66-F-263M

COUNTY RD. #31

HAMLET OF SALTFORD

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February 7, 1966.

Our Ref: 5-7-12

County of Huron,
Highways Department,
GODERICH, Ontario.

66 - F - 263 M

Attention: Mr. J. W. Britnell, P.Eng.
County Engineer.

Re: Soil Engineering Studies, County Road #31,
Hamlet of Saltford, Ontario.

Dear Sirs:

This letter accompanies our report on the soil engineering studies carried out at the above site.

We find that the site is underlain by a thin veneer of sand followed by a deep deposit of very stiff overconsolidated clay. It is the properties of this latter deposit which govern design and construction of any proposed works. The engineering properties for the clay are given in the body of the report.

Our study has shown that the existing slopes have an adequate factor of safety against deep-seated shear failure in the clay but that there is unstability against shallow sliding or sloughing when the ground water table is high. This accounts for the gradual "creep" type of movement which has occurred. Probably this is largely seasonal and would be aggravated during the spring thaw conditions. We believe that the present slope stability problems could be eliminated by the installation of drainage, in particular the construction of an interceptor drain at the top of the slope to collect the water which emanates from the uppermost sand deposit.

As a guide to the design of the proposed road improvements our report includes suggested schemes for widening on both the uphill and downhill sides. The choice between these tentative proposals is based on the desired geometry of the road and on economics and is, therefore, beyond the scope of our report.

We trust that this report contains the information you require. However, should you wish to discuss any of these aspects further, we would be pleased if you would call on us.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED,

K. H. King

K. H. King, P.Eng.,
Director.

REGISTERED PROFESSIONAL ENGINEER
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KHK/is

COUNTY OF HURON

HIGHWAYS DEPARTMENT

GODERICH, ONTARIO.

# REPORT

# ON

# SOIL ENGINEERING STUDIES

# FOR

# COUNTY ROAD #31

# HAMLET OF SALTFORD, ONTARIO.

SUBMITTED BY

DOMINION SOIL INVESTIGATION LIMITED
77 CROCKFORD BOULEVARD
SCARBOROUGH - ONTARIO

REFERENCE
5-7-12
JANUARY - 1966

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# APPENDICES

## INTRODUCTION

Dominion Soil Investigation Limited was retained by the County of Huron Highways Department to carry out soil engineering studies along a portion of County Road No. 31 near the intersection of King's Highway No. 21 in the Hamlet of Saltford.

The site in question is known as Saltford Hill where the County road ascends the north bank of Maitland River, rising about 100 feet in a distance of about 2,000 feet. Within this length, the existing road requires improvement of grade, alignment and, in particular, width. The question of improving the road is aggravated by the existing problem of the stability of the slope. This has manifested itself by gradual movements of the slope over a period of years, causing loss of support to the road on the downhill side and gradual encroachment on the uphill side. Over one short length, the uphill side of the road has been retained by a concrete gravity retaining wall which also shows some evidence of movement.

In general, the purpose of this investigation and study was to determine the subsurface conditions at the site and the geotechnical properties of the encountered soil strata as pertaining to the stability of the present and future slopes. Specifically the aim of the study was to give general recommendations for the widening of the road in order to maintain stable slopes on either side of the road and to alleviate the present unstable portions.

To explore the subsurface conditions, a total of nine boreholes was put down at the locations indicated on the attached site plan. The depths of the boreholes ranged between a few feet and approximately 100 feet. In view of the hard consistency of the subsoil only disturbed soil samples could be obtained, and these were recovered continuously or at close intervals of depth. The laboratory testing therefore was limited to the determination of the Index Properties of the different soil strata, although an attempt was made to determine the shear strength parameters of the main soil type using partially disturbed samples and triaxial testing techniques. The results of the borings and laboratory tests are described in the appropriate sections of this report.

A centerline profile and typical cross sections of the slope taken perpendicular to the centerline of the existing road were obtained and supplied by the County of Huron, who also established the ground surface elevations at the locations of the boreholes.

## SITE AND GEOLOGY

The site is located east of the town of Goderich on the north bank of Maitland River. The river at this point is deeply entrenched in a clay plain and flows in a wide and flat gravelly bed. The valley is about 130 feet deep with steep banks cut in unconsolidated material, except for the bottom six to ten feet where limestone is exposed.

The eastern shore of Lake Huron is bordered by a narrow, approximately 3 to 4 mile wide, strip along which the land rises from 650 feet to about 800 or 850 feet above sea level. This is a massive clay till plain modified by narrow strips of sand and shallow lacustrine clay deposits, deposited by the now extinct glacial Lake Warren. The waters of Lake Warren inundated this area towards the middle of the last glacial period and were confined between the Wyoming moraine, running parallel to and about 4 miles east of the present shore line, and the retreating glacier then occupying the Lake Huron basin. The twin shorelines of Lake Warren can be seen and traced north of Goderich on aerial photographs.

DESCRIPTION OF THE BORINGS

Between the 16th and 24th of August, 1965, nine boreholes were put down at the approximate locations indicated on the attached site plan (Drawing No. 1).

The borings were performed with a skid-mounted diamond drill machine adapted for soil sampling and testing, except boreholes No. 3, 4, 5 and 6, which, because of the difficulties involved in getting access to their locations, were advanced by a small, portable, power-auger machine. However, in view of the hard consistency of the subsoil these latter holes had to be abandoned at shallow depths. The remainder of the boreholes (Boreholes No. 2, 4A, 5A and 7) were carried to depths ranging between 14 and 21 feet and one (B.H. No. 1) was extended 95 feet below ground surface.

The boreholes were lined with 2-3/8 I.D. steel pipe casing which was driven to the depth of sampling and the soil inside the casing was removed by washing.

Disturbed soil samples were recovered by a 2-inch O.D. split-spoon sampler driven into undisturbed ground by a 140-lb. hammer falling freely 30 inches. The number of blows required to drive the sampler 12 inches into the ground, after an initial set of 6 inches, was recorded as the Standard Penetration Resistance or "N" value. There is an empirical but well-established relationship between the relative density of granular soils and the "N" values. It is also possible to estimate from the "N" values the consistency of cohesive strata. Several attempts were made to obtain undisturbed thin-walled samples. However, in view of the hard consistency of the subsoil, all attempts failed.

The recovered samples were classified visually in the field and with a minimum of additional disturbance put into airtight jars and shipped to the laboratory for further examination and testing. Upon completion of the test holes, and to facilitate prolonged water-level readings, piezometers were installed in the boreholes at various depths. The piezometers were surrounded and the boreholes backfilled with granular material. Depending on the soil stratigraphy bentonite clay plugs were provided around the plastic tubes in the boreholes near the ground surface and/or at greater depth in order to eliminate the infiltration of the water from the ground surface or pervious strata encountered at higher elevations.

## GENERALIZED SOIL CONDITIONS

On the evidence of the exploratory boreholes put down along the existing County road, it can be stated that the north bank of the Maitland River is built up of a massive clay till sheet covered only by a thin veneer of topsoil, and at some places by shallow layers of natural or artificial granular deposits.

Borehole No. 1, which penetrated the subsurface to a depth of 95 feet, indicates that this till sheet is relatively homogeneous and continuous and that it is interrupted only by a thin, approximately 4 foot thick coarse sand and gravel stratum encountered between elevations 685 and 681 feet. Underlying the clay till at elevation 628 feet, i.e. about 90 feet below the ground surface, another sand and gravel stratum was encountered. Since the borehole was terminated within this stratum its total thickness is not known, but it is believed that this stratum forms the river bed and that it extends to the bedrock

The clay till has a brown to a pale brownish-grey colour and is believed to be highly calcareous. As indicated by a few hydrometer tests (Enclosures No. 10, 11, 12 & 13) the till consists of about 15% sand and gravel; 65% silt and 20% clay.

The consistency limits of the clay till were determined by Atterberg tests and are as follows:

| | |
|---|---|
| Liquid Limit: | 16 to 34% |
| Plastic Limit: | 11 to 16% |
| Plasticity Index: | 5 to 18 |

On the basis of the average test results the till can be described as a clay of low plasticity. The activity of the clay, i.e. the ratio of the Plasticity Index to the clay fraction ( 2μ), is about 0.5.

The natural moisture content ranges between 11 and 21 percent. The liquidity index, which relates the moisture content to the liquid and plastic limits by the relationship $L.I. = \frac{M.C. - P.L.}{L.L. - P.L.}$ varies from less than 0 to 0.37, indicating a very stiff to very hard consistency.

Approximately the same range of consistency can be inferred from the Standard Penetration Tests which gave "N" values ranging from 19 to over 100 blows per foot.

The bulk density or natural unit weight of the soil ranges between 118 and 142 pounds per cubic foot, but an average value of 130 P.C.F. is more characteristic.

Because of the lack of undisturbed samples an attempt was made to determine the shear strength parameters of the clay till in terms of total and effective stresses using partially disturbed soil samples. However in view of the overconsolidated nature of the soil, its hard consistency and low sensitivity, the disturbance appeared to influence the results only to a small degree and it is believed that the results, especially those in terms of effective stresses, can be used with reliance. The shear strength parameters to be used in the design will be discussed later.

## GROUND WATER CONDITIONS

Because the position of the ground water table has an important bearing on the stability of the slopes, seven piezometers and water observation pipes were installed in five of the boreholes. The water levels in these installations were regularly observed during the field work and periodically since. The latest set of water level readings are plotted on the records of boreholes and the day-to-day observations are tabulated in Table 1 of Appendix I.

The observations indicate that generally the water table lies 10 to 15 feet below the ground surface. But since the borings were carried out during the dry season it is possible that these observations reflect a low, if not the lowest, position of the ground water table. Further observations seem therefore to be desirable.

## SHEAR STRENGTH PROPERTIES IN RELATION TO DESIGN

In many soil engineering problems, particularly those involving slope stability, it is necessary to consider the shear strength of the soil and the stability calculations in terms of both total and effective stresses. The total stresses refer to undrained conditions where no separate account is taken of pore-water pressures in the soil, it being assumed that the pore pressures in the laboratory test sample represent those occurring in the field. This is reasonably true under certain conditions, limited mainly to those prevailing at the end of construction. However, when considering the long-term stability it is necessary to take into account the effect of the pore-water pressures in the soil since these change with time, generally attaining equilibrium under the natural ground water conditions. The laboratory tests are performed either under undrained conditions with measurements of pore-water pressures, as in the present case, or under drained conditions when the pore-water pressure is equal to zero. Both types of test give essentially similar results in which the shear strength of the soil is expressed in terms of effective stresses.

### a.) Total Stresses

The analysis is carried out in terms of total stress using the value of $C_u$ obtained from undrained tests. The sample of soil is tested under undrained conditions and the shear strength at failure is taken as ½ of the unconfined compression strength or the maximum deviator stress $(\sigma_1 - \sigma_3)$ if the test is carried out in the triaxial apparatus.

The undrained shear strength of the clay till encountered at the site was measured on four partially disturbed split-spoon samples. The obtained $C_u$ values range between 4200 and 5800 pounds per square foot. Since the tested samples were disturbed the laboratory strength measured is therefore expected to be below that which may exist in the ground. It has been suggested that for many soils the strength of "normal" samples may only be 40% of that for "perfect" samples. [2]

There is an established relationship between the undrained shear strength ($C_u$) of cohesive soils and the Standard Penetration Resistance, "N" (Terzaghi and Peck 1948) which is shown on Enclosure No. 14. Since this relationship was published it has become apparent that in the majority of cases the shear strength of the soil is underestimated if based on the "N" values. Also shown on Enclosure No. 14 are the results of the present tests. The values plot generally near to the Terzaghi-Peck line indicating that in the absence of a sufficient number of tests on undisturbed samples, the undrained shear strength of the clay till may be approximated from the "N" values, bearing in mind that the results will be on the safe if not overconservative side.

b.) Effective Stresses

Consolidated-undrained tests were carried out on three samples with the pore pressures measured at the base of the sample. The samples were consolidated under an all-around stress greater than the existing overburden pressure. After the consolidation was completed the cell pressure ($\sigma_3$) was raised in order to avoid the development of large negative pore pressures when applying the deviator stress.

The pore pressures recorded when the all-around pressure is applied gives the value of the parameter B. ($B = \Delta u/\Delta\sigma_3$). The values of

B are given on Enclosure No. 18. It should be noted here that for a fully saturated soil B is usually equal to 1.0.

Then the sample was sheared by increasing the axial load - deviator stress ($\sigma_1 - \sigma_3$) - and keeping the cell pressure ($\sigma_3$) constant. The test was carried out at a slow rate of strain (0.0015 in./min.) until the maximum value of the deviator stress was reached. Simultaneously the magnitude and variation of pore pressure was also measured.

The stress-strain curves as well as the variation of the pore pressure 'u' with axial strain are plotted on Enclosures 15, 16 & 17.

In determining the values of c' and Ø' in any test in which shear occurs under undrained conditions, an ambiguity arises about the state of stress to be denoted by the term "failure". Values of c' and Ø' almost equal to their maximum values, are found to be mobilized at strains appreciably smaller than the strain required to produce the maximum deviator stress. The increase in deviator stress occurring after this point is almost entirely the consequence of the drop in pore pressure. A prolonged drop in pore pressure will result in values of c' and Ø' based on the maximum deviator stress ($\sigma_1' - \sigma_3'$) being slightly less than their peak values measured at the maximum principal stress ratio ($\sigma_1' / \sigma_3'$).

This is illustrated on Enclosures Nos. 18 and 19 where the triaxial test results are presented in the form of Mohr stress circles observed at both the maximum values of $\sigma_1' - \sigma_3'$ and $\sigma_1'/\sigma_3'$. The failure envelopes drawn to these stress circles indicate a Ø' value of 30° and 31° and c' values of 6 and 7 P.S.I. respectively.

In the analysis of the long-term stability of the slope and earth retaining structures Ø' was assumed to be 31°, but different values of c' were used, depending on the nature of the assumed conditions as discussed in the following paragraphs.

DISCUSSION

1. Analysis of the stability of the existing slopes.

A typical cross-section of the existing slopes was taken by the engineers of the County at station 3 + 00 and is reproduced on Figure 1 of Appendix IV. As shown here the slope of the bank on the uphill side is about 29.5$^{o}$ (1-3/4 in 1) and on the downhill side it is at an angle with the horizontal plane of about 21 degrees (2½ in 1).

At the present time on both sides of the road the slope is densely covered by 1 to 1½ foot diameter trees with a heavy undergrowth consisting of small brushes, young trees and other vegetation. The trees are generally vertical over the entire slope indicating no recent major earth movements. Signs of unstability however were noticed in the areas of the existing earth retaining structures which indicate some movement. The existing concrete retaining wall in the area of borehole No. 6 shows signs of forward tilting, the amount of which is estimated to be about 2 inches at the top. In other areas the uphill side of the slope is retained by wooden posts driven into the subsoil. In these areas also signs of bulging and movement can be noticed probably due to the deterioration of the posts and/or surface sloughing. The flat floor of the valley lies at about elevation 610 feet and there are no signs of heaving at the toe of the slope.

The stability of the existing slopes was analyzed in terms of both total and effective stresses. The total stress analysis, which was carried out with the assumption that the average undrained shear strength of the clay till is 3,000 lbs. per square foot, gave a safety factor greater than 3.

The results of the effective stress analysis are shown on Figures 1 and 2. In the analysis it was assumed that the unit weight of the soil is 130 lbs. per cubic foot. The angle of shearing resistance $\phi' = 31$ degrees and the apparent cohesion $c' = 7$ p.s.i., that is, approximately 1,000 lbs. per square foot. The ground water table was assumed to lie 15 feet below the ground surface - measured in the vertical direction - which correspond to the conditions found during the investigation. As shown on Figure 1 the lowest value of the Factor of Safety against failure occurring along a circular arc was calculated to be 1.5. The effect of the position of the ground water level on the stability is also demonstrated by plotting the Factors of Safety for the critical slip circle for different water levels. As illustrated here the Factor of Safety decreases to 1.1 when the water level rises to the ground surface. To secure a safety factor of 1.3, which is considered to be the minimum desirable value, the ground water table has to be kept 5 feet below the ground surface. Because of the flatter slopes on the downhill side the danger of failure occurring along deep-seated circular arcs is less. Here the minimum value of the safety factor was found to be 2 for the condition when the water level is 12 feet below the ground surface, and 1.6 when the water level is assumed to be at the ground surface. This is shown on Figure 2.

In the above analysis the laboratory value of $c' = 1{,}000$ lbs. per square foot was used. In overconsolidated stiff, fissured, and weathered clays, the values of c' which correspond to equilibrium in the field, as determined from the analysis of actual slip failures, is usually less than that obtained in laboratory tests on samples from the same soil. (References 3, 6 and 8). These observations indicate that the value of c' changes and decreases with time. This change takes place over a great number of years and can be expressed possibly only on a geological time scale. Furthermore the cohesion is subjected to changes with the variation of the moisture content as a result of the fluctuation of the ground water level. With an increase in the moisture content the value of c' decreases. For the above reasons most authorities suggest that in design the value of the cohesion intercept be neglected completely usually accepting a low Safety Factor which is only slightly greater than unity. It is interesting to note that if in the present case c' is taken equal to zero the factor of safety decreases from 1.5 to 0.9 and for the worst piezometric condition from 1.1 to 0.3. In other words failure of the slope should have occurred in the past. The fact that there are no signs of deep-seated shear failures occurring in the past indicates that some value of c' must be still active. The actual value of the effective cohesion cannot be estimated, but from calculations for the limiting case of equilibrium (safety factor of 1) it is inferred that c' is at least equal to or greater than 250 lbs. per square foot.

It is reasonable to assume that in the top 5 or 6 feet, to which depth the clay is greatly weathered and fissured, the cohesion is negligible. The <u>failure</u> surface under these circumstances would be generally <u>parallel to the ground surface</u>. Assuming that the critical slip surface is 6 feet below the ground surface and that the water table rises to the ground surface, the factor of safety against sliding is only 0.55. By lowering the water table the safety factor increases. A factor of safety of 1 is obtained when the water table is 5 feet below the ground surface. Since it is likely that during a wet season the water table could rise above this level, failure in forms of surface sloughing is indicated. It is believed that failure of the existing slopes is greatly retarded and prevented by the heavy vegetation cover which protects the surface of the slope. Again, because of the flatter slopes on the downhill side the conditions here are less critical and the ground water table could rise as high as 18 inches below the ground surface before reaching the critical failure conditions.

To increase the stability of natural slopes often imposes great difficulties and generally leads to costly measures. The remedial measures which can be undertaken to improve slope stability are to flatten the slopes and drainage, especially if it is demonstrated that the lack of stability is due to pore-water or its effects.

Flattening the slopes is only practical and economical when the height of the slope is not excessive. In the present case the slopes on the uphill side would have to be flattened at least to 2½ in 1. This, however, would mean not only a large amount of excavation but also that the top of the slope would be moved back by about 50 feet for which we believe there is insufficient space. Therefore to increase the stability of the slopes by flattening them is not considered to be practical.

As discussed above, there is sufficient safety factor against both deep-seated slip failures and surface sloughing if the position of the ground water is controlled and kept below a critical level. This critical level, on the basis of the analyses, appears to be 5 feet below the ground surface. Therefore, if a drainage system can be devised and constructed which will secure that the water table will not rise above the critical level, the stability of the slopes can be secured. These drains could be a series of counter-fort drains running perpendicular to the centreline of the road, or alternatively interceptor drains laid parallel to the road. In the present case the construction of counter-fort drains may not only be more expensive but also they could create additional instability because their construction would necessitate the removal of the protecting vegetation from the face of the slopes. Since much of the ground water is believed to be fed by the sand and gravel stratum encountered in borehole No. 1 at the top of the slope, it is believed that the ground water table can be effectively controlled by the construction of an interceptor drain laid parallel to the top of the slope. The invert of this interceptor drain should be at least 1 foot below the bottom of the sand and gravel stratum, that is, approximately at a depth of 10 feet. In addition to this, drains should be installed parallel to the road behind the earth retaining structures as discussed later.

2. Widening of the Road

The average width of the existing road is about 26 feet and it is believed that after construction it will be widened to about 40 feet. The necessary road width could be obtained by either cutting on the uphill side or by placing fill on the downhill side. These alternative methods are shown as Scheme I and Scheme II on Figure 3 of Appendix IV, and are discussed in more detail below.

a.) Widening on the Uphill Side

The possibility of widening the road on the uphill side was investigated and it was found to be feasible. By cutting into the slope the factor of safety against rotational slip failure will not be decreased, in fact, because of the removed weight of the excavated earth it will be slightly increased. To secure the long term stability of the excavation face, a retaining wall will have to be constructed. The new height of the retaining wall would be about 17 feet above the road level. For the design of the wall the following design values are recommended.

| | |
|---|---|
| Allowable bearing pressure: | 3 tons per square foot |
| Maximum edge pressure: | 3½ tons per square foot |
| Unit Weight of Natural Soil: | 130 lbs. per cubic foot |
| Angle of Shearing Resistance: | $\phi$ = 30 degrees |
| Cohesion: | C = 250 lbs. per square foot |
| Coefficient of friction between the base of the foundation and the subgrade | = 0.3 |
| Recommended safety factor against horizontal sliding: | S.F. = 2.0 |

Preliminary calculations indicate that in order to secure a Factor of Safety of 2 against horizontal sliding it will be necessary to construct a shear key below the general foundation level, and also the water level behind the wall will have to be kept at least five feet below the road level. A sketch showing the general layout of the retaining structure and the proposed drain is shown on Figure 4. The drain should be surrounded, and the space between the wall backfilled, with a select granular filter material for which the grading requirements are shown on Enclosure No. 10. The grading of the recommended filter material is such that it will prevent the silt or fine particles from the subgrade soil from being washed into the drain and at the same time it will be pervious enough to allow the free movement of the ground water.

b.) Widening on the Downhill Side

The alternative methods of widening the road on the downhill side are shown on Figures 3a and 3b. In both cases part of the existing road on the uphill side is filled-in to eliminate the reconstruction of the existing retaining walls and to improve the conditions on this side. Thus the greatest portion of the new road width is gained on the downhill side by filling. The filling could be achieved by free dumping on top of the benched natural slopes and allowing the fill material to assume its angle of repose. This, in the case of predominantly granular materials, would probably be about 30 degrees, that is a slope of about 1 vertical in 1 3/4 horizontal. The success of such operation will largely depend on the actual degree of incline of the natural slope but in any event it will require large quantities of fill. The layout of the subdrains required for stability purposes are shown on Figure 3b.

The alternative method to the above scheme is to retain the fill by some kind of retaining structure. It is believed that probably the most economical type of retaining structure would be a cantilevered steel sheet-pile wall driven into the natural subgrade to a depth necessary to secure the stability of the wall. The tentative layout of such construction is shown on Figure 5. The cantilevered height of this sheet pile wall will be about 12 feet above natural grade and will probably have a total length of 25 to 30 feet. The design of the sheeting could be based on the following properties.

| | |
|---|---|
| Unit Weight of the compacted fill and natural subgrade: | 130 lbs. per cubic foot |
| Submerged unit weight of the above materials: | 68 lbs. per cubic foot |
| Angle of Shearing Resistance: | $\phi$ = 31 degrees |
| Cohesion: | C = 250 lbs. per square foot |
| Wall friction between sheeting and the soil: | $\delta$ = 15 degrees |

In the design allowance should be made for concentrated wheel loads or an equivalent line load positioned near to the top of the sheet-pile wall. To prevent the buildup of water pressure behind the sheeting a drain pipe should be installed at the position indicated on the sketch and it should be surrounded by a granular filter material. The limits of this filter media should be based on the piping and permeability ratio of the fill material. The requirement of the piping ratio is to prevent silt or fine particles from the subgrade soil being washed into the filter material. The permeability ratio is to ensure that the filter material will be sufficiently permeable to permit the movement of water. It is customary to determine the suitability of the filter material on the basis of its 15% size (the particle size in millimeters of which only 15% of the material is finer). The U.S. Corps of Engineers recommends that the 15% size of the filter material should lie between 5 times the 15% size of the subgrade material and 5 times the 85% size of the subgrade material. The limit for the coarse particles of the filter material is based on the size of the holes in the pipes in case of perforated pipes, or the gaps in case of open-jointed pipes. The 85% size of the filter material must be greater than twice the size of this gap. In the case of porous concrete pipes this requirement is unnecessary. If the particle-size distribution of the subgrade soil and the gap between the drain pipes are such that it is not practical to find one filter material which will meet all of the requirements, then it may be necessary to employ two filter materials, a coarse material placed around the pipe and a finer one between the coarse filter material and the subgrade soil.

## CONCLUSIONS

The investigation indicates that in the studied area the north bank of the Maitland River is built up of massive very stiff to hard clay deposits.

The shear strength parameters of the clay were established in the laboratory by consolidated, undrained triaxial compression tests with pore pressure measurements, and the effective angle of shearing resistance $\phi'$ was found to be equal to $31^{\circ}$ and the effective cohesion $c'$ to be 1000 P.S.F. Using these parameters and the pore water conditions established by field observations, the stability of the existing slopes was analyzed and it was found that the Factor of Safety against rotational slip failure is 1.5. The Factor of Safety against surface sloughing or shallow shear failures occurring along slip planes parallel to the face of the slope is 1.06. The analysis has also indicated that both safety factors decrease if the water level rises and for the limiting case when the water level reaches the ground surface the safety factor against deep-seated failures is reduced to 1.1, and to 0.55 for the case of surface slides. This accounts for the present "creep" movements of the slope which occur under adverse ground water conditions. To secure an adequate safety factor for both cases it was found that measures will have to be adopted to keep the water level at least 5 feet below the ground surface. To achieve this it is recommended that interceptor drains be installed at the top of the slope and behind the new earth retaining structures, parallel to the centreline of the road.

The possibility of widening the road on the uphill or the downhill side was investigated and it was found that both schemes are feasible. Both schemes will likely involve the construction of earth retaining structures for which design values are given in the text. In the stability of these retaining structures the position of the ground water table again appears to play an important role emphasizing the importance of subdrains.

Since from the soil mechanics point of view, both schemes appear to be feasible, the selection between the alternatives should be based on economical studies which are beyond the scope of this report.

DOMINION SOIL INVESTIGATION LIMITED,

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I. P. Lieszkowszky, P.Eng.,
Project Engineer.

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# A P P E N D I X  I

## GROUND WATER LEVEL OBSERVATIONS

### TABLE 1.

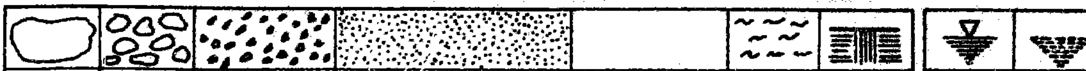
| Borehole No. | Ground Surface Elevation | Piezometer Tip at Depth Ft. | Water Level at Depth | Date | Remarks |
|---|---|---|---|---|---|
| 1 | 718.6 | No. 1: 37' | 10.75 | Aug. 19/65 | |
| | | | 11.82 | " 20/65 | |
| | | | 12.92 | " 23/65 | |
| | | | 12.82 | " 24/65 | |
| | | | 15.0 | Sept. 2/65 | |
| 1 | | No. 2: 29' | 10.2 | Aug. 19/65 | |
| | | | 8.5 | " 20/65 | |
| | | | 11.58 | " 23/65 | |
| | | | 11.58 | " 24/65 | |
| | | | 13.0 | Sept. 2/65 | |
| 1 | | Standpipe: 10' | 7.5 | Aug. 19/65 | |
| | | | 7.67 | " 20/65 | |
| | | | 7.82 | " 23/65 | |
| | | | 7.82 | " 24/65 | |
| | | | 8.85 | Sept. 2/65 | |
| 2 | 697.6 | 12' | Dry | Aug. 19-Sept. 2/65 | |
| 4A | 687.8 | 18' | 10.2 | Aug. 24/65 | |
| | | | 15.1 | Sept. 2/65 | |
| 5A | 666.5 | 18' | 5.2 | Aug. 20/65 | |
| | | | 11.75 | " 23/65 | |
| | | | 12.33 | " 24/65 | |
| 7 | 645 | 14.5' | 11.25 | Aug. 25/65 | |
| | | | Dry | Sept. 2/65 | Blocked at 12.5' |

# A P P E N D I X   II

E n c l o s u r e s

# LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

## SOIL COMPONENTS AND GROUND WATER CONDITIONS.



| BOULDER | COBBLE | GRAVEL COARSE | GRAVEL FINE | SAND COARSE | SAND MEDIUM | SAND FINE | SILT | CLAY | ORGANICS | BEDROCK | GROUND WATER LEVEL | DEPTH OF CAVE-IN |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 | RC | Rock core | TP | Piston, thin walled tube sample |
| CS | Sample from casing | % | Recovery | TW | Open, thin walled tube sample |
| ChS | Chunk sample | SS | Split spoon 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"ϕ, 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 | $\gamma$ | 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 | $\phi$ | Angle of int. friction | |
| PI % | Plasticity index | $C_v$ | Coeff. of consolidation | C' | Cohesion | in terms of effective stress |
| LI | Liquidity index | $m_v$ | Coeff. of volume compressibility | $\phi'$ | Angle of int. friction | |

## 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: |
|---|---|
| Very loose | 0 – 15 % |
| Loose | 15 – 35 % |
| Compact | 35 – 65 % |
| Dense | 65 – 85 % |
| Very dense | 85 – 100 % |

| COHESIVE SOILS: | C lbs/sq.ft. |
|---|---|
| Very soft | less than 250 |
| Soft | 250 – 500 |
| Firm | 500 – 1000 |
| Stiff | 1000 – 2000 |
| Very stiff | 2000 – 4000 |
| Hard | over 4000 |

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1 (0'-50')

OUR REFERENCE No. 5 - 7 - 12

CLIENT COUNTY OF HURON
PROJECT COUNTY ROAD Nº 31
LOCATION GODERICH, ONT.
DATUM ELEVATION GEODETIC

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE $2\,^{3}/_{8}$
DATE AUG. 16, 1965

2

| ELEVATION ft | DEPTH ft | STRATIFICATION DESCRIPTION | SAMPLE NUMBER | TYPE | N or Advancement of Sampler | REMARKS |
|---|---|---|---|---|---|---|
| 718·6 | 0 | GROUND SURFACE | | | | |
| | | 6" TOPSOIL | | | | Bentonite Plug |
| 715·0 | | Dense Pale Brown SANDY GRAVEL with some silt | | | | Standpipe |
| | 5 | | 1 | S.S. | 77 | |
| 710·0 | 9·0 | | | | | |
| | 10 | Hard CLAY TILL | 2 | S.S. | 71 | |
| 705·0 | | | | | | |
| | 15 | Brown / Brown-Grey | 3 | S.S. | 21 | Piezometer Nº 2 |
| 700·0 | | | | | | |
| | 20 | Grey | 4 | S.S. | 33 | |
| 695·0 | | | | | | Bentonite Plug |
| | 25 | (clay and silt with some sand and fine gravel) | 5 | S.S. | 35 | |
| 690·0 | | | | | | |
| | 30 | | 6 | S.S. | 34 | Bentonite Plug |
| 685·0 | 33·7 | | | | | |
| | 35 | Dense, Grey COARSE SAND & FINE GRAVEL | 7 | S.S. | 59 | Piezometer Nº 1 |
| 680·0 | 38·0 | | | | | |
| | 40 | Very Hard Grey CLAY TILL | 8 | S.S. | 132 | EXTRAPOLATED 'N' VALUE 66/6" |
| 675·0 | | | | | | |
| | 45 | | 9 | S.S. | 100 | Sand Backfill (to bottom of hole) |
| 670·0 | | | | | | |
| | 50 | CONT. ON NEXT ENCLOSURE | | | | Note: W.L. as on Sept. 2, 1965 |

PENETRATION RESISTANCE blows per foot: 20 40 60 80 100
SHEAR STRENGTH lbs sq ft
CONSISTENCY: 10 20 30 40

VERTICAL SCALE: 1 IN. TO 5 FT

**DOMINION SOIL INVESTIGATION LIMITED**

MADE D.A.M. CH'D.

OUR REFERENCE NO. 5 - 7 - 12

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1 (cont.) (50' - 95·4')

CLIENT: COUNTY OF HURON
PROJECT: COUNTY ROAD Nº 31
LOCATION: GODERICH, ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 3/8"
DATE: AUG. 16, 1965

ENCLOSURE NO. 3

| ELEVATION ft | DEPTH ft | STRATIFICATION DESCRIPTION | SAMPLE NUMBER | TYPE | N or Advancement of Sampler | REMARKS |
|---|---|---|---|---|---|---|
| | 50 | Hard to Very Hard Grey CLAY TILL (clay, silt some sand and fine gravel) | 10 | S.S. | 47 | |
| 665·0 | 55 | | 11 | S.S. | 41 | |
| 660·0 | 60 | | 12 | S.S | 36 | |
| 655·0 | 65 | | 13 | S.S. | 38 | |
| 650·0 | 70 | | 14 | S.S. | 77 | GRAIN SIZE DISTR. (ENCL. 10) |
| 645·0 | 75 | | 15 | S.S. | 67 | |
| 640·0 | 80 | | 16 | S.S. | 250 | GRAIN SIZE DISTR (ENCL. 11) EXTRAPOLATED 'N' VALUE 107/5·5" |
| 635·0 | 85 | | 17 | S.S. | 300 | EXTRAPOLATED 'N' VALUE 100/4" |
| 630·0 | 90 | Very Dense Grey MEDIUM to COARSE SAND | 18 | S.S. | 560 | EXTRAPOLATED 'N' VALUE 140/3" |
| 625·0 | 95 | | 19 | S.S. | 235 | EXTRAPOLATED 'N' VALUE 98/5" |
| | 95·4 | END OF BOREHOLE | | | | |
| 620·0 | 100 | | | | | |

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

CONSISTENCY water content %: PL W LL
10 20 30 40

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 2, 3, 4 & 5

OUR REFERENCE NO. 5 - 7 - 12

CLIENT: COUNTY OF HURON
PROJECT: COUNTY ROAD Nº 31
LOCATION: GODERICH, ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: AUGERING
DIAMETER OF BOREHOLE: 3 1/2"
DATE: B.H. Nº 2 - AUG. 18, 1965
B.H. Nº 3 - AUG. 19, 1965
B.H. Nº 4 - AUG. 18, 1965
B.H. Nº 5 - AUG. 19, 1965

ENCLOSURE NO. 4

Columns: ELEVATION ft. | DEPTH ft. | STRATIFICATION DESCRIPTION | STRATIFICATION SYMBOL | SAMPLES (NUMBER, TYPE, N of Advancement of Sampler) | PENETRATION RESISTANCE blows per foot 20 40 60 80 100 / SHEAR STRENGTH 1000 lbs sq.ft 1 2 3 4 5 | CONSISTENCY water content % (PL, W, LL) 10 20 30 40 | REMARKS

## BOREHOLE Nº 2

| Elevation | Depth | Stratification Description | Sample Number | Type | N | Remarks |
|---|---|---|---|---|---|---|
| 697.6 | 0 | GROUND SURFACE | | | | |
| | | ROAD FILL SAND, GRAVEL | | | | Bentonite Plug |
| 695.0 | 1.5 | Loose to Compact Multicoloured, GRAVEL embedded into CLAYEY SILT matrix, some organic matter. PROBABLY FILL | 1 | S.S. | 6 | |
| | 5 | | 2 | S.S. | 12 | Piezometer |
| 690.0 | 8.5 | | 3 | S.S. | 20 | Sand Backfill |
| | 10 | Very Stiff to Hard Brown CLAY TILL | 4 | S.S. | 22 | γ = 128 P.C.F; $C_u$ = 5200 P.S.F |
| 685.0 | 13.4 | | 5 | S.S. | 106 | |
| | 15 | END OF BOREHOLE | | | | HOLE DRY - SEPT. 2, 1965 |

## BOREHOLE Nº 3

| Elevation | Depth | Stratification Description | Sample Number | Type | N |
|---|---|---|---|---|---|
| 704.2 | 0 | GROUND SURFACE | | | |
| | | 6" TOPSOIL | | | |
| | | Very Stiff, Brown CLAY TILL | 1 | S.S. | 18 |
| 700.0 | 3.5 | END OF BOREHOLE | | | |
| | 5 | | | | |

## BOREHOLE Nº 4

| Elevation | Depth | Stratification Description | Sample Number | Type | N |
|---|---|---|---|---|---|
| 682.5 | 0 | GROUND SURFACE | | | |
| | | 3" TOPSOIL | | | |
| 680.0 | 2.5 | weathered organic | | | |
| | | Very Stiff, Brown CLAY TILL | 1 | S.S. | 24 |
| | 5 | | 2 | S.S. | 26 |
| | | END OF BOREHOLE | | | |
| 675.0 | | | | | |
| | 10 | | | | |

## BOREHOLE Nº 5

| Elevation | Depth | Stratification Description | Sample Number | Type | N |
|---|---|---|---|---|---|
| 666.5 | 0 | GROUND SURFACE | | | |
| | | BOULDERS | | | |
| 665.0 | | Hard, Brown CLAY TILL | 1 | S.S. | 38 |
| | 2.5 | END OF BOREHOLE | | | |
| | 5 | | | | |
| 660.0 | | | | | |

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: D.A.M. CH'D.

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 4 A

OUR REFERENCE NO. 5 - 7 - 12

CLIENT: COUNTY OF HURON
PROJECT: COUNTY ROAD Nº 31
LOCATION: GODERICH, ONT.
DATUM ELEVATION: GEODETIC

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 3/8"
DATE: AUG. 23, 1965

ENCLOSURE NO. 5

| ELEVATION ft. | DEPTH ft. | STRATIFICATION DESCRIPTION | SAMPLES NUMBER | TYPE | BLOWS PER FOOT | REMARKS |
|---|---|---|---|---|---|---|
| 687.8 | 0 | GROUND SURFACE | | | | |
| | | TOPSOIL | | | | |
| | 1.0 | weathered organic | 1 | S.S | 12 | Bentonite Plug |
| 685.0 | | | 2 | S.S. | 27 | |
| | 4.0 | Very Stiff to Hard CLAY TILL (clay, silt with some sand and fine gravel) | 3 | S.S. | 19 | GRAIN SIZE DISTR. (ENCL. 12) |
| | 5 | | 4 | S.S. | 30 | |
| 680.0 | | | 5 | S.S. | 27 | Piezometer |
| | 10 | | 6 | S.S. | 26 | |
| | | | 7 | S.S. | 25 | Sand Backfill |
| 675.0 | | | 8 | S.S. | 59 | |
| | 15 | | 9 | S.S. | 35 | GRAIN SIZE DISTR. (ENCL. 13) |
| | | 11 to 14 ft. OCCASIONAL THIN LENSES OF FINE SAND | 10 | S.S. | 32 | |
| 670.0 | | | 11 | S.S. | 21 | |
| | 19.0 | | 12 | S.S. | 30 | γ = 129 P.C.F. |
| | 20 | END OF BOREHOLE | | | | |

PENETRATION RESISTANCE blows per foot: 20 40 60 80 100
SHEAR STRENGTH 1000 lbs. sq. ft.: 1 2 3 4 5
CONSISTENCY water content %: PL W LL; 10 20 30 40

VERTICAL SCALE: 1 IN. TO 5 FT.

**DOMINION SOIL INVESTIGATION LIMITED**

MADE D. A. M. CH'D.

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 5A & 6

OUR REFERENCE NO. 5 - 7 - 12

CLIENT COUNTY OF HURON
PROJECT COUNTY ROAD Nº 31
LOCATION GODERICH, ONT.
DATUM ELEVATION GEODETIC

METHOD OF BORING B.H. 5A, WASHBORING - 2 3/8"
B.H. 6, AUGERING - 3 ½"
DIAMETER OF BOREHOLE
ENCLOSURE NO. 6
DATE B.H. 5A, AUG. 20, 1965
B.H. 6, AUG. 18, 1965

Columns: 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 1000 lbs sq ft 1 2 3 4 5 | CONSISTENCY water content % ($P_L$, W, $L_L$) 10 20 30 40 | REMARKS

## BOREHOLE Nº 5A

| Elevation | Depth | Stratification Description | Sample No. | Type | Advancement | Remarks |
|---|---|---|---|---|---|---|
| 666.5 | 0 | GROUND SURFACE | | | | |
| 665.0 | 1.0 | BOULDERS, GRAVEL | | | | Piezometer |
| | 5 | weathered organic | | | | |
| 660.0 | 6.0 | Hard Brown - Grey Mottled CLAY TILL | 1 | S.S. | 46 | |
| | | | 2 | S.S. | 59 | |
| 655.0 | 10 | | 3 | S.S. | 43 | Bentonite Plug |
| | | | 4 | S.S. | 45 | Cu = 5800; γ = 118.0 P.C.F. |
| 650.0 | 15 | | 5 | S.S. | 50 | Sand Backfill |
| | 20 | | 6 | S.S. | 66 | γ = 133 P.C.F. |
| 645.0 | 21.5 | END OF BOREHOLE | | | | |
| | 25 | | | | | |

## BOREHOLE Nº 6

| Elevation | Depth | Stratification Description | Sample No. | Type | Advancement |
|---|---|---|---|---|---|
| 658.2 | 0 | GROUND SURFACE | | | |
| | | 6" TOPSOIL | | | |
| 655.0 | | Very Stiff to Hard, Brown CLAY TILL | 1 | S.S. | 14 |
| | 5 | | 2 | S.S. | 34 |
| | 6.5 | END OF BOREHOLE | | | |
| 650.0 | 10 | | | | |

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CH'D.

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 7 . . .

OUR REFERENCE NO. 5 - 7 - 12

CLIENT COUNTY OF HURON
PROJECT COUNTY ROAD Nº 31
LOCATION GODERICH, ONT.
DATUM ELEVATION GEODETIC

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE 2 3/8"
DATE AUG. 24, 1965

ENCLOSURE NO. 7

| ELEVATION ft | DEPTH ft | STRATIFICATION DESCRIPTION | SAMPLE NUMBER | TYPE | "N" or Advancement of Sampler | REMARKS |
|---|---|---|---|---|---|---|
| 645± | 0 | GROUND SURFACE | | | | |
| | | 2" ASPHALT<br>6" GRAN. BASE COURSE | | | | |
| | | Multicoloured SAND, GRAVEL CLAY & SILT PROBABLY FILL | 1 | S.S. | 6 | NO RECOVERY |
| 640.0 | 5 | | 2 | S.S. | 13 | NO RECOVERY |
| | | | 3 | S.S. | 17 | |
| 635.0 | 10 | Very Stiff Brown CLAY TILL | | | | Bentonite Plug |
| | | | 4 | S.S. | 25 | Piezometer |
| | | | 5 | S.S. | 21 | Sand Backfill |
| 630.0 | 15 | | | | | |
| | | | 6 | S.S. | 28 | γ = 135 P.C.F. |
| | 16.5 | END OF BOREHOLE | | | | |
| 625.0 | 20 | | | | | |

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

CONSISTENCY water content % PL W LL — 10 20 30 40

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE D. A. M. CH'D:

# APPENDIX III

| SAMPLE DETAILS | | | | CONSISTENCY | | | | | UNDRAINED COMPRESSION | | UNIT WEIGHT | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| BOREHOLE | SAMPLE | TYPE | AVERAGE DEPTH (FEET) | NATURAL WATER CONTENT (%) | LIQUID LIMIT (%) | PLASTIC LIMIT (%) | PLASTICITY INDEX | LIQUIDITY INDEX | SHEAR STRENGTH (P.S.FT.) | AXIAL STRAIN AT FAILURE (%) | (P.C.FT.) | |
| 1 | 2 | S.S. | 10·0 | 12·4 | 23·3 | 13·9 | 9·4 | — | | | 142 | Consolidated, Undrained Triaxial Test with P.P. Measurement. (Encl. № 15) |
| | 3 | S.S. | 15·0 | 14·3 | 21·7 | 13·4 | 8·3 | 0·18 | | | | |
| | 4 | S.S. | 20·0 | 12·1 | 24·2 | 14·0 | 10·2 | — | | | 140 | —"— (Encl. № 17) |
| | 5 | S.S. | 25·0 | 13·6 | 25·6 | 13·3 | 12·3 | 0·02 | | | | |
| | 6 | S.S. | 30·0 | 13·6 | 25·2 | 13·7 | 11·5 | — | | | | |
| | 8 | S.S. | 40·0 | 15·4 | 21·0 | 12·9 | 8·1 | 0·37 | | | | |
| | 9 | S.S. | 45·0 | 16·1 | 29·0 | 14·5 | 13·9 | 0·12 | | | | |
| | 10 | S.S. | 50·0 | 17·1 | 31·3 | 15·4 | 15·2 | 0·12 | | | | |
| | 11 | S.S. | 55·0 | 20·8 | 33·6 | 16·0 | 17·6 | 0·27 | | | | |
| | 12 | S.S. | 60·0 | 16·6 | | | | | | | | |
| | 13 | S.S. | 65·0 | 18·1 | | | | | | | | |
| | 14 | S.S. | 70·0 | 12·7 | 24·0 | 14·6 | 9·4 | — | | | | Grain Size Distribution (Encl. № 10) |
| | 15 | S.S. | 75·0 | 13·5 | | | | | | | | |
| | 16 | S.S. | 80·0 | 8·7 | 15·7 | 11·2 | 4·5 | — | | | | Grain Size Distribution (Encl. № 11) |
| | 17 | S.S. | 85·0 | 11·8 | 25·1 | 14·5 | 10·6 | — | | | | |
| 2 | 4 | S.S. | 10·0 | 14·0 | | | | | 5184 | 12·5 | 128 | |
| 3 | 1 | S.S. | 2·0 | 13·1 | | | | | | | | |
| 4 A | 1 | S.S. | 1·0 | 12·1 | 28·4 | 14·0 | 14·4 | — | | | | |
| | 2 | S.S. | 2·5 | 12·2 | | | | | | | | |
| | 3 | S.S. | 4·0 | 10·6 | 26·5 | 13·9 | 12·6 | — | | | | Grain Size Distribution (Encl. № 12) |
| | 4 | S.S. | 5·5 | 10·6 | | | | | | | | |
| | 5 | S.S. | 7·0 | 12·8 | 30·5 | 14·9 | 15·6 | — | | | | |
| | 7 | S.S. | 10·0 | 15·0 | | | | | | | | |
| | 8 | S.S. | 11·5 | 15·5 | | | | | | | | |
| | 9 | S.S. | 13·0 | 17·3 | 24·2 | 15·8 | 8·4 | 0·18 | | | | Grain Size Distribution (Encl. № 13) |
| | 10 | S.S. | 14·5 | 18·0 | 33·9 | 15·8 | 18·1 | 0·12 | | | | |
| | 11 | S.S. | 16·0 | 18·9 | | | | | | | | |
| | 12 | S.S. | 17·5 | 17·7 | | | | | 4220 | 20 | 129 | |

# TABLE OF LABORATORY TEST RESULTS

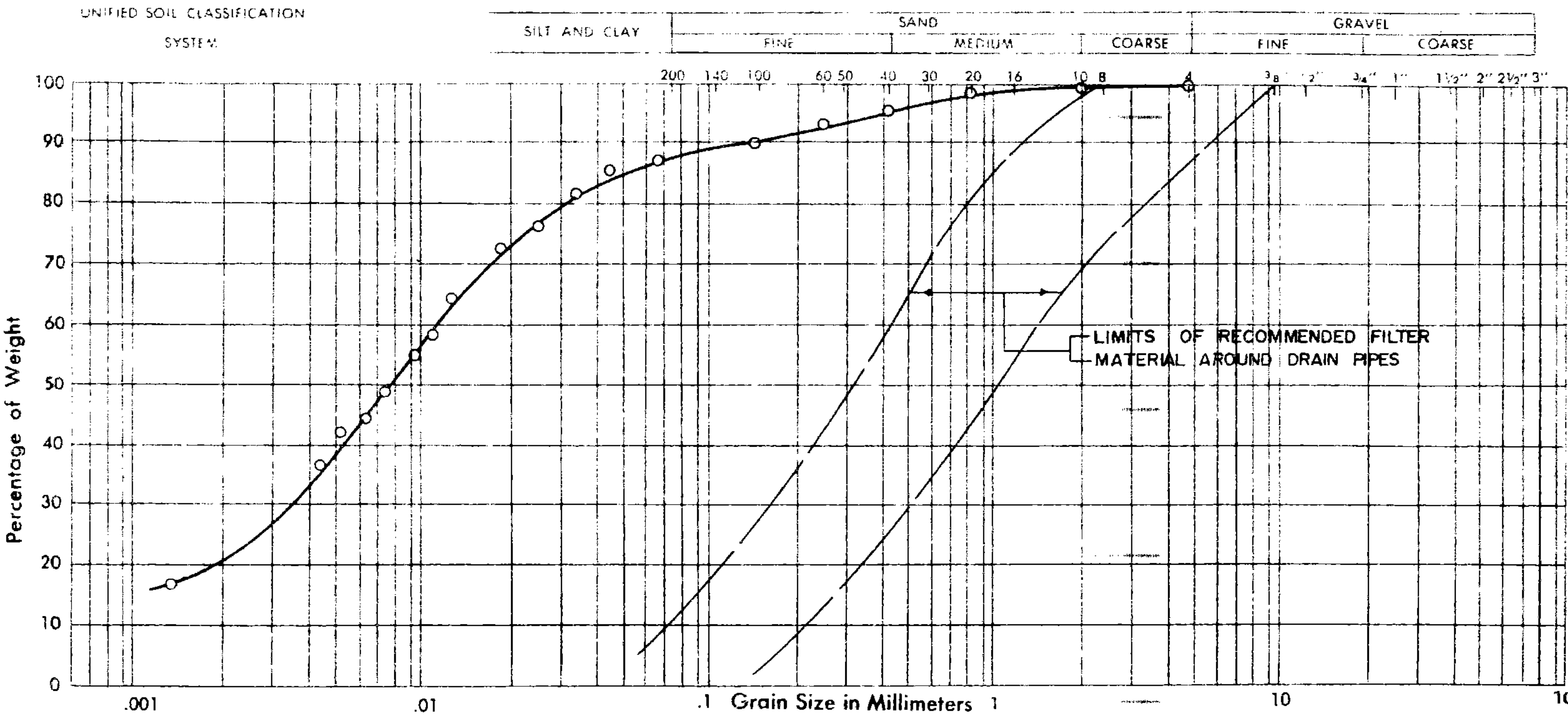
| SAMPLE DETAILS | | | | CONSISTENCY | | | | | UNDRAINED COMPRESSION | | UNIT WEIGHT | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| BOREHOLE | SAMPLE | TYPE | AVERAGE DEPTH (FEET) | NATURAL WATER CONTENT (%) | LIQUID LIMIT (%) | PLASTIC LIMIT (%) | PLASTICITY INDEX | LIQUIDITY INDEX | SHEAR STRENGTH (P.S.FT.) | AXIAL STRAIN AT FAILURE (%) | (P.C.FT.) | |
| 5 A | 1 | S.S. | 5·0 | 13·9 | 26·7 | 14·2 | 12·5 | — | | | | |
| | 2 | S.S. | 7·5 | 13·3 | | | | | | | | |
| | 3 | S.S. | 10·0 | 13·4 | | | | | | | | |
| | 4 | S.S. | 12·5 | 15·9 | | | | | 5825 | 13·3 | 118 | |
| | 5 | S.S. | 15·0 | 13·9 | | | | | | | | |
| | 6 | S.S | 20·0 | 12·6 | 22·6 | 13·1 | 9·5 | — | 4250 | 16 | 133 | |
| 7 | 6 | S.S. | 15·0 | 14·2 | | | | | | | 135 | Consolidated, Undrained Triaxial Test with P.P. Measurement |

# TABLE OF LABORATORY TEST RESULTS

# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 5 - 7 - 12




Enclosure No. 10

PROJECT: HURON COUNTY ROAD Nº 31

LOCATION: GODERICH

BOREHOLE NO.: 1

SAMPLE NO.: 14

DEPTH OF SAMPLE: 70' - 71·5'

ELEVATION OF SAMPLE: 648' ±

COEFFICIENT OF UNIFORMITY ~12

COEFFICIENT OF CURVATURE

**Classification of Sample and Group Symbol:**

CLAYEY SILT with some sand

(TILL)

CL

PLASTIC PROPERTIES:

LIQUID LIMIT % = 24

PLASTIC LIMIT % = 14·6

PLASTICITY INDEX % = 9·4

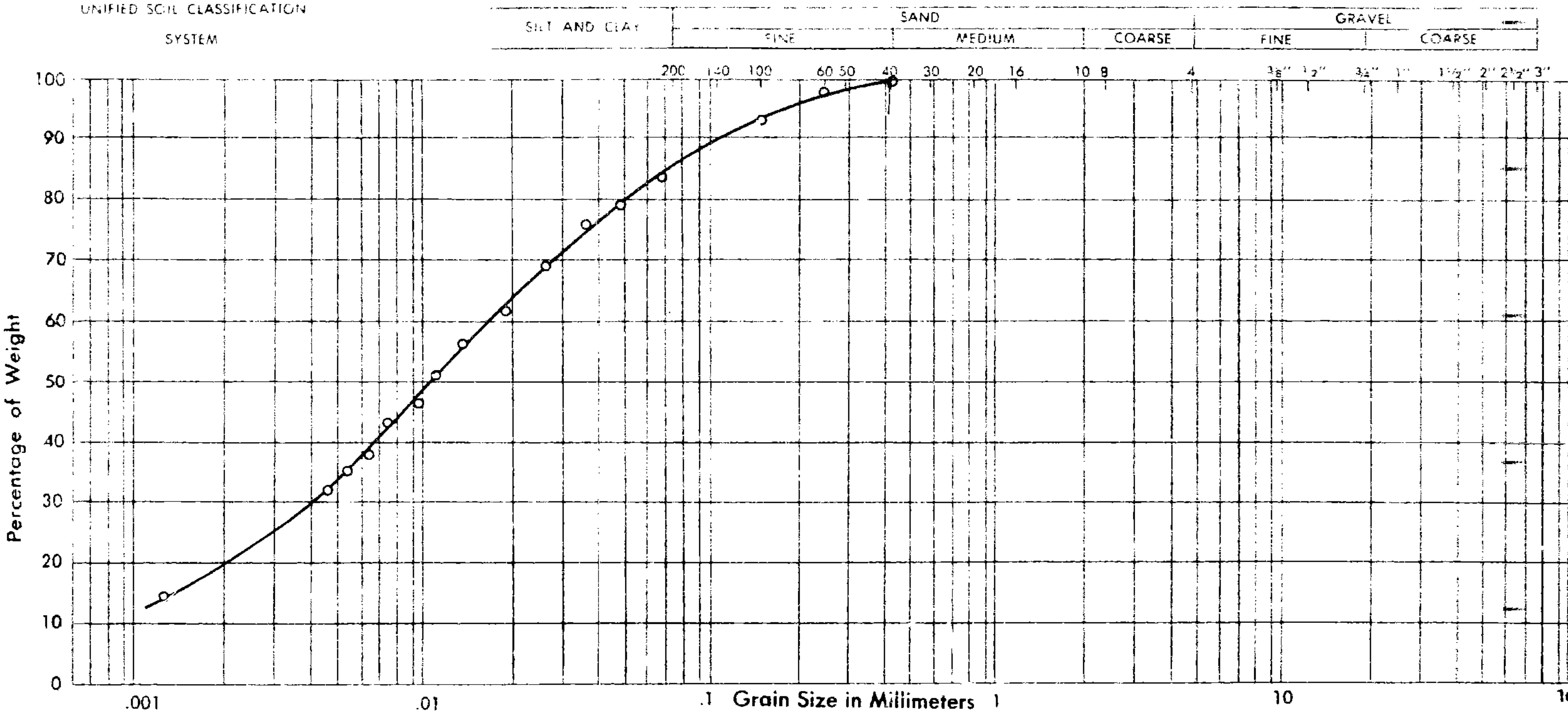
MOISTURE CONTENT % = 12·7

ACTIVITY = 0·47

# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO 5 - 7 - 12




Enclosure No. 11

PROJECT HURON COUNTY ROAD № 31

LOCATION GODERICH

BOREHOLE NO · 1

SAMPLE NO · 16

DEPTH OF SAMPLE: 80' — 80' - 11"

ELEVATION OF SAMPLE: 638' ±

COEFFICIENT OF UNIFORMITY ~ 20

COEFFICIENT OF CURVATURE

**Classification of Sample and Group Symbol:**

CLAYEY SILT with some fine sand

( T I L L )

CL – ML

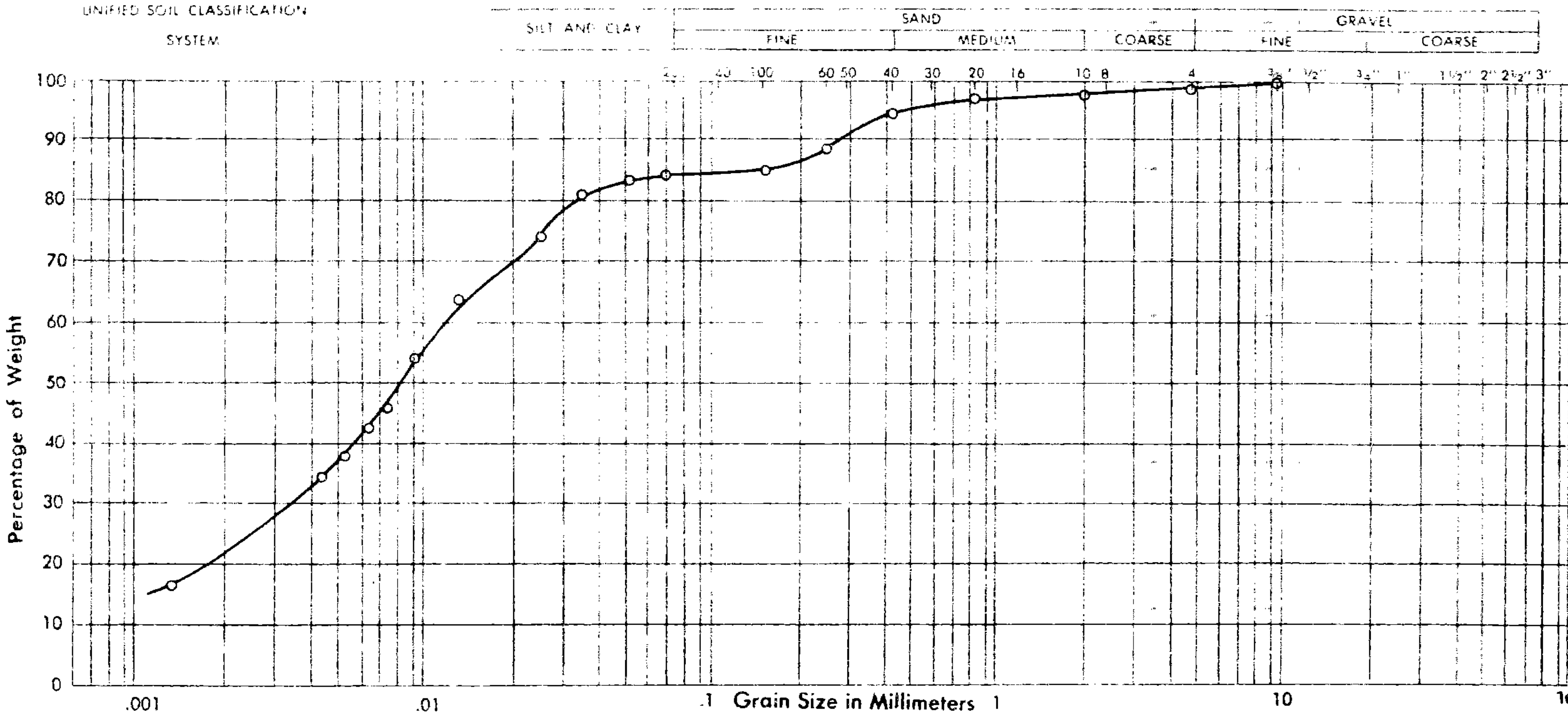
PLASTIC PROPERTIES:

| | | |
|---|---|---|
| LIQUID LIMIT | % = | 15·7 |
| PLASTIC LIMIT | % = | 11·2 |
| PLASTICITY INDEX | % = | 4·5 |
| MOISTURE CONTENT | % – | 8·7 |
| ACTIVITY | = | 0·23 |

# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 5 - 7 - 12




PROJECT: HURON COUNTY RD. Nº 31

LOCATION: GODERICH

BOREHOLE NO.: 4 A

SAMPLE NO.: 3

DEPTH OF SAMPLE: 4' - 5.5'

ELEVATION OF SAMPLE: 683' ±

COEFFICIENT OF UNIFORMITY ~ 13

COEFFICIENT OF CURVATURE

**Classification of Sample and Group Symbol:**

CLAYEY SILT with some sand and a trace of gravel

(T I L L)

C L

PLASTIC PROPERTIES:

LIQUID LIMIT % = 26.5

PLASTIC LIMIT % = 13.9

PLASTICITY INDEX % = 12.6

MOISTURE CONTENT % = 10.6

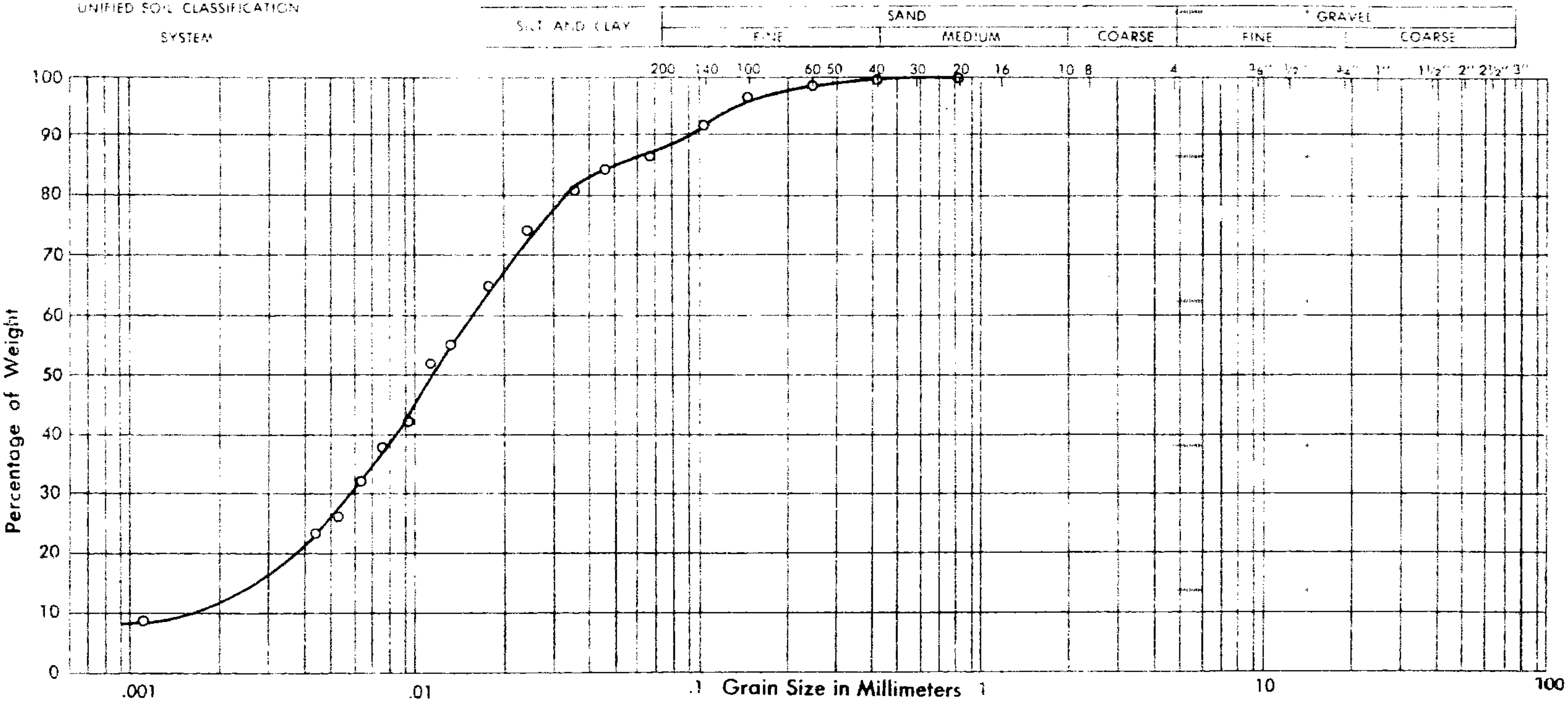
ACTIVITY = 0.5

Enclosure No. 12

# DOMINION SOIL INVESTIGATION LIMITED

GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 5 - 7 - 12




Enclosure No. 13

PROJECT HURON COUNTY RD. № 31

LOCATION GODERICH

BOREHOLE NO. 4 A

SAMPLE NO.: 9

DEPTH OF SAMPLE: 13' - 14.5'

ELEVATION OF SAMPLE: 674' ±

COEFFICIENT OF UNIFORMITY ~ 8

COEFFICIENT OF CURVATURE

PLASTIC PROPERTIES:

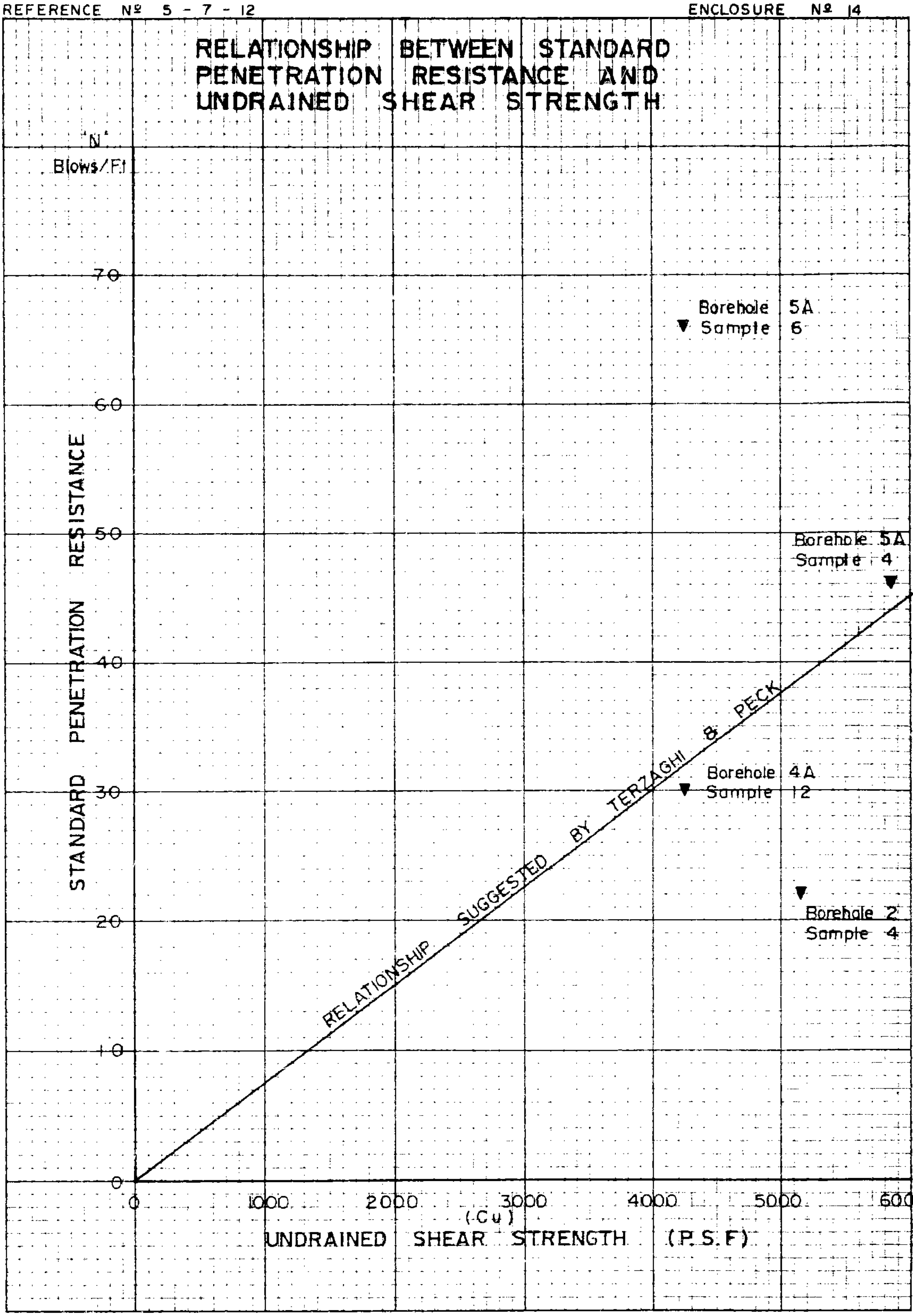
| | | | |
|---|---|---|---|
| LIQUID LIMIT | % | = | 24·2 |
| PLASTIC LIMIT | % | = | 15·8 |
| PLASTICITY INDEX | % | = | 8·4 |
| MOISTURE CONTENT | % | = | 17·3 |
| ACTIVITY | | = | 0·75 |

**Classification of Sample and Group Symbol:**

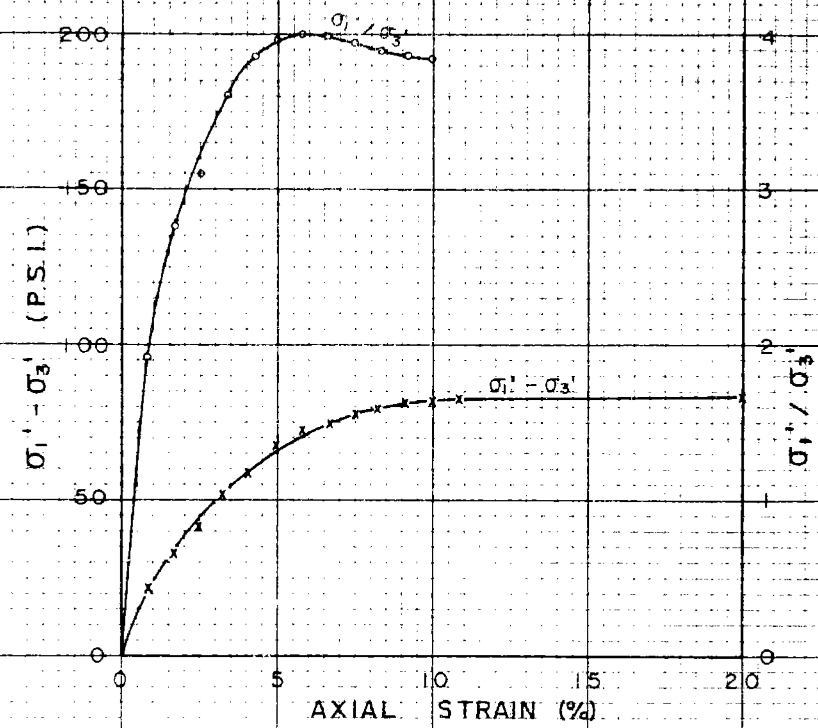
SILT with some clay and fine sand

(TILL)

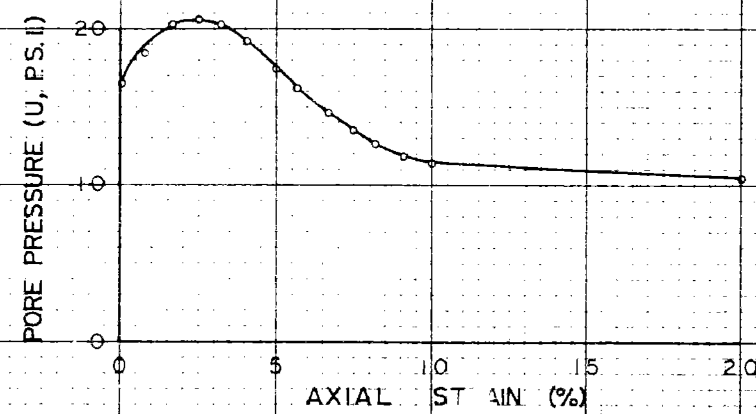
CL


RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE AND UNDRAINED SHEAR STRENGTH
'N'
Blows/Ft
STANDARD PENETRATION RESISTANCE
0
10
20
30
40
50
60
70
0
1000
2000
3000
4000
5000
6000
(Cu)
UNDRAINED SHEAR STRENGTH (P.S.F)
RELATIONSHIP SUGGESTED BY TERZAGHI & PECK
Borehole 5A
Sample 6
Borehole 5A
Sample 4
Borehole 4A
Sample 12
Borehole 2
Sample 4

Test № 1
Borehole 1
Sample 2

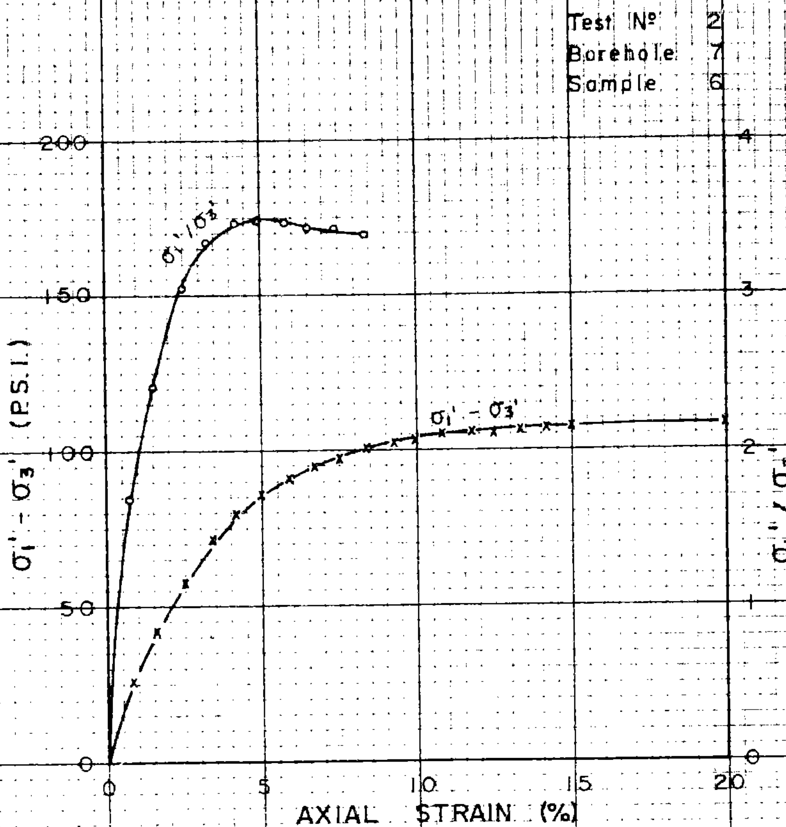



STRESS - STRAIN CURVES

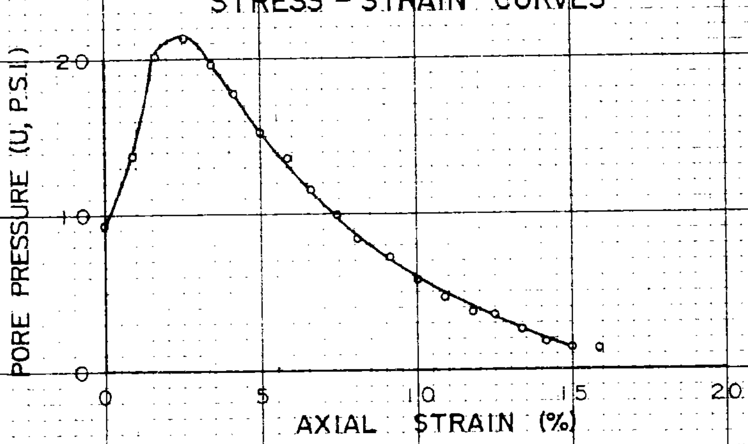



VARIATION OF PORE PRESSURE WITH STRAIN

CONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST
WITH PORE PRESSURES MEASURED

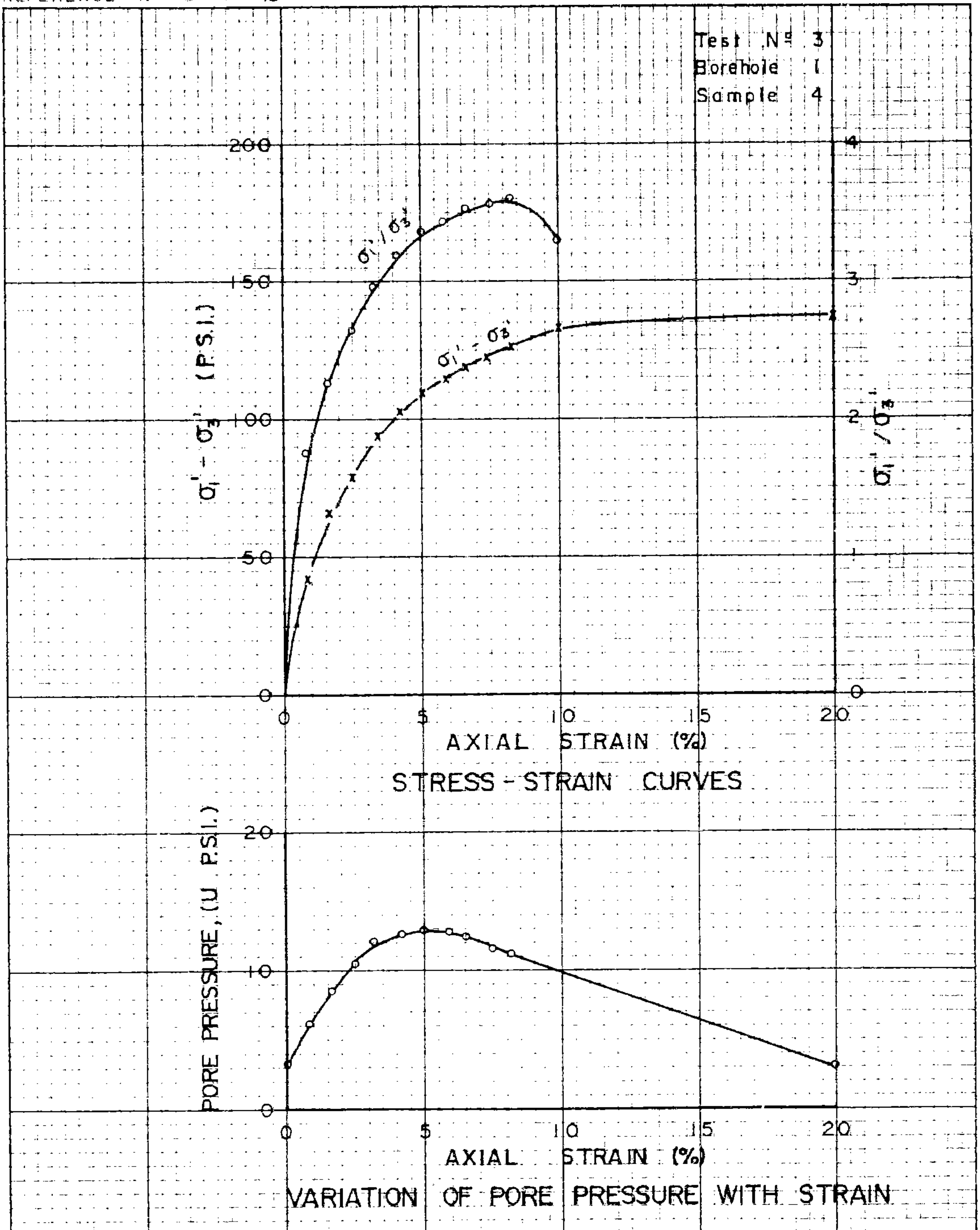



STRESS - STRAIN CURVES




VARIATION OF PORE PRESSURE WITH STRAIN

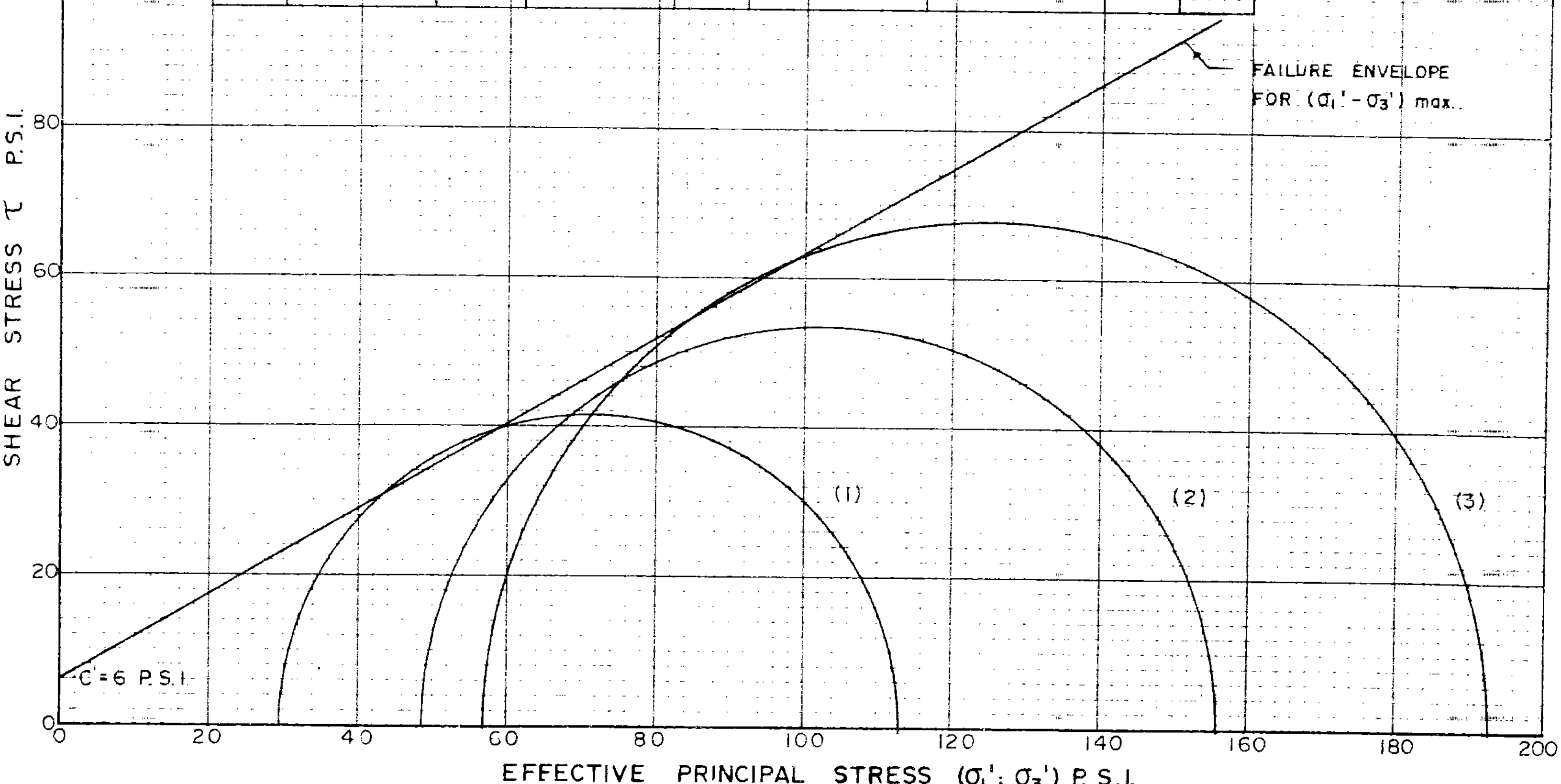
CONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURES MEASURED




**CONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURES MEASURED**

# MOHR'S CIRCLES FOR TRIAXIAL COMPRESSION TEST

| TEST Nº | CONSOLIDATION PRESSURE (P.S.I.) | CELL PRESSURE (P.S.I.) | INITIAL PORE PRESSURE (P.S.I.) | B | DEVIATOR STRESS AT FAILURE $\sigma_1' - \sigma_3'$ (P.S.I.) | PORE PRESSURE AT FAILURE (P.S.I.) | $\sigma_1'$ (P.S.I.) | $\sigma_3'$ (P.S.I.) |
|---|---|---|---|---|---|---|---|---|
| 1 | 15 | 30 | 6·4 | 0·43 | 83 | 0·2 | 112·8 | 29·8 |
| 2 | 25 | 50 | 9·6 | 0·38 | 107·5 | 1·5 | 156 | 48·5 |
| 3 | 30 | 60 | 3·1 | 0·10 | 136 | 3·0 | 193 | 57 |

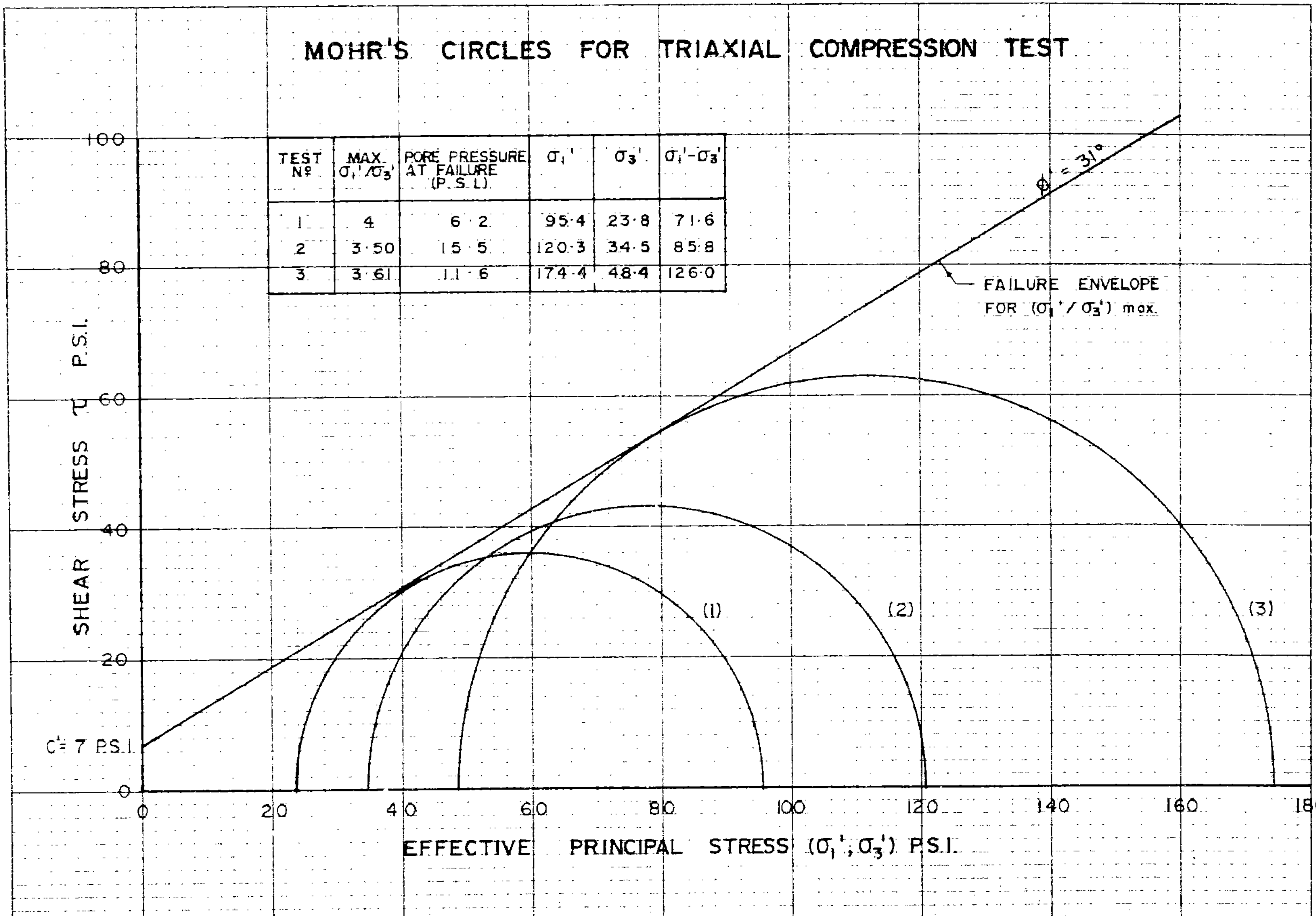



REFERENCE Nº 5-7-12

ENCLOSURE Nº 18

# MOHR'S CIRCLES FOR TRIAXIAL COMPRESSION TEST

| TEST Nº | MAX. $\sigma_1'/\sigma_3'$ | PORE PRESSURE AT FAILURE (P.S.I.) | $\sigma_1'$ | $\sigma_3'$ | $\sigma_1'-\sigma_3'$ |
|---|---|---|---|---|---|
| 1 | 4 | 6·2 | 95·4 | 23·8 | 71·6 |
| 2 | 3·50 | 15·5 | 120·3 | 34·5 | 85·8 |
| 3 | 3·61 | 11·6 | 174·4 | 48·4 | 126·0 |




REFERENCE Nº 5-7-12

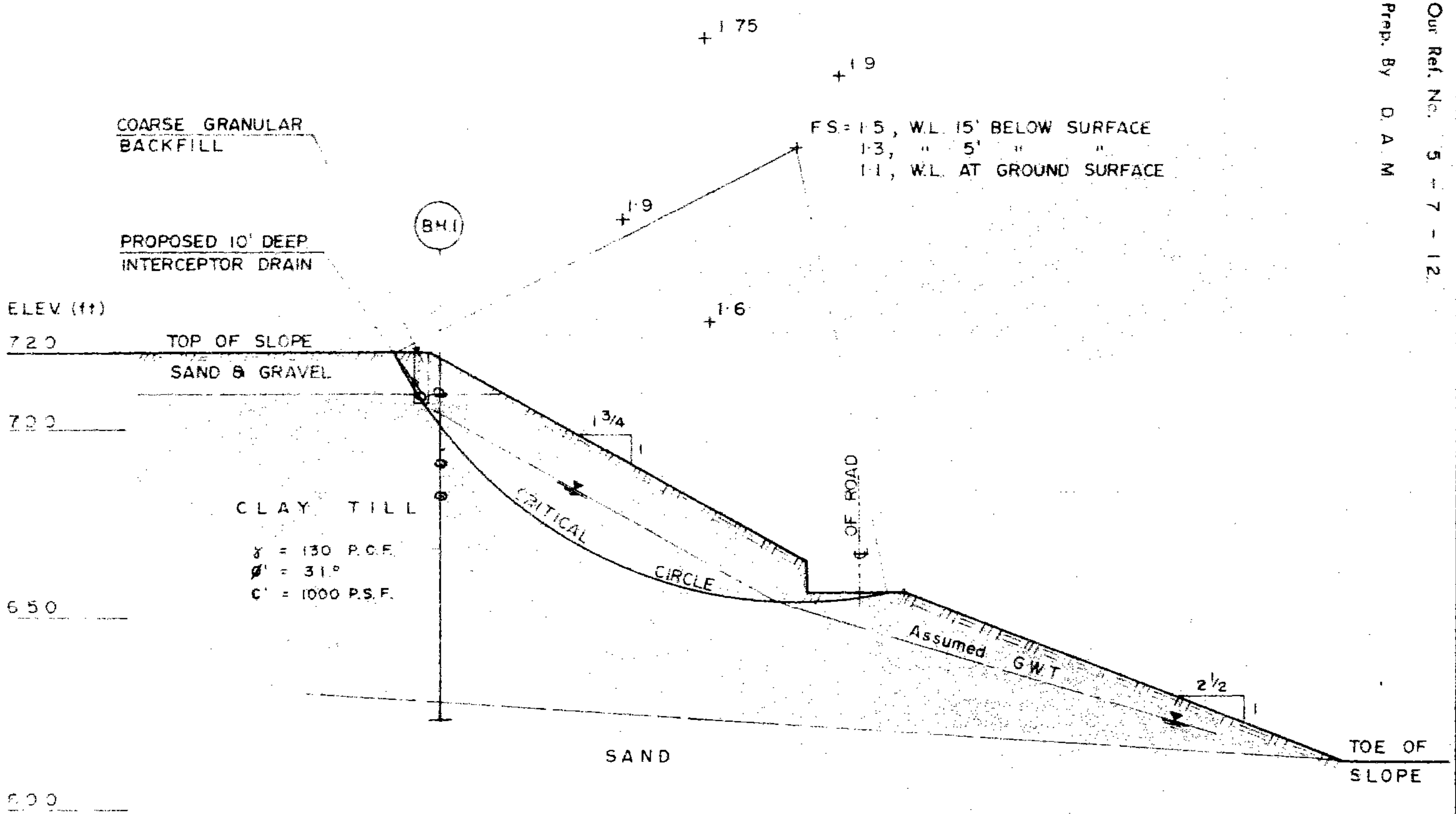
ENCLOSURE Nº 19

# APPENDIX IV

Our Ref. No. 5 - 7 - 12

Prep. By D. A. M

Figure Nº 1




## CROSS SECTION AT STATION 3 + 00

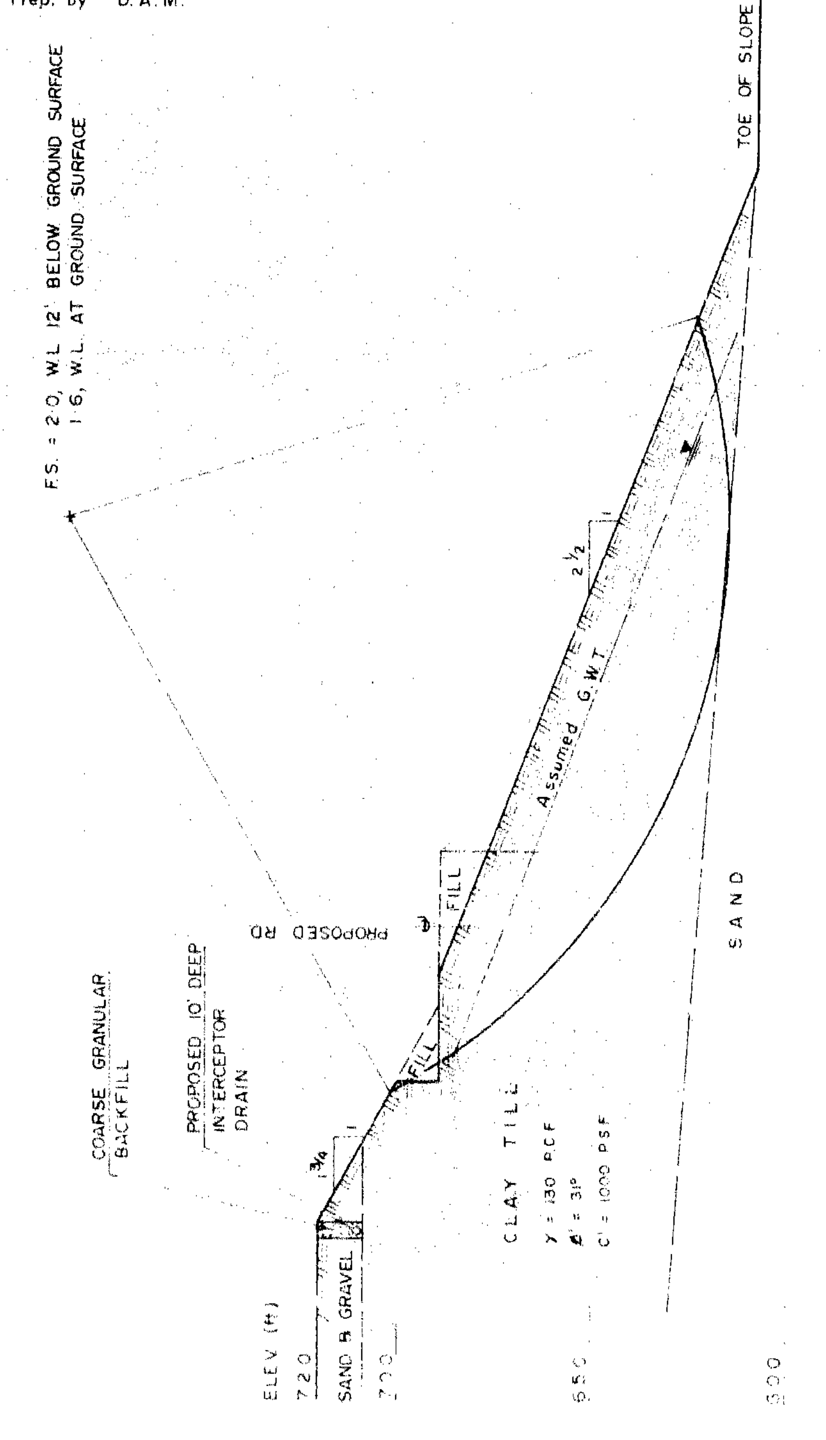
## ANALYSIS OF SLOPE STABILITY ON UPHILL SIDE EFFECTIVE STRESSES

SCALE: 1" = 40 feet

Our Ref. No. 5 - 7 - 12

Prep. By D. A. M.

Figure Nº 2




ASSUMED CROSS SECTION AT STATION 8+50

ANALYSIS OF SLOPE STABILITY ON DOWNHILL SIDE AFTER CONSTRUCTION

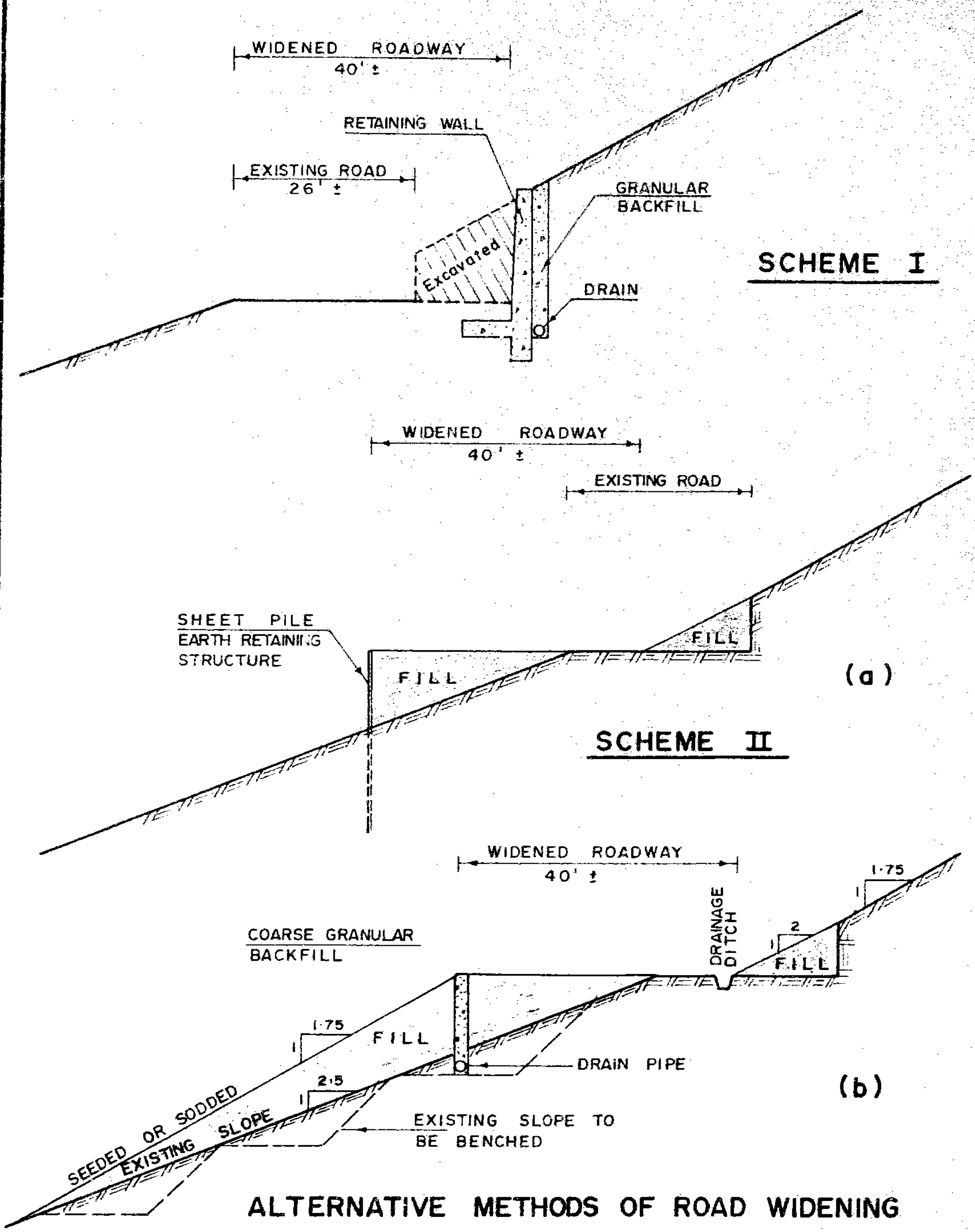
(EFFECTIVE STRESSES)

SCALE: 1" = 40 feet

Our Ref. No. 5 - 7 - 12

Figure № 3

Prep. By D. A. M.




ALTERNATIVE METHODS OF ROAD WIDENING

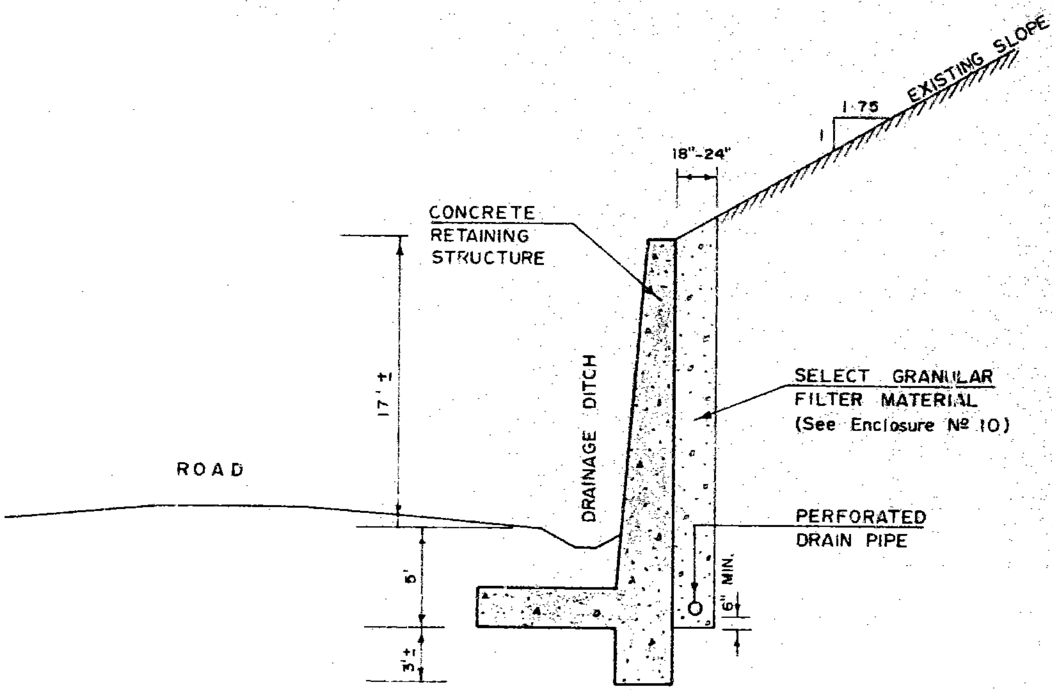
SCALE: 1" = 20 feet

DOMINION SOIL INVESTIGATION LIMITED

Our Ref. No. 5 - 7 - 12

Figure Nº 4

Prep. By D. A. M.




NOTE: STRUCTURAL DETAILS AND DIMENSIONS PRELIMINARY, SUBJECT TO FINAL DESIGN.

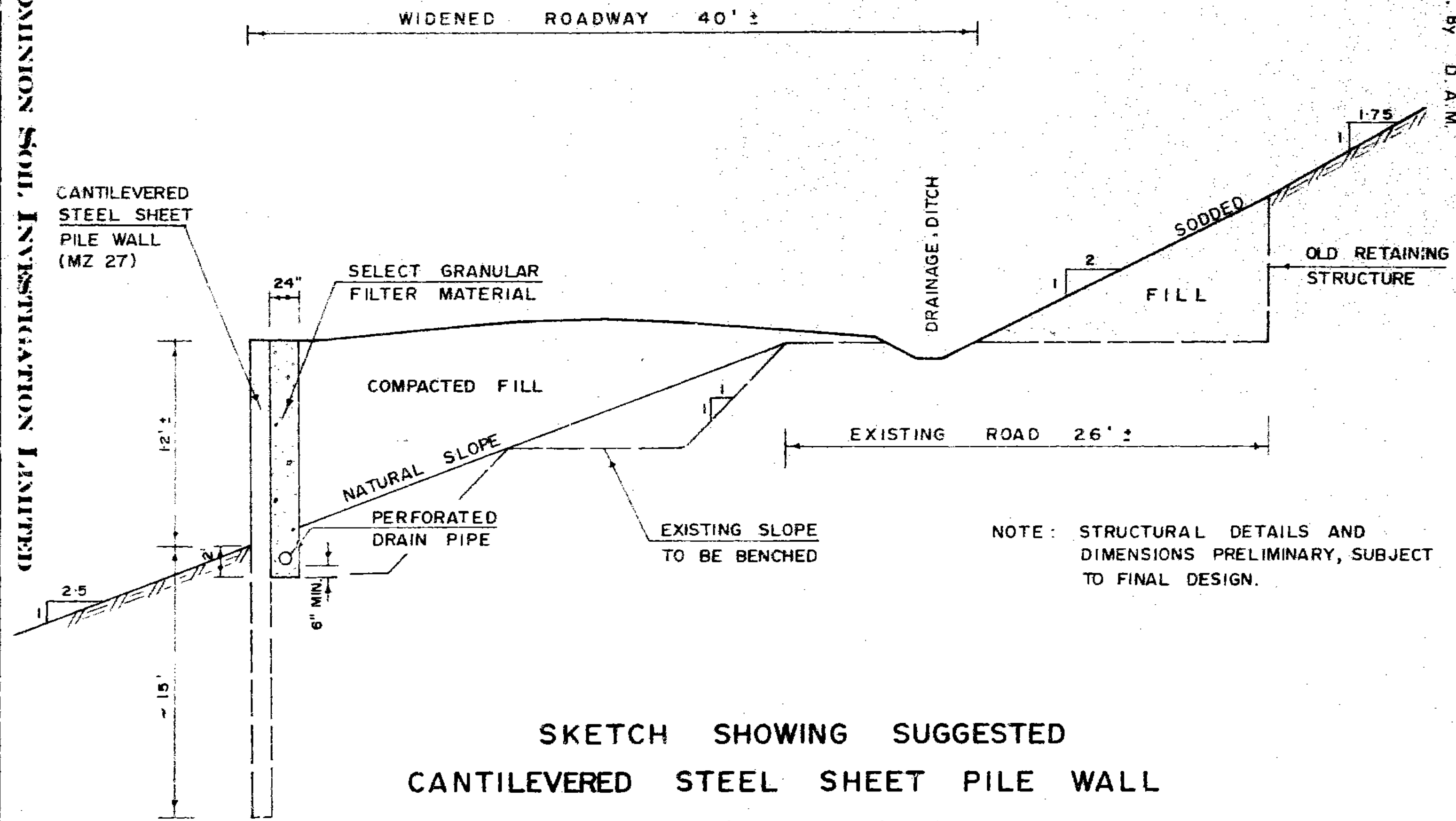
SKETCH SHOWING SUGGESTED LAYOUT OF RETAINING STRUCTURE AND DRAINS ON UPHILL SIDE

Dominion Soil Investigation Limited

Our Ref. No. 5 - 7 - 12

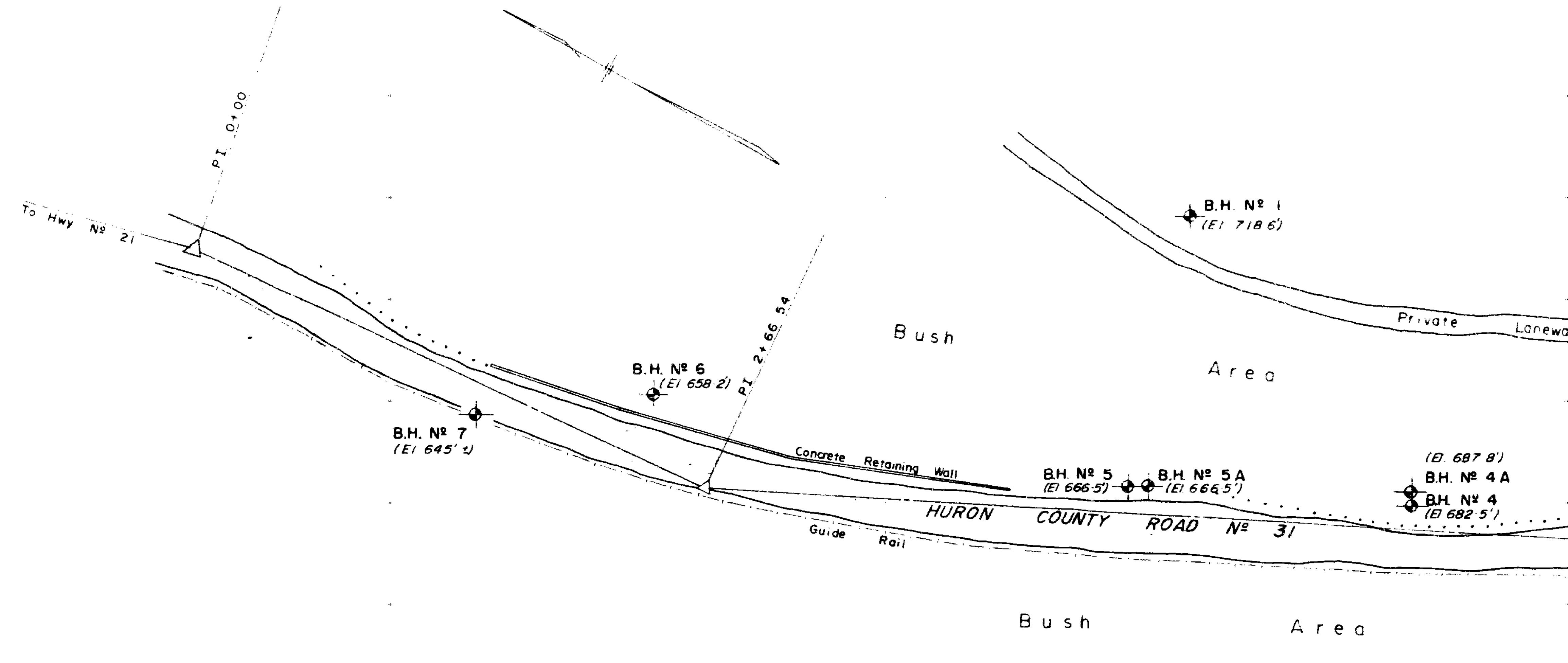
Prep. By D. A. M.

Figure № 5



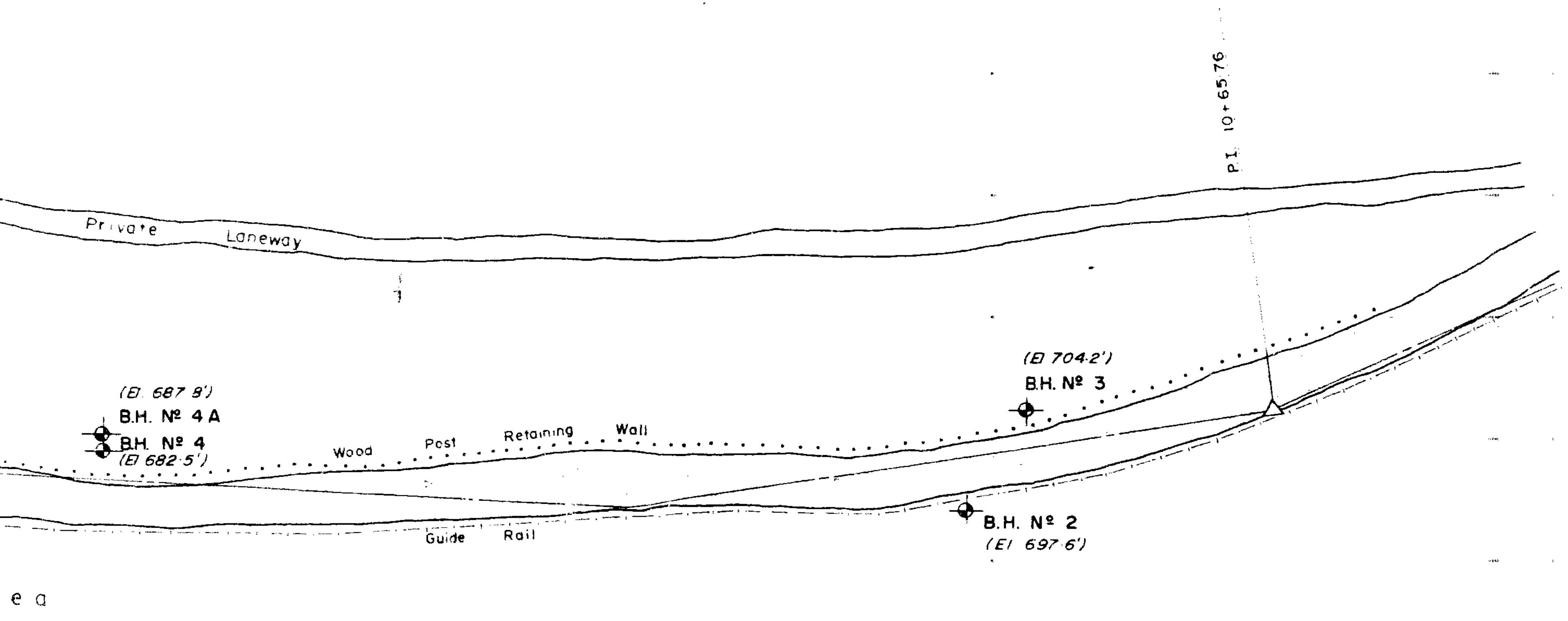

SKETCH SHOWING SUGGESTED
CANTILEVERED STEEL SHEET PILE WALL
ON DOWNHILL SIDE

DOMINION SOIL INVESTIGATION LIMITED




BOREHOLE LOCATION P

SCALE: 1 inch = 50 feet




LOCATION PLAN

1 inch = 50 feet

| DOMINION SOIL INVESTIGATION LTD. | | |
|---|---|---|
| BOREHOLE LOCATION PLAN | | |
| Drawn By: D.A.M. | Date: FEB. 3, 1965 | Reference Nº 5-7-12 |
| Drawing Nº 1 | Scale: 1" = 50 feet | Approved: |

cc: Foundations Office (Rm. 110)

Hwy. 401 & Keele St.,
Downsview, Ontario.

Materials and Testing Division

May 20, 1966

Mr. J. W. Britnell, County Engineer,
County of Huron,
Court House,
Goderich, Ontario.

Dear Mr. Britnell:

Re: County Rd. No. 31 (Saltford Hill), Huron County

In his memorandum of February 16, 1966, Mr. J. G. Tillcock, District Engineer, Stratford, has conveyed to us your request that the Department review the report prepared by Dominion Soil Investigation Ltd., regarding the soil investigation for the County Road No. 31 (Saltford Hill), and that at a later stage, a meeting be held at the site to discuss the problem.

This is to recapitulate the discussion that took place at the site on May 17, 1966, and also to summarize the conclusions and suggestions given verbally to you by the writer.

The above mentioned investigation has disclosed that the soil of which the main portion of the hill is composed, is a clayey type till material, very dense and of a hard consistency. This material seems to be underlain by a granular deposit at about elevation 630 - i.e., at the valley level. This is, however, insufficiently documented to be accepted as an undisputed fact.

The investigation has also disclosed the presence of a dense sand and gravel layer on the top of the hill. The thickness is about 9 feet. However, here again, the evidence is insufficient to substantiate the presence of this layer all over the top of the hill.

All the evidence contained in the report and pertaining to the main material - i.e., the clayey till, indicates that it is a strong and basically stable material. In view of the sampling technique used, the writer would not put any emphasis on the

cont'd. /2 ...

stability calculations presented in the report, but would rather rely on the field evidence and road maintenance record.

It can be said that the performance of the road, in general, can be considered as satisfactory. According to the available information, the amount of maintenance required to keep the road in satisfactory condition has been relatively small.

There are two areas in which evidence of relatively unsatisfactory performance is visible. These are the two extreme sides of the road - the downhill and the uphill sides. The downhill side as compared to the uphill side, can be considered as the one of inferior performance. This is evidenced by cracks running more or less parallel to the centre-line of the road, and by small localized road subsidences. The inclined positions of some of the guard rail posts indicate certain small lateral movements of the shoulder of the road.

Similar signs, but to a much lesser degree, are found on the uphill side of the road.

Part of the uphill slope along the road is supported by a retaining wall. This wall seems to have been built at various times - i.e., one part of the wall seems to be older than the remainder. In this older part of the wall, signs of slope movement are clearly visible, as evidenced by the forward tilting of the wall, partial cracking, and also by some differential movements of two parts of the old wall.

The above is a short description of the actual facts as they are believed to be at present.

As stated earlier, the road is in generally good condition. The described signs of relatively inferior performance can be attributed to a number of reasons, some of which will be dealt with below:

There is no provision for drainage either on the uphill or the downhill side of the road, and the water running downhill has to find its own way. Naturally, this condition cannot but result in areas of lesser density or lower strength, leading to a decrease in stability and possibly in eventual smaller or larger failures.

Although the road was built mostly as a cut in the side of the hill, some parts of the downhill side could conceivably be fill. This fill could be either a clayey, or some sort of a

cont'd. /3 ...

granular material. The compaction or density - i.e., the strength of this portion of the road, could have changed through the years and the change due to the geometry of the site and the influence of water, would be for the worse. As a consequence, some settlements have taken place resulting in the aforementioned longitudinal cracks. However, no major movements did take place, nor are there any signs of an incipient failure to be found on the downhill side of the road.

Due to the lack of surface drainage facilities, some damage was also caused to the uphill edge of the road. In one or two locations frost seems to have also had some detrimental effects.

The movements of the retaining wall can be attributed also to a number of reasons, but at this stage it would be practically impossible to establish the real cause of trouble.

The writer would like to mention here, that although the slope material has been identified as very dense and hard and therefore, stable, slow movements do take place in such materials. Usually, these movements are very slow, and from an engineering point of view, do not represent a problem. Only when the rate of movement becomes noticeable does such a case become the concern of the engineer. Based on available evidence, the writer would classify the case in question as one which should not cause too much of a problem during what is generally considered as the expected life-span of an engineering structure.

In connection with the long-term stability problem of the entire slope or parts of it, the need of vegetation preservation cannot be overemphasized. It is also drawn to your attention that any changes in the present conditions would have to be thoroughly analyzed and their possible effects on the slope stability would also have to be considered.

As discussed earlier, provisions and appropriate measures have to be undertaken to take care of the surface waters. Care should be taken that the sources of water discharge be also properly protected to prevent the creation of new areas of weakness.

Since it is intended to improve the road as far as width is concerned, thus bringing it up to the desired standard as closely as possible, either cutting into the uphill slope or filling on the downhill slope will have to be done. It is the writer's opinion that cutting into the uphill slope would represent a better solution. This opinion is based on the following reasoning:

cont'd. /4 ...

The performance of the existing walls, in view of their age, is considered to be acceptable. The lack of knowledge about the details of construction and performance of drainage behind and through the walls, makes their performance even more acceptable.

On the other hand, the widening of the road towards the downhill side, would also require building of retaining walls whose performance, however, due to the sloping ground in front of them, would probably be inferior to the ones mentioned earlier.

In either case, a flexible type of wall with proper and adequate provisions for drainage, is suggested.

In view of the otherwise relatively satisfactory performance of the road, no other major work apart from providing effective surface drainage on both sides of the road, would be suggested.

It is believed that the above basically covers the questions that you have raised at the meeting of May 17. Should there be any other problems that you would like to discuss, please feel free to call on this Office.

Yours very truly,

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Mr. J. G. Tillcock

Foundations Office
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

# MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
Department of Highways,
Downsview, Ontario.

FROM: J. G. Tillcock,
District Engineer,
Stratford, Ontario.

DATE: February 16th., 1966.

OUR FILE REF.

IN REPLY TO

SUBJECT: Re: District #3 County of Huron Soils Report on County Road #31 (Saltford Hill).

As per our telephone conversation of this afternoon I am forwarding herewith a copy of the above Soils Report which was prepared by Dominion Soils Investigation Limited for the County of Huron.

I would appreciate having any comments or suggestions you might care to make in regard to the recommendations contained in this Report.

The County Engineer has requested that someone from our Department meet him at the site to discuss this Report and proposal. If you could arrange to visit this location sometime this spring when weather conditions are suitable and your schedule permits it would be appreciated.

Should you wish to contact the County Engineer direct, for any reason, he is : Mr. J. W. Britnell, County Engineer, County of Huron, Court House, Goderich, Ontario - (Phone 524-7412).

J. G. Tillcock,
District Engineer.

JGT/P