

53-62-59.

REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
STABILITY ANALYSES
RELOCATED GHOST RIVER BRIDGE
HIGHWAY 101
W.P. 73-60
NEAR MATHESON ONTARIO

Distribution:

- 10 copies - Department of Highways, Ontario,
Downsview, Ontario.
- 2 copies - Geoccon Ltd,
Rexdale, Ontario.

January 12th, 1961

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Rexdale, Ontario,
January 12th, 1961.

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Stability Analyses,
Relocated Ghost River Bridge,
Near Matheson, Ontario.

Dear Sirs:

This letter reports the results of additional stability analyses carried out at the above site at your request. A soil investigation and stability analysis was carried out in July, 1960 for a new bridge located on the existing centre line, and the results were presented in our report S7124, dated October 6th, 1960. From these studies, it was concluded that the alignment would have to be shifted to the north, if a grade of 894 was to be used for the bridge, or else a limiting grade of 890 would be necessary. After considering these alternatives, the Department of Highways, Ontario, decided to relocate the centre line to the north by an amount varying from 33 to 38 feet.

Geocon Ltd was requested to carry out further stability analyses on the relocated embankments and the results of these analyses are discussed below and are shown on the attached Drawing S7175-1. The design values used throughout are those quoted in report S7124 and shown in the Design Data Table on Drawing S7175-1.

General

The case of rapid drawdown from elevation 878 to 850 was considered in the calculations, this being more than half the total water level variation in Ghost River at the site. It is doubtful whether saturation of the river banks would occur during the short period of the highest water level stages and thus the rapid drawdown considered is believed to be conservative. The ground water level in the banks of the river, as observed in the bore holes during the investigation, was near ground surface. Thus the case of steady seepage, with such a high ground water level, would prove to be almost as critical as the rapid drawdown case assumed.

Lateral Stability

The lateral stability will be lowest in the southeast quadrant adjacent to the meander of the river and the stability of the slope was checked in this area. Assuming the grade at elevation 894, a 40 foot wide roadway and an embankment fill slope of 2 to 1, stability analyses at the critical section at about station 42+45 gave computed factors of safety for a deep circle of 2.0 and 1.6, for the total and effective stress analyses respectively. For a shallow circle, the computed factors of safety were 1.7 and 1.5 for the total and effective stress analyses respectively. On the basis of these results, it is considered that the lateral stability is adequate and that the grade can be constructed to elevation 894 at the proposed relocated crossing.

Longitudinal Stability

After several trials, the longitudinal section as shown on Drawing S7175-1 was chosen. The crests of the embankments are located

Longitudinal Stability (continued)

at stations 40+15 and 42+15. For a deep circle through the varved silt and clay in the west bank the computed factors of safety were 2.1 and 1.3 for the total and effective stress analyses respectively. Considering an embankment fill slope of 2 to 1, the computed factors of safety for a shallow circle were 1.6 and 1.3 for the total and effective stress analyses. For the east bank of clayey silt, the factors of safety obtained were 1.4 and 1.3 for both a deep circle and a shallow circle by total and effective stress analyses respectively. All the computed factors of safety for longitudinal stability include an addition of 10 percent for end effects. On the basis of these results, the longitudinal section shown on Drawing S7175-1 is considered stable. The overall slopes are about 15 degrees for the west bank and 17 degrees for the east bank.

The toe of the embankments, where they occur below high water level, should be protected against scour by the addition of a rip-rap cover. The soil strata comprising both banks are very susceptible to scour and so consideration should be given to the placing of rip-rap protection on the banks in the area where the river cross-section is constricted by the addition of the embankments and bridge piers.

Side Slopes

All shallow circles for embankment heights up to 14 feet were found to have factors of safety in excess of 1.3 both by the total and effective stress analyses, making no allowance for end effects. It is therefore considered that an embankment slope of 2:1 is stable for the majority of the embankment. The ground level in one area in the north-east quadrant appears to occur a few feet below elevation 880 and thus

Department of Highways, Ontario,
January 12th, 1961,
Page 4.

Side Slopes (continued)

the embankment height might exceed 14 feet; however this is based on rather limited topographical information. If the ground level is below elevation 880, additional measures such as the addition of a small berm or the flattening of the slope would have to be considered. Rip-rap protection would be required on the toes of slopes occurring below high water.

We trust that this letter, which was written by Mr. F. J. Heffernan and checked by Mr. J. L. Seychuk, gives all the information required on the stability of the relocated embankments. If, however, we can be of any further service, please call us.

Yours very truly,

GEOCON LTD



J. L. Seychuk, P. Eng.,
District Soils Engineer.

JLS/dw
S7175

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HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7632

Rexdale, Ontario,
October 6th, 1960.

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1425 WEST PENDER ST.
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TEL. MU. 1-8926

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations,
Proposed Ghost River Bridge,
W.P. 73-60,
Highway 101,
Matheson, Ontario.

Dear Sirs:

This letter accompanies our detailed report covering the investigation carried out for the above site.

We find that the site is covered by soft to firm varved silt and clay on the west bank and soft clayey silt on the east bank, the thickness of each stratum ranging from 10 to 30 feet. The varved silt and clay and the clayey silt are underlain by very loose to loose silt. A stratum of compact sand, gravel and boulders or glacial drift underlies the silt and generally covers limestone bedrock.

The design of the embankments are discussed in the report and the recommended slopes for adequate stability are shown.

It is recommended that the proposed bridge structure over Ghost River be founded on end bearing timber or steel piles driven to bedrock or to practical refusal in the sand, gravel, and boulder stratum. If timber piles are adopted they should be provided with shoes to minimize tip damage in the glacial drift.

We believe that this report gives all the information for safe and economical foundation and embankment design. If we can be of any further service, however, please do not hesitate to call us.

Yours very truly,

GEOCON LTD

V. Milligan
V. Milligan, P. Eng.,
Assistant Chief Engineer.

VM/dw
S7124

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HALIFAX

MONTREAL

TORONTO

VANCOUVER

S7124
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED GHOST RIVER BRIDGE
W.P. 73-60
HIGHWAY 101
NEAR MATHESON ONTARIO

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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario (by letter of authorization dated July 22nd, 1960) to investigate and report on the soil conditions at the site of the proposed Highway #101 bridge over Ghost River, some 30 miles east of Matheson, Ontario.

The object of the investigation was to determine and interpret the soil conditions as they affect the foundations of the bridge and the stability of the highway approach embankments.

A description of the procedure, site and geology and detailed accounts of the soil and water conditions at the site (D.H.O., W.P. 73-60), are given in Appendix I of this report. The results of the in-situ and laboratory testing are shown on the Office Reports on Soil Exploration in Appendix I and on the figures in Appendix II. A drawing showing the borehole locations, together with the inferred soil stratigraphy is located in the pocket at the rear of this report.

SUMMARIZED SOIL CONDITIONS

The granular fill which comprises the embankment on the east side of the bank ranges in thickness from 3 or 4 feet on the north slope to 10 feet on the south slope. Underlying this fill on the east bank is a stratum of clayey silt with some organic matter which extends down to about elevation 858. This clayey silt is of soft to firm consistency, though generally soft and there is no noticeable increase in strength with depth. On the west bank, a stratum of soft to firm varved clay and silt underlies the road sub-base and extends down to elevation 858. From the results of the strength tests, it is estimated that the stratum has been preconsolidated under a pressure of about 500 pounds per square foot in excess of the existing overburden pressure. Underlying both the varved

silt and clay and the clayey silt is a stratum of very loose to loose light grey silt. The upper boundary of the stratum is reasonably level but the lower boundary slopes from east to west from elevation 849 at borehole 4 to elevation 820 at borehole 2. A layer of glacial drift over 15 feet in thickness was encountered in three of the boreholes. The glacial drift consisting of sand, gravel and boulders is generally of a loose to compact nature. Sound limestone bedrock was encountered underlying this glacial drift in one borehole and the light grey silt in another.

DISCUSSION

General

It is proposed to replace the existing one-lane steel Bailey bridge which spans the Ghost River with a larger and more suitable structure. The existing bridge is located on Highway 101, some 30 miles east of Matheson, Ontario at station 41+00, County of Harker. The site is shown on Drawing S7124-1, located in the pocket at the rear of this report. The new bridge will be two lanes wide and will be placed at a higher elevation than the present bridge in order to flatten the highway grades immediately east and west of the bridge location.

On Profile No. C-1741, the proposed grade at the bridge is given as about 894. However, it is understood that this grade line may be altered in the light of the soil conditions at the site. The minimum elevation of the bridge deck for hydrology consideration is about elevation 890. It is further understood that the bridge will probably be a three-span simply supported bridge with timber trestle approaches.

General (continued)

For stability computations the embankment top width has been taken as 40 feet for a two lane highway with side slopes of 2 horizontal to 1 vertical.

Slope Stability

It is known from local information that the high water level in Ghost River is about elevation 883.0 and that low water level is at 870.8. From stability analyses, it was found that an effective stress analysis considering rapid drawdown is the critical case. The water table in the ground is probably near elevation 883.0 for much of the year so that the steady seepage case could be almost as critical as the rapid drawdown case.

In the analyses, the ground water level has been taken as intersecting the front slope of the bank at about elevation 880 on the west side and about elevation 882 on the east bank. These elevations correspond closely with the water level as observed in the boreholes. All analyses considered the elevation of the embankment to be 890.0.

A conventional effective stress analysis for the west bank, with the embankment front slope beginning at station 40+55 as shown on Drawing S7124-2, gave a factor of safety of 0.72 including an allowance of 10 percent for end effects. However, because of the large central angle for this circle, it is believed that the conventional method is not sufficiently accurate. The rigorous method, which takes into account the forces between the slices, gave a factor of safety of 1.1 including 10 percent for end effects on the same circle. For the embankment front slope beginning at station 40+45, the factor of safety by the rigorous method is 1.25, including 10 percent for end effects, and

Slope Stability (continued)

this is considered an adequate long term factor of safety for the rigorous method. The slope is about 16 degrees or approximately three quarters of ϕ' , which is a reasonable value for a moderately shallow circle with the water table in the slope near or at the surface. The corresponding factor of safety for the total stress analysis was greater than 2. The large difference between the factors of safety obtained by effective and total stress analysis is due partly to the slight overconsolidation of the silt and clay stratum and partly to the varved nature of the material. In the undrained shear strength, the silt layer of the varve adds greatly to the 'undrained' strength while the effective angle of shearing resistance, ϕ' , seems to be governed mainly by the clay layer.

On the east bank, with the embankment front slope starting at station 42+23, a rigorous effective stress analysis gave a factor of safety of 1.1, including an allowance of 10 percent for end effects. This factor of safety is considered too low and it is suggested that the embankment slope be made to begin at station 42+33 to ensure adequate protection. The slope of the bank would then be about 16 degrees. The corresponding factor of safety for the total stress analysis is above 1.5.

It is recommended that the slope down to the river along the south east side of the road be limited to 16 degrees for long term stability. The total stress analysis for the bank shown on Drawing S7124-2 has a factor of safety above 1.5. Drawing S7124-2 shows the recommended slopes on the east and west banks.

If it is considered advantageous to raise the embankment height from elevation 890 to elevation 894, it would be necessary to relocate the road some 15 feet northwards for adequate long term

Slope Stability (continued)

stability of the lateral sections. Also it would be necessary to move the embankment some 15 feet from the river on either side in order to maintain the slope at 16 degrees.

Because of the high liquidity index of the silt layers, above 1.0 on the west bank, and the sensitivity of the varved silt and clay, it is recommended that the piles through the west bank be driven before the additional fill is placed.

It is further recommended that the slopes be provided with a gravel blanket at least 3 feet in thickness to prevent sloughing due to frost. This gravel blanket should be provided with a rip-rap cover up to about elevation 883, the extreme high water level, in order to prevent scour of the banks and a consequent reduction in slope stability.

Foundations

Two types of foundations for the proposed structure have been examined. These are friction piles and end-bearing piles driven to bedrock or to adequate resistance in the sand, gravel and boulder stratum. Because of the low shear strength of the upper most stratum on either bank, spread footings were not considered.

Considering the low shear strength of both the varved clay and silt on the west bank and the clayey silt on the east bank, an adhesion between the pile and the soil of about 300 to 400 pounds per square foot is assumed along the pile shaft. Based on past experience with similar silts, an adhesion through the light grey silt of 300 to 400 pounds per square foot should also be taken for design. Therefore, a 40 foot timber pile, 8 inches in average diameter and founded wholly

Foundations (continued)

above the glacial drift stratum would have an allowable capacity of about 10 tons if a factor of safety of 2 is applied. This may be satisfactory for supporting timber trestle approaches; however, it appears to be an uneconomical solution for carrying the abutment load. Furthermore, settlement of these piles could be considerable. End bearing piles, therefore, offer the most feasible solution.

Because of the high boulder content of the glacial drift stratum, steel piles, either H piles or closed end tubular piles, are considered the most suitable type for either penetrating the boulder stratum to bedrock or for deriving sufficient resistance within the glacial drift stratum. However, because of the high ground water table and the relatively short distance to an adequate bearing stratum, end bearing timber piles may offer an economical alternate solution. It probably will not be necessary to splice the timber piles under the central piers.

End bearing piles would be about 40 to 55 feet in length. For example, a 10 inch average diameter timber pile and a 12 inch diameter pipe pile of 3/8 inch thickness are considered here. It is estimated that if a Vulcan No. 2 hammer is used which has a rated energy of 7250 ft. lb. the timber pile will have an allowable load of 15 tons at a set of .25 inches per blow and the steel pipe pile will have an allowable load of 45 tons at a set of .12 inches per blow.

Settlement

Computations were carried out to determine the probable total consolidation settlement that will occur at the approach embankment due to consolidation of the underlying clay stratum. If the embankment is raised to elevation 890, the load imposed will be

DISCUSSION (continued)

7.

Settlement (continued)

about 600 pounds per square foot. The settlement of the embankment will be of the order of 3 inches based on assumed consolidation characteristics. This relatively low value for embankment settlement is due partly to the light loading, which is roughly equivalent to the probable past consolidation pressure and partly to the moderate total thickness of compressible clay layers. The silt layers of the varves and the underlying silt stratum have low compressibility characteristics.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by about 40 feet of soft clay and loose silt overlying compact glacial drift and limestone bedrock.
2. The water level at the site during the investigation was generally about elevation 881, that is, within a few feet of ground surface.
3. Stability analyses show that the slopes must be limited to about 16 degrees for long term stability. For grades higher than elevation 890, relocation of the roadway towards the north will be necessary. ✓
4. The slopes should be covered with a granular blanket and rip-rap cover to prevent erosion and scour.
5. It is recommended that the bridge structure be carried on end bearing timber or steel piles driven to practical refusal in the glacial drift stratum or on bedrock.
6. Settlements of the approach embankment will be moderate.

PERSONNEL

8.

The field work was carried out under the supervision of Mr. J. Wong. This report was written by Mr. F. J. Heffernan and reviewed by Mr. V. Milligan.

FJH/dw
S7124



F. J. Heffernan,
Soils Engineer.

APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

PROCEDURE

The field work for this investigation was commenced on July 25, 1960 and completed on August 17, 1960. A total of 5 boreholes with adjacent dynamic penetration tests were put down at the site using a skid-mounted machine drill rig. The borings were put down in HX and BX sizes to a maximum depth of about 90 feet. Bedrock was proved in two boreholes by core drilling in AXT size to a maximum depth of 10.0 feet. In-situ vane testing was carried out in 3 of the boreholes.

The location of the boreholes together with the inferred soil stratigraphy are shown on Drawing S7124-1 located in the pocket at the rear of this report. A detailed log of each boring is given on the Office Reports on Soil Exploration in this Appendix.

The laboratory testing of soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto and the results are shown on the Office Reports on Soil Exploration in this Appendix and on the Figures in Appendix II. The samples remaining after testing will be stored until March 1st, 1961, at which time you will be contacted regarding their disposal.

Elevations are referred to Geodetic Datum and were obtained by running levels from a nearby Department of Highways, Ontario bench mark. The elevation of this bench mark, which is located on a pine stump 112 feet north of station 40+82, was given as 888.22. The longitudinal cross section was supplied to us by the Department of Highways, Ontario, while several lateral cross-sections were taken by our field engineer along the critical south-east side of the road.

SITE AND GEOLOGY

The intersection of Ghost River and Highway No. 101 is located some 30 miles east of Matheson, in the County of Harker,

Ontario. The region, which is generally covered by heavy bush, has an undulating topography. The principal drainage is to the north with Ghost River flowing into Lake Abitibi.

From available geological information and local reconnaissance, it is known that extensive deposits of varved clay and silt exist and that sand deposits overlie this silt and clay in the region west of Ghost River. These post glacial deposits are generally underlain by a stratum of glacial drift.

Bedrock in the region is generally of igneous nature, though a narrow band of older sedimentary rocks are found along the Destor-Porcupine fault which runs east-west across the northern portion of Harker County. Highway No. 101 generally follows the course of the fault in the region of the site investigated.

SOIL CONDITIONS

The principal soil strata encountered by the borings are as follows:

Fill

A layer of fill was encountered at borehole 1 and 2 and it is about 6 inches in thickness. The fill material consisted of gravel in a matrix of silty clay at borehole 1 and sand at borehole 2.

Firm Brown Silty Clay

Underlying the sand fill at borehole 1 and extending to the surface at borehole 4 is a layer of brown silty clay. The thickness of the layer is 8.0 feet at borehole 1 and 9.0 feet at borehole 4. This material was not encountered at boreholes 2 and

Firm Brown Silty Clay (continued)

3. The brown clay in borehole 1 is similar to the underlying varved silt and clay except that the varving is not so pronounced in the brown clay. It is believed that the brown colour is the result of oxidation and desiccation following a temporary drawdown of the water table. The clay in borehole 4 overlies the dark grey clayey silt which is probably of more recent geological origin and this clay may be a fill.

Atterberg limits run on a sample from borehole 4 gave a liquid limit of 69 per cent, plasticity index of 37 per cent, and a natural moisture content of 42 per cent.

Wet unit weight determinations gave an average value of 116 pounds per cubic foot. A wet unit weight of 115 pounds per cubic foot and a submerged unit weight of 52 pounds per cubic foot have been used for design purposes.

Soft to Firm Varved Dark Grey Silty Clay and Light Grey Clayey Silt

Underlying the brown silty clay in borehole 1 and the sand fill in borehole 2 is a stratum of varved dark grey silty clay and light grey clayey silt. The thickness as encountered at the boreholes was 15.0 feet at borehole 1 and 29.5 feet at borehole 2. This stratum was not encountered at boreholes 3 and 4, the boreholes on the eastern side of Ghost River. The separate layers of the varve consist of dark grey silty clay and light grey clayey silt. These layers are generally distinct and horizontal throughout. The dark grey silty clay layers are generally about $\frac{1}{2}$ inch in thickness throughout the depth while there is a trend for the silt layers to increase in thickness with depth from about $\frac{1}{8}$ to 2 inches. The clay layers comprise approximately 30 to 40 per cent of the total thickness of the stratum.

Soft to Firm Varved Dark Grey Silty Clay
and Light Grey Clayey Silt (continued)

The liquid limits obtained from the dark grey silty clay varied between 69.9 and 71.0 while the plasticity index ranged generally from 42.8 to 48.2. Two Atterberg limit tests were run on samples of the silt bands resulting in liquid limits of 40.8 and 28.0 and plasticity indices of 18.0 and 5.9 respectively. On Figure 1 of Appendix II, typical values of the Atterberg limits are plotted on the plasticity chart together with the Casagrande "A" line. The dark grey silty clay may, therefore, be classified as an inorganic clay of high plasticity while the light grey clayey silt may be classified as an inorganic clay of low plasticity.

The liquidity index was computed to have an average value of 1.00 for the dark grey silty clay and 1.15 for the light grey clayey silt.

A typical grain size distribution curve for each of the separate layers is plotted on Figure 2 of Appendix II. These show that the dark grey silty clay contains about 92 per cent clay sizes and the light grey clayey silt contains about 25 per cent clay sizes.

The activity, which is the ratio of plasticity index to the percentage of grain sizes less than .002 millimeters is about 0.5, which is in the inactive zone for clays.

The shear strength of the stratum was determined by in-situ field vane testing and undrained triaxial compression tests carried out in the laboratory. These shear strengths are plotted against elevation on Figure 6 of Appendix II. The shear strength line used for design is also given on this figure. It may be seen

Soft to Firm Varved Dark Grey Silty Clay
and Light Grey Clayey Silt (continued)

that the shear strength increases with depth from a value of about 400 pounds per square foot at elevation 880 to about 700 pounds per square foot at elevation 861. The plot of shear strength versus elevation suggests that the stratum has been only slightly preconsolidated.

By using a preconsolidation load of the order of 500 pounds per square foot and taking the water table to be at about elevation 881, it is possible to compute for the portion of the shear line, between about elevation 880 and 861, a $(c/p)_n$ value of 0.31. This value is in good agreement with the measured $(c/p)_n$ figure for clays of similar plasticity.

The sensitivity of the varved clay and silt, as determined by in-situ vane testing in the undisturbed and remoulded states ranges from about 2 to 5.

No undrained triaxial compression tests with pore pressure measurement were carried out on samples of the varved silty clay and clayey silt. However, from experience with similar material and from the results of tests on similar material at Pain Killer Creek, it would be conservative to take C' as zero, and ϕ' as 22 degrees for long term stability calculations.

Soft Dark Grey Clayey Silt

Extending to the surface at borehole 3A, and underlying clay fill at borehole 4, is a stratum of dark grey clayey silt with some organic material. The thickness is 16.2 feet at borehole 4 and 21.0 feet at borehole 3. The thickness of the stratum at penetration test 3 is inferred to be about 20 feet. The organic

Soft Dark Grey Clayey Silt (continued)

material contained in the clayey silt generally consists of sound wood chips. Occasional deposits of sandy silt were also encountered within the stratum.

Atterberg limits run on a sample of the clayey silt resulted in a liquid limit of 37.5 and a plasticity index of 21. The corresponding moisture content was 27.5 per cent.

Wet unit weight determinations gave an average value of 111 pounds per cubic foot at a corresponding moisture content of about 37 per cent. For design purposes a wet unit weight of 110 pounds per cubic foot and a submerged unit weight of 47 pounds per cubic foot have been used.

Three typical grain size distribution curves for this stratum are shown on Figure 3 of Appendix II. Two curves are typical of the clayey silt portions, while the curve for the coarser material is probably typical for the sections of high sand size concentration.

The shear strength of the stratum was determined by undrained triaxial compression tests carried out in the laboratory. The values obtained are plotted versus elevation on Figure 7 of Appendix II. An average strength of 500 pounds per square foot has been used in design.

Though the average undrained shear strength as measured in the laboratory is considerably less than 500 pounds per square foot, this figure is still considered to be a conservative estimate of the in-situ shear strength. It is believed that samples from this stratum were partly disturbed due to the sand and organic matter content.

Soft Dark Grey Clayey Silt (continued)

No undrained triaxial compression tests with pore pressure measurements were carried out on samples of the dark grey silt. However, conservative practice would suggest taking C' as zero and ϕ' as 25 degrees for long term stability computations.

Very Loose to Loose Light Grey Silt

Underlying the varved grey clay and silt in boreholes 1 and 2 on the west bank of the river and the dark grey clayey silt in boreholes 3 and 4 on the east bank of the river is a stratum of light grey silt. The thickness of the stratum ranges from 9.8 feet at borehole 4 to 36.2 feet at borehole 2. Grain size distribution tests were run on three typical samples from this stratum and the results are shown on Figure 5 of Appendix II. This shows that the material consists predominantly of coarse silt sizes. The material is considered to derive the greater part of its undrained strength from the frictional component.

Standard penetration tests carried out in the stratum gave "N" values ranging from zero to 17 with an average value of 5. However, it is considered that "N" values may be unreliable in uniform silts and thus the relative density of the stratum is inferred to be very loose to loose.

A liquid limit of 24 and a plasticity index of 7 was obtained from one test on a more cohesive sample of the stratum.

The results of undrained triaxial tests with pore pressure measurements are shown on Figure 8 of Appendix II. The angle of effective stress, ϕ' , as determined by these tests is about 43 degrees. However, if the deviator stress was modified to take into account the effect of dilatancy during shear, it is con-

Very Loose to Loose Light Grey Silt (continued)

sidered that the residual angle of effective stress would be about 35 degrees. For design purposes, it is recommended that C' be taken as zero and ϕ' as 35 degrees.

Loose to Compact Sand, Gravel and Boulders

Underlying the light grey silt in boreholes 2, 3A and 4 is a stratum of sand, gravel and boulders. This stratum was not encountered in borehole 1. Boreholes 2 and 4 were probably completed within this stratum. However, where the stratum was encountered it had a minimum thickness of 15.5 feet as encountered in borehole 3A. The proportion of boulders varied greatly both in the horizontal and vertical direction. Occasional sand pockets were found within the stratum. It is believed that this material is a glacial drift.

The results of grain size distribution tests on samples from this stratum are shown on Figure 6 of Appendix II. These are not typical of the stratum as a whole as they do not take into account the many cobbles and boulders present in the stratum which cannot be recovered in soil sampling.

From the results of the dynamic penetration tests the relative density of the material is estimated to be loose to compact.

Bedrock

Bedrock was encountered below the light grey silt in borehole 1 and below the sand, gravel and boulder stratum in borehole 3A. It was proved by core drilling in AXT size in boreholes 1 and 3A for depths of 10.1 and 5.2 feet respectively. The bedrock is com-

Bedrock (continued)

posed of grey limestone and from visual examination of the core, and from the high core recovery obtained, it is believed that the limestone is generally sound.

WATER CONDITIONS

The water level as observed in boreholes 1, 2 and 3 ranged from elevation 880.2 at borehole 1 to elevation 881.7 at borehole 3. At borehole 3, a slight artesian condition was observed when the borehole penetrated the sand, gravel and boulder stratum at 45 foot depth, and the water level in the casing was observed to rise 4.5 feet above ground level overnight.

3AL
$$\begin{array}{r} 4.5 \\ 877.1 \\ \hline 881.6 \end{array} \checkmark$$

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance, Blows/ft.</u>
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7124 BORING # 1 DATUM GEODETIC CASING HX & BX
 BORING DATE JULY 27 & 28, 60 REPORT DATE SEPTEMBER 2, 60 COMPILED BY ATL CHECKED BY E
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

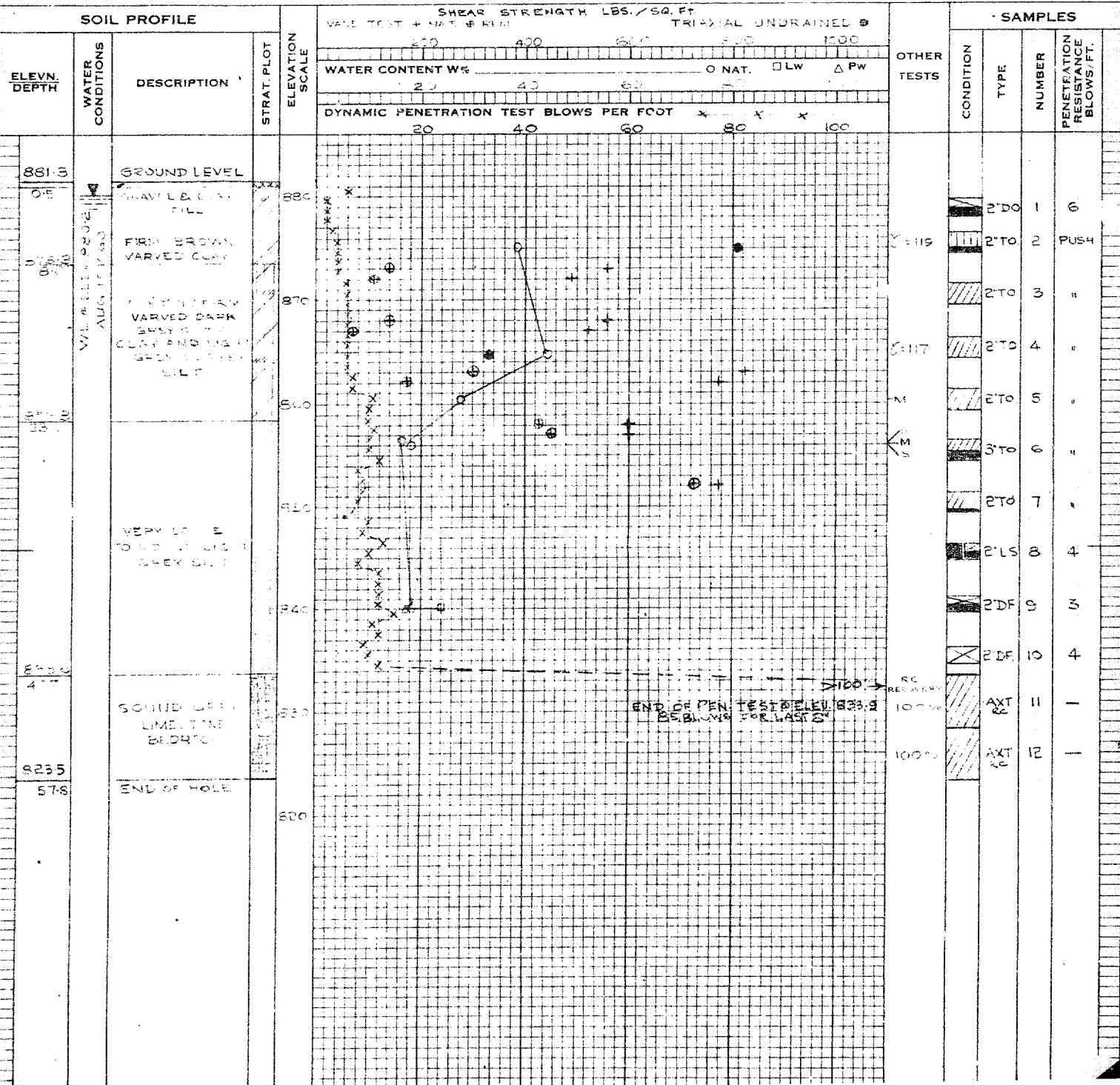
F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION

WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7124 BORING # 2 DATUM GEODETIC CASING HX # 3X
 BORING DATE JULY 29 - AUG 6, 60 REPORT DATE SEPT 6, 60 COMPILED BY AT:DS CHECKED BY FSH
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

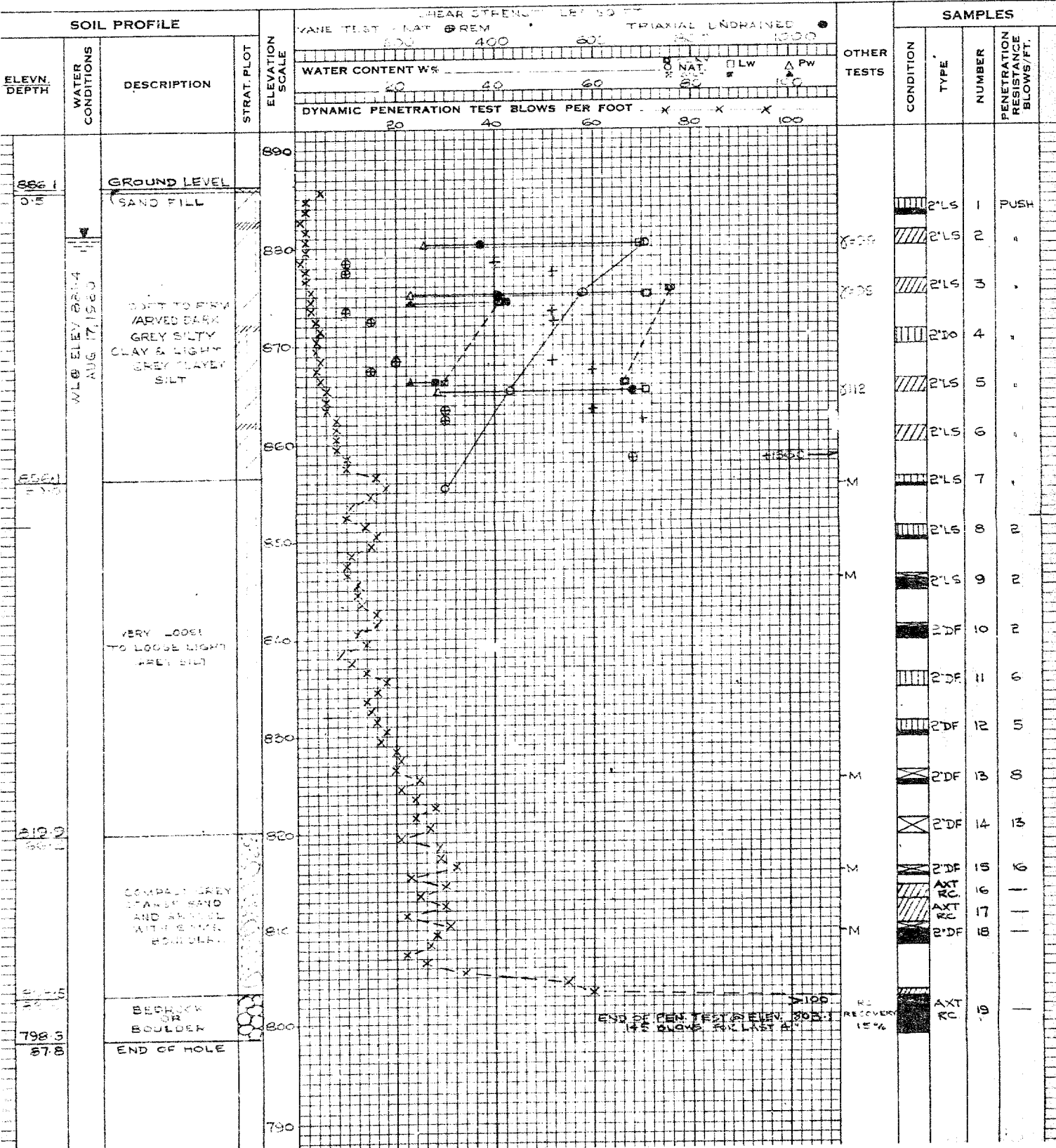


SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS



DISTURBED
 FAIR
 GOOD
 LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE				OTHER TESTS		SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW ▲ Pw	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
8498 0-0		GROUND LEVEL		0-0					
8798 40		LOOSE SAND FIL WITH WOOD CHIPS END OF HOLE		40				1	0
8600 238		PROBABLY SOFT CLAYEY SILT WITH WOOD CHIPS		238					
8420 418		PROBABLY LOOSE LIGHT GREY SILT		418					
8285 450		PROBABLY SAND, GRAVEL AND BOULDERS END OF PEN TEST		450					

OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

DISTURBED
FAIR
GOOD
LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Qc - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

[illegible]

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7124 BORING # 4 DATUM GEODETIC CASING HX # BX
 BORING DATE AUG 15 - 17, 60 REPORT DATE SEPTEMBER 6, 60 COMPILED BY AT & DS CHECKED BY F. J. J.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FCIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
884.2		GROUND LEVEL		890
875.0 9.0		FIRM BROWN CLAY		880
859.0 25.2		LOFTY TO FIRM DARK GRAY CLAYEY SILT W/ WOOD CHIPS		870
843.2 39.0		VERY LOOSE TO LOOSE LIGHT GREY SILT		860
833.2 51.0		BOULDER WITH SOME SAND & GRAVEL		850
		END OF HOLE		840
				830

SHEAR STRENGTH (LB/50 FT)
 VANE TEST - NAT & REM
 200 400 600 800 1000

WATER CONTENT W%
 10 40 60 80 100
 NAT. ELW ΔPw

DYNAMIC PENETRATION TEST BLOWS PER FOOT
 20 40 60 80 100

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT
CS	1	—	
2TO	2	PUSH	
2'LS	3	"	
3TO	4	"	
2'LS	5	4	
2'LS	6	2	
2'LS	7	10	
2'LS	8	2	
AX-RC	9	—	
"	10	—	
"	11	—	
"	12	—	

END OF PEN. TEST ELEV. 842.5
 NO BLOWS FOR LAST 1'

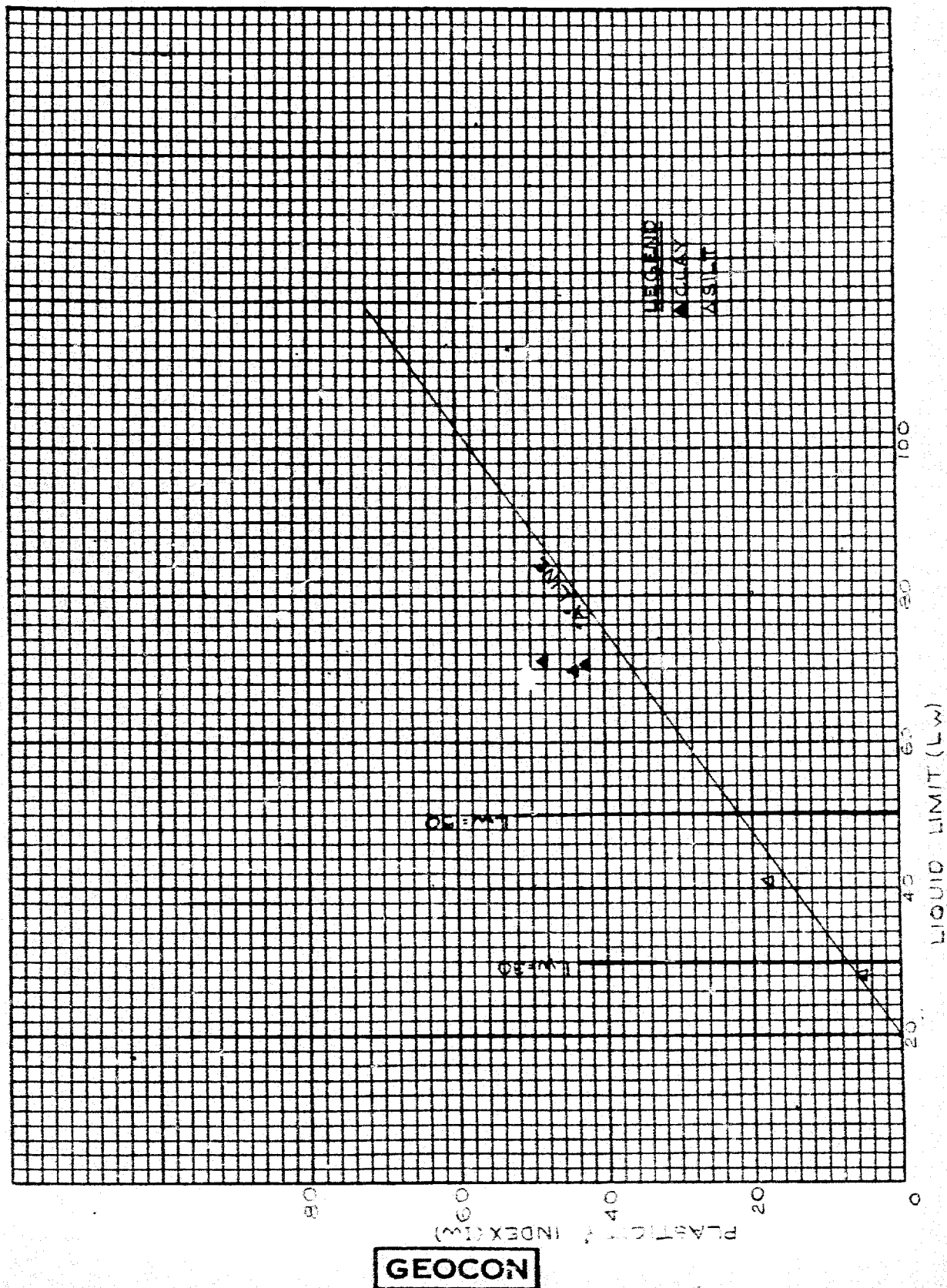
APPENDIX II

FIGURES - LABORATORY TESTING

PLASTICITY CHART

VARVED SILTY CLAY AND CLAYEY SILT

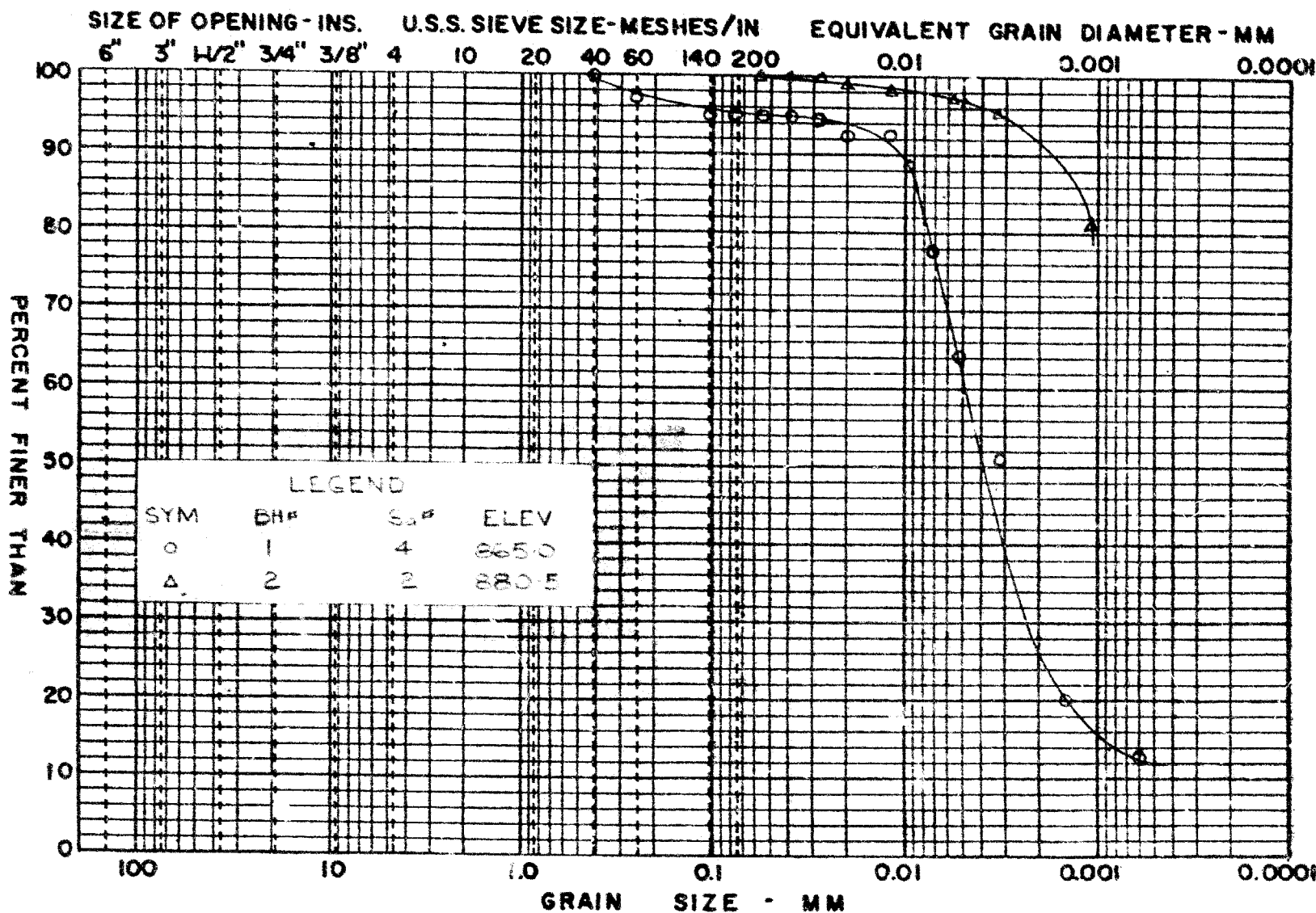
APPENDIX II
FIGURE 1
PROJECT S.7124



GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 2
PROJECT S 7124

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



M.I.T. GRAIN SIZE SCALE

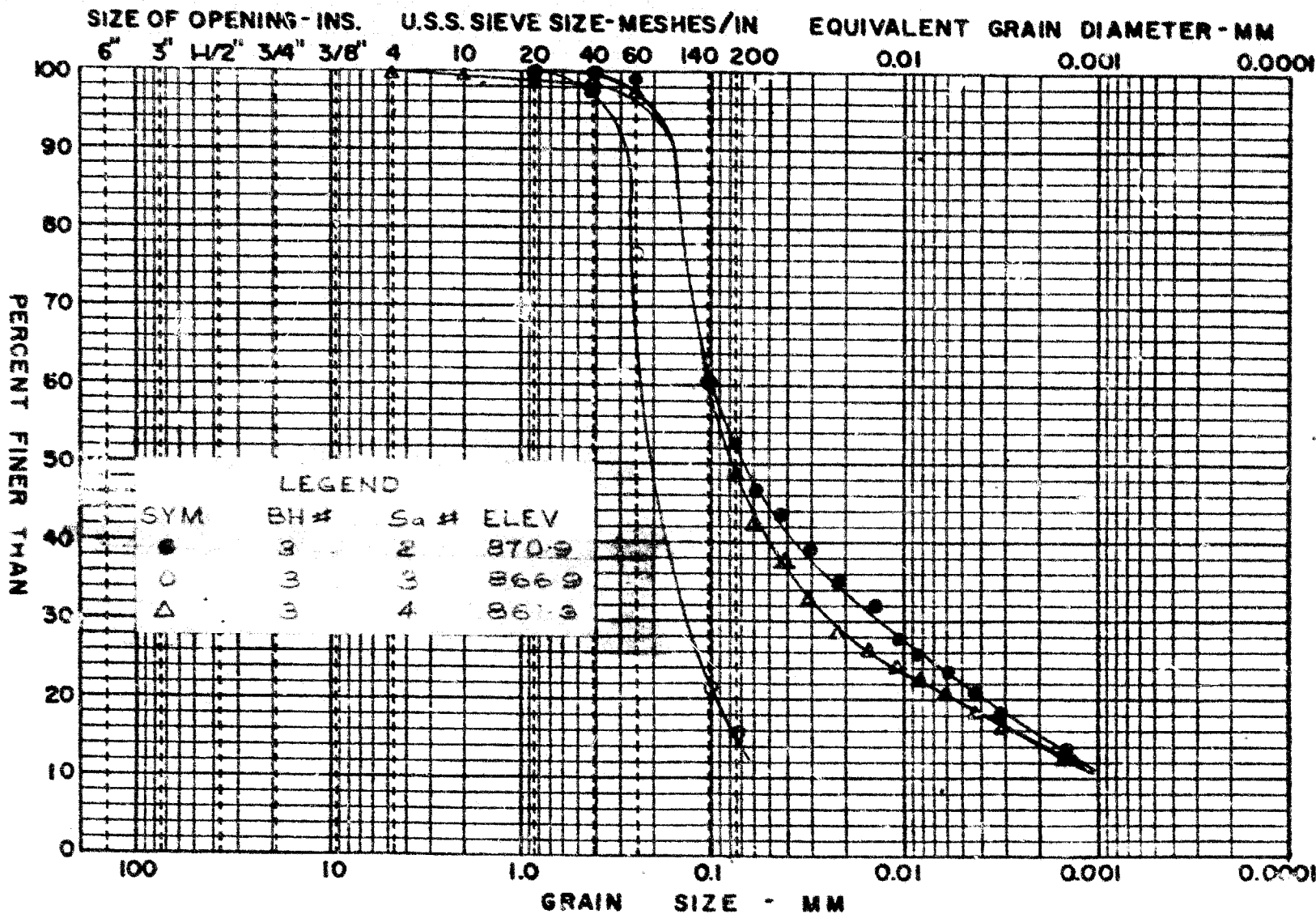
GRAIN SIZE DISTRIBUTION

APPENDIX II

FIGURE 3

PROJECT S7124

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



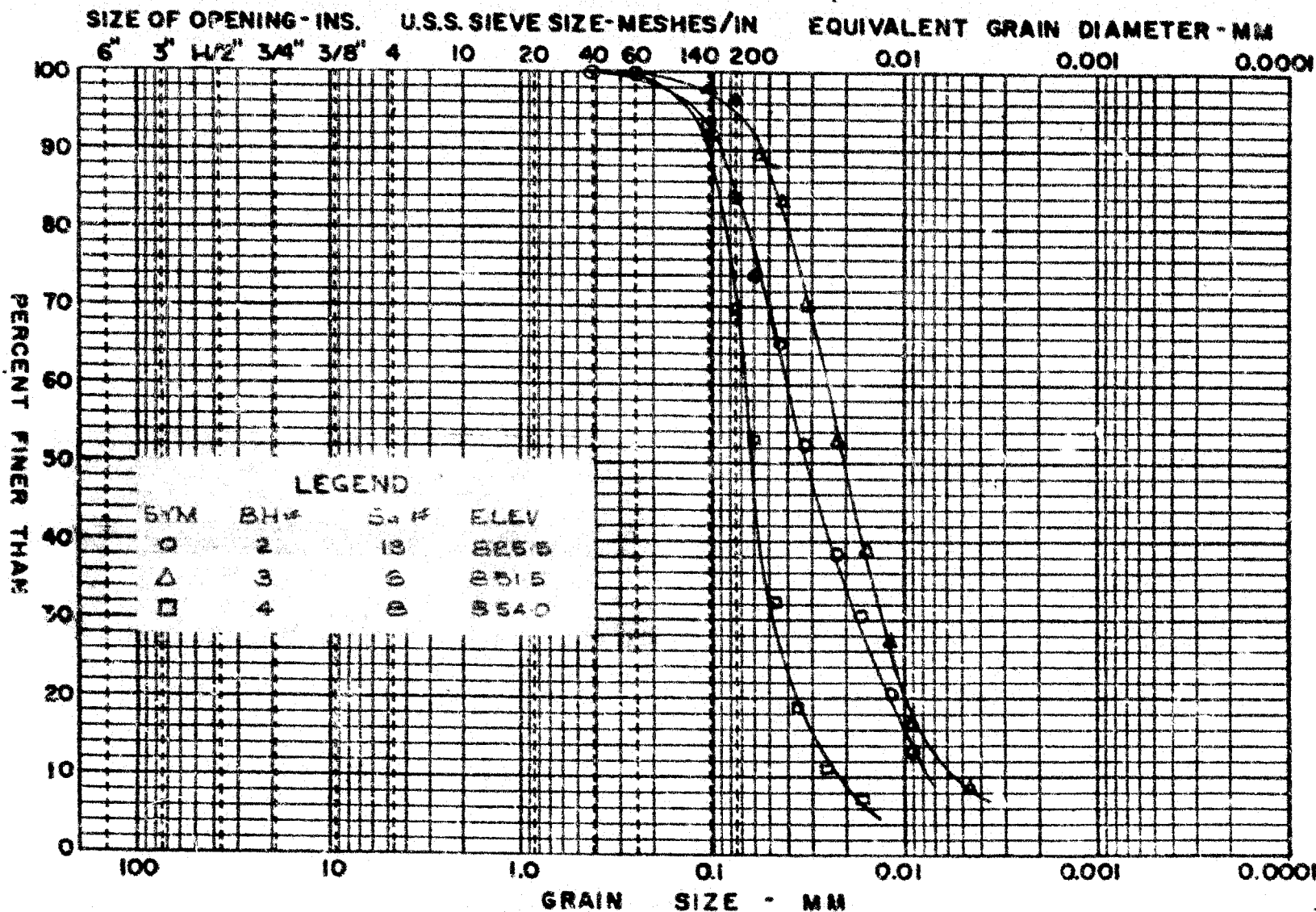
M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX I
FIGURE 4
PROJECT 57124

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



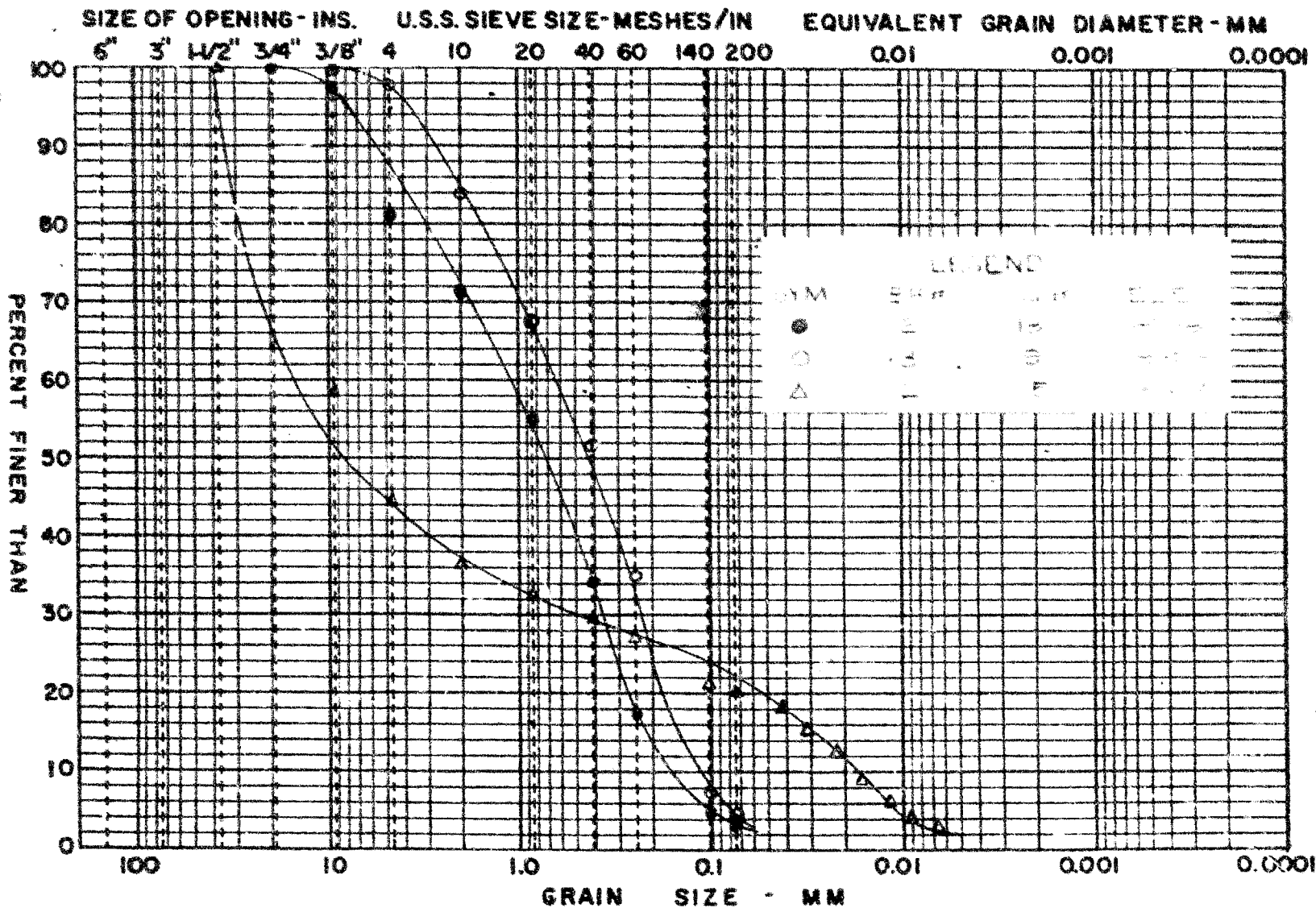
M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 5
PROJECT S 7124

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

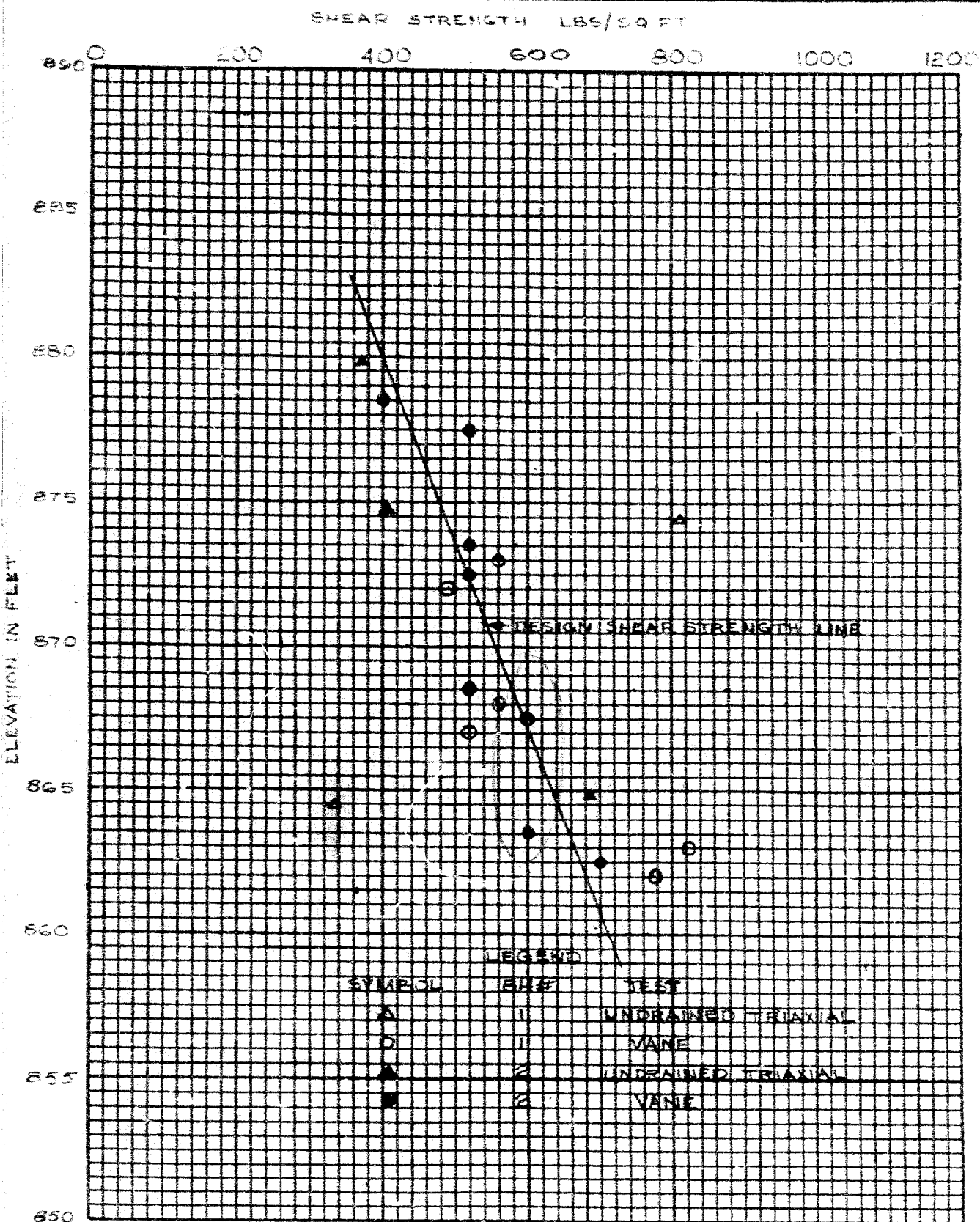


GEOCON

SHEAR STRENGTH VS ELEVATION

VARVED SILTY CLAY AND CLAYEY SILT

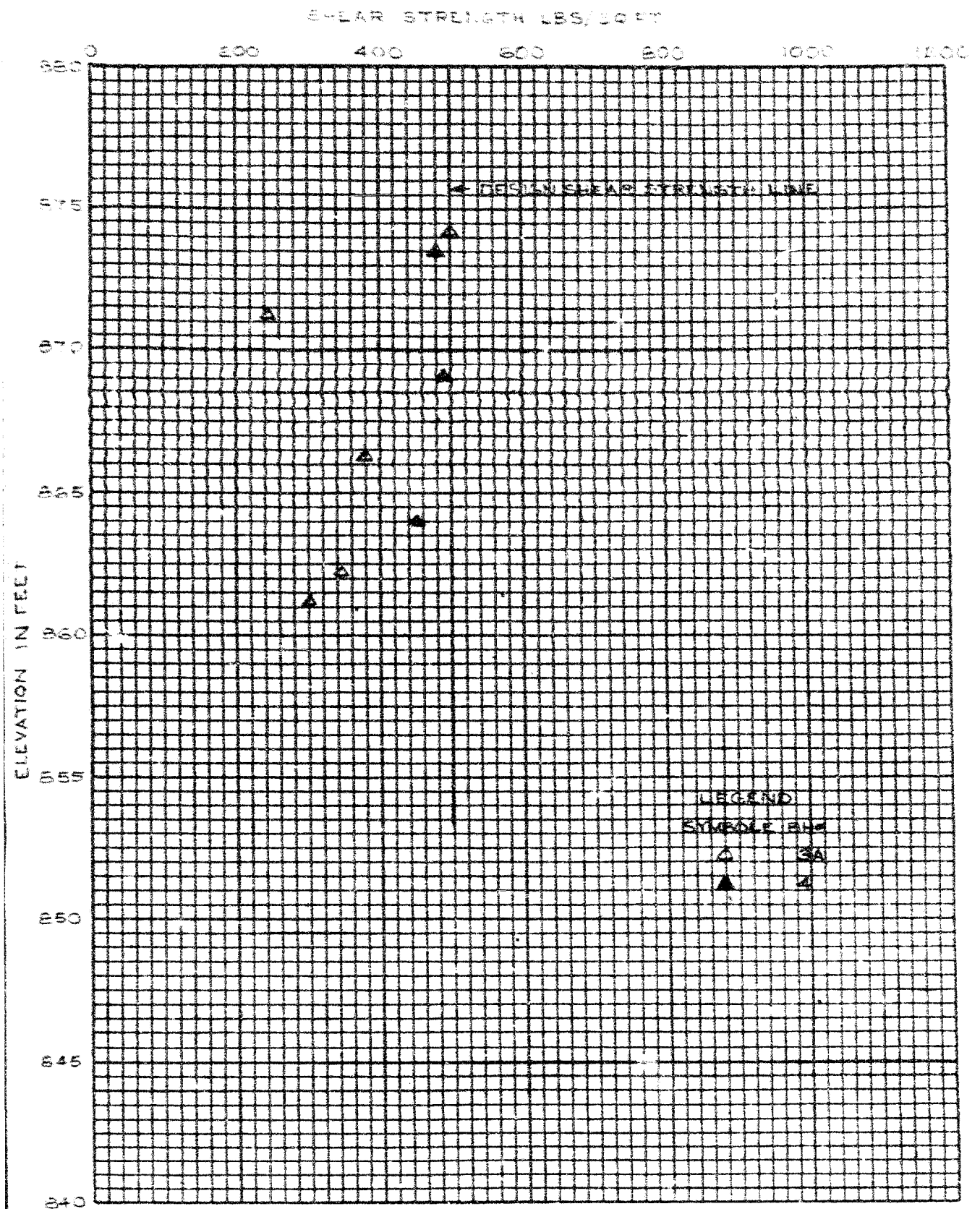
APPENDIX II
FIGURE 6
PROJECT S7124



UNDRAINED SHEAR STRENGTH VS ELEVATION

DARK GRAY CLAYEY SILT

APPENDIX II
FIGURE 7
PROJECT S7124

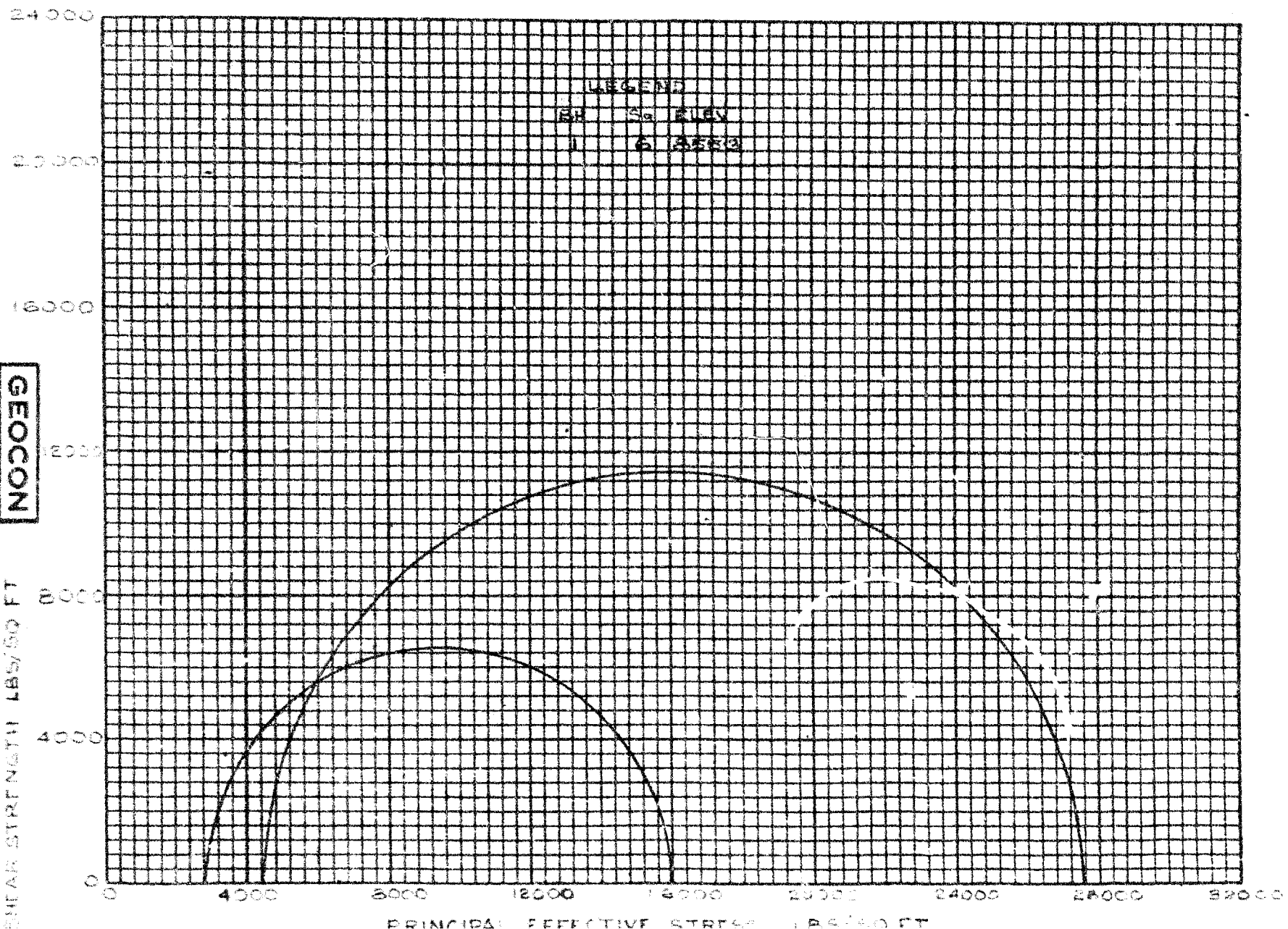


CONSOLIDATED TRIAXIAL COMPRESSION TESTS

WITH POKE PRESSURE MEASUREMENTS

NOTES CIRCLES

APPENDIX II
FIGURE 8
PROJECT S7124



GEOCON

11 09/587 11/07/1975 REVISED

Mr. A. A. Toye,
Bridge Engineer,
Materials & Research Section.

October 27, 1960,
Re: Foundation Report by
Gordon, Limited.

Attention: Mr. D. McDermid

Re: Proposed Great River Bridge
Highway 101 Sta. 41-40
Near Matheson, Ontario.
1073-60

This memo accompanies the foundation report for the above site prepared by Gordon, Ltd. The data, calculations, and recommendations, have been reviewed by the Foundation Section. We are in agreement with the recommendations given in the report with the exception of the acceptability of large displacement type piles. Steel H piles should be used in order to minimize disturbance and the subsequent generation of high pore water pressure which could lead to slope failures during construction.

L. G. Goddard

L. G. Goddard
Principal Foundation Engineer

LSH/tt

C.C. A. A. Toye (4)
B. A. Trengrove
C. B. Ramsay
D. L. Hunter
E. S. Chapman
F. L. Cairns

A. Att
Foundations Office
General Files

Department of Highways

COPY

For the Information of:

Mr. L. G. Soderman,
Principals Soils & Foundation Eng
Department of Highways,
Room 107 Lab. Bldg., Downsview,
Hydrology Section,
November 30th, 1960.

MEMORANDUM TO:

Mr. S. McCombie,
Bridge Planning Engineer,
Downsview, Ontario.

Re: Ghost River on Kings Hwy #101
W.P. 73-60
BN 389

The Soils Branch reports extremely unstable banks and substrate along this river necessitating very long, flat slopes and berms.

The rise of flood has been found to be 13 feet as a perennial occurrence, and the banks are very vulnerable to scour. The river rises fast and carries a considerable amount of debris in the form of tree trunks and such like.

The following recommendations are made in the light of the above conditions.

Length	5-40 foot spans
Skew	29° left
Soffit	to not less than elevation 890.00
Grade	at centre line it is set at about 894.40+

The recommended length of bridge will result in 5 foot ballast walls which could be taken care of by raking piles. The pile bents should be protected by sheathing against damage by ice and debris.

The range of protection should be from elevation 871 to 884.

The above protection applies as follows:

The two inner piers sheathed on both sides. The next two piers on the river side only. Suitable cutwaters should be provided on the two inner piers on the upstream side or better still an additional pile driven in front of the piers and connected to the pier by a suitable sloping member reinforced, say, with an old railway rail.

Bank protection should be as recommended in the soil report.

B. Wilkie

B. Wilkie,
Bridge Hydrology Engineer.

BN/ek

c.c. R. Chapman
L. Soderman

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Mr. B. S. Chapman,
District Engineer,
New Liskeard, Ontario.

Materials & Research Division,
(Foundation Section)

July 19, 1962.

SITE VISIT - JULY 17, 1962
By K. Y. Lo,
Supervising Foundation En

Attn: Mr. E. A. Fletcher, Const. Engr.

Re: Ghost River Bridge - W.P. 73-60
District No. 14.

This is to confirm the verbal recommendations discussed with Mr. E. A. Fletcher, Construction Engineer, New Liskeard District at the site on July 17, 1962.

(a) The existing Bailey bridge is exerting considerable pressure on the sheet piles of the excavation for the pier on the east bank. This pressure should be relieved as soon as possible. The location for a new 60-ft. span Bailey bridge approximately 30' south of the existing one, is suitable from our point of view. This provides a better alternative than the proposal to carry the supports of the present one 20 ft. back on each side.

(b) All permanent slopes should be trimmed to $3\frac{1}{2} : 1$ right up both banks at the site of the new bridge except for the slopes below the piers. This is to be maintained at 2:1 as shown on the design drawing.

(c) In order to increase stability for construction purposes, the rip-rap from the piers down to the river bed should be placed as soon as possible.

(d) No driving of wooden piles for construction purposes should be permitted.

cont'd. /2

Mr. E. S. Chapman, Dist. Engr.
New Liskeard.
Attn: Mr. E. A. Fletcher, Const. Engr.

July 19/62


(e) Recommendations (a) to (c) should be carried out before driving of 'H' piles for the piers.

(f) The recommended sequence of driving of 'H' pile for the piers is shown in the sketch attached hereto.

KYL/WdeF
Attach.

cc: Messrs. H. A. Tregaskes
A. McKim

Foundations office
Gen. Files.


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

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL PHOTOGRAPH

PILE DRIVING SEQUENCE

W.P. 73 - 60

H¹

I¹⁵

H¹⁰

H¹⁹

I²⁷

H²³

H²

I¹⁴

H⁹

H¹⁸

I²⁶

H²²

H³

I¹³

H⁸

H¹⁷

I²⁵

H²¹

H⁴

I¹²

H⁷

H¹⁶

I²⁴

H²⁰

H⁵

I¹¹

H⁶

61-F-211

W.P. [#] 73-60

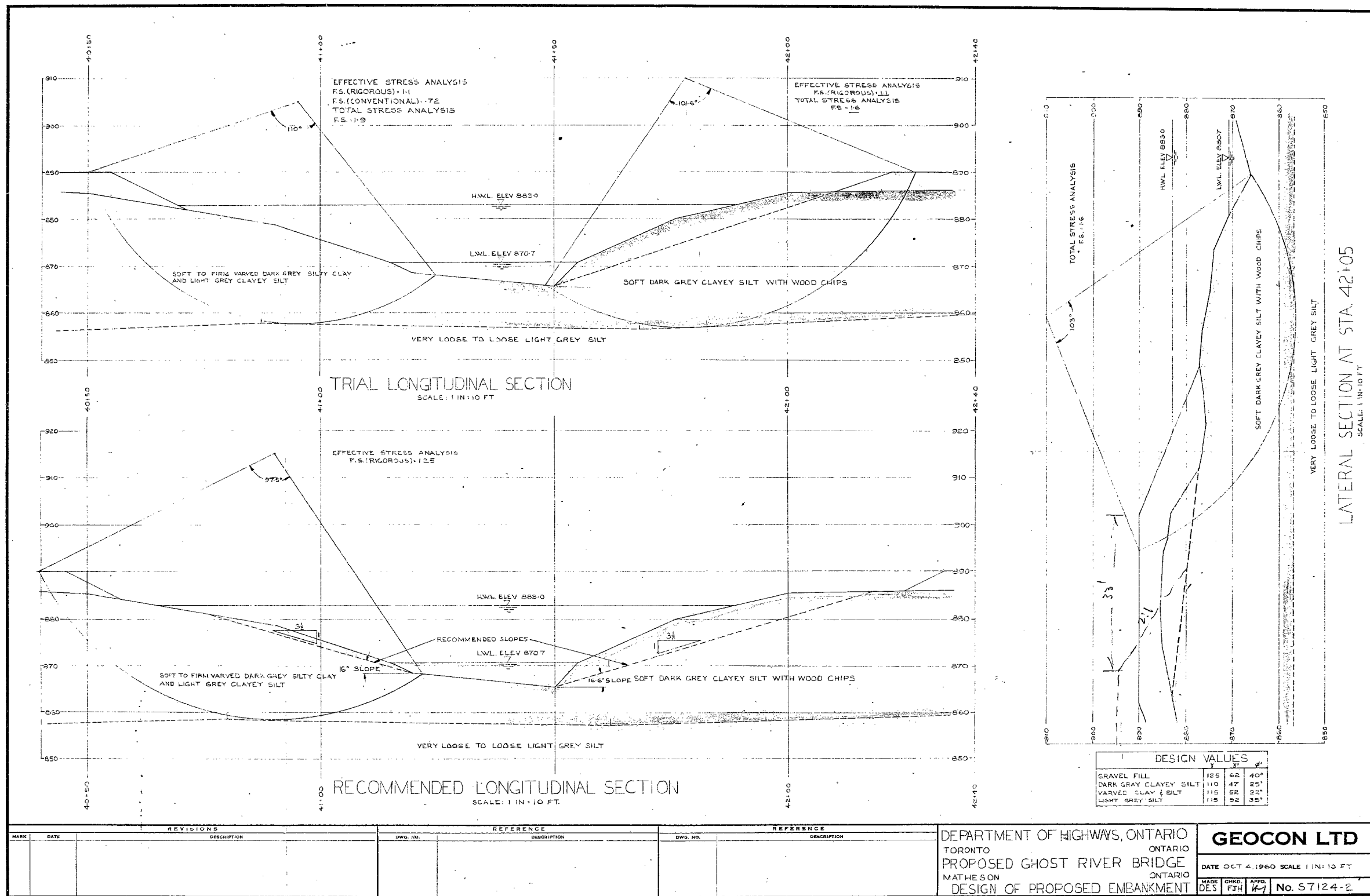
HWY. [#] 101

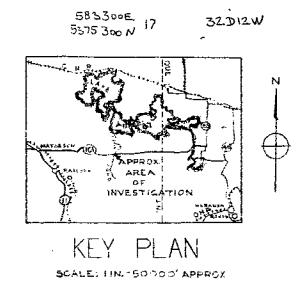
PROP. GHOST

RIVER BRIDGE,

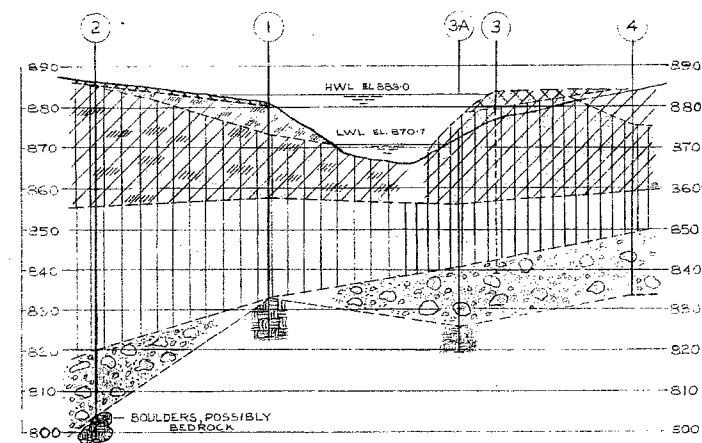
NEAR,

MATHESON





PLAN
SCALE: 1 IN. = 40 FT.



SCHEMATIC SECTION ALONG CENTRE LINE
SCALE: HORIZONTAL: 1 IN = 40 FT. VERTICAL: 1 IN = 20 FT.

LEGEND

- 3 BOREHOLE WITH PENETRATION TEST IN
PLAN
END OF HOLE
END OF PENETRATION TEST

STRATIGRAPHY

- | | |
|--|--|
| | LOOSE GRANULAR FILL |
| | FIRM BROWN CLAY |
| | SOFT TO FIRM VARVED SILTY CLAY & CLAYED SILT |
| | SOFT CLAYEY SILT WITH SOME ORGANIC MATTER |
| | VERY LOOSE TO LOOSE LIGHT GREY SILT |
| | COMPACT TO DENSE SAND, GRAVEL, & EQUIVALENT |
| | SOUND GREY UNCONSOLIDATED BEDROCK |

MARK		DATE	REVISIONS		DWG. NO.		REFERENCE		DWG. NO.		REFERENCE		DEPARTMENT OF HIGHWAYS, ONTARIO		GEOCON LTD	
			DESCRIPTION				DESCRIPTION				DESCRIPTION		TORONTO		DATE 55-12-1000 SCALE AS SHOWN	
													PROPOSED GHOST RIVER BRIDGE		MADE DS	
													MATHESON		CHKD. APPD.	
													BORING PLAN AND SOIL STRATIGRAPHY		No. 3712	