problems due to unexpected variation in lithology or structure need be anticipated.

STRATA	ELEVATION	THICKNESS	SECTION	LITHOLOGY
Silurian				
Lockport	622(?)-660+	135'+	Boreholes 13, 20	Creamy to light grey dolomite with chert nodules.
Rochester	617-622	5'±	Webster's Falls	Dark grey soft fissile shale with 2" beds of grey, sandy limestone.
Clinton	604-617	13'	Webster's Falls	Massive, light grey, coarsely crystalline dolomite (8'). Dense dark grey dolomite in thinner even beds (5').
Medina				
Thorold	585-600	15'	Borehole 25	Light grey, quartzose sandstone.
Grimsby	572-585	15'	Borehole 26	Red sandstone and sandy shale.
(Cabot Head (Manitoulin	510-572	62'	Boreholes 10, 11, 12	Mostly grey shale with thin inter-beds of grey sandstone or dolomite.
Whirlpool	500-510	10'	In stream valley 200' N.E. of highway bridge 300'W of B.H.1	Light grey quartzose sandstone.

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STRATA	ELEVATION	THICKNESS	SECTION	LITHOLOGY
Ordovician				
Queenston	0(?)-500	500'±	Boreholes 3 to 8, 23.24	Brick-red, fine-grained argillite shale.

John H. Low, Ph.D., P.Eng.

SOIL CONDITIONS

Details of the soil conditions encountered are given on the borehole logs. In the following paragraphs, the relevant properties of the various strata are described.

TOPSOIL OR FILL

These two deposits are discussed under the same heading because both are of recent origin; they are unconsolidated and occupy the top portion of the substrata. The topsoil is mostly sandy, dark coloured, interwoven with roots and contains a fair amount of decomposed organic matter. The fill is mainly granular, although in Borehole No. 9, the clay content was quite high.

ROCK FRAGMENTS TO ROCK FLOUR

Angular boulders, cobbles, gravel, sand and silt - irregularly intermixed in various proportions - are the components of this stratum. They can either be found immediately below the organic topsoil and/or fill and, in this case, the deposit is a recent loose talus material; or they comprise part of the glacial till, in which case the deposit is a fairly dense, preconsolidated stratum. Typical grain-size distribution curves are presented on Enclosure No. 30: with the exception of the gradation curve of the combined samples Nos. 4, 5, 6 and 7 from Borehole No. 3, all diagrams represent the grain-size distribution of the rock fragments and rock flour. (It should be noted that all grains designated "coarse gravel" and larger were discarded from the soil mass.)

The material is fairly well graded, most of the particles are coarser than the No. 200 sieve size (0.074 millimeters) and the clay content is negligible. It lacks plasticity and is well draining. Slight cementation caused by calcareous agents leached out from above was noticed in some instances.

CLAYEY TILLS

These deposits were variable in appearance; sometimes a homogeneous, smooth textured and one-coloured sample was obtained, but in most instances, the soil was mottled and had the appearance of an uncemented conglomerate in which either the coarse fragments were embedded in a cohesive till mass or the coarse fragments were in contact with one another and the voids between them were filled up with clayey stuff. The deposit is not well draining and is quite plastic: the liquid limit varies between 19.4 and 47.9 percent; the plasticity index between 6.9 and 26.5 percent; hence the till is designated as a CL-ML to CI material; that is, clay-silt mixture to clay of intermediate plasticity. Typical gradation curves are presented on Enclosures Nos. 30 and 31.

.... Page 24

The natural moisture content of the clayey tills is near the plastic limit; the average liquidity index (defined as the ratio of water content minus plastic limit divided by the plasticity index) being about 0.2. Very few samples of regular geometrical shape were recovered from the boreholes owing to the disturbance caused by gravel and cobbles, but in two cases (B.H. 6/Sa. 7 and B.H. 23/Sa.2), the specimens were fairly cylindrical and the following engineering properties were obtained: Unit weight: 145.5 and 128.0 pcf; natural moisture content: 14.0 and 24.3 percent; and by assuming a spec. gravity of 2.75, the computed void ratios: 0.35 and 0.65 respectively. Unconfined compression tests were also performed on these samples and the undrained shear strengths (defined as one-half of the unconfined compression strength) were 10400 psf with a strain at failure of 4.7 percent (B.H. 6/Sa.7) and 2575 psf with a strain at failure of 5 percent (B.H. 23/Sa.2). CLAYEY TILLS (CONT'D)

The effective shear strength parameters (C' and Ø') were determined on the samples obtained from the test pit. In order to avoid the difficulties of measuring pore-water pressures in partially saturated samples, consolidated-drain triaxial compression tests were performed. The effective angle of shearing resistance is 35° and the effective cohesion is 288 psf. These results were compared with published data of tests performed on similar materials. In the Research Conference on Shear Strength of Cohesive Soils held in 1960 in Boulder, Colorado, Messrs. A. Bishop and L. Bjerrum presented a paper "The Relevance of the Triaxial Test to the Solution of Stability Problems" in which they reported C' and Ø' of 170 psf and 32½° respectively for an over-consolidated clay of low plasticity (Ref. 7). In the Proceedings of the 14th Canadian Soil Mechanics Conference held in 1960 (Ref. 11), Messrs. L. G. Soderman, T. C. Kenney and A.K. Loh (Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario) suggested a correlation between the plasticity index and effective angle of shearing resistance. According to this, clay tills of low plasticity can have a Ø' of 35° or more. For the range of the plasticity index measured on the present samples, the range of Ø' to be expected is 35° to 30°. Therefore, it is concluded that the shear strength parameters obtained in the drained triaxial compression tests are acceptable and can be used in the stability analyses.

This deposit is already a transition to shale and with justification, it could be called a "soft shale". Its unit weight is very high (135 pcf average) and the natural moisture content very low (12.5 percent average) yielding a void ratio of 0.46. HARD, DRY, RED CLAY

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GROUND WATER CONDITIONS

Several measurements of the position of the ground water table were made and the table below presents a summary of the findings:

B.H.NO	WATER LEVEL BELOW SURFACE (1964) MAY 15	MAY 20	MAY 21	MAY 25	JUNE 30
1	27' - 0"	27' - 2"	27' - 2"	27' - 8"	27' - 6"
2	23' - 2"	24' - 5"	24' - 2"	25' - 3"	28' - 6"
3	none	none	none	none	none
4	27' - 0"	29' - 2"	29' - 8"	none	none
5 Blue	41' - 8"	41' - 0"	42' - 4"	42' - 5"	none
Yellow	none	none	none	none	none
6 Red	none	none	none	none	none
Blue	29' - 6"	29' - 4"	30' - 2"	30' - 4"	30' - 4"
7 Red	30' - 2"	31' - 7"	30' - 8"	31' - 1"	31' - 4"
White	30' - 2"	31' - 0"	31' - 1"	31' - 6"	30' - 8"
8 Red	25' - 8"	26' - 5"	26' - 4"	26' - 11"	27' - 0"
9	-	-	15' - 0"	17' - 10"	18' - 8"
10	-	-	17' - 0"	17' - 4"	18' - 6"
11	26' - 0"	28' - 8"	28' - 8"	29' - 10"	31' - 0"
12 White	-	22' - 0"	22' - 0"	22' - 10"	25' - 0"
Red	-	6' - 0"	6' - 0"	6' - 1"	6' - 5"
13	-	-	27' - 8"	1' - 4" *	16' - 4"
20	-	-	-	11' - 10"	19' - 0"
22	25' - 10"	28' - 0"	28' - 4"	30' - 4"	34' - 0"
23 Red	none	none	none	none	none
Blue	30' - 2"	none	none	none	none
24	10' - 6"	11' - 4"	11' - 7"	13' - 1"	14' - 2"
25	8' - 3"	9' - 7"	9' - 10"	9' - 7"	9' - 0"
26	13' - 10"	13' - 10"	13' - 8"	13' - 10"	13' - 8"

*The high water level was probably due to rain water which entered the borehole from above.

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In addition to the above observations, "Drainage Experiments" were also performed in Boreholes Nos. 3, 4, 5, 6, 7, 8, 22, 23 and 24. (These are the holes which are located at the area where the proposed embankment will be the highest). The experiments consisted of pouring water into the tubes and observing the rate of dissipation which revealed - at least qualitatively - the relative permeability of the substrata.

<u>B.H. No. 3</u>

The water disappeared very quickly.

<u>B.H. No. 4</u>

The water level subsided steadily but at a decreasing rate.

<u>B.H. No. 5</u>

<u>Blue Line</u>: The water level subsided rather slowly.

<u>Yellow Line</u>: The water disappeared immediately.

<u>B.H. No. 6</u>

<u>Red Line</u>: The water disappeared immediately.

<u>Blue Line</u>: The water level in the tube rose about one foot and stayed there.

<u>B.H. No. 7</u>

<u>Red Line</u>: The water level subsided slowly to its original position.

<u>White Line</u>: The water level subsided quickly to its original position.

<u>B. H. No. 8</u>

<u>Red Line</u>: The water level subsided to its original position very quickly.

<u>White Line</u>: The water dissipated immediately.

<u>B.H. No. 22</u>

The water level quickly subsided to its original position.

.... Page 27

B.H. No. 23

Red Line: The water disappeared immediately.

Blue Line: The water level rose to within $2\frac{1}{2}$ ft. from the surface and subsided steadily but very slowly.

B. H. No. 24

The water level rose several inches, then subsided fairly fast to its original position.

It is our intention to observe and record the position of the water table at regular intervals because the ground water level will probably attain its highest position after long rainy periods or during the spring thaw. The highest position of the water level has to be taken into account when the factor of safety of the slope with respect to sliding is being computed.

.... Page 28

REFERENCES

(1) Procedures for Testing Soils, ASTM, April 1958. pp 186 to 198. (Unified Soil Classification System - by A. A. Wagner).

(2) Terzaghi and Peck: Soil Mechanics in Engineering Practice - John Wiley and Sons, New York 1948.

(3) The Physiography of Southern Ontario by L.J.Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press.

(4) Pleistocene Geology of the Hamilton Map Area by P.F.Karrow - Ontario Department of Mines, Geological Circular No. 8, Toronto 1959.

(5) Principles of Engineering Geology and Geotechnics by D. P. Krynine and W. R. Judd. McGraw Hill Book Co. 1957.

(6) The Use of the Slip Circle in the Stability Analysis of Slopes - by A. W. Bishop - Geotechnique Vol. V - March 1955.

(7) The Relevance of the Triaxial Test to the Solution of Stability Problems by A. W. Bishop and L. Bjerrum - Res. Conf. on Shear Strength of Cohesive Soils, Boulder, Colorado, 1960.

(8) Fundamentals of Soil Mechanics - by D. W. Taylor, 1956.

(9) Pore Pressure and Suction in Soil by D. Croney and J. D. Coleman. Conf. on Pore Pressure and Suction in Soils, London, 1961.

(10) Slope Stability Analysis - I.B.M. Scientific Datacentre, Toronto, Ont. Program prepared by Mr. F.B.D. Simpkin, P.Eng.

(11) Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario by L. G. Soderman, T. C. Kenney and A.K. Loh. Prov. XIV, Canadian Soil Mechanics Conference, October 1960.

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E n c l o s u r e s

Enclosure No. 1.

LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
Ø > 8"	3"	3/4"	4.76mm	2.0	0.42	0.074	0.002 >	NO SIZE LIMIT				

U.S. Standard Sieve Size: No 4 No 10 No 40 No 200

SAMPLE TYPES.

AS Auger sample
CS Sample from casing
ChS Chunk sample
RC Rock core
% Recovery
SS Split spoon sample
TP Piston, thin walled tube sample
TW Open, thin walled tube sample
WS Wash sample

SAMPLER ADVANCED BY static weight : w
" pressure : p
" tapping : t

OBSERVATIONS MADE WHILE CORING
Steady pressure
No pressure
Intermittent pressure

Washwater returns
Washwater lost

PENETRATION RESISTANCES.

SYMBOL:

DYNAMIC PENETRATION RESISTANCE : to drive a 2" ø, 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia. split spoon sampler 1 foot into the ground, expressed in blows per foot.

EXTRAPOLATED -N- VALUE 322

The energy for the penetration resistances is supplied by a 140 lb. hammer falling 30 inches

SOIL PROPERTIES.

W %	Water content	γ	Natural bulk density (unit weight)	k	Coeff. of permeability
LL %	Liquid limit	e	Void ratio	C	Shear strength — in terms of total stress
PL %	Plastic limit	RD	Relative density	ϕ	Angle of int. friction — in terms of total stress
PI %	Plasticity index	C_v	Coeff. of consolidation	C'	Cohesion — in terms of effective stress
LI	Liquidity index	m_v	Coeff. of volume compressibility	ϕ'	Angle of int. friction — in terms of effective stress

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL / UNCONFINED COMPRESSION TEST: ◉ ●St

LABORATORY / FIELD VANE TEST: X St + St

POCKET PENETROMETER TEST: - St

Strain at failure is represented by direction of stem: 20% (up), 5% (right), 10% (down), 15% (left)

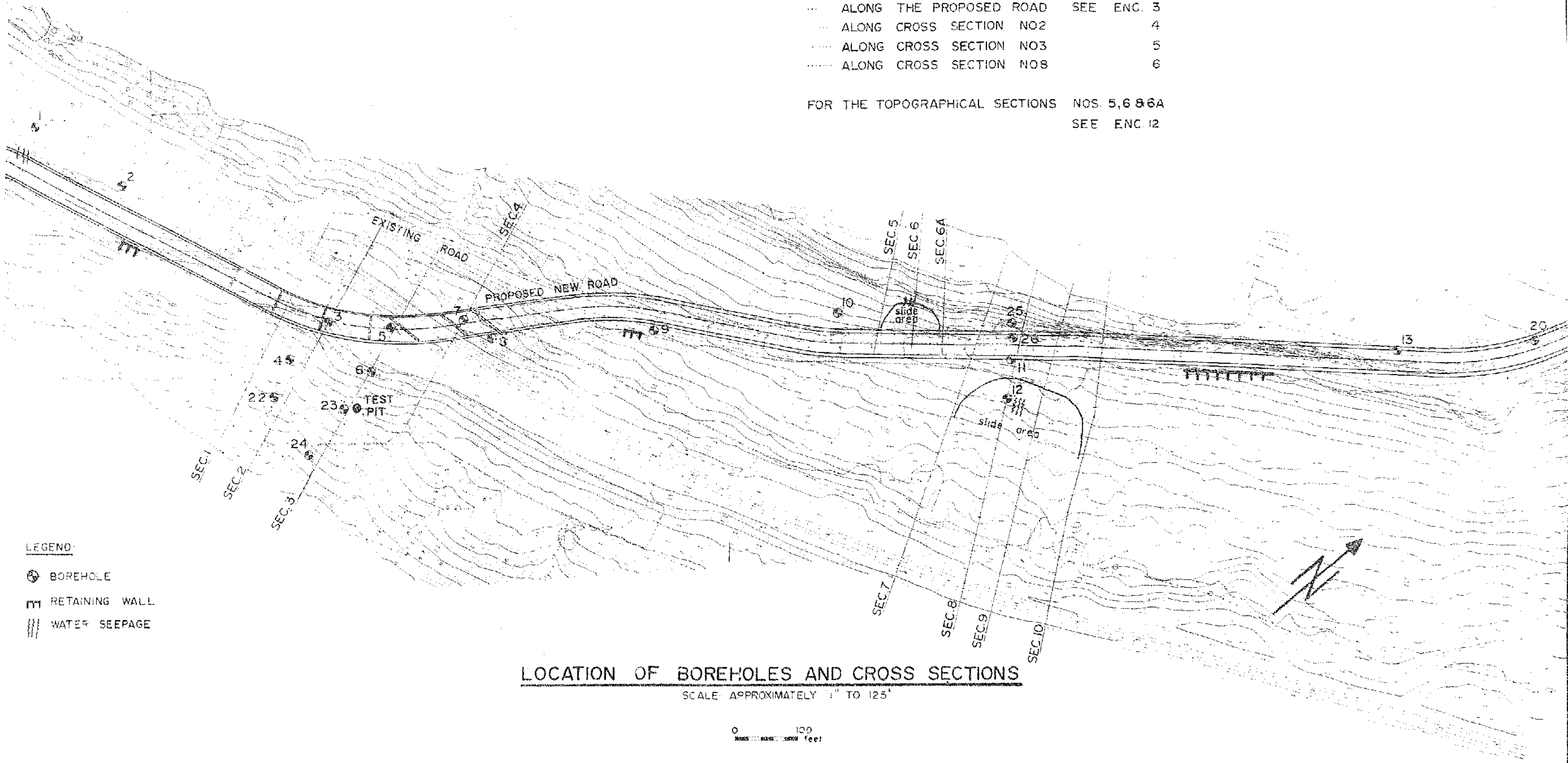
St : sensitivity = $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

SOIL DESCRIPTION.

COHESIONLESS SOILS :	RD :	COHESIVE SOILS :	C lbs/sq.ft.
Very loose	0 – 15 %	Very soft	less than 250
Loose	15 – 35 %	Soft	250 – 500
Compact	35 – 65 %	Firm	500 – 1000
Dense	65 – 85 %	Stiff	1000 – 2000
Very dense	85 – 100 %	Very stiff	2000 – 4000
		Hard	over 4000

FOR THE STRATIGRAPHICAL SECTION
ALONG THE PROPOSED ROAD SEE ENC. 3
ALONG CROSS SECTION NO2 4
ALONG CROSS SECTION NO3 5
ALONG CROSS SECTION NO8 6

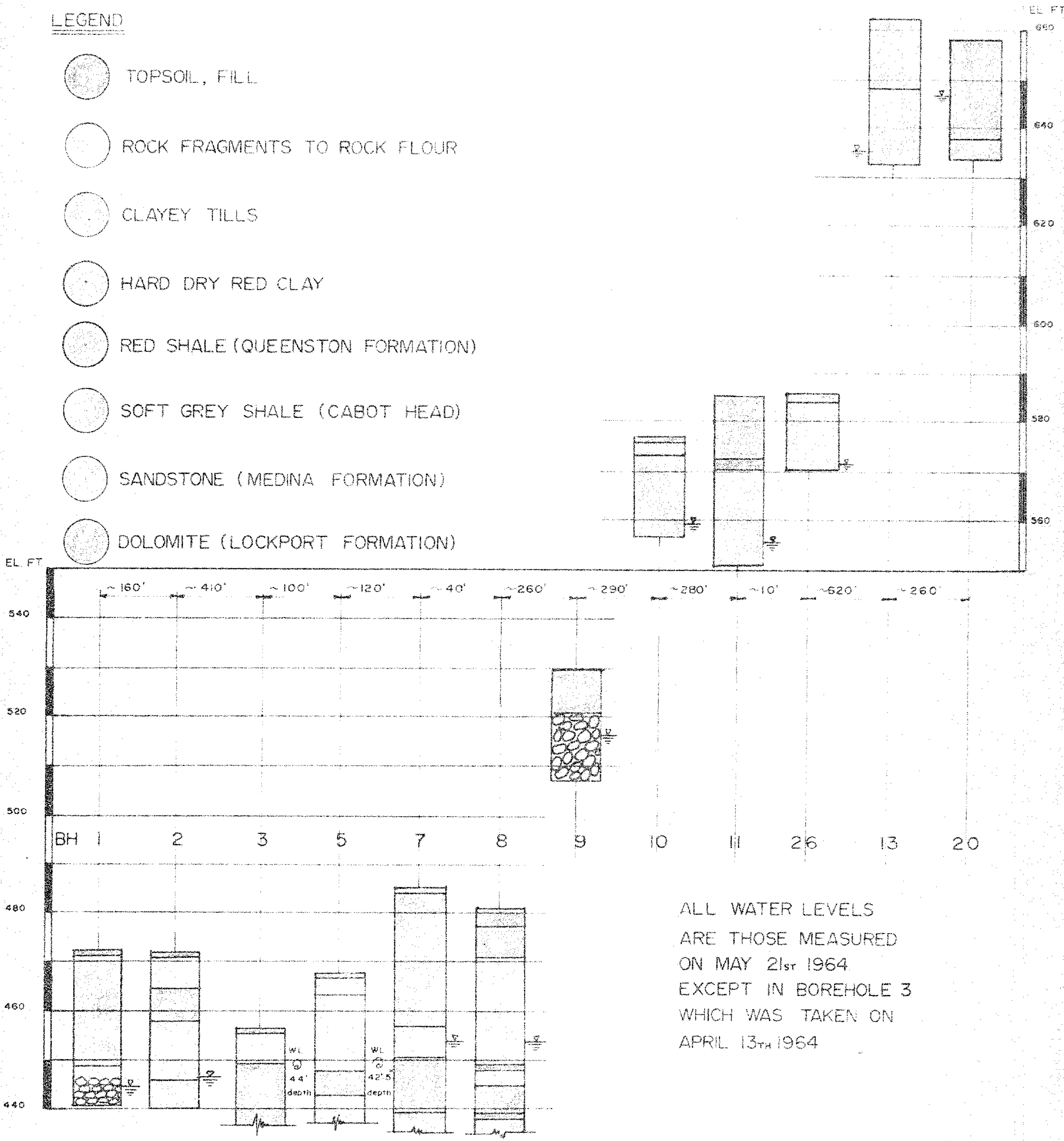
FOR THE TOPOGRAPHICAL SECTIONS NOS. 5,6 & 6A SEE ENC. 12



LOCATION OF BOREHOLES AND CROSS SECTIONS

SCALE APPROXIMATELY 1" TO 125'

0 100 feet



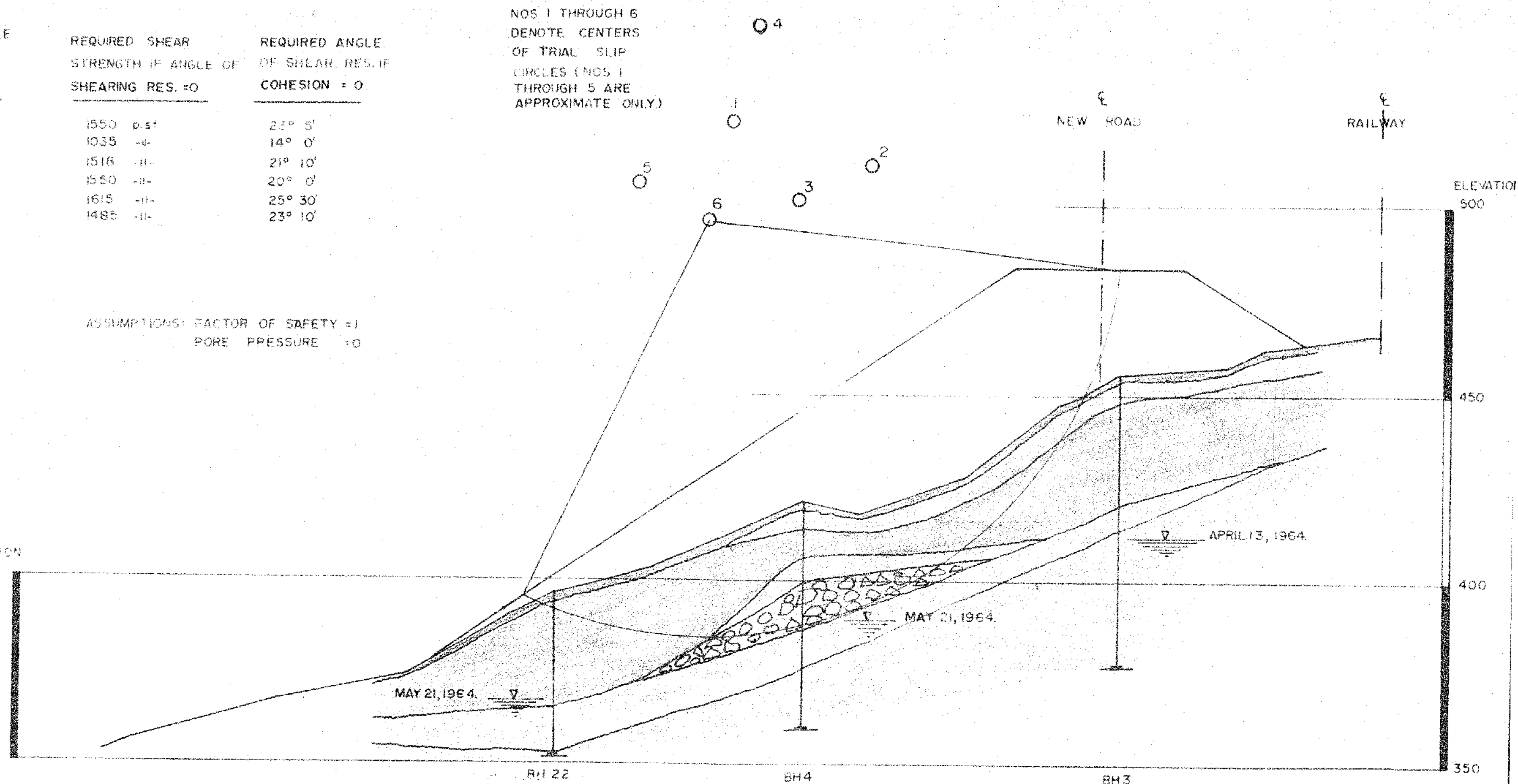
STRATIGRAPHICAL SECTION ALONG THE PROPOSED ROAD

SCALES: HORIZONTAL NOT TO SCALE · VERTICAL 1" TO 20'

CIRCLE NO	REQUIRED SHEAR STRENGTH IF ANGLE OF SHEARING RES. =O	REQUIRED ANGLE OF SHEAR RES. IF COHESION = O
1	1550 p.s.f	23° 5'
2	1035 -II-	14° 0'
3	1518 -II-	21° 10'
4	1550 -II-	20° 0'
5	1615 -II-	25° 30'
6	1485 -II-	23° 10'

NOS 1 THROUGH 6 DENOTE CENTERS OF TRIAL SLIP CIRCLES (NOS 1 THROUGH 5 ARE APPROXIMATE ONLY.)

ASSUMPTIONS: FACTOR OF SAFETY =1
PORE PRESSURE =0



STRATIGRAPHICAL SECTION NO 2

SCALE: 1" TO 30'

LEGEND

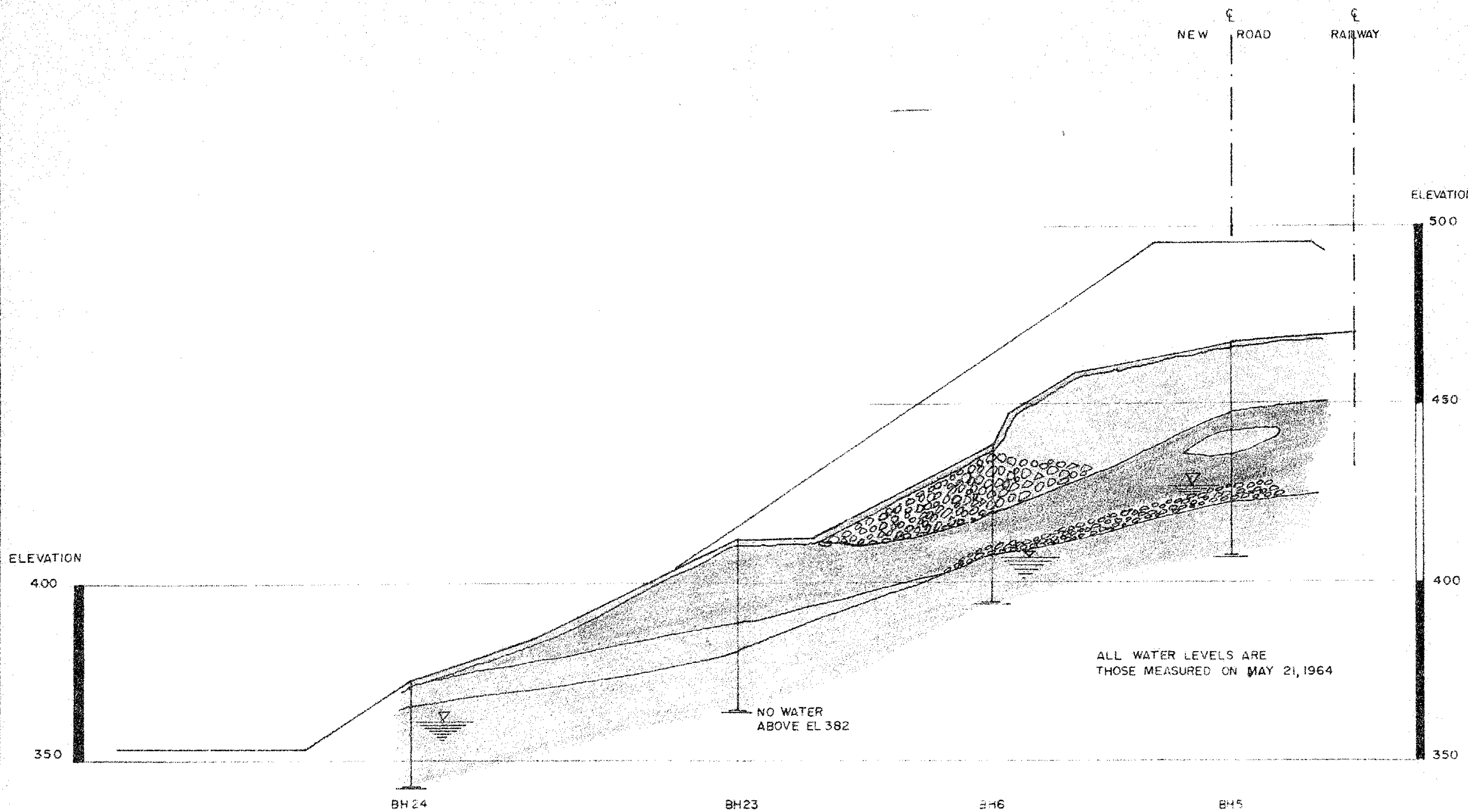
TOPSOIL

ROCK FRAGMENTS TO ROCK FLOUR

CLAYEY TILL

HARD DRY RED CLAY

RED SHALE (QUEENSTON FORMATION)

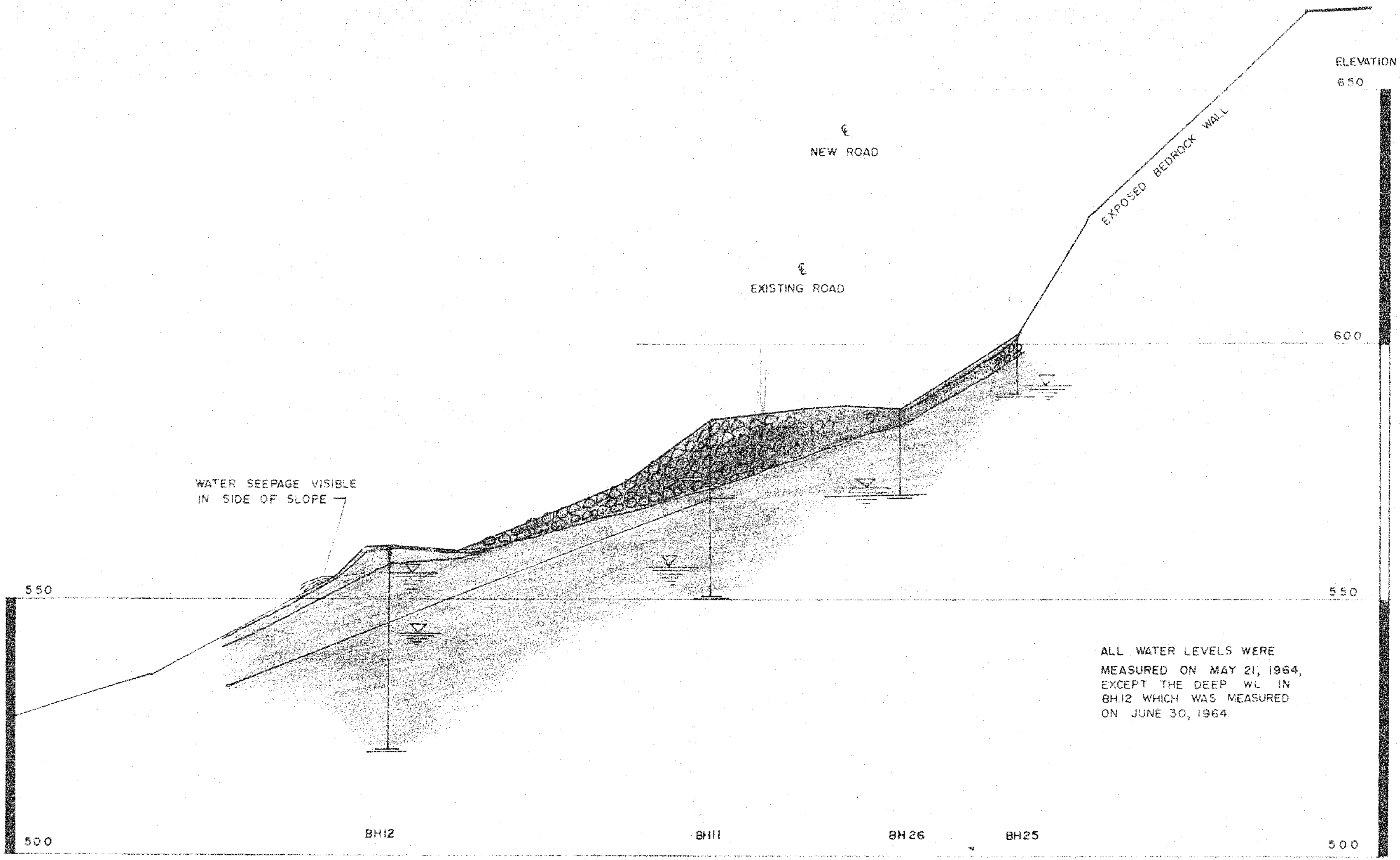


STRATIGRAPHICAL SECTION NO3

SCALE: 1" TO 30'

LEGEND

- TOPSOIL, FILL
- ROCK FRAGMENTS TO ROCK FLOUR
- CLAYEY TILLS
- HARD DRY RED CLAY
- RED SHALE (QUEENSTON FORMATION)



STRATIGRAPHICAL SECTION NO8

SCALE: 1" TO 20'

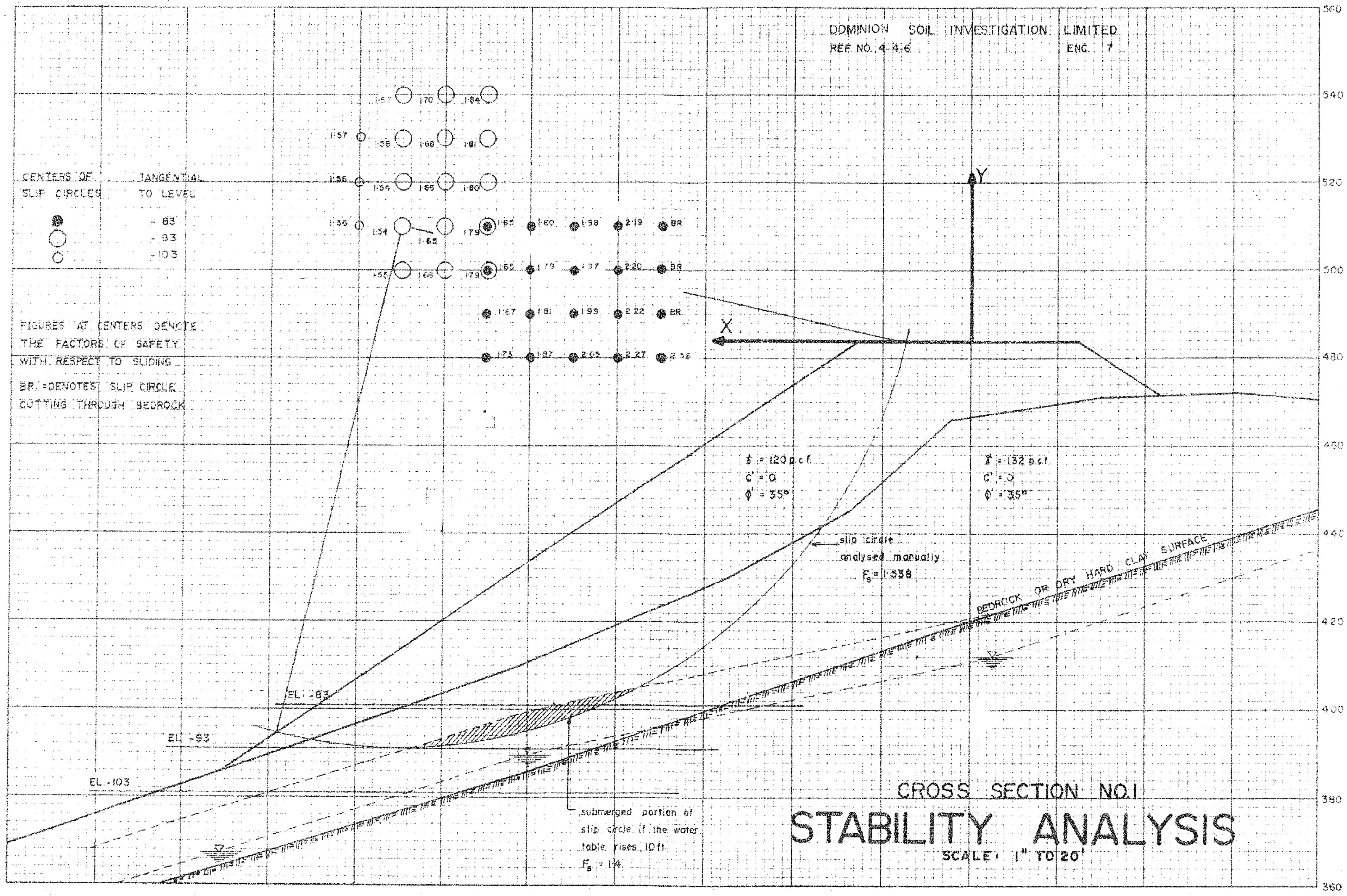
TOPSOIL, FILL

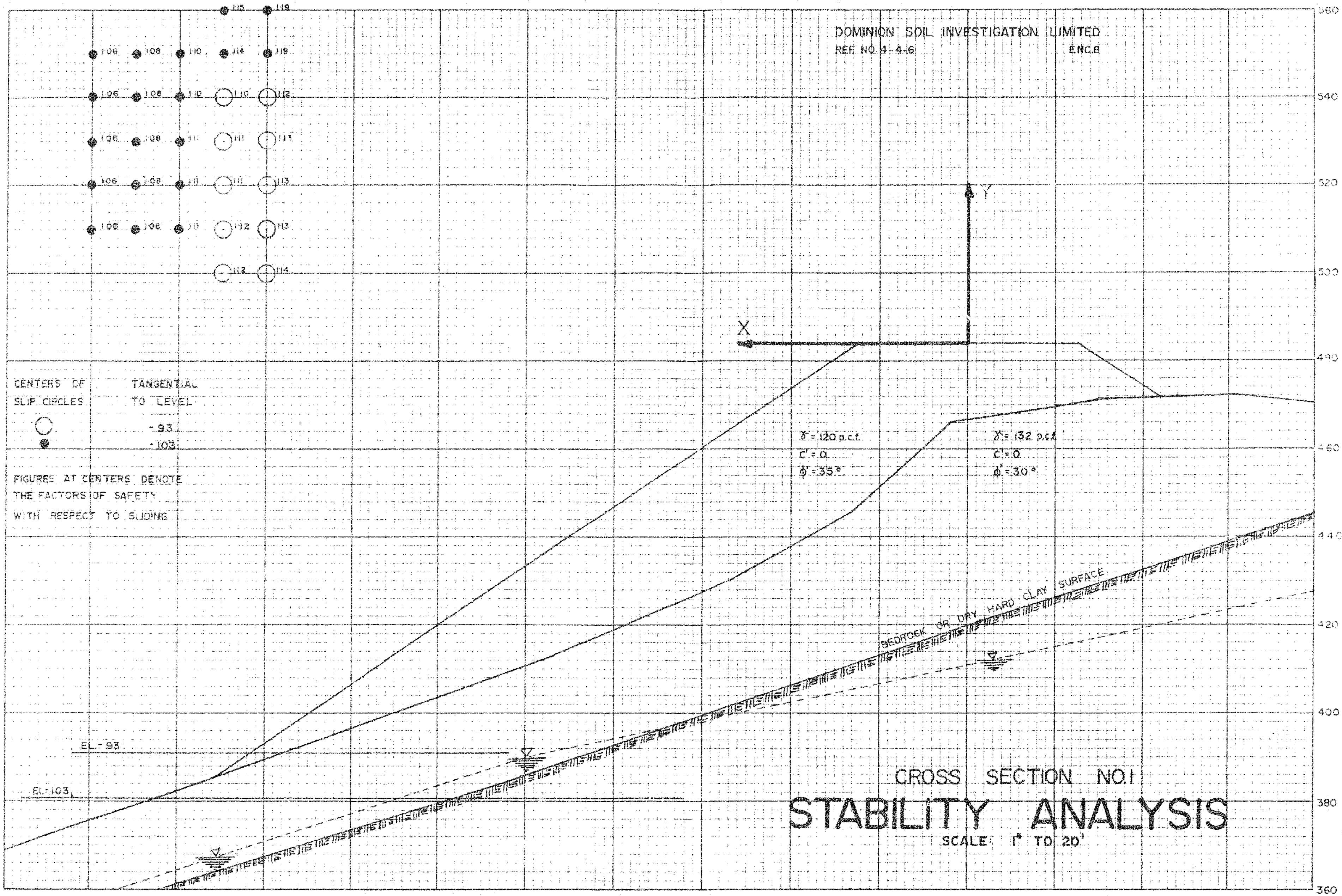
ROCK FRAGMENTS TO ROCK FLOUR

CLAYEY TILLS

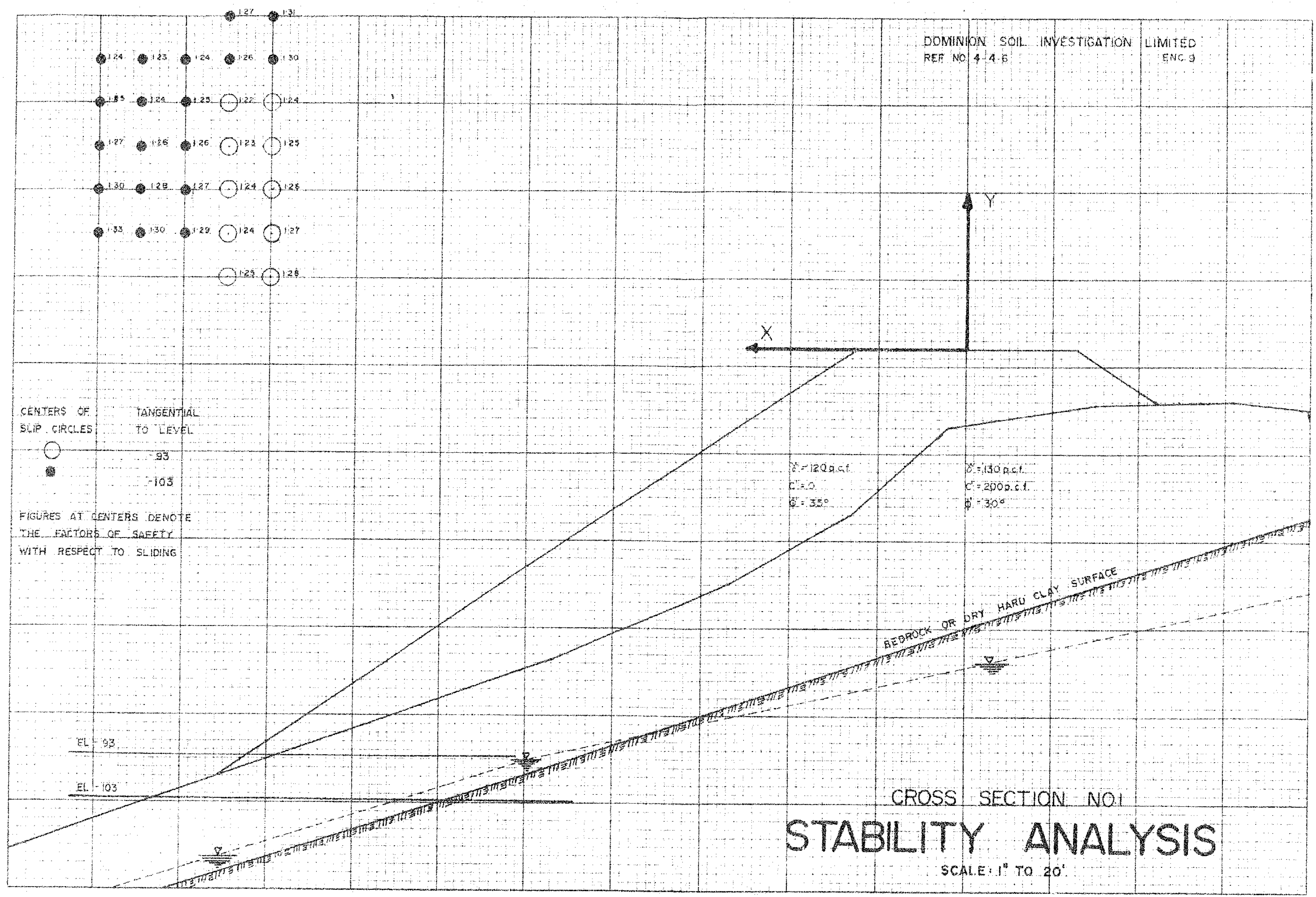
SOFT GREY SHALE (CABOT HEAD)

SANDSTONE (MEDINA FORMATION)

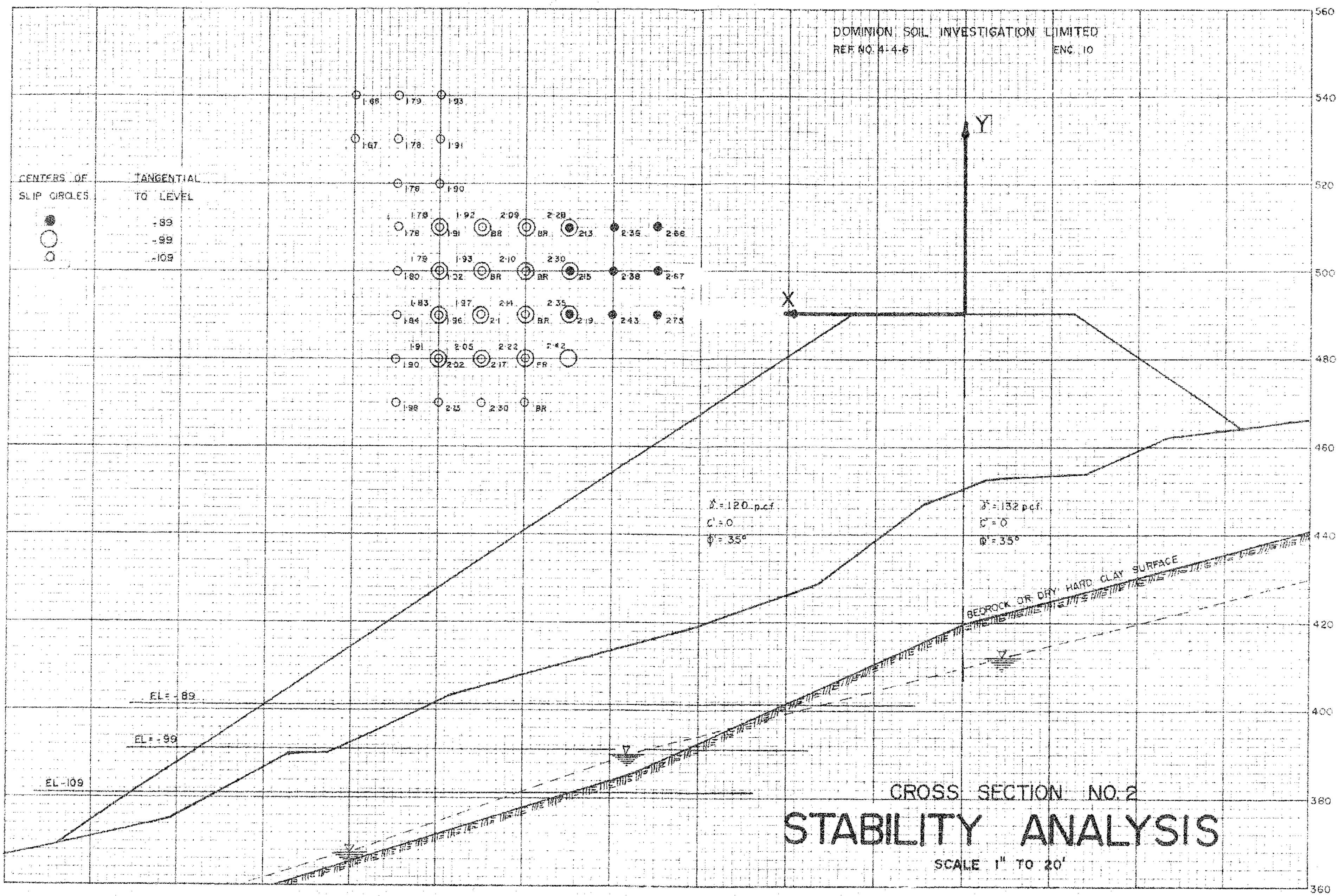

DOMINION SOIL INVESTIGATION LIMITED
REF. NO. 4-4-6
ENC. 7
CENTERS OF SLIP CIRCLES
TANGENTIAL TO LEVEL
- 83
- 93
- 103
FIGURES AT CENTERS DENOTE THE FACTORS OF SAFETY WITH RESPECT TO SLIDING
BR.=DENOTES SLIP CIRCLE CUTTING THROUGH BEDROCK
1.57
1.70
1.84
1.57
1.56
1.68
1.81
1.56
1.54
1.66
1.80
1.56
1.54
1.65
1.79
1.65
1.80
1.98
2.19
BR
1.55
1.66
1.79
1.65
1.79
1.97
2.20
BR
1.67
1.81
1.99
2.22
BR
1.73
1.87
2.05
2.27
2.56
Y
X
γ = 120 p.c.f.
C' = 0
φ' = 35°
γ = 132 p.c.f.
C' = 0
φ' = 35°
slip circle analysed manually
F_s = 1.538
BEDROCK OR DRY HARD CLAY SURFACE
EL. -83
EL. -93
EL. -103
submerged portion of slip circle if the water table rises 10 ft.
F_s = 1.4
CROSS SECTION NO.1
STABILITY ANALYSIS
SCALE 1" TO 20'
560
540
520
500
480
460
440
420
400
380
360



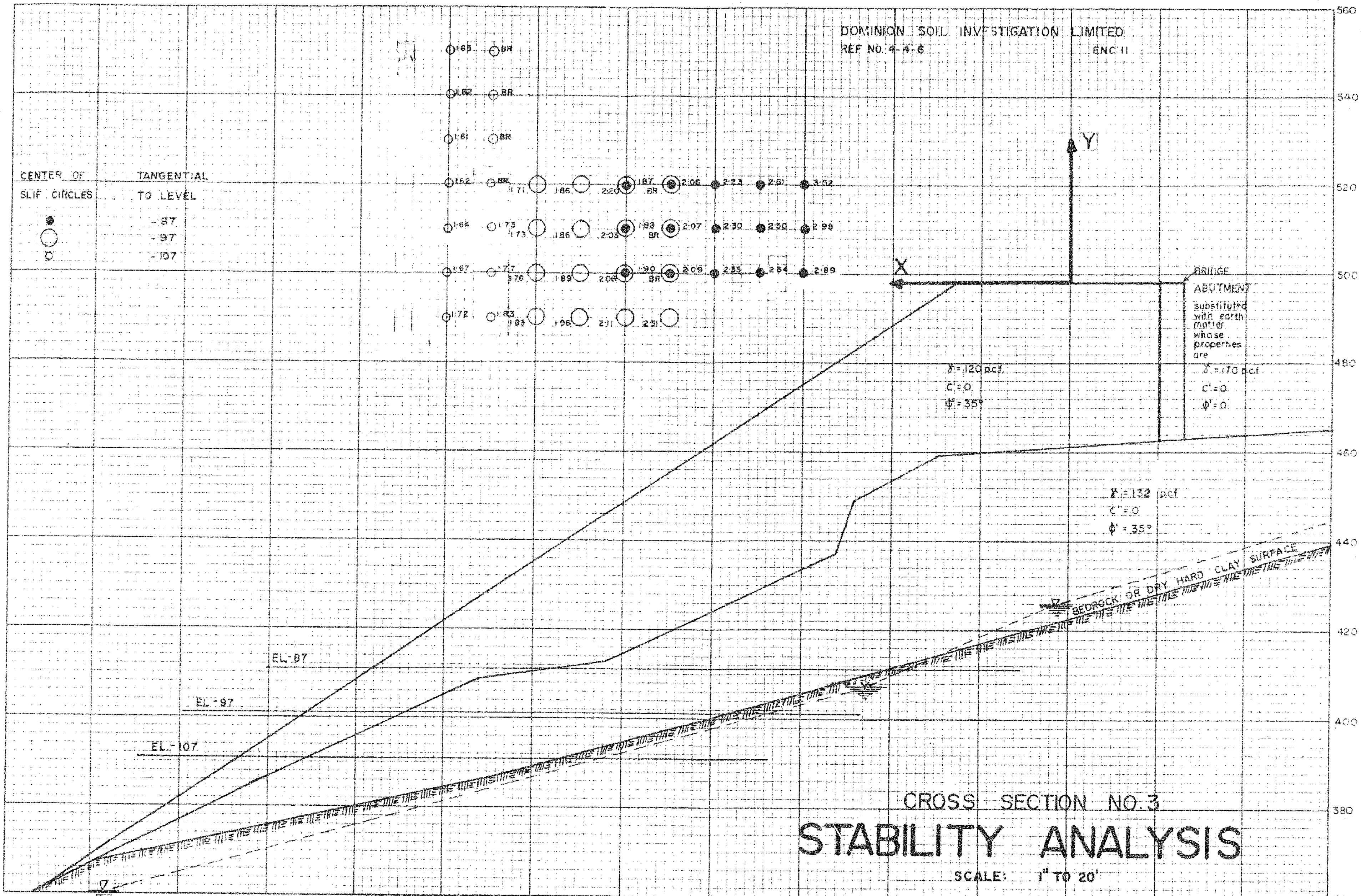
DOMINION SOIL INVESTIGATION LIMITED
REF NO. 4-4-6
ENG.8
CENTERS OF SLIP CIRCLES
TANGENTIAL TO LEVEL
-93
-103
FIGURES AT CENTERS DENOTE THE FACTORS OF SAFETY WITH RESPECT TO SLIDING
γ = 120 p.c.f.
c' = 0
φ' = 35°
γ = 132 p.c.f.
c' = 0
φ' = 30°
BEDROCK OR DRY HARD CLAY SURFACE
EL -93
EL -103
X
Y
CROSS SECTION NO.1
STABILITY ANALYSIS
SCALE 1" TO 20'
560
540
520
500
480
460
440
420
400
380
360



DOMINION SOIL INVESTIGATION LIMITED
REF NO 4-4-6
ENC. 9
1.27 1.31
1.24 1.23 1.24 1.26 1.30
1.25 1.24 1.25 1.22 1.24
1.27 1.26 1.26 1.23 1.25
1.30 1.28 1.27 1.24 1.25
1.33 1.30 1.29 1.24 1.27
1.25 1.28
Y
X
CENTERS OF SLIP CIRCLES
TANGENTIAL TO LEVEL
93
103
FIGURES AT CENTERS DENOTE THE FACTORS OF SAFETY WITH RESPECT TO SLIDING
γ = 120 p.c.f.
c = 0
φ = 35°
γ = 130 p.c.f.
c = 200 p.c.f.
φ = 30°
BEDROCK OR DRY HARD CLAY SURFACE
EL. 93
EL. 103
CROSS SECTION NO.1
STABILITY ANALYSIS
SCALE: 1" TO 20'



CROSS SECTION NO. 2
STABILITY ANALYSIS
SCALE 1" TO 20'



CROSS SECTION NO. 3

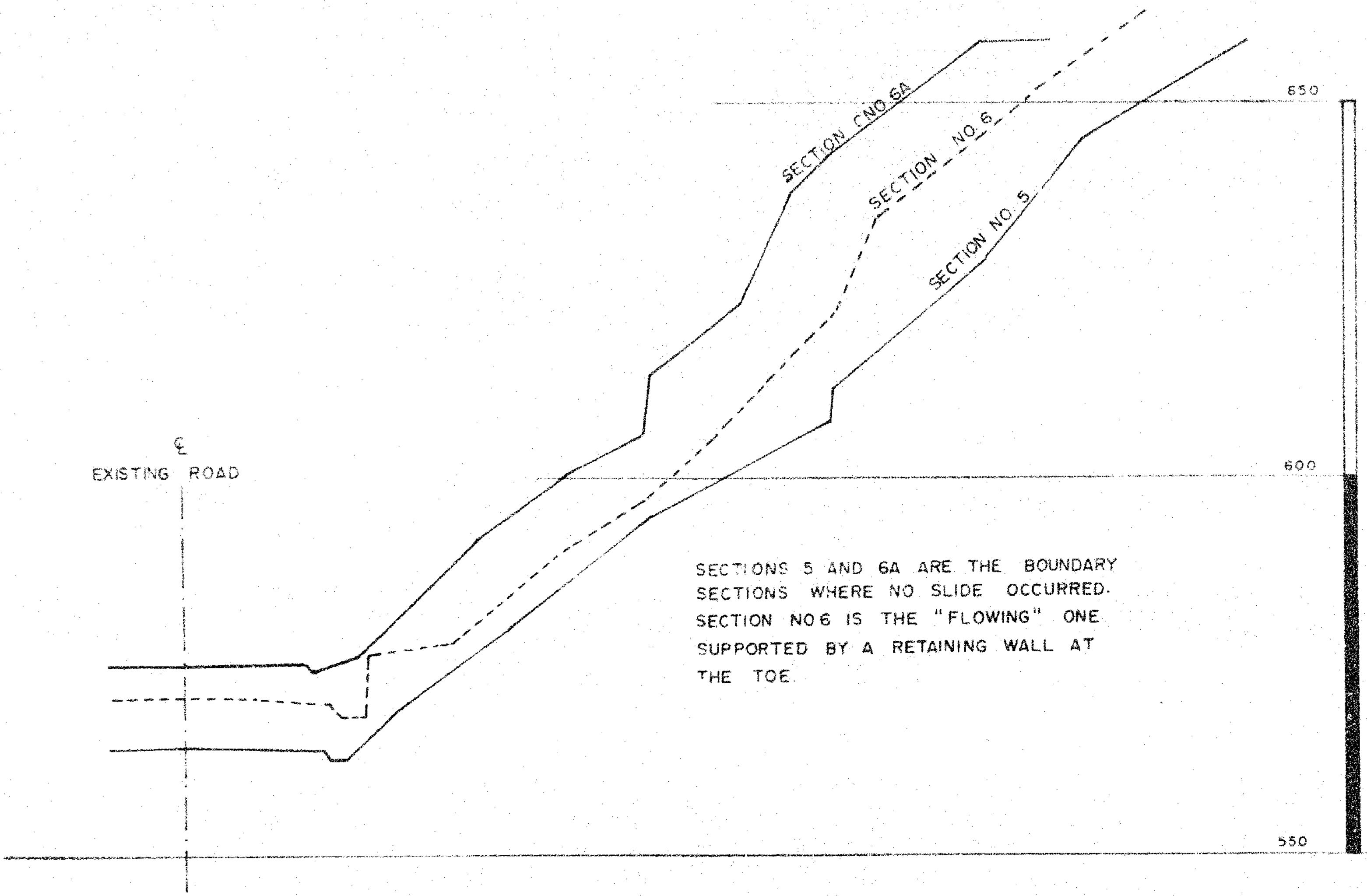
STABILITY ANALYSIS

SCALE: 1" TO 20'

DOMINION SOIL INVESTIGATION LIMITED

Our Ref. No. 4-4-6

Prep. By L.K.



TOPOGRAPHICAL SECTIONS 5, 6 & 6A

SCALE: 1" TO 20'

Enclosure No. 12

GEOTECHNICAL DATA SHEET FOR BOREHOLE ...1...

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MAY 8, 9, 1964.

ENCLOSURE NO. 13

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	N- or Advancement of Sampler	PENETRATION RESISTANCE blows per foot (20 40 60 80 100) / SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % (PL W LL; 20 40 60 80)	REMARKS
472.3	0							
		TOPSOIL						
470.0		densely packed brown CLAYEY AND SANDY TILL with embedded angular gravel (excess clay)						
	5		1	SS	32			DRYBORING / WASHBORING
465.0								
	10		2	SS	31			
460.0								
	15		3	SS	21			
455.0								
	20	excess sand	4	SS	29			
450.0								
	25	well graded LIMESTONE FRAGMENTS	5	SS	32			
445.0		LIMESTONE COBBLES AND BOULDERS	6	RC 24%				
	30		7	RC 23%				PLASTIC TUBE INSTALLED MAY 9, 1964 TIP EL. 441.9
440.0		END OF BOREHOLE						
435.0								

DETAILS OF EXTRAPOLATED N-VALUES

Sa#	Blows
1	15/6" 16/6" refusal
5	16/6" 50/6" refusal

VERTICAL SCALE: 1 IN. TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD:

GEOTECHNICAL DATA SHEET FOR BOREHOLE 2

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MAY 7, 8, 1964

ENCLOSURE NO. 14

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	"N" or Advancement of Sampler	REMARKS
471.6	0	TOPSOIL				
470.0		well graded SILTY ANGULAR GRAVEL AND SAND (limestone talus)				DRYBORING / WASHBORING
465.0	5		1	SS	68	
	10	densely packed brown CLAYEY AND SANDY TILL with embeddeded angular gravel				
460.0			2	SS	34	
	15	compact well graded SILTY SANDY ANGULAR GRAVEL (limestone fragments)				
455.0			3	SS	19	
	20	(probably talus material)				
450.0			4	SS	18	
	25					
445.0		mottled (red - green-grey - brown) CLAYEY TILL	5	SS	19	
	30					PLASTIC TUBE INSTALLED MAY 8, 1964 TIP EL. 440.1
440.0			6	SS	37	
		END OF BOREHOLE				
	35					
435.0						

PENETRATION RESISTANCE blows per foot: 20 40 60 80 100
SHEAR STRENGTH lbs. sq. ft.

CONSISTENCY water content %: PL W LL — 20 40 60 80

DETAILS OF EXTRAPOLATED N-VALUES

Sa#	Blows
4	8/6" 10/6" 15/4"

OUR REFERENCE NO. 4-4-6

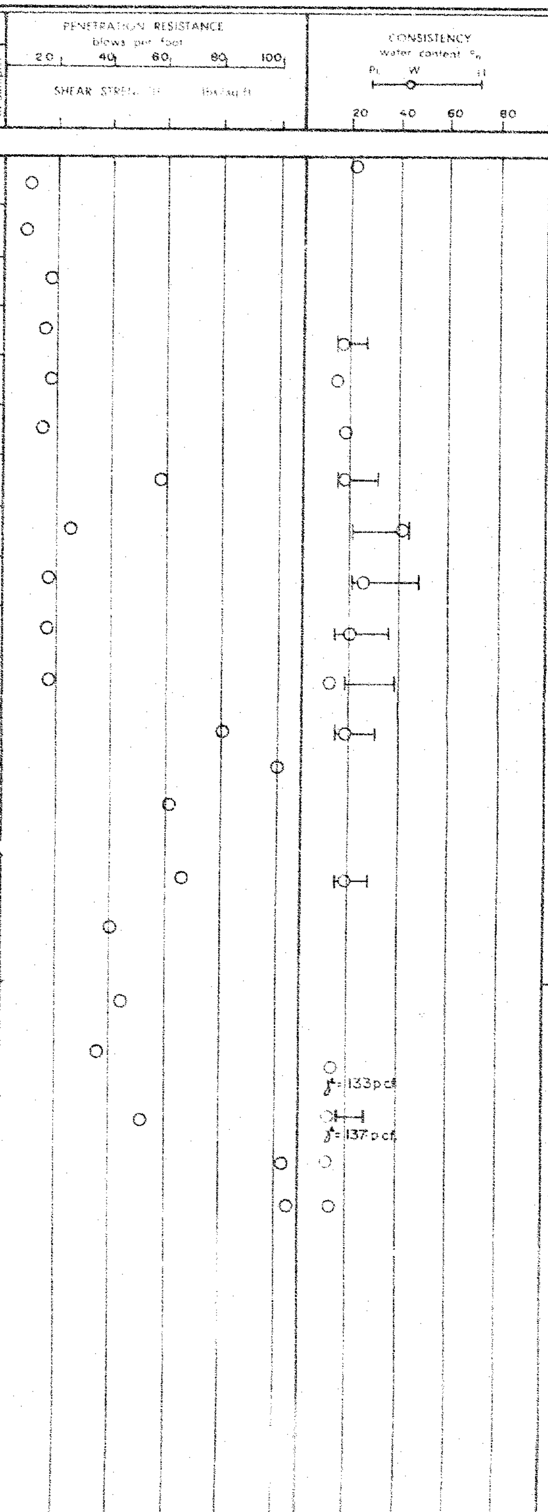
GEOTECHNICAL DATA SHEET FOR BOREHOLE . 3 . . . (0'-55')

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS, ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 9-13, 1964.

ENCLOSURE NO. 15

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	N or Advancement of Sampler
456.3	0				
455.0		TOPSOIL	1	SS	9
		loose dry buff ANGULAR GRAVEL AND SAND	2	SC	7
	5		3	SS	17
450.0			4	SS	15
		wet pocket			
	10	compact ANGULAR GRAVEL AND SAND AND SILTY CLAY OR SANDY CLAY	5	SS	17
445.0			6	SS	14
			7	SS	58
	15	organic matter	8	SS	26
440.0			9	SS	17
	20	(talus material, the two components are irregularly intermixed)	10	SS	16
435.0			11	SS	17
		glacial till	12	SS	80
	25		13*	SS*	100*
430.0			14	SS	62
			15	SS	66
	30				
425.0			16	SS	40
			17	SS	44
	35				
420.0		hard dry red CLAY with occasional grey clay layers	18	SS	36
	40		19	SS	53
415.0			20	SS	169
APRIL 13,1964			21	SS	240
	45	red SHALE with green mottlings (Queenston formation)	22	RC	81 %
410.0			24	RC	80%
	50				
405.0			24	RC	93%
	55				



REMARKS

DRYBORING / WASHBORING (at 10')

LOST WATER FROM 21'- 10" TO 22'- 0"

PIEZOMETER INSTALLED APRIL 16, 1964 TIP EL 423.02

DETAILS OF EXTRAPOLATED N-VALUES

S#	Blows
13	100/3½"
20	27/6"
	55/6"
	19/1"
21	100/5"

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD: [illegible]

OUR REFERENCE NO. 4-4-6

GEOTECHNICAL DATA SHEET FOR BOREHOLE 3 (55'-79'-7")

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 9-13, 1964

ENCLOSURE NO. 16

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	N or Advancement of Sampler	PENETRATION RESISTANCE blows per foot (20, 40, 60, 80, 100) / SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % PL W LL (20, 40, 60, 80)	REMARKS
	55								
400·0		red SHALE		25	RC 84%				
395·0	60			26	RC 85%				
390·0	65			27	RC 92·5%				
385·0	70			28	RC 75%				
380·0	75			29	RC 80%				
375·0	80	END OF BOREHOLE							

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K   CHD: [illegible]

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 4 . . (0'-60')

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 9-15, 1964

ENCLOSURE NO. 17

Elevation ft	Depth ft	Stratification Description	Sample Number	Type	Advancement of Sampler (blows per foot)	Remarks
420.0	0	TOPSOIL	1	SS	54	HOLE WAS RELOCATED BECAUSE OF BOULDERS AT SHALLOW DEPTHS
		compact dry buff SILTY SANDY GRAVEL angular limestone fragments	2	SS	75	
415.0	5		3	SS	34	
			4	CS	—	
		very stiff brown CLAYEY TILL with embedded angular gravel	5	SS	22	CASING DRILLED FROM 0' TO 30'
410.0	10		6	SS	25	
			7	SS	26	
		clay layer				
		dense SANDY ANGULAR GRAVEL	8	SS	100	
405.0	15		9	SS	62	
		wet	10	SS	59	
			11	SS	46	
400.0	20		12	SS	REFUSAL	
		GRAVEL, COBBLES AND BOULDERS (mainly limestone)	13	RC 40%		
395.0	25		14	RC 21.7%		PIEZOMETER INSTALLED APRIL 16, 1964 TIP EL 389.12
390.0	30		15	RC 39%		
			16	RC 33%		
385.0	35	hard dry red CLAY with grey pockets and embedded angular gravel	17	SS	48	
			18	SS	31	
380.0	40		19	SS	39	
			20	SS	49	
375.0	45	red SHALE with one 10" bleached green section at 52.5' (Queenston formation)	21	RC 71%		
			22	RC 36.2%		
370.0	50		23	RC 88.5%		LAST RUN OF CORING 55'-60' 98% RECOVERY END OF HOLE AT 60' DEPTH.
365.0	55					

PENETRATION RESISTANCE: blows per foot 20, 40, 60, 80, 100; SHEAR STRENGTH lbs/sq.ft.

CONSISTENCY: water content % PL W LL; 20, 40, 60, 80

VERTICAL SCALE 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L K CH'D: [illegible]

OUR REFERENCE NO. 4-4-6

GEOTECHNICAL DATA SHEET FOR BOREHOLE 5 (0'–60')

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 17, 1964.

ENCLOSURE NO. 18

Elevation ft: 467.9 (depth 0), 465.0, 460.0, 455.0, 450.0, 445.0, 440.0, 435.0, 430.0, 425.0, 420.0, 415.0
Depth ft: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55

Penetration resistance (blows per foot): 20, 40, 60, 80, 100 — Shear strength lbs/sq ft
Consistency: water content %: PL, W, LL — 20, 40, 60, 80

Stratification description	Samples: Number	Type	N or Advancement of Sampler	Remarks
TOPSOIL				
loose wet red organic SILTY SAND with rock fragments	1	SS	19	
	2	SS	23	
loose to compact SILTY AND SANDY GRAVEL (angular limestone fragments - talus) — white	3	SS	26	DRYBORING / WASHBORING
	4	SS	15	
black	5	SS	8	
buff	6	SS	27	
	7	SS	R	
	8	SS	R	
red	9	SS	26	
very stiff red and grey CLAYEY TILL	10	SS	26	
	11	SS	39	
compact damp brown and red GRAVELLY SANDY SILT (talus material)	12	SS	20	(YELLOW) PIEZOMETER INSTALLED APRIL 20, 1964. TIP EL. 439·07
	13	SS	35	
	14	SS	26	
hard damp CLAYEY TILL with embedded gravel	15	SS	30	
grey / red	16	SS	53	
LIMESTONE BOULDERS	17	RC	100%	(BLUE) PIEZOMETER INSTALLED APRIL 20, 1964 TIP EL. 424·46
	18	RC	61%	
red SHALE (Queenston formation)	19	RC	100%	
	20	RC	85%	LAST RUN OF CORING 55'-60' 100% RECOVERY END OF HOLE AT 60' DEPTH

VERTICAL SCALE 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CH'D: Kolli

GEOTECHNICAL DATA SHEET FOR BOREHOLE 6

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: CORING
DIAMETER OF BOREHOLE: 2⅞"
DATE: APRIL 21, 23, 24, 1964

ENCLOSURE NO. 19

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	Blows / % Advancement of Sampler	PENETRATION RESISTANCE (blows per foot 20 40 60 80 100) / SHEAR STRENGTH x 100 lbs. sq. ft. (0 25 50 75 100 125)	CONSISTENCY water content % (PL W LL) (20 40 60 80)	REMARKS
439.13	0	TOPSOIL						CASING DRILLED FROM 0'-33'
435.0	5	GRAVEL COBBLES AND BOULDERS in sand and silt matrix (mostly limestone talus)		RC				
			1	RC				
430.0	10		2	RC				
			3	RC				
425.0	15		4	RC				(RED) PIEZOMETER INSTALLED APRIL 24, 1964 TIP EL. 418.67
420.0	20		5	RC				
		hard brown dry SANDY CLAY TILL	6	SS	39			
415.0	25		7	SS	65		γ = 145.5 p.c.f.	
410.0	30	BOULDERS	8	RC 21 %				
		red SHALE with grey shale layers (Queenston formation)	9	RC 75 %				
			10	RC 87 %				
405.0	35		11	RC 82 %				
			12	RC 100%				
400.0	40		13	RC 92 %				(BLUE) PIEZOMETER INSTALLED APRIL 24, 1964 TIP EL. 394.6
395.0	45	END OF BOREHOLE						
390.0	50							

VERTICAL SCALE: 1 IN. TO 5 ft.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD: [illegible]

OUR REFERENCE NO. 4-4-6

GEOTECHNICAL DATA SHEET FOR BOREHOLE 7 (0'-60')

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION DUNDAS, ONT.
DATUM ELEVATION GEODETIC

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE $2\frac{7}{8}$"
DATE APRIL 23, 24, 27, 28, 1964

ENCLOSURE NO. 20

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	Blows per 0.5' Advancement of Sampler	REMARKS
485.3	0	TOPSOIL				
		reddish-brown CLAYEY GRAVEL AND COBBLES, limestone fragments embedded into stiff clay, glacial till	1	CS		
480.0	5	gravel and sand pocket	2	SS	12	DETAILS OF EXTRAPOLATED N-VALUES: So 7, Blows 29/9" 92/6"
475.0	10		3	SS	31	DRYBORING / WASHBORING
470.0	15		4	SS	37	
465.0	20		5	SS	66	
460.0	25	greenish grey	6	SS	82	
455.0	30	dense buff SANDY GRAVEL AND COBBLES (mainly limestone)	7	SS	184	
			8	SS	49	
			9	RC 47%		LOST WATER AT 33'-0"
450.0	35	hard grey CLAYEY TILL with embedded gravel cobbles and boulders	10	SS	38	(WHITE) PIEZOMETER INSTALLED APRIL 30, 1964 TIP EL. 445.3
445.0	40		11	RC 47%		
			12	SS	51	
440.0	45		13	RC 23.4%		
			14	SS	102	
		red SHALE (Queenston formation)	15	RC 20%		LOST WATER AT 51'-0"
435.0	50		16	RC 49%		(RED) PIEZOMETER INSTALLED APRIL 28, 1964. TIP EL. 431.3
	55		17	RC 42%		

SAMPLE 17 TO 56'-4"
LAST RUN OF CORING 56'4"-60' 87% RECOVERY
END OF HOLE AT 60' DEPTH

VERTICAL SCALE 1 IN TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE L.K. CHD:

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 8 . . .

OUR REFERENCE NO 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 30, 1964
MAY 1,2, 1964

ENCLOSURE NO 21

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	N or Advancement of Sampler	PENETRATION RESISTANCE blows per foot (20 40 60 80 100) / SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % (PL W LL; 20 40 60 80)	REMARKS
481.1	0							
480.0		black SAND with organics cinders and boulders						
475.0	5	grayish CLAYEY GRAVEL AND COBBLES limestone fragments embedded into stiff clay	1	SS	19			DRYBORING / WASHBORING
470.0	10	greyish SANDY ANGULAR GRAVEL, COBBLES AND BOULDERS (limestone talus)	2	SS	74			(WHITE) PLASTIC TUBE INSTALLED MAY 2, 1964 TIP EL 466.1
465.0	15		3	SS	27			LOST WATER AT 15'-1"
			4	SS	41			
460.0	20		5	SS	49			
			6	RC				
			7	SS	38			LOST WATER AT 23'-6"
			8	RC 58%				
455.0	25		9	SS	54			
			10	RC 70%				
450.0	30		11	SS	65			
		grey stiff CLAYEY TILL						
445.0	35	dense reddish SAND	12	SS	64			(RED) PIEZOMETER INSTALLED MAY 2, 1964 TIP EL. 444.8
440.0	40	grey hard CLAYEY TILL with angular sand gravel and cobbles	13	SS	43			
		hard red dry CLAY						
	45	red SHALE with green mottlings (Queenston formation)	14	SS	100			
435.0			15	RC 87%				
430.0	50		16	RC 80%				
	55	END OF BOREHOLE						

DETAILS OF EXTRAPOLATED N-VALUES

Sa #	Blows
5	20/5" 40/1"
7	19/6" 60/1"
9	27/6" 40/2"
14	100/4"

VERTICAL SCALE 1 IN TO 5 ft

DOMINION SOIL INVESTIGATION LIMITED

MADE L. K. CHD [illegible]

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 9 & 10

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: BH 9 MAY 20, 1964; BH 10 MAY 19, 1964

ENCLOSURE NO. 22

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	NUMBER	TYPE	N - or Advancement of Sampler	PENETRATION RESISTANCE blows per foot (20 40 60 80 100) / SHEAR STRENGTH lbs/sq.ft.	CONSISTENCY water content % PL W LL (20 40 60 80)	REMARKS
		BH9						
529.9	0	GRAVELLY SANDY ROAD FILL						
		reddish CLAYEY FILL traces of organic matter, gravel, cobbles	1	SS	19			
525.0	5		2	SS	23			DRYBORING
520.0	10	BOULDERS AND COBBLES INTERMIXED WITH CLAY	3	RC	89%			CASING DRILLED TO 10FT.
			4	RC	32%			
515.0	15		5	SS	31			
			6	RC	11%			PLASTIC TUBE INSTALLED MAY 20, 1964. TIP EL. 510.1
510.0	20		7	SS	53			
		END OF BOREHOLE						
505.0	25							
		BH10						
577.1	0	TOPSOIL						CASING DRILLED FROM 0' TO 4'
575.0		stiff red CLAY						
	5	soft dark grey fissile SHALE with 2" beds of grey DOLOMITE (Cabot head)	1	RC	58%			
570.0			2	RC	50%			
	10							
565.0			3	RC	10%			
	15							
560.0			4	RC	88%			PLASTIC TUBE INSTALLED MAY 19, 1964 TIP EL. 557.9
	20	END OF BOREHOLE						
555.0								
	25							

VERTICAL SCALE 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE U.K. CHD.

GEOTECHNICAL DATA SHEET FOR BOREHOLE ..11...

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MAY 13, 14, 1964

ENCLOSURE NO. 23

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	TYPE	N or Advancement of Sampler	REMARKS
585.3	0					
		loose damp grey GRAVEL-SAND FILL	1	SS	6	DRYBORING / WASHBORING
580.0	5		2	SS	5	
575.0	10		3	SS	27	
		bluish grey SANDY SILT TILL	4	SS	240	
570.0	15	soft bluish-grey SHALE with thin beds of dark grey sandstone and dolomite (cabot head)	5	RC 60%		DETAILS OF EXTRAPOLATED N VALUES: Sa 4, Blows 40/2"
			6	RC 77%		
565.0	20		7	RC 36.6%		
560.0	25		8	RC 56.6%		
555.0	30		9	RC 63%		
			10	RC 87.5%		PLASTIC TUBE INSTALLED MAY 14, 1964 TIP EL 555.5
550.0	35	END OF BOREHOLE				

PENETRATION RESISTANCE blows per foot: 20 40 60 80 100; SHEAR STRENGTH lbs/sq ft

CONSISTENCY water content %: PL W LL; 20 40 60 80

VERTICAL SCALE: 1 IN. TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD: [signature]

GEOTECHNICAL DATA SHEET FOR BOREHOLE 12

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MAY 14, 15, 1964.

ENCLOSURE NO. 24

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION
560·9	0	
560·0		TOPSOIL soft brown CLAYEY SILT
	5	stiff weathered reddish and grey CLAYEY TILL
555·0		
	10	hard dry grey CLAY with embedded angular gravel
550·0		
	15	
545·0		bluish grey soft SHALE with occasional thin beds of grey sandstone and dolomite (cabot head)
	20	
540·0		
	25	WATER LEVEL IN BEDROCK JUNE 30, 1964
535·0		
	30	
530·0		
	35	
525·0		
	40	
520·0		END OF BOREHOLE
	45	

SAMPLES NUMBER	TYPE	N or Advancement of Sampler
1	SS	4
2	SS	6
3	CS	—
4	SS	8
5 A B	SS	11
6	SS	33
7	SS	90
8	SS	120
9	RC 38·8%	
10	RC 51·6%	
11	RC 55%	
12	RC 60%	
13	RC 100%	

PENETRATION RESISTANCE blows per foot 20 40 60 80 100
SHEAR STRENGTH lbs. sq. ft.

CONSISTENCY water content % PL W LL
20 40 60 80

REMARKS

(RED) PIEZOMETER INSTALLED MAY 15, 1964 TIP EL. 552·9

DRYBORING
WASHBORING

(WHITE) PLASTIC TUBE INSTALLED MAY 15, 1964 TIP EL. 521·5

DETAILS OF EXTRAPOLATED N-VALUES

Sa #	Blows
8	60/6"

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD.: [illegible]

GEOTECHNICAL DATA SHEET FOR BOREHOLE 13 & 20

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: BH 13, MAY 21, 1964
BH 20, MAY 22, 1964

ENCLOSURE NO. 25

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	N or Advancement of Sampler	REMARKS
BH.13						
662.1	0	damp hard brown CLAYEY TILL with red shale, silt, and gravel enclosures	1	CS	—	DRYBORING / WASHBORING
660.0			2	SS	48	
655.0	5		3	SS	61	
650.0	10		4	SS	74	
645.0	15	grey DOLOMITE (Lockport formation)	5	RC 100%		
			6	RC 100%		
640.0	20		7	RC 94%		
635.0	25		8	RC 92%		PLASTIC TUBE INSTALLED MAY 21, 1964 TIP EL. 633.1
630.0	30	END OF BOREHOLE				
BH.20						
658.0	0	TOPSOIL				
650.0		hard damp brown CLAYEY TILL	1	SS	34	DRYBORING / WASHBORING
	5		2	SS	63	γ = 137 p.c.f.
645.0	10		3	SS	85	
640.0	15		4	SS	100	PLASTIC TUBE INSTALLED MAY 22, 1964 TIP EL. 638.7
635.0	20	grey DOLOMITE (Lockport formation)	5	RC		DETAILS OF EXTRAPOLATED N-VALUES: Sa# 4, Blows 50/6"

PENETRATION RESISTANCE: blows per foot 20, 40, 60, 80, 100
SHEAR STRENGTH x 100 lbs. sq. ft. 25, 50, 75, 100, 125

CONSISTENCY: water content % PL W LL; 20, 40, 60, 80

VERTICAL SCALE: 1 IN. TO 5 ft.

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CHD: [signature]

GEOTECHNICAL DATA SHEET FOR BOREHOLE 22

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MAY 4 TO 6, 1964

ENCLOSURE NO. 26

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	TYPE	"N" or Advancement of Sampler	PENETRATION RESISTANCE (blows per foot)	REMARKS
395·1	0	TOPSOIL					CASING WAS DRILLED TO 30FT.
		compact to dense ANGULAR GRAVEL SAND AND BOULDERS AND reddish-brown SILTY SANDY CLAY (talus material, the two components are irregularly intermixed) glacial till	1 & 2	RC fragments			
390·0	5	*excess clay*	3	SS	18	18	
385·0	10		4	SS	34	34	
380·0	15		5	SS	35	35	
375·0	20		6	SS	38	38	
			6A	RC			
370·0	25	*excess sand and gravel*	7	SS	39	39	
365·0	30	hard dry red CLAY transition to shale	8	SS	39	39	
360·0	35		9	SS	120	120	PLASTIC TUBE INSTALLED MAY 6, 1964 TIP EL. 357·6
355·0	40		10	SS	126	126	
			11	SS	REFUSAL		
350·0	45	END OF BOREHOLE					

PENETRATION RESISTANCE: blows per foot 20 40 60 80 100; SHEAR STRENGTH lbs/sq ft

CONSISTENCY: water content % PL W LL; 20 40 60 80

VERTICAL SCALE: 1 IN. TO 5 ft

DOMINION SOIL INVESTIGATION LIMITED

MADE: L.K. CH'D: Vollo

GEOTECHNICAL DATA SHEET FOR BOREHOLE 23

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 27, 28, 29, 30, 1964

ENCLOSURE NO. 27

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	TYPE	2" or Advancement of Sampler	REMARKS
413.8	0	TOPSOIL				CASING WAS DRILLED TO 32 FT
410.0	5	weathered	1	SS	13	
		stiff to hard red brown and greenish grey CLAYEY AND SILTY TILL, embedded boulders and cobbles	2	SS	17	γ = 128 pcf
405.0	10		3	SS	24	
400.0	15		4	SS	61	(RED) PIEZOMETER INSTALLED APRIL 30, 1964 TIP EL. 399.05
395.0	20		5	SS	150	
390.0	25	hard dry red CLAY with greenish fragments and boulders	6	SS	110	
385			7	RC		
	30		~~8~~	~~SS~~	~~REFUSAL~~	(BLUE) PIEZOMETER INSTALLED APRIL 30, 1964 TIP EL. 381.8
380.0			9	RC		
	35	red SHALE with greenish sections up to 12" thick (Queenston formation)	10	RC 100%		NO WATER ABOVE EL. 382.0
375.0	40					
370.0	45		11	RC 100%		
365.0	50		12	RC 100%		
		END OF BOREHOLE				
360.0	55					

PENETRATION RESISTANCE (blows per foot): 20, 40, 60, 80, 100
SHEAR STRENGTH x 100 lbs/sq ft: 25, 50, 75, 100, 125
CONSISTENCY (water content %): PL, W, LL; 20, 40, 60, 80

DETAILS OF EXTRAPOLATED N-VALUES

Sa#	Blows
3	8/4"
5	50/4"
6	60/6"
	55/6"
	54/2½

VERTICAL SCALE 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE L.K. CH'D: [illegible]

GEOTECHNICAL DATA SHEET FOR BOREHOLE 24 . . .

OUR REFERENCE NO. 4-4-6

CLIENT: HAMILTON SUBURBAN ROAD COMMISSION
PROJECT: SYDENHAM ROAD
LOCATION: DUNDAS ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE:

ENCLOSURE NO. 28

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION
372.43	0	TOPSOIL
370.0		very stiff to hard red CLAY with occasional grey spots
365.0		red SHALE with sections of bleached green argillite (Queenston formation)
	30	END OF BOREHOLE

SAMPLES NUMBER	TYPE	N or Advancement of Sampler
1	SS	16
2	SS	36
3	RC 100%	
4	RC 25%	
5	RC 66.6%	
6	RC 66.6%	
7	RC 86%	

PENETRATION RESISTANCE blows per foot 20 40 60 80 100 — SHEAR STRENGTH lbs/sq ft

CONSISTENCY water content % PL W LL — 20 40 60 80

REMARKS:
CASING WAS DRILLED TO 5FT.
PLASTIC TUBE INSTALLED MAY 2, 1964 TIP EL. 354.7

GEOTECHNICAL DATA SHEET FOR BOREHOLES 25 & 26

OUR REFERENCE NO 4-4-6

CLIENT HAMILTON SUBURBAN ROAD COMMISSION
PROJECT SYDENHAM ROAD
LOCATION DUNDAS ONT.
DATUM ELEVATION GEODETIC

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE 2 7/8"
DATE BH 25 & 26 MAY 12, 1964

ENCLOSURE NO 29

Headings: ELEVATION ft | DEPTH ft | STRATIFICATION DESCRIPTION | STRATIFICATION SYMBOL | SAMPLES (NUMBER, TYPE, Advancement of Sampler) | PENETRATION RESISTANCE blows per foot 20 40 60 80 100; SHEAR STRENGTH lbs sq ft | CONSISTENCY water content % PL W LL 20 40 60 80 | REMARKS

BH. 25

Elevation ft	Depth ft	Stratification Description	Sample Number	Type	Remarks
602.3	0	TOPSOIL loose angular ROCK FRAGMENTS			CASING WAS DRILLED TO 4FT
600.0	5	light grey quartzose SANDSTONE (Medina formation)	1	RC 83%	
595.0	10	(water level)	2	RC 40%	PLASTIC TUBE INSTALLED MAY 12, 1964 TIP EL 591.8
590.0	15	END OF BOREHOLE			

BH. 26

Elevation ft	Depth ft	Stratification Description	Sample Number	Type	Blows	Remarks
586.0	0	ROADFILL AND ROCK TALUS				CASING WAS DRILLED TO 4FT
585.0		red SANDSTONE and SANDY SHALE (Medina formation)	1	SS	24	
580.0	5		2	RC 36%		
			3	RC 56.6%		
575.0	10		4	RC 56.6%		PLASTIC TUBE INSTALLED MAY 12, 1964 TIP EL 571.5
570.0	15	END OF BOREHOLE				
	20					

DETAILS OF EXTRAPOLATED N-VALUE

Sa	Blows
1	12/6"
	50/2"

VERTICAL SCALE 1 IN. TO 5 FT

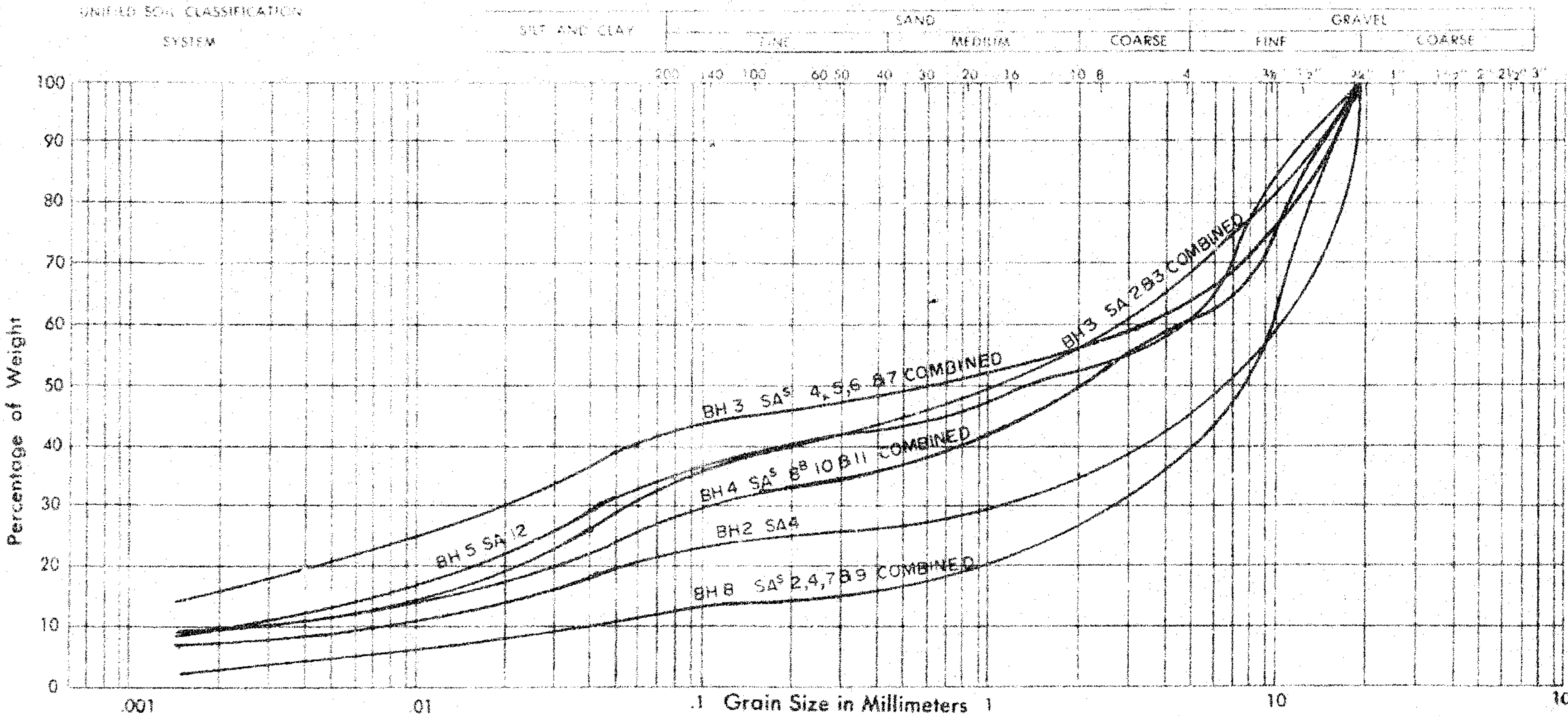
DOMINION SOIL INVESTIGATION LIMITED

MADE L.K. CHD. Koclue

DOMINION SOIL INVESTIGATION LIMITED

GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 4-4-6



PROJECT SYDENHAM ROAD

LOCATION: DUNDAS ONT

BOREHOLE NO.:	2	3	3	4	5	8
SAMPLE NO.	4	2-3	4-7	8,10,11	12	2,4,7,9

DEPTH OF SAMPLES: 2' — 27'

ELEVATION OF SAMPLES 400 — 470

COEFFICIENT OF UNIFORMITY non applicable

COEFFICIENT OF CURVATURE non applicable

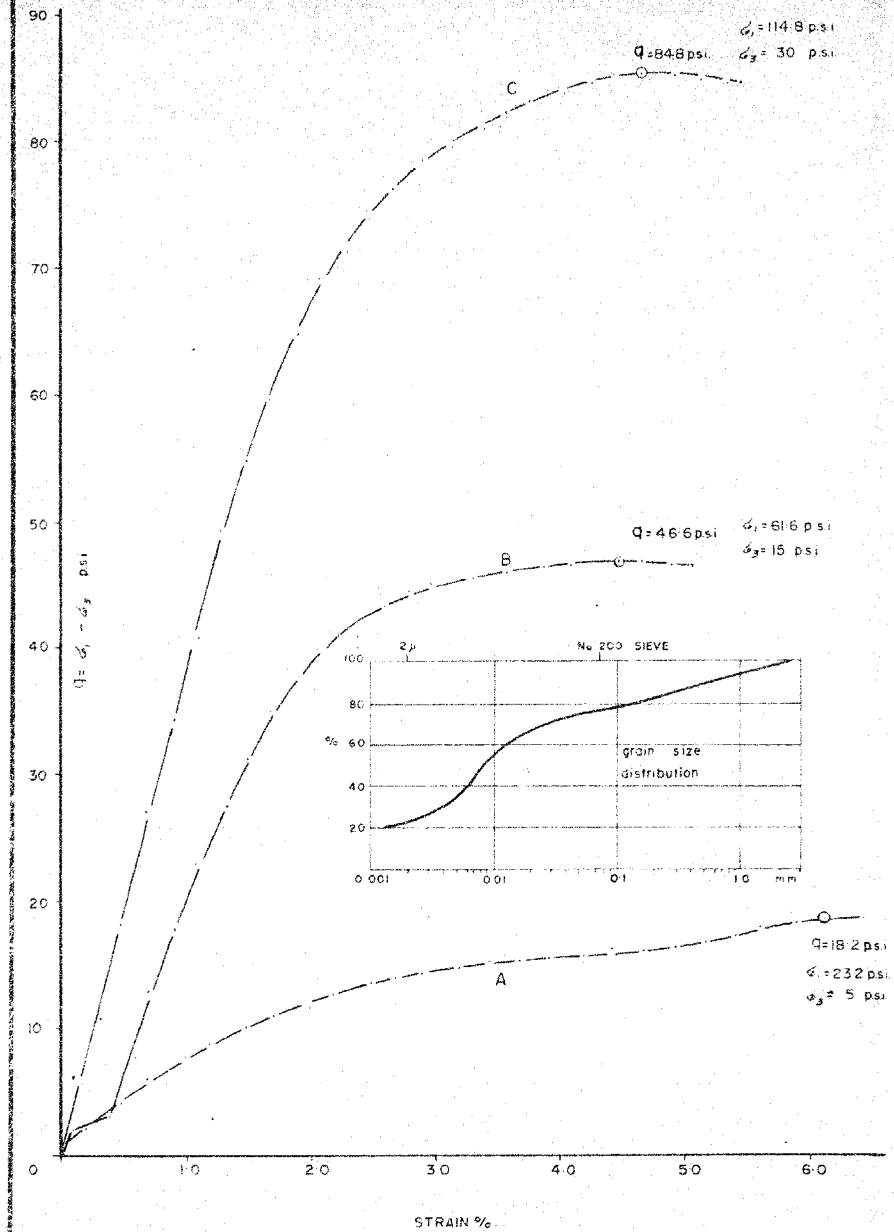
Classification of Sample and Group Symbol:

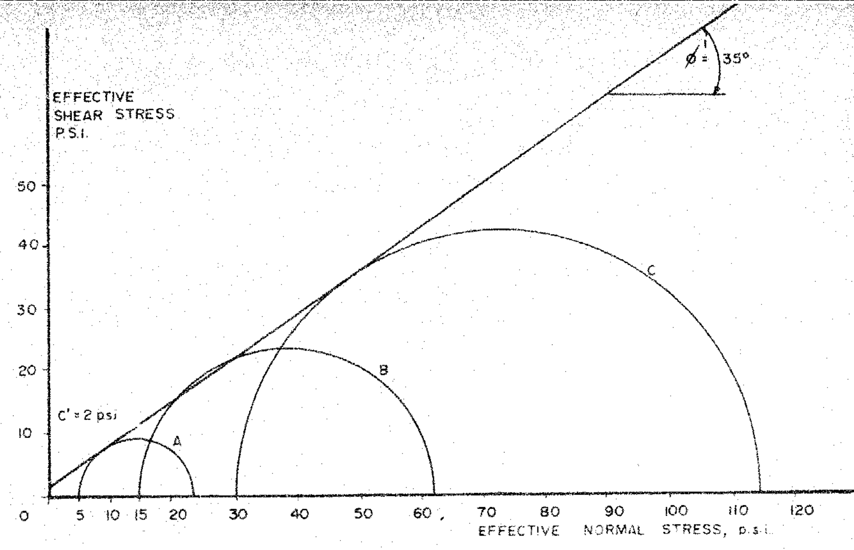
BH 3 So^s 4 – 7 CL	ALL OTHERS GS—SM
SANDY CLAY	ROCK FRAGMENTS & ROCK FLOUR

PLASTIC PROPERTIES: BH 3 So^s 4-7 comb

LIQUID LIMIT	%	= 26·6 — 30·3
PLASTIC LIMIT	%	= 15·1 — 15·0
PLASTICITY INDEX	%	= 11·5 — 15·3
MOISTURE CONTENT	%	= 16·8 — 18·6
ACTIVITY	= ("normal")	0·77 — 1·0

Enclosure No. 30





CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST RESULTS

MATERIAL REDDISH BROWN SILTY CLAY TILL WITH SOME SAND AND TRACES OF GRAVEL. THE STRUCTURE IS FISSURED AND CONTAINS MINOR ROOTS

L.L.: 30.8% P.L. = 18.1% P.I. = 6.6% γ_{ov} = 132.8 p.c.f. e = 0.43 Sr = 83%

W = 11.2 – 17.2% Wav = 13.5% SPECIFIC GRAVITY ~ 2.67 gr/cm^3

TEST DATA

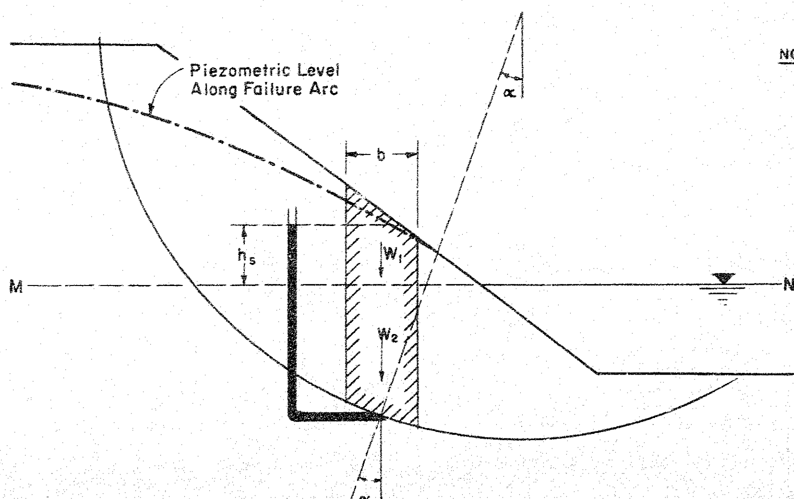
CIRCLE DESIGNATION		A	B	C
CONSOLIDATION PRESSURE	P.S.I.	5	15	30
SAMPLE HEIGHT x DIA.	IN.	4.00 x 2.00	4.00 x 2.00	4.08 x 2.00
UNIT WEIGHT	P.C.F.	133.2	131.0	136.1
WATER CONTENT				
before test (1)	%	11.7	11.2	17.2
after consolidation (2)	%	13.0	11.6	17.1
after test (2)	%	12.9	11.7	17.1
after test (1)	%	15.0	16.0	16.2
MAX. DEVIATOR STRESS	PSI.	18.2	46.6	84.8
STRAIN @ FAILURE	%	6.12	4.5	4.75
RATE OF STRAIN	in/min	0.00033	0.00036	0.00036

NOTE (1) DETERMINED BY WEIGHING THE SAMPLE
(2) DETERMINED FROM BURETTE READING

SLICE NO.	OVERTURNING ELEMENTS b	h	Area	γ or γ'	Weight $W = W_1 + W_2$	α	Sin α	① $\pm(W_1+W_2)\sin\alpha$	COHESIVE ELEMENTS c'	② bc'	FRICTIONAL ELEMENTS h_s	$u_s = \gamma_w h_s$	bu_s	$W_1+W_2-bu_s$	tan ϕ'	③ $(W_1+W_2-bu_s)\tan\phi'$	④ ② + ③	$K = \frac{\sec\alpha}{1+\frac{\tan\phi'\tan\alpha}{F}}$ 1st Trial F = 1.25	2nd Trial F = 1.31	K × ④ 1st Trial F = [illegible]	2nd Trial F = [illegible]
1	[illegible]	[illegible]	102	130	13250	38	.616	8170	—	[illegible]	—	—	—	—	[illegible]	7650		.938	[illegible]	7175	7200
2	10	[illegible]	[illegible]	[illegible]	[illegible]	31	.515	7300		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
3	7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
4	12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	15.5	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
6	10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10	[illegible]	[illegible]		[illegible]					[illegible]	5060		[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	3	[illegible]	[illegible]	[illegible]	3	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]					[illegible]	[illegible]		[illegible]	[illegible]	819	[illegible]
																42089				[illegible]	[illegible]
								[illegible]													

$$F = \frac{\Sigma[K \times ④]}{\Sigma①} = \text{[illegible]}$$

FINAL F = [illegible]



NOTES

1. $W = W_1 + W_2$
2. W_1 is the weight of that portion of a slice above line MN and is computed from bulk density.
3. W_2 is the weight of that portion of a slice below line MN and is computed from submerged density.
4. h_s refers only to the height of piezometric level above line MN. However in the case of slopes with no partial submergence h_s would be measured from the failure arc.

SOIL PROPERTIES

Type	γ	γ'	c'	ϕ'
	130		0	30
	[illegible]			[illegible]

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

CALCULATIONS BY: C. K	EFFECTIVE STRESS ANALYSIS			W.P.
CHECKED BY:	CIRCLE NO. 1	CIRCLE LOCATION	RADIUS 127	JOB
DATE Aug 27-'64	LOCATION			SHEET NO.

X-37
REV. NOV. 62

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT.

FRICTIONAL ELEMENTS						④	$K = \frac{\sec \alpha}{1 + \frac{\tan \phi' \tan \alpha}{F}}$		K x ④	
h_s	$u_s = \gamma_w h_s$	bu_s	$W_1 + W_2 - bu_s$	$\tan \phi'$	③ $(W_1+W_2-bu_s)\tan\phi'$	② + ③	1st Trial F = 1	2nd Trial F =	1st Trial F = 1.04	2nd Trial F =
				0.466	6170		.928		5730	
					6660		.916		6030	
					3840		.910		3430	
					5750		[illegible]		5240	
					3690		[illegible]		3400	
					2480		[illegible]		2310	
					4090		.940		3840	
					442		[illegible]		442	
					662		[illegible]		662	
					33784				31264	

$$F = \frac{\Sigma [K \times ④]}{\Sigma ①} =$$

FINAL F =

1. $W = W_1 + W_2$
2. W_1 is the weight of that portion of a slice above line MN and is computed from bulk density.
3. W_2 is the weight of that portion of a slice below line MN and is computed from submerged density.
4. h_s refers only to the height of piezometric level above line MN. However in the case of slopes with no partial submergence h_s would be measured from the failure arc.

SOIL PROPERTIES				
Type	γ	γ'	c'	ϕ'
	130			25°

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

CALCULATIONS BY:	EFFECTIVE STRESS ANALYSIS			W. P.
CHECKED BY:	CIRCLE NO.	CIRCLE LOCATION	RADIUS	JOB
DATE	LOCATION			SHEET NO.

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

GEORGE E. GOMME
MINISTER

cc: Foundations Files (Rm. 110)

A. T. C. McNAB
DEPUTY MINISTER

A48

Hwy. 401 & Keele St.,
Downsview 464, Ontario.
Tel. 248-3282
(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Office

November 3, 1970

Mr. R. C. Fraser,
Commission Engineer,
Hamilton-Wentworth Suburban Roads Commission,
Court House,
Hamil' n, Ontario.

Attention: Mr. H. G. Pattison,
Assistant Commission Engineer

Re: Sydenham Road Bridge

Dear Sirs:

Following is a summary of the main points of our discussions held on October 30, 1970, during our meeting at the above mentioned site.

(1) Remedial measures for the recent failure which took place during excavation for the North pier, should be designed so as to ensure future stability of the escarpment slope above the failed area, as well as the failed area itself. Unloading of the lower slope (which is the present failed area), might decrease the stability of the escarpment slope above this, since it acts as a toe berm.

(2) Excavations for foundations must be carried out in properly sheeted and braced excavations, otherwise, there is the danger of precipitating slope failures as has occurred already.

(3) It appears that some caissons may not have been socketed into the bedrock as recommended in the foundation report. If this is the case, it must be regarded as unacceptable from a foundation point of view.

Yours very truly,

K. G. Selby

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

KGS/MdeF

cc: Messrs. K. L. Kleinsteiber
B. H. Newington
Foundations Files
Gen. Files
C.C. Parker & Assoc. Ltd., Hamilton
Dominion Soil Investigation Ltd. - Toronto

[illegible]
[illegible]

Union Station
Toronto 116, Ontario
11 August 1970.

[illegible]

Mr. D. C. Cramm, P. Eng.
Bridge Department Manager
C.C. Parker and Associates Limited
Consulting Professional Engineers
688 Queensdale Avenue, East
Hamilton, Ontario.

Re: Sydenham Road Overhead Bridge, Mile 5.68,
Dundas Subdivision

Dear Sir:

With reference to your letter of the 14th ultimo concerning the construction of a future track south of the two existing mainline tracks in the vicinity of subject structure.

Kindly be advised that we have, as yet, not prepared plans for the profile and cross section of the embankment to support the future track.

When the time comes to construct the embankment for the third track, you may be ensured, that the experiences gained in the construction of subject structure with respect to slope stability will certainly be taken into consideration.

Yours truly

E. Jeronimus
Bridges and Structures Engineer

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: FILE.
(c. c. All in attendance)

FROM: Mr. H. Orlando,
Municipal Roads Office,
Admin. Bldg.

ATTENTION:

DATE: June 29th, 1970.

OUR FILE REF.

IN REPLY TO

SUBJECT: Hamilton-Wentworth Suburban Roads Commission
Road #5 (Sydenham Road) at C. N. R.

A meeting was held in Downsview June 26th, 1970, to review the status of the engineering on this project with particular emphasis on foundations for the new structure.

Those in attendance were:

R.C. Fraser, Commission Engineer.
D. Cramm, C.C. Parker & Associates.
Ken King, Dominion Soils.
A. G. Stermac, Foundations Section.
K. G. Selby, Foundations Section.
Doug Smith, Soils Section.
B. H. Newington, District Municipal Engineer (#4)
K. L. Kleinsteiber, Bridge Municipal Engineer.
B. R. Heavysege, Construction Methods Engineer.
H. Orlando, Plans Approval Engineer.

The following is a summary of the discussion at this meeting.

A short structure involving extensive approach fills was first considered the most economical and best suited if the stability of the high fills over the talus deposit could be fully assessed.

Roadwork was contracted in 1969, including the approaches over the talus slope, and indicators were placed in the approach area to measure movement. Considerable movement was measured and the fill operation at the approach was terminated. The rockfill was moved to form a berm which would add to the stability of the talus and the fill site was restored to original contour and stability. A slide that recently occurred on this contract was not at the bridge site but rather further upgrade. This was believed caused by blasting error which brought down too heavy a rock load causing failure to the ledge supporting the original road.

The design has been revised to a 6 span bridge with considerable reduction to the approach fills. A final contract proposed for this year will include bridge, granular backfill to structure, some granular base, paving etc.

There is no approach instability now and none is expected.

Blasting is completed and there should be none on this second stage contract.

No movement was observed at the bridge site (along the railway tracks) when the movement occurred with the placement of fill for the approach.

It was agreed plugs should be placed on the new bridge and tied down to permit future measurements, and also that slope indicators be placed in the talus adjacent to the approach.

The depth of talus at the bridge site is of the order of 50'.

Bore holes were taken for the foundation investigation at the end of each pier. The data from these borings provide the slope of the shale surface below the talus.

Caissons will be carried through the talus and 4' into the shale formation. There was some discussion on the advantage of 4 caissons with footing for each pier rather than 2 caissons, one for each individual column. There is little advantage having the 4 caissons per pier and there may be some disadvantage because of smaller size and added expense for grouting outside liners which may be left in place. This matter will be reviewed by the bridge consultant.

The railway have requested lateral clearance for one future track. The Commission Engineer and consultant will obtain more data from the railway, such as grade, to determine what effect this future track may have on the structure approaches.

No further investigation, except as noted above, has been recommended.

It is believed there will be no further problem with foundations; however, there is no surety.

H. Orlando

H. Orlando,
Plans Approval Engineer.

HO/amn.

A. G. Stermac

Mr. H. Greenland,
District Engineer.
Hamilton.

A. Rutka,
Materials and Testing Engineer

Mr. B. H. Newington

October 9, 1969

Your memo. Oct./6/69

Sydenham Road Bridge Project

The minutes of the meeting of the Hamilton-Wentworth Suburban Roads Commission dealing with the Sydenham Road Bridge Project are not too clear. The minutes indicate that the Department is requiring that the fill be removed, when in fact, we are only recommending that part of the fill be removed. This recommendation was contained in a letter written by Mr. K. Selby to Mr. R. C. Fraser, on September 8, 1969, and you have a copy of it. Attached to this letter was the recommended cross-sections at various locations, to achieve the kind of stability that existed prior to the placement of any of the fill. I am of the opinion that this letter of September 8, 1969 is adequate for the Commission and another letter should not be required.

In speaking with some of the Commission members recently, they were under the impression that all of the fill already placed had to be removed. This is by no means the case, and a quick review of the recommended cross-section will bear this out. It is quite possible that some of the excess fill can be bulldozed beyond the existing fill location and hence the amount of fill that will have to be loaded and hauled away might be kept to a minimum.

It has been reported that cracking has appeared on the top of the fill and if this is the case, then the fill should be corrected as soon as possible.

AR

A. Rutka,
Materials and Testing Engineer

AR/jm

cc: A. G. Stermac
T. J. Kovich

HAMILTON-WENTWORTH SUBURBAN ROADS COMMISSION

Committee Room #1, Court House
Hamilton, September 24th, 1969

The Commission met at 9:30 a.m. on the above date with Commissioner Alderson, Applegath, Freel, Green and Vernon; and R. C. Fraser, Commission Engineer, present.

Commissioner Green, Chairman, presiding.

Minutes of the meetings held on August 27th and September 16th were adopted as circulated to the members on motion of Commissioner Vernon and Commissioner Applegath.

Business arising from the minutes was dealt with as follows:

a) Reconstruction of Fruitland Road. Mr. Fraser reported on further problems encountered in acquiring property required for the project from T. E. Griffiths and W. Griffiths.

After discussion he was authorized to offer Mr. T. E. Griffiths the equivalent in cash of a fence if Mr. Griffiths requires one between his property and Fruitland Road.

b) Extension of Fruitland Road up the Escarpment. Tabled pending report of McCormick, Rankin & Associates on October 8th.

c) Sydenham Bridge project. Mr. Fraser advised that the Department of Highways was now requiring that the fill be removed.
The members of the Commission agreed that a letter was required from the Department to this effect and that the Engineer should prepare a tender for the work.

d) Widening of Barton Street, Saltfleet Township. Tabled pending Engineering Study to be submitted by W. Strok & Associates.

The Secretary read a letter from H. A. Woodgate, Engineer for the Township of Saltfleet, inquiring if sufficient land has been acquired on Barton Street to enable the Township to construct footpaths in certain areas.

It was moved by Commissioner Vernon, seconded by Commissioner Alderson, that this letter be tabled until the next meeting when Mr. Stroks' report will be presented.

Carried.

e) Realignment of the Brock Road, West Flamborough Township. Tabled.

Department of Highways Ontario
Copy for the information of
K. SELBY

~~Mr. F. R. Marshall~~
Subsidies Engineer
Municipal Roads
Downsview, Ontario

H. Greenland
District Engineer
Hamilton, Ontario

September 12, 1969

Additional By-Law Approval - County of Wentworth

Since discussing the matter of the Wentworth County Roads Program and the need for additional funds with you by telephone on September 5, 1969, I have met with the County Engineer and our discussions indicate that a total additional approval of $100,000.00 would be warranted. The items making up this expenditure are the Gray's Road storm sewer - $43,000.00, property acquisition - $10,000.00, and the modification of the Sydenham Road Bridge embankment - $47,000.00.

The last mentioned item was not discussed at the time the Suburban Commission in company with the Road Committee Chairman and Warden met with the Minister. However, since that time it has been recommended by Materials and Testing that the height of embankment should be reduced. I have not yet received the report on this subject which I understand is on the way to me at the moment but I have discussed it with Mr. Selby of the Foundations group and it is his feeling that it would be best to carry out this work prior to the onset of wet weather. While no definite statement can be made as to whether or not there will be some type of failure, certainly the likelihood of any failure will be reduced if the work is done before the embankment material becomes saturated.

As far as the County is concerned, the work on the Sydenham embankment ranks behind the Gray's Road sewer in priority so that if you were unable to find funds for the Sydenham work, the additional approval would be reduced to $53,000.00. I might say further that the $47,000.00 estimated for the Sydenham work is a fairly rough estimate and I would hope to give you a more precise figure in the near future. However, I believe it is sufficiently close that approval could be given on the basis. Subsidy on this amount should be based on 80%.

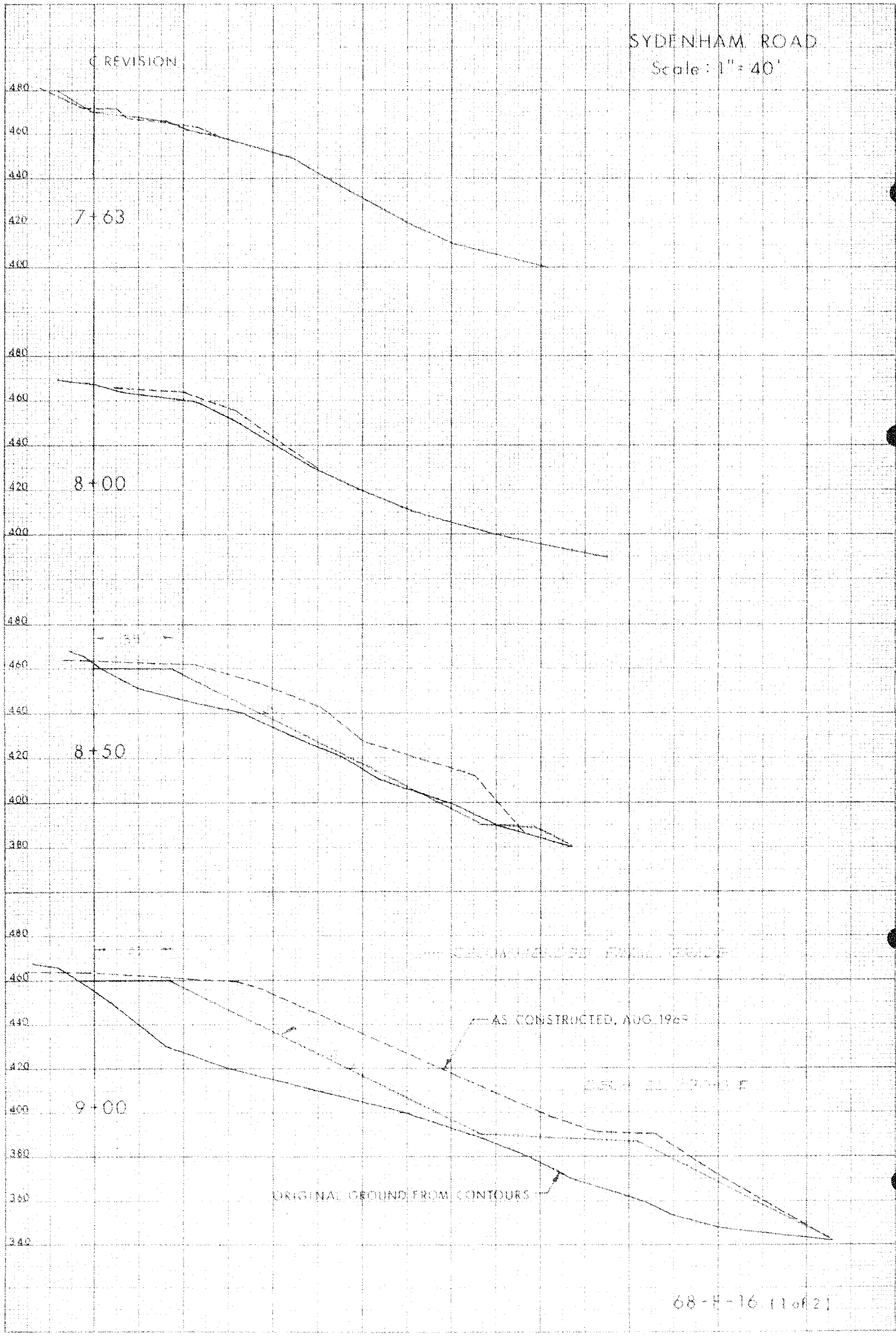
H. Greenland
District Engineer

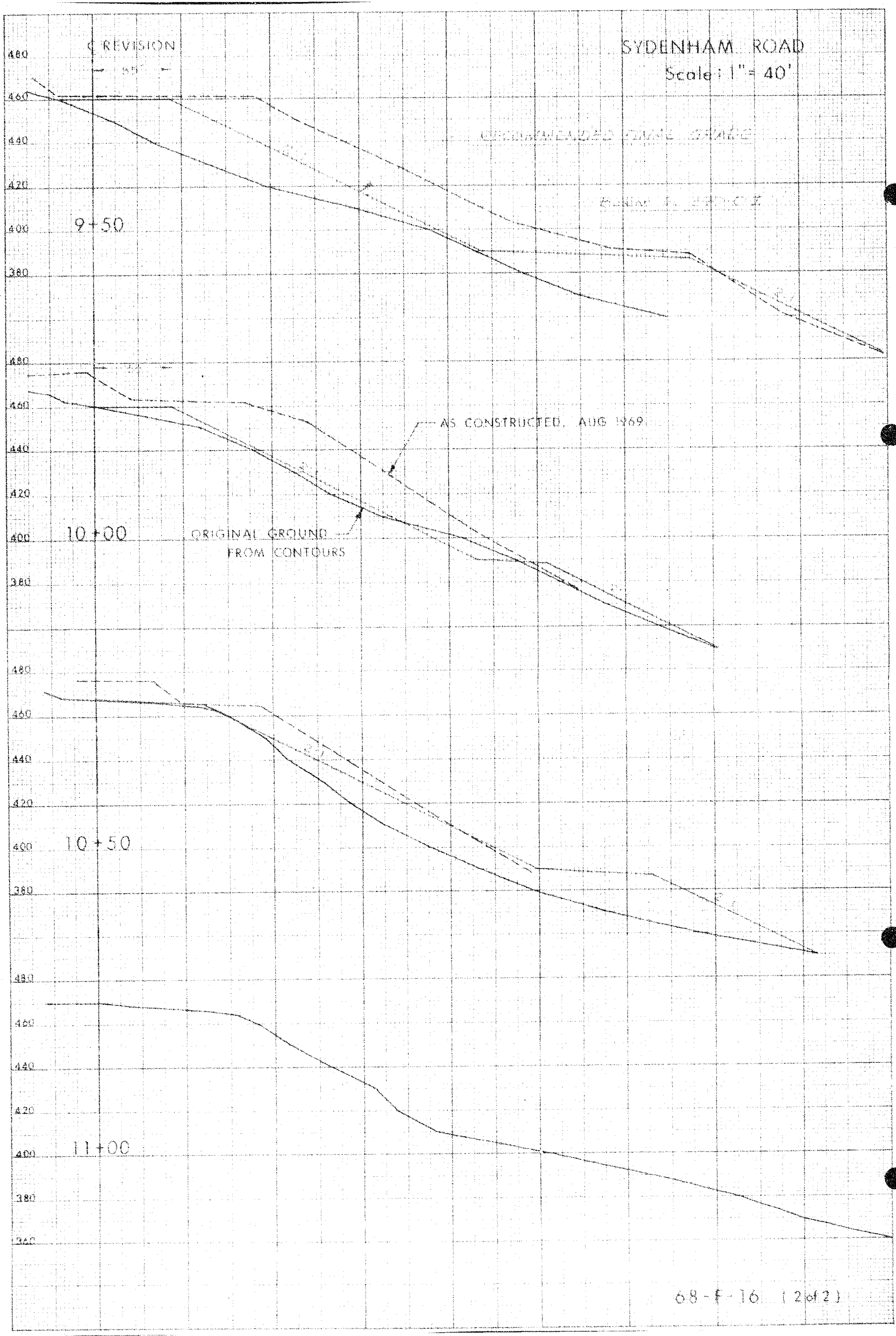
Per: B. H. Hewington

B. H. Hewington
District Municipal Engineer

BHH:lo

c.c. K. Selby


SYDENHAM ROAD
Scale : 1" = 40'
℄ REVISION
480
460
440
420
400
7+63
480
460
440
420
400
8+00
480
460
440
420
400
380
8+50
480
460
440
420
400
380
360
340
9+00
AS CONSTRUCTED, AUG. 1969
ORIGINAL GROUND FROM CONTOURS
68-H-16 (1 of 2)


SYDENHAM ROAD
Scale: 1" = 40'
C REVISION
'85
AS CONSTRUCTED, AUG 1969
ORIGINAL GROUND FROM CONTOURS
9+50
10+00
10+50
11+00
480
460
440
420
400
380
360
68-F-16 (2 of 2)

63-F-16

R. C. FRASER, B.A.Sc., P.ENG.
COMMISSION ENGINEER

HAMILTON-WENTWORTH SUBURBAN ROADS COMMISSION

COURT HOUSE HAMILTON, ONTARIO 525-9111

July 11, 1969

Mr. K. Selby
Ontario Department of Highways
Downsview, Ontario

<u>Re: Sydenham Road Bridge</u>

Dear Sir:

Further to our meeting on July 2nd at the site of the proposed Sydenham Road Bridge, I wish to advise that we plan to regrade the embankment approach to the bridge on a grade sloped at 2:1 from the toe of the berm. The grade formed will intersect the uppermost part of the berm and will require filling above the top of the berm immediately adjacent to the embankment.

I trust that this method will accomplish the intended purpose discussed at the meeting. If there is anything undesirable about this proposal will you please notify me immediately .

Yours very truly,

Robert C. Fraser
Commission Engineer

RCF:hb

SUPER IMPOSED DOCUMENT MAY APPEAR AS MULTI-FEED ON FILM.

Telephone:
248-3266

Downsview, Ontario

May 14, 1969

Mr. R. C. Fraser,
County Engineer,
County of Wentworth,
Court House,
HAMILTON, Ont.

Dear Mr. Fraser, **Re: Sydenham Road**

The above mentioned site was visited by myself, Mr. B. Newington and Mr. [illegible] Selby on May 7th, 1969. Discussions were held with Mr. H. Pattison of your office and Mr. K. King of Dominion Soil Investigation Ltd., regarding the construction of a berm at the toe of the south approach fill. The decision to build this berm was made since lateral movements had occurred and were observed in our slope indicators whilst the embankment was still only about half completed. These slope indicators are now inoperative since they have been sheared off some 12 ft. below the original ground surface. Further lateral movements have been monitored on a line of stakes placed at the toe of the fill.

At the present time it is not possible to predict with any degree of certainty whether or not the berm as designed, will have the desired result of stabilizing the approach fill when this is built to its full height. It will, therefore, be necessary to carry out careful observations during future construction and it may prove necessary to enlarge the size of the berm. This situation arises from the fact that the extreme variations of soil properties and parameters in the escarpment overburden make the application of conventional methods of stability analysis very difficult.

As mentioned in our letter of February [illegible], 1969, the type and length of the future bridge structure are entirely dependent on the overall stability of the approach embankment. In our view, the most critical situations insofar as stability is concerned, will occur during very wet periods in the fall and the spring. The entire drainage pattern of this area of the escarpment will have been affected by the grading operations which are at present being carried out and the effect of the new drainage patterns may not be immediately evident.

/cont'd....

In view of the foregoing, we would recommend that any decition regarding the type, length and construction of the bridge structure be postponed until a suitable period, after successful completion and subsequent performance of the south approach embankment, has elapsed. We believe that the performance of this embankment can be properly assessed after the 1970 spring rainy season.

Yours truly,

A. Rutka,
Materials and Testing Engineer

AR/jm

cc: H. Greenland
K. Kleinsteiber
A. G. Stermac
T. J. Kovich
File

A. G. Stermac

Telephone:
248-3266

Downsview, Ontario

February 18, 1969

Mr. R. C. Fraser,
County Engineer,
County of Wentworth,
Court House,
HAMILTON.

Dear Mr. Fraser,

Re: Sydenham Road

This will confirm the discussions held on the site with Mr. H. Pattison, Mr. K. Selby and myself on February 12, 1969, regarding the proposed bridge structure over the C.N.R. crossing. We understand that it is your present intention to start construction of the bridge in May of this year and that some design work has already been carried out.

As you know, there is a great deal of concern about the stability of the approach fills to this structure and for this reason the approach fills are being placed in advance of the structure as a means of determining if suitable stability can be achieved. Slope indicators have been installed to observe if intermittent earth movements take place during and after the construction of the embankment. The type and length of the bridge structure is, of course, entirely dependent on the overall stability of the embankment and on the nature of any earth movements.

In this connection, it would appear advisable to defer any decision as to the type and length of structure until the approach fills have been placed and the slope indicator readings analyzed. We would naturally expect some internal settlement and adjustment of the approach fill, due to winter construction, but this type of movement would not be of serious consequence as long as there is no movement of the subsoil beneath the embankment.

We believe that earth movements, if they are to occur, should develop before the completion of the approach fills; however, artificial bridging may occur because of frozen pockets and lenses

/cont'd......

in the soil and these probably will not be thawed until early summer. Naturally, the longer the embankment can remain before bridge construction takes place, the more assurance there will be of a stable embankment since the existing sub-surface drainage patterns will have been disturbed by the new construction and the effect of the new drainage patterns may not be immediately evident.

In summary, the length and type of the structure will be dependent upon the performance of the approach fill embankments. The earth approach fills are being constructed to determine whether the subsoil can sustain the additional loads imposed without any movement. Slope indicators have been installed to detect any possible movement. Therefore it would seem appropriate to withhold any decision on the type and length of the structure until the fill has been in place for a suitable period of time and the slope readings analyzed.

Yours truly,

A. Rutka,
Materials and Testing Engineer

AR/jm

cc: A. G. Stermac
K. L. Kleinsteiber
H. Greenland
T. J. Kovich
C. Wilmot

HON. GEORGE E. GOMME
MINISTER

A. T. C. McNAB
DEPUTY MINISTER

Hwy. 401 & Keele St.,
Downsview, Ontario.
Tel. 248-3282
(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Office

January 30, 1969

Mr. R. C. Fraser, B.A.Sc., P.Eng.,
Commission Engineer,
Hamilton - Wentworth Suburban Roads Commission,
Court House,
Hamilton, Ontario.

Dear Sir:

Re: Sydenham Road, Suburban Road #5,
C.N.R. Crossing.

The above mentioned site was visited by the writer and Mr. H. Pattison of your office on January 15, 1969. During our visit, certain observations were made and recommendations were given verbally to Mr. Pattison. This letter confirms recommendations made on site and summarizes, also, the main points of our discussions.

(1) Excavation of Keys on Escarpment Face

Excavation for the key at the toe of the fill revealed the presence of numerous layers and pockets of loose or soft material. At other locations on the face of the escarpment, grubbing operations also revealed the presence of similar poor material, some of it organic. It is recommended that all such material on the face of the escarpment be removed prior to placing fill material. To achieve this, it will be necessary to construct larger keys than formerly contemplated. It is further recommended that wherever excavation of loose soil is carried out on the escarpment face, that horizontal bases are formed to act as additional keys.

(2) Fill Material

It is understood that there will not be enough rock fill material available from the cuts to complete the embankments for the bridge approaches. It is therefore intended to construct the embankments in alternate 2-ft. thick layers of rock fill and earth

(2) Fill Material - (cont'd.) ...

fill. In our opinion, the entire embankment should be constructed of rock fill, but if this is not possible, we would recommend that the lower portion be constructed using all of the available rock fill, and that the upper portion remaining be constructed of earth fill. In addition, we recommend that a minimum 2-ft. thick layer of rock fill be placed on the face of the escarpment to act as a drainage medium under the earth fill.

(3) Rock Cuts

In the rock cut sections, north of the future bridge, excavations will be made in dolomite and shale bedrock. The shale underlies the dolomite. The design calls for an approximate vertical face in the dolomite and a 1-1/4:1 slope in the shale. It should be borne in mind that this type of shale weathers rapidly, and to a considerable depth, on exposure to the elements. In a relatively short time it is believed that, with the present design, the shale will weather and disintegrate under the dolomite to such an extent that the base of the latter will be exposed. This will, of course, increase the danger of falling rocks. This situation prevails now in the existing rock cuts. In our opinion, a bench about 10 ft. wide should be constructed on the shale upper contact, which would provide more protection against frost and other eroding agencies, for the shale layers under the dolomite. It would also provide against boulders falling on the road and reduce this danger considerably. In addition, it would also provide space for a drainage ditch, since the shale surface is a natural drainage path for water percolating through the permeable overlying dolomite.

(4) Slope Indicators

With regard to the reading of the slope indicators, which were installed for the purpose of checking the stability of the embankment, we suggest that you advise us at the stage when the embankment is approximately half completed. We will then make arrangements to carry out the necessary readings and observations. We would also suggest that some readings of the piezometers, which were installed by your soils consultant, be taken during the placing of fill material.

Yours very truly,

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

KGS/MdeF

cc: Messrs. K. L. Kleinsteiber
T. J. Kovich
B. H. Newington
Foundations Files
Gen. Files

DOMINION SOIL INVESTIGATION LIMITED

CONSULTING SOIL & FOUNDATION ENGINEERS

HEAD OFFICE
104 CROCKFORD BLVD
SCARBOROUGH, ONT.
CANADA
TELEPHONE 751-6565

BRANCH OFFICE
359 QUEENS AVE
LONDON, ONT
TELEPHONE 433-3551

ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD
38 BRENTFORD ROAD
KINGSTON 5, JAMAICA
WEST INDIES

Our Ref. No: 8-1-14

26th June 1968.

Hamilton-Wentworth Suburban Roads
Commission,
Court House,
Hamilton, Ontario.

Attention: Mr. H. Pattison, P.Eng.,
Assistant Commission Engineer.

Re: Sydenham Road Reconstruction

Dear Sirs,

Further to your letter of 25th June 1968 to Mr. A.G. Stermac of the Department of Highways, we would like to advise you that in our letter of 7th June 1968 a typographical error has been made and the proposed side slopes of the embankment were erroneously given as 2 horizontal in 1 vertical instead of the correct values of 1½ horizontal in 1 vertical.

Please accept our apologies for this error.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

I.P. Lieszkowszky, P.Eng.,
Chief Engineer.

IPL/me

cc: Mr. A.G. Stermac, P.Eng.,
Department of Highways, Ontario.

R. C. FRASER, B.A.Sc., P.ENG.
COMMISSION ENGINEER

HAMILTON - WENTWORTH SUBURBAN ROADS COMMISSION

COURT HOUSE HAMILTON ONTARIO 529-9111

June 25, 1968

Mr. A. Stermac,
Principal Foundations Engineer,
Department of Highways,
Downsview, Ontario.

Re: Proposed reconstruction of Sydenham Road

Dear Sir:

As discussed with Mr. Selby of your office by telephone on June 20th, we would be pleased to have your office perform the slope deflection readings and interpretations of same.

Enclosed is copy of letter and a sketch showing the area on which the embankment is to be built from Dominion Soil. I would like to draw your attention to paragraph 4, in which they mention that the side slopes of the proposed embankment should be 2 horizontal to 1 vertical. To date our contract drawings and quantity take off are based on 1½ horizontal to 1 vertical on the basis of stone fill. I would appreciate your comments on this at your earliest convenience, as it will be necessary for us to acquire more property for a 2:1 slope.

Please find enclosed preliminary plan No. 5-621, construction drawing 5-682 and sections 5-643, 4, 5 and 6.

Very truly yours,

Harold Pattison

Assistant Commission Engineer

HP/c

Enc.

cc: Mr. I. P. Lieszkowszky

DOMINION SOIL INVESTIGATION LIMITED
CONSULTING SOIL & FOUNDATION ENGINEERS

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ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
39 BRENTFORD ROAD
KINGSTON 5, JAMAICA
WEST INDIES

Our Ref. No: 8-1-14

7th June 1968.

Mr. R.C. Fraser, P.Eng.,
County Engineer,
County of Wentworth,
Court House,
Hamilton, Ontario.

Re: Sydenham Road Reconstruction
Proposed Test Embankment

Dear Mr. Fraser,

As requested by you over the telephone we are pleased to forward you a sketch plan showing the location and extent of the proposed test fill.

On the south side of the CNR tracks the test fill should be brought up to its finished grade between stations 8 + 25 and 10 + 25, that is along a 200 ft. long section.

On the north side the crest of the proposed test embankment should be 110 ft. long extending from Station 12 + 60 to 13 + 70.

In both cases the width of the test embankment should be equal to the final proposed embankment width and the side slopes should be 2 horizontal in 1 vertical.

The existing slope should be stripped and benched before placing the new fill and the entire construction of the embankment should be in accordance with the current Standards of the Department of Highways, Ontario (Standard No. DD-414).

During the entire duration of the construction and the placing of the test embankment it will be necessary to make daily readings on the slope indicators, the location of which is indicated on the sketch plans. May we suggest that you contact the Department of Highways and enquire about the availability of their equipment and personnel for carrying out these readings. Should they be unable to assist you we would be glad to carry out the instrumentation and the control of the test embankment ourselves.

We trust that the enclosed sketch plan and the above brief notes are sufficient for your present requirements. Should you require, however, further assistance and information please do not hesitate to call.

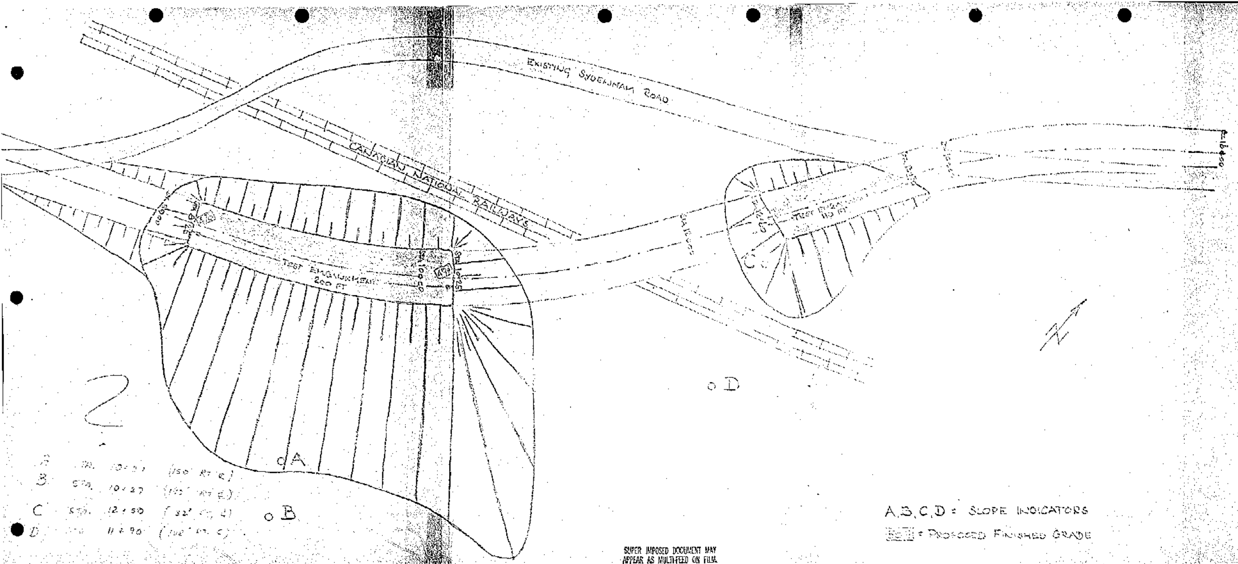
Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

I.P. Lieszkowszky, P.Eng.,
Chief Engineer.

IPL/me
Enclosure.

cc: Mr. K. Selby, P.Eng.,
Department of Highways, Ontario.



A, B, C, D = Slope Indicators

= Proposed Finished Grade

Hamilton Suburban Roads Commission

SYDENHAM ROAD RECONSTRUCTION

— PROPOSED TEST EMBANKMENTS —

Prepared by:
Dominion Soil Investigation Limited
164 Crockford Blvd., Scarborough

Job No. 8-1-14 June 1968

COUNTY OF WENTWORTH

SYDENHAM ROAD

Feb 9/68.

Suggestion for meeting

1) Line should be staked out

2) Toe of embankment slope (2:1 slopes) should be staked out to enable the placement of slope indicators in an area where they will not interfere excessively with construction

3) Discussion of building only part of fill

4) Discussion of measures to be undertaken if instability is becoming evident.

COUNTY OF WENTWORTH
SYDENHAM ROAD

NOTE

- COUNTY HAS DECIDED TO BUILD THE ROAD AND THE BRIDGE.
- THE SUGGESTIONS BY D.S.I. TO INSTALL SLOPE INDICATORS HAS BEEN AGREED UPON.
- D.H.O. PARTICIPAS 80% IN THE COST OF THE PROJECT.
- D.S.I. WILL DRILL THE HOLES, D.H.O. WILL SUPPLY & INSTALL THE SLOPE INDICATORS

- A MEETING SHOULD BE HELD TO DECIDE ON THE APPROPRIATE LOCATIONS FOR THE SLOPE INDICATORS.

- THE INSTRUMENTATION SHOULD PROVIDE INFORMATION ON:

  (a) THE PRESENT STATE OF STABILITY OF THE SLOPE AND

  (b) THE BEHAVIOR OF SLOPES WHEN APPROACH EMBANKMENTS WILL BE PLACED.

- TWO OR THREE SLOPE INDICATORS SHOULD BE INSTALLED.

- MEETING SHOULD BE HELD SOMETIME BETWEEN FEB 12 & 16/68

A.G. STERMAC

FEB 9. 1968.

COURT HOUSE
HAMILTON, ONTARIO

Feb. 2, 1968

Tom

Could you please review the Stanley report so that we can get together in the near future & see what possible solutions might be best employed to carry out our intended work on the escarpment.

Some of the questions I have to offer are

1. Can excavation of talus be a solution if the material is replaced over the clay layer with rock fill?
2. Would increasing the length of the structure help? Stanley thinks it can't be carried to rocky layers.
3. Would there be any merit in employing another consultant

[illegible] all of the [illegible]
[illegible] (G. E. M.)

Shall we consider a [illegible]
[illegible] ordinary [illegible]
[illegible] Bridge

The [illegible] problem will have to [illegible]
[illegible] opportunity to discuss the problem
again

Yours truly,
[illegible]

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

DOMINION SOIL INVESTIGATION LIMITED
CONSULTING SOIL & FOUNDATION ENGINEERS

HEAD OFFICE
504 CROCKFORD BLVD.
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ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
39 BRENTFORD ROAD
KINGSTON 5, JAMAICA
WEST INDIES

Our Ref. No: 8-1-14 29th January 1968.

Mr. R.C. Fraser, P.Eng.,
County Engineer,
County of Wentworth,
Court House,
Hamilton, Ontario.

Re: Road Embankments along the Niagara Escarpment

Dear Mr. Fraser,

At your request, we have studied the soil reports on the existing New Mountain Road in Stoney Creek and on the proposed Sydenham Road reconstruction near Dundas. The purpose of the review was to re-examine the conclusions on the stability of the talus slope in the light of the information furnished by the Department of Highways of Ontario at a meeting held in their offices on the 18th January 1968; namely, that the slope along the escarpment is unstable, even in an unloaded condition. In the following paragraphs we will outline our tentative conclusions and suggest possible future courses of action for your consideration.

1. Proposed Sydenham Road Reconstruction, Dundas

Slope indicators installed by DHO elsewhere in the talus slope have proven, that the talus slope is inherently unstable and is creeping downwards at a slow rate. Because of similarities of geology and topography, unstable conditions might prevail also at the Sydenham Road site.

Despite this analogy, there is as yet no conclusive evidence that the talus slope at the proposed Sydenham Road site is unstable. However, to achieve the certainty which is a prerequisite of sound engineering design, the following measures are recommended:

Two slope indicators should be installed in the vicinity of sections 2 and 3 (see our report Ref. No: 4-4-6, Enclosure No. 2) where the natural slope is the steepest and highest, and where the height of the proposed embankment would also be the maximum. These slope indicators, together with the previously installed piezometers, should be observed over a minimum period of one year whereafter the results would be evaluated. Depending on the results, several alternatives exist.

If the slope were inherently unstable, evidently no stru ure could be built thereon because the downward creeping talus would drag the piled or caisson foundations with itself. The proposed scheme, in this event, could be executed only after the slope has been stabilized, provided this were practicable at all. On the other hand, if the existing slope were stable, the road reconstruction scheme in its present form could be adopted after an eventual test section of the embankment has performed satisfactorily, or the scheme could be modified by replacing the fill with an elevated structure.

Simultaneously with the installation of slope indicators, one borehole should also be put down to test the suitability of the soil for eventual stabilization. (Further details on this will be set forth in the next section).

2. Existing New Mountain Road, Stoney Creek

We have studied the report prepared by Messrs. E.M. Peto Associates Limited, Reference No. 65188, dated 3rd February 1966, and the writer visited the site on 22nd and 26th January 1968. Furthermore, we are in the possession of DHO diagram ("Movement of slope indicator vs. time, Ref. 65-F-128") which illustrates the unstable conditions prevailing in the talus slope.

On 26th January 1968 the following observations were made: The railway tracks appeared to have been dislocated to the north. The dislocation began at around station 21 + 00 and reached its maximum value (2 to 3-inches) at around station 17 + 50. All stations are in reference to DHO drawing 65-F-128 A, dated 1st April 1966.

Our Ref. No: 8-1-14

Only scant data are at present in our possession about the movement of the railway tracks and its correlation with the road construction operations, therefore it is virtually impossible to go beyond the conclusions of Messrs. E.M. Peto Associates Ltd: "the placing of the fill on the existing slope has tended to decrease the factor of safety of the slope to some extent, but not critically" (see Report etc. Ref. 65188, Introductory letter by Mr. E.M. Peto, P.Eng., p. 2). Consequently, at this stage, we would like to confine our suggestions to those positive measures, which might be capable of permanently remedying the situation. In particular, consideration should be given to stabilization of the slope by (a) cement grouting and/or (b) lime.

Cement grouting could be successfully employed at those locations where mainly granular talus was encountered. In this method, using a predetermined pattern of grout holes, cement mortar would be injected into the voids of the rock fragments. On the other hand, stabilization by lime (or other chemicals) has been successfully applied for soils containing excess amounts of silt or clay, and in 1967 lime stabilization was employed with favourable results to stop an active landslide at a residential area in Iowa, U.S.A.

To examine the feasibility of these methods, we recommend that two borings should be made at the unstable road section. From these borings fresh samples could be taken which, apart from the usual soils engineering analyses, should also be analysed as to the chemical and mineralogical composition and reactivity with chemicals.

In the possession of the test data could the decision on the stabilization be made.

Should you be interested in the above suggestions, we would be glad to perform further research and a cost analysis for your consideration. Trusting that we were able to satisfy your present requirements,

We remain,

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

REGISTERED PROFESSIONAL ENGINEER
L.S. ROLKO
PROVINCE OF ONTARIO

L. S. Rolko

L.S. Rolko, P.Eng., A.M. ASCE.

LSR/me

DOMINION SOIL INVESTIGATION LIMITED

CONSULTING SOIL & FOUNDATION ENGINEERS

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LONDON, ONT.
TELEPHONE: [illegible]

ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
[illegible] BRENTFORD ROAD
KINGSTON [illegible], JAMAICA
WEST INDIES

Our Ref. No: 8-[illegible]

MINUTES OF THE MEETING
held at the office of the Department of Highways, Ontario,
Foundation Section, at 10.00 am, [illegible] January 1968.

Present: from the Department of Highways, Ontario,
Materials and Testing Division, Foundation Section:

Mr. A.G. Stermac, P.Eng., Principal Foundation Engineer
Mr. K. Selby, P.Eng., Senior Foundation Engineer

from the Hamilton Suburban Roads Commission:

Mr. R.C. Fraser, P.Eng., County Engineer
Mr. H. Pattison, P.Eng., Senior Design Engineer

from Dominion Soil Investigation Limited:

Mr. L.S. Rolko, P.Eng.

Re: Sydenham Road Reconstruction near Dundas, Ontario.
Soils Report by Dominion Soil Investigation Limited
Reference No: [illegible]

Mr. Stermac opened the meeting with a brief description of the events that took place on the New Mountain Road which scales the Niagara escarpment near the town of Stoney Creek. The highway fill failed between Stations 19 + 00 and 20 + 50 and considerable movements of the Toronto, Hamilton and Buffalo Railway line, which runs along the road at an about 100 ft. lower level, occurred. This may indicate a general instability of the talus material overlying the bedrock.

Therefore, the Department wishes to call the attention of the Hamilton Suburban Roads Commission and Dominion Soil Investigation Limited to the necessity of re-examining the conclusions for the proposed Sydenham Road reconstruction in the light of this new information.

Mr. Selby pointed out that near Stoney Creek the soil stratigraphy was similar, if not better, than the one revealed in the soils report (Ref. No: 4-4-6) prepared by Dominion Soil Investigation Limited. Furthermore, although the soils engineering analyses are correct for the New Mountain Road near Stoney Creek and indicate the escarpment to be stable, the experience with the railway line contradicts this conclusion. Therefore the DHO, on the basis of slope indicator measurements, surmises that there is a steady downward creep of the talus stratum which precludes the possibility of building any structure along the escarpment.

Mr. Stermac outlined the philosophy of the Department's analysis: a talus slope, consisting of broken rock fragments which accumulated at the toe of the escarpment, is at the state of limit equilibrium, i.e. the factor of safety against slip failure is close to unity. Any surcharge could easily cause failure on such a slope and even bridge piers could be dragged downwards by the creeping talus material.

The analogy between the Stoney Creek and Sydenham Road sites is very close. At the latter, a similar talus slope was encountered, consequently, the same conclusions are applicable.

Mr. Rolko — pointed out that several careful perambulations of the Sydenham project area were made in 1964 and with the exception of two areas, where poor drainage caused localized creep, there were no signs of slope movement. All trees were vertical and the heavily travelled railway and the road appeared to be stable.

Mr. Selby — countered this observation with the notion that in the event of parallel translation of the talus slope the trees would preserve their vertical position.

Mr. Stermac — emphasized that the safety of the proposed new and 30 ft. high embankment would be doubtful. He suggested that possibly a better site could be found for this road relocation. He also mentioned the construction of a test section whereby the stability analysis would be performed by a full scale model test.

Mr. Selby — indicated that the conclusions based on the results of the test section may still be negative. He also suggested that the embankment could probably be built at a flatter slope than now proposed.

Mr. Rolko — questioned if the stability of the talus mass could be preserved by grouting.

Mr. Stermac — expressed his reservation in connection with this method. He would not advise it.

Mr. Fraser — expressed his intention to decide on the matter in the light of the new information.

The meeting was adjourned at approximately 11.30 am.

DOMINION SOIL INVESTIGATION LIMITED

CONSULTING SOIL & FOUNDATION ENGINEERS

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TELEPHONE: 433-3851

ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
39 BRENTFORD ROAD
KINGSTON 5, JAMAICA
WEST INDIES

Our Ref. No: 8-1-14

25th January 1968.

Mr. A.G. Stermac, P.Eng.,
Materials and Testing Division,
Department of Highways, Ontario,
Downsview Avenue,
Downsview, Ontario.

Dear Mr. Stermac,

Enclosed please find your copy of the Minutes of the Meeting held on 18th January 1968.

Please let the undersigned know, at your convenience, if you are in disagreement with the Minutes or wish to add to them.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

L.S. [illegible], P.Eng., A.M. ASCE.

LSR/mc
Enclosure.

cc: Foundations Office (Rm. 110)

Mr. G. C. E. Burkhardt, P.Eng.,
Municipal Bridge Checking Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

September 24, 1965

Your Memo -- Sept. 22/65

County of Wentworth,
C.N.R. Overhead Bridge,
Twp. W. Flamborough,
Lot 16, Conc. I,
Structure Site No. 36-202,
Your File No. BA 2155.

The foundation report on the above site, submitted by the Soils Consultant, Dominion Soil Investigation Limited, has been reviewed. We agree in general, with the recommendations contained therein, but would like to draw your attention to the following points:

(1) It has been recommended on page 8 of the report that observations for any sign of incipient failure and of piezometers during construction, should be made. It should be ensured that either the Consultant or the Foundation Section will carry out this operation.

(2) The limit of retaining walls has not been specified on page 10, and the design for drainage system (page 11) not detailed. Since both items are of great importance to the stability of the slopes, care should be taken that details be incorporated in the final design.

We believe that the foregoing comments are adequate; however, should additional information be required, please feel free to contact our Office.

KYL/MdeF

K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

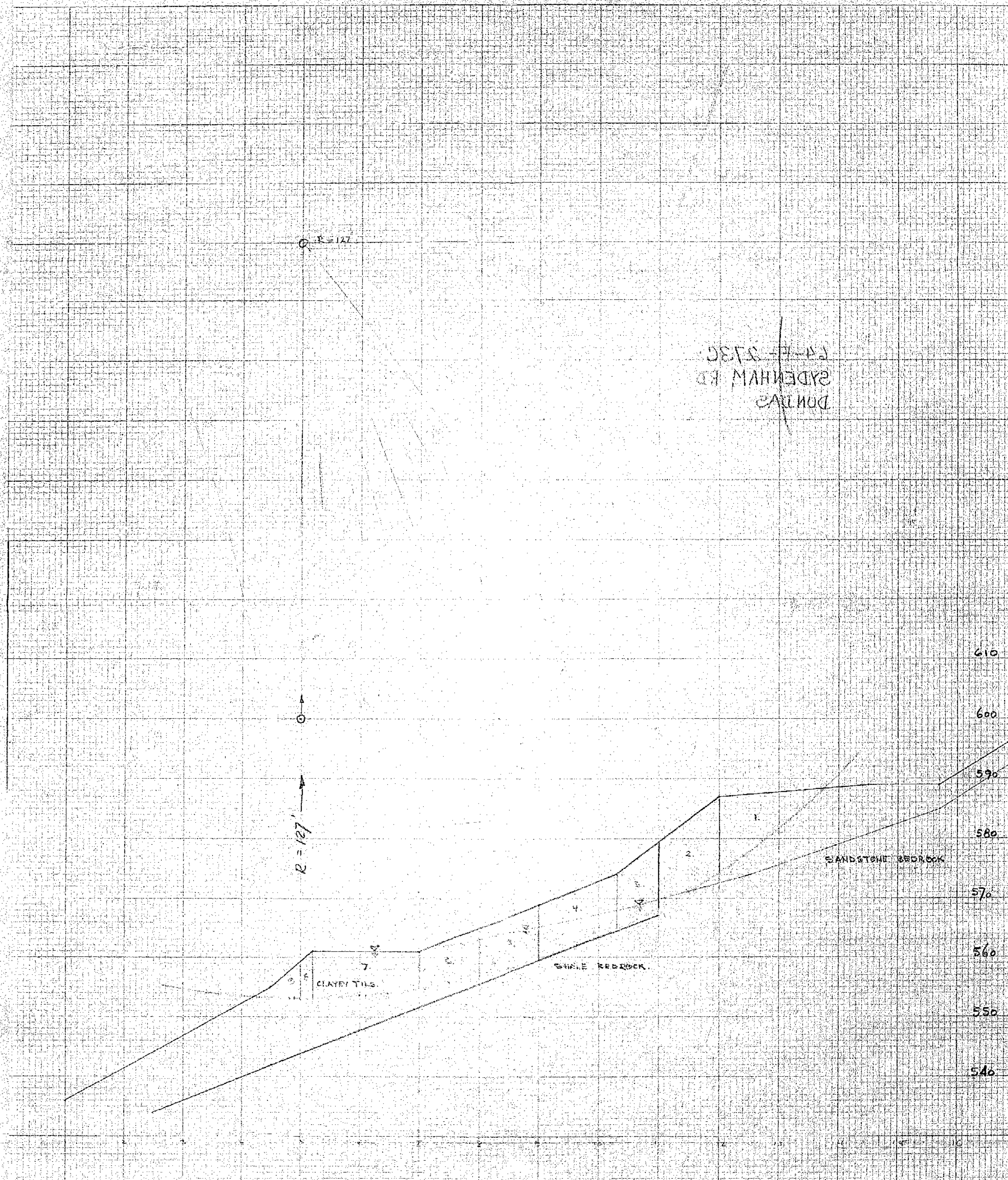
cc: Foundations Office
Gen. Files

P.S. -- Report and plan returned herewith, as requested.

64-F-273C

SYDENHAM RD.

DUNDAS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SANDSTONE BEDROCK
SHALE BEDROCK
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