

Mr. S. A. Davis,
Bridge Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 137, Lab. Bldg.

Attention: Mr. L. MacCoshie

September 8, 1966

SEP - 8 1966

FOUNDATION INVESTIGATION REPORT BY:
Gordon, Limited, Consulting Engineers -
Proposed Highway 7 and 15 Underpass,
District 3 (Ottawa) - M.F. 905-64 -

Attached, please find the report for the above structure, prepared and submitted by the consultant, Gordon, Ltd.

We have reviewed the report and have found the factual information adequate and well presented. We are also in agreement with the recommendations contained in the report. It is our opinion that it should be endeavored to place all the footings on bedrock, thus eliminating any possible differential settlements.

Should you, in the course of your further design work, require additional information, or would like to discuss some aspects of the foundation work, please feel free to call on this Office.

A. C. Starnes

A. C. Starnes,
PRINCIPAL FOUNDATION ENGINEER

AMJ/MSF

Attach.

cc: Memo. S. A. Davis (C)
E. J. Trepanier
J. A. Warren
E. C. Miller
C. Scott
C. E. Robertson
J. E. Crueger
A. Watt

Foundations Office
Gen. Files

66-F-2126

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Rexdale, Ontario,
HEAD OFFICE September 6, 1966.

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Department of Highways, Ontario,
Materials and Testing Division,
Highway 401 and Keele Street,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations,
Proposed C.N.R. Grade Separation,
Highway 15 and 7,
Ottawa, Ontario.

Dear Sirs:

This letter accompanies our detailed report on the above investigation.

We find that the site is covered by up to 6 feet of generally compact fill which is underlain by a stratum of very stiff to firm silty clay. The silty clay is underlain by loose to compact silt till then bedrock. The total encountered overburden thickness ranged from about 27 to 30 feet over the site. The actual soil and groundwater conditions are described in detail in the report.

Based on the findings of this investigation, it is recommended that foundations for the grade separation structure be founded directly on bedrock, as discussed. Foundations for retaining walls may be carried in part in the overburden strata or on piles to bedrock as discussed in the

report. A permanent drainage system, and temporary dewatering during construction will be required as discussed.

We believe that this report contains the information required from this investigation. However, should you require further information or if we can be of assistance in the application of the findings to design as the latter develops, we would be pleased if you would give us a call.

Yours very truly,

GEOCON LTD



M. A. J. Matich, P. Eng.,
President.

GEOCON

T7916

REPORT

TC

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED C. N. R. GRADE SEPARATION

HIGHWAY 15 & 7

OTTAWA

ONTARIO

Distribution:

11 copies Department of Highways, Ontario,
Downsview, Ontario.

3 copies Geocon Ltd

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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter and by Purchase Order No. 534821 both dated August 2nd, 1966, to carry out a foundation investigation for the proposed C.N.R. grade separation at Highway 15 and 7 at Bell's Corners near Ottawa, Ontario.

The purpose of this investigation was to determine and interpret the soil and ground water conditions as they affect the design and construction of the proposed C.N. R. grade separation.

SUMMARIZED SOIL CONDITIONS

The site is covered by a layer of generally granular, compact fill about 4 to 6 feet thick. The fill is underlain by very stiff to firm grey brown to grey silty clay about 15 to 23 feet thick. The silty clay is separated from bedrock by a stratum of loose to compact grey silt till about 2 to 10 feet thick. Total encountered overburden thickness ranged from about 27 to 30 feet and the bedrock surface is generally flat over the site. Observed ground water level was variable over the site at the time of the investigation.

DISCUSSION

General

It is understood that it is proposed to construct an underpass along Highway 15 and 7 over which the existing C.N.R. track will

General (continued)

pass. The location of the proposed grade separation is shown on drawing T7916-1 located at the rear of this report. Presently the C.N.R. and Highway 15 and 7 intersect at grade at about elevation 286.5 and at a skew angle of about 57 degrees. Proposed profile grade of Highway 15 and 7 at the grade separation is about 262.5, that is about 24 feet below existing grade.

At the time of writing this report, further data such as proposed structural details and the like are not available to us; however, for the purposes of this discussion it is considered that the grade separation structure will incorporate closed abutments in conjunction with a simply supported superstructure.

Foundations

The soil conditions at the site consist of about 4 to 6 feet of fill underlain by about 15 to 23 feet of very stiff to firm silty clay. Bedrock is separated from the silty clay by loose to compact silt till. Observed bedrock elevation ranges between elevations 255.8 and 257.2 over the site.

Irregardless of the type of structure used a number of general recommendations from a soil mechanics standpoint follow.

Foundations (continued)

(a) It is recommended that all footings founded in overburden be provided with at least 5 feet of earth cover for frost protection purposes; however, where footings are founded on bedrock the frost cover may be reduced to three feet.

(b) The base of footing excavations which expose the silty clay or silt till stratum should be protected by a thin mud mat of lean concrete placed immediately after the excavation has reached final grade since these strata are susceptible to softening from surface water and construction traffic. In this regard, attention is drawn to the sensitive nature of the clay.

(c) It is recommended that backfill to abutments, wing walls or retaining walls where they are involved consist of at least 5 feet of well graded free draining non-frost susceptible clean granular material compacted to at least 95 percent of modified A.A.S.H.O. density. With this provision, a coefficient of lateral earth pressure of 0.4 is recommended for the case where footings to retaining structures

Foundations (continued)

are founded on bedrock. However, should the proposed structure be constructed integrally as a rigid frame a coefficient of lateral earth pressure of 0.5 is recommended for the case of the abutments.

(d) Footings founded on bedrock may be designed for an allowable bearing value of 10 tons per square foot. Abutments, wing walls, or retaining walls founded on bedrock should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to bedrock surface of 0.8. Additional resistance may be readily obtained by keying or doweling into bedrock.

With grade as proposed and with consideration given to frost protection requirements the approximate foundation elevation at the proposed structure would be about 257.5, that is in the loose to compact silt till stratum. In view of the relatively loose condition of the till and the close proximity of foundation elevation to bedrock surface, it is recommended that the abutments for the grade separation structure be founded on bedrock.

Foundations (continued)

For the case where wing walls are used adjacent to the abutments in conjunction with permanent cut slopes forming the sides of the excavation, it is recommended that the foundations for the wing walls also be founded on bedrock due to the proximity of bedrock surface to foundation elevation.

Permanent cut slopes forming the sides of the excavation will involve slopes in the very stiff to firm grey brown to grey silty clay and the loose to compact silt till strata. As the distance from the main structure increases the slopes will occur only in the grey brown portion of the clay stratum. Based on undrained shear strengths a slope of 1 vertical to 2 horizontal is stable with an adequate factor of safety for the maximum height of slope involved in this case.

A consolidated undrained triaxial test was carried out on a sample of the grey brown silty clay and the results are shown on Figure 6 of Appendix II. The test results indicate the effective angle of internal friction ϕ' to be 30 degrees and the effective cohesion c' to be about 0.5 tons per square foot. Based on published data for similar clays in the general area it is apparent that the angle of internal friction ϕ' as indicated above is too high. However, measured angles of internal friction for sensitive clay in the Ottawa area have been found to range

Foundations (continued)

from 17 to 34 degrees, depending on interpretation of tests results involving such factors as strain, failure criterion and the like.

Since the clay in the area is known to be moderately overconsolidated, and considering that toe drainage will be provided to the slope, a finished slope of 1 vertical to 3 horizontal should give adequate factor of safety against instability of the long term. It is recommended that permanent cut slopes however, be provided with a surficial layer of granular material or grass cover as protection against erosion.

Should retaining walls be used to permanently support the sides of the excavation, it is recommended that foundations for the retaining walls be founded directly on bedrock adjacent to the abutments and to a distance from the abutments such that the requirements of the proposed highway grade makes founding directly on bedrock impractical. As the height of retaining walls decreases with distance from the main structure, excavation to bedrock will at some point, not be practical. At this point, the foundations for the retaining walls could as one alternative, be transferred to piles end bearing on bedrock. In this regard, a number of pile types would be suitable such as steel H or tube piles, treated timber piles and precast concrete piles.

Foundations (continued)

As another alternative, consideration could be given to the use of spread foundations within the clay stratum with a suitable joint between the sections on bedrock and clay. However, in view of the variation in strength of the clay with depth the actual allowable bearing value to be used in the design of spread foundations would be dependent on factors such as footing elevation and size. For guidance in preliminary design and for continuous footings less than 10 feet in width, the net allowable bearing value at elevation 270 or below may be taken as 0.75 tons per square foot. At elevation 280 the net allowable bearing value for preliminary design may be taken as 2.0 tons per square foot. If consideration is given to this alternative or to the use of piles the overall stability of the retaining structure should be checked for the final configuration.

It is recommended as discussed above that an adequate construction joint in the retaining wall be provided at the point of transition between footings founded on bedrock and in overburden to allow for differential settlement which will probably occur at this location.

It is recommended that positive permanent subsurface drainage be provided behind retaining structures and around footings to

Foundations (continued)

avoid build up of hydrostatic pressures. For subsurface drainage purposes, perforated or open jointed drainage pipes could be used if embedded in suitable filter material, and lead to a sump or sumps, from which accumulated water would probably have to be removed by pumping since practical gravity drainage does not appear to be available at the site.

Construction

It is assumed that facilities to detour the road traffic and to detour or maintain the rail traffic at the proposed site will be provided prior to construction of the grade separation, or that the existing railway would be maintained by means of a temporary pile supported trestle. The temporary structure would probably be best founded on piles end-bearing on the bedrock surface or within the silt till stratum. A variety of pile types would be suitable in either timber concrete or steel. The choice would be dependent on economic considerations, and perhaps on the suitability of the piles for incorporation into the trestle structure itself.

Construction of the proposed underpass will involve excavation through the very stiff to firm silty clay and the loose to compact silt till. In the case of an open cut or sheeted excavation, dewatering may be achieved by the procedure of pumping from filter

Construction (continued)

equipped sumps maintained in advance of the excavation at all times.

A large inflow of water is not anticipated due to the relatively impermeable nature of the overburden strata.

If unsupported excavations are used during construction, an overall height of embankment of about 30 feet (measured from bedrock surface to top of existing C.N.R. fill) could exist. It is recommended therefore that the sides of the temporary excavations be cut back to 1 vertical to 2 horizontal in the clay.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by about 4 to 6 feet of fill which is underlain by a stratum of very stiff to firm grey brown to grey silty clay from about 15 to 23 feet thick. Bedrock is separated from the silty clay stratum by a stratum of loose to compact silt till. Total encountered overburden thickness ranged from about 27 to 30 feet.
2. The observed ground water level was variable over the site at the time of the investigation.
3. In view of the close proximity of proposed footing elevation to bedrock surface and the relatively loose nature of the till stratum, it is recommended that foundations for abutments and adjacent

- retaining walls be founded directly on bedrock. Foundations for retaining walls may be founded in part in the overburden strata or on piles end-bearing on bedrock as discussed in the report.
4. Construction of foundations carried out in unsupported excavations may involve an embankment height of about 30 feet. Recommendations for temporary side slopes of this excavation are given herein.
 5. Where permanent cut slopes in the clay are required, it is recommended that they be cut to 1 vertical to 3 horizontal, as discussed in the report.
 6. A number of general recommendations from a soil mechanics standpoint pertinent to design and construction are given in the report.

PERSONNEL

The field work for this investigation was carried out under the supervision of Mr. H. M. Padgett. This report was written by Mr. H. L. MacPhie, checked by Mr. D. B. Oates and reviewed by Mr. M. A. J. Matich, P. Eng.

H. L. MacPhie

H. L. MacPhie, P. Eng.,
Senior Soils Engineer.

D. B. Oates

D. B. Oates, P. Eng.,
District Engineer.

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APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

GEOCON

PROCEDURE

The field work for this investigation was carried out between August 3rd and August 12, 1966. A total of four boreholes with accompanying dynamic penetration tests and an additional two dynamic penetration tests were put down using a standard skid mounted machine drill rig.

Sampling of the overburden was carried out using a 2 inch O.D. split spoon sampler and 2 inch Osterberg tubes. The depth of overburden encountered in the four boreholes ranged from about 27 to 30 feet and bedrock was proved for a depth of at least 10 feet, by core drilling in AXT size in three of the boreholes.

A complete log of each borehole and each dynamic penetration test is given on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes and dynamic penetration tests are shown on Drawing T7916-1 located at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Toronto Soil Mechanics laboratory of Geocon Ltd. The results of laboratory testing are shown on the Figures in Appendix II and on the Office Reports on Soil Exploration in this Appendix. All samples remaining after testing will be stored until September 1st, 1967 at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic datum. The bench used is located 45 feet left of Station 195+73 of Highway 15 and 7. The bench mark consists of a nail and washer in the south root of a 2.7 foot diameter maple tree and has a given elevation of 242.68.

SITE AND GEOLOGY

The site is located at the intersection of the Canadian National Railway and Highway 15 and 7, about 3/4 miles west of Bells Corners near Ottawa, Ontario. Presently the Railway crosses Highway 15 and 7 at grade which is at about elevation 286.5. The natural ground level in the area of the intersection is generally flat lying and the Railway and Highway are founded on embankments about 4 feet above the adjacent natural ground level.

Available geological information and previous experience in the general area indicates that the area is covered by glacial marine deposits of stratified sand, silt and clay laid down on the bottom of the Champlain Sea in the Pleistocene epoch. The glacial marine deposits are generally underlain by a relatively thin deposit of glacial till as the unstratified deposit originating from ice-sheet deposition. Available geological information further indicates that bedrock in the area is dolomite or sandstone of the Beekmantown group, Ordovician Period.

The principal soil strata encountered in the boreholes are as follows:

Compact Brown Granular Fill

The site was found to be overlain by a layer of essentially brown granular fill ranging in observed thickness from about 4 to 6 feet. The fill at boreholes 1 and 5 was sand and gravel road fill, however, at borehole 1 the bottom portion of the fill contained ashes and partly decayed wood. The fill at borehole 3 contained some ashes and cinders and some surficial railway ballast was observed to exist in the area of borehole 3. The fill encountered in borehole 6 consisted of sand and gravel.

Three standard penetration resistances determined in the fill layer gave "N" values of 15, 18, and 24 blows per foot. From the results of the above "N" values and from the results of the dynamic penetration tests, it is believed that the fill is generally compact.

Very Stiff to Firm Grey Brown to Grey Silty Clay

Underlying the fill at all borehole locations is a stratum of silty clay ranging in encountered thickness from about 15 to 23 feet. This stratum was fully penetrated in all boreholes. Generally about the top 10 feet of this stratum is a grey brown silty clay with distinct light to dark brown mottles. The mottling is more pronounced near the

Very Stiff to Firm Grey Brown to Grey Silty Clay (continued)

surface of the top 10 feet of this stratum. From visual and tactile observation of the samples recovered, the distributed silt and fine sand content in the grey brown portion of this stratum is higher than that in the underlying grey silty clay.

Examination of air dried samples of the grey silty clay forming the bottom portion of this stratum indicate that fine sand sizes exist in the form of very thin partings, occurring at irregular intervals. The fine sand partings were not observed to occur throughout the grey silty clay and were about 1 inch apart where they were observed to exist.

Five mechanical analysis tests carried out on samples recovered from the silty clay stratum indicated that the samples tested contained from 9 to 15 percent fine sand sizes, from 48 to 57 percent silt sizes and from 28 to 43 percent clay sizes. The resulting grain size distribution curves for the above mechanical analysis tests are shown on Figures 3 and 4 in Appendix II. It is pointed out that generally the mechanical analysis tests were carried out on samples recovered from the grey brown mottled portion of this stratum and that the underlying grey silty clay probably contains more clay sizes than indicated above.

Very Stiff to Firm Grey Brown to Grey Silty Clay (continued)

Ten natural water contents determined on samples from this stratum gave values ranging from 38.6 to 54.6 percent with an average of about 45 percent. Atterberg limits determined on nine samples of this stratum gave values of liquid limit ranging from about 36 to 82 and values of plastic limit ranging from about 15 to 25, the corresponding range in natural moisture content is that given above. The above limits tests indicate that this material is an inorganic clay of medium to high plasticity and the results of the limits tests are shown on the plasticity chart on Figure 2 of Appendix II.

Seven wet unit weights determined on samples from this stratum gave values ranging from about 106 to 113 pounds per cubic foot with an average of 110 pounds per cubic foot.

Seven standard penetration resistances determined in the silty clay stratum gave "N" values ranging from 2 to 16 blows per foot with an average of about 10 blows per foot.

Laboratory vane shear strength tests carried out on undisturbed samples from this stratum gave shear strengths ranging from 650 to greater than 2000 pounds per square foot where the latter value is the upper limit of shear strength that the laboratory vane apparatus can measure. The average of the laboratory vane shear strengths

Very Stiff to Firm Grey Brown to Grey Silty Clay (continued)

was 1360 pounds per square foot. Remoulded laboratory vane shear strengths varied from 50 to 250 pounds per square foot with an average of about 130 pounds per square foot. Based on the results of the remoulded laboratory vane shear strength tests the sensitivity of this stratum ranges from about 6 to 16 with an average of about 10.

In-situ field vane shear strengths were determined in the silty clay stratum at boreholes 5 and 6 and the results gave shear strengths ranging from 575 to 2380 pounds per square foot with an average of about 1300 pounds per square foot. The corresponding remoulded in-situ field vane shear strengths gave values ranging from about 120 to 660 pounds per square foot. Based on the results of the remoulded in-situ field vane shear strengths the sensitivity of this stratum ranges from about 3 to 7 with an average of about 4.5.

Unconfined compression tests carried out on 5 samples from the silty clay stratum gave shear strengths (taken as one-half the unconfined compression strength) ranging from about 700 to 1150 pounds per square foot. One further unconfined compression test carried out on a sample from this stratum gave a shear strength of about 330 pounds per square foot, however, this value of

Very Stiff to Firm Grey Brown to Grey Silty Clay (continued)

shear strength is not believed to be representative of the shear strength of this stratum since laboratory vane and in-situ field vane shear strengths determined adjacent to the location of this test indicate considerably higher values than 330 pounds per square foot.

The results of the shear strength testing with the exception of remoulded shear strengths are shown as a plot of shear strength versus elevation on Figure 1 in Appendix II. Based on the shear strength testing, the consistency of this stratum is believed to be very stiff to firm and generally stiff to firm.

Loose to Compact Grey Silt Till

Underlying the silty clay stratum described above is a stratum of grey silt till. This stratum was encountered in all boreholes, however, at borehole 1 this stratum was identified by observations in the field of the return wash water. The encountered thickness of this stratum ranged from 1.6 to 9.7 feet. The till stratum was fully penetrated in boreholes 1, 5, and 6 and borehole 3 was terminated in this stratum. The boundary between the till and overlying silty clay is not distinct since the upper portion of the till stratum exhibits a cohesive nature. The till has a matrix of clay and silt, binding sand and gravel.

Loose to Compact Grey Silt Till (continued)

The maximum observed gravel sizes were about 1 inch and the gravel particles were generally subangular in shape.

Two mechanical analysis tests carried out on representative samples recovered from this stratum indicated the samples tested to consist respectively of 8 and 12 percent gravel sizes, 28 and 44 percent sand sizes, 42 and 32 percent silt sizes and 22 and 12 percent clay sizes. The resulting grain size distribution curves for the above mechanical analysis tests are shown on Figure 5 of Appendix II.

Two laboratory vane tests carried out on undisturbed samples from the upper part of this stratum gave apparant shear strengths of 900 and 800 pounds per square foot. Values of shear strength and this method of testing are not appropriate for this type of soil; however, they do indicate a low relative density as discussed later.

Two standard penetration resistances determined in the till stratum gave "N" values of 12 and 13 indicating the relative density of this stratum to be compact. However, based on the results of dynamic penetration tests and examination of undisturbed samples of this stratum it is believed that the relative density of this stratum varies from loose to compact.

Bedrock

Bedrock was encountered and proved by core drilling to a depth of at least 10 feet in boreholes 1, 5, and 6. Borehole 3 was advanced to refusal probably on bedrock surface. Encountered bedrock surface elevation varied between elevations 255.8 and 257.2 and borehole 3 was terminated at elevation 254.9. Therefore, it is believed that bedrock surface is generally flat over the site. Loss of return water during core drilling operations was observed at elevation 251.9 in borehole 5 and at elevation 254.9 in borehole 6.

Bedrock was identified as dolomite.

WATER CONDITIONS

Ground water levels were observed in the cased boreholes during drilling operations. Subsequent to drilling operations water levels were observed in standpipes installed in boreholes 3, 5, 6, and in a piezometer installed in borehole 1.

Observed water levels were found to be extremely variable over the site. At boreholes 5 and 6 where bedrock was cored to a depth of at least 10 feet the ground water level subsequent to drilling operations was observed to be at elevations 252.6 and 256.4 respectively. That is slightly below bedrock surface. No water level observations were recorded for boreholes 5 and 6 during drilling

operations. At borehole 1 where bedrock was also cored the ground water level subsequent to and during drilling operations was observed to be below elevation 256.9. At borehole 3 where bedrock was not cored the ground water level was observed to vary between elevations 274.8 and 276.3 with the last observation indicating the ground water level to be at elevation 275.7.

Based on the above water level elevations there may be a perched water level in the overburden strata at about elevation 276 as indicated by the water level in borehole 3. The relatively low water levels observed in boreholes 1, 5, and 6 may be indicative of a water level in the bedrock since these boreholes were cored and since the relatively low permeability of the overburden strata prevents excessive loss of ground water to the underlying bedrock.

In any event, it is recommended that a ground water level at about elevation 276 be anticipated to exist over the site.

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:

Consistency	U-Strength Tons/sq. ft.	Relative Density	Standard Penetration Resistance, Blows/ft.
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".

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7316 GEN. TEST 2 DATUM GEODETIC CASING —
 BORING DATE AUG. 6, 1966 REPORT DATE AUG. 16, 1966 COMPILED BY J.W.A. CHECKED BY H.L.M.
 SAMPLER HANMER WT. — LBS. DROP — INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

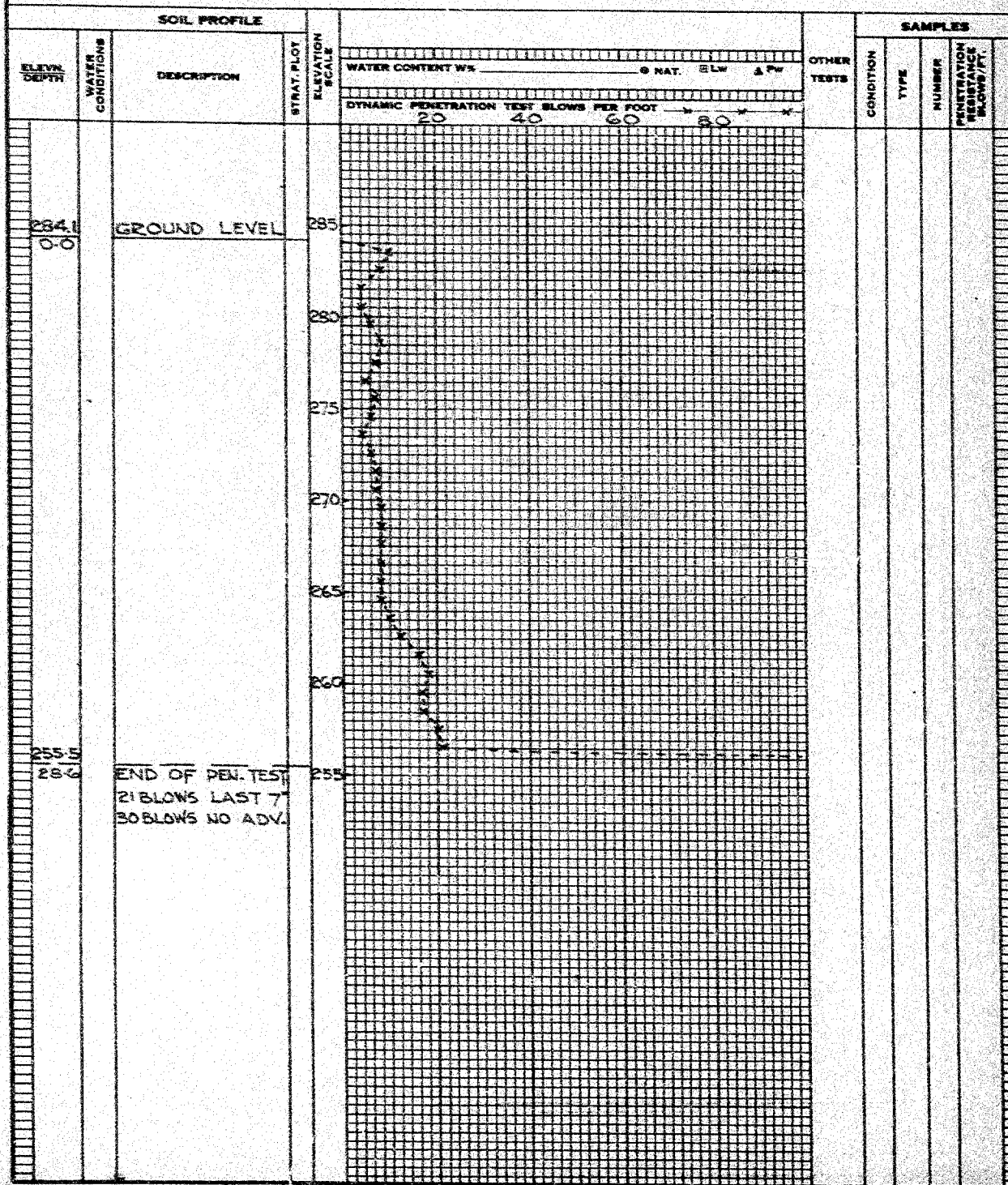
☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOSE

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
 OC - TRIAXIAL CONSOLIDATED UNDRAINED
 O - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7916 BORING # 3 DATUM GEODETIC CASING BX
 BORING DATE AUG 16, 1966 REPORT DATE AUG. 15, 1966 COMPILED BY J.W.A. CHECKED BY H.L.M.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

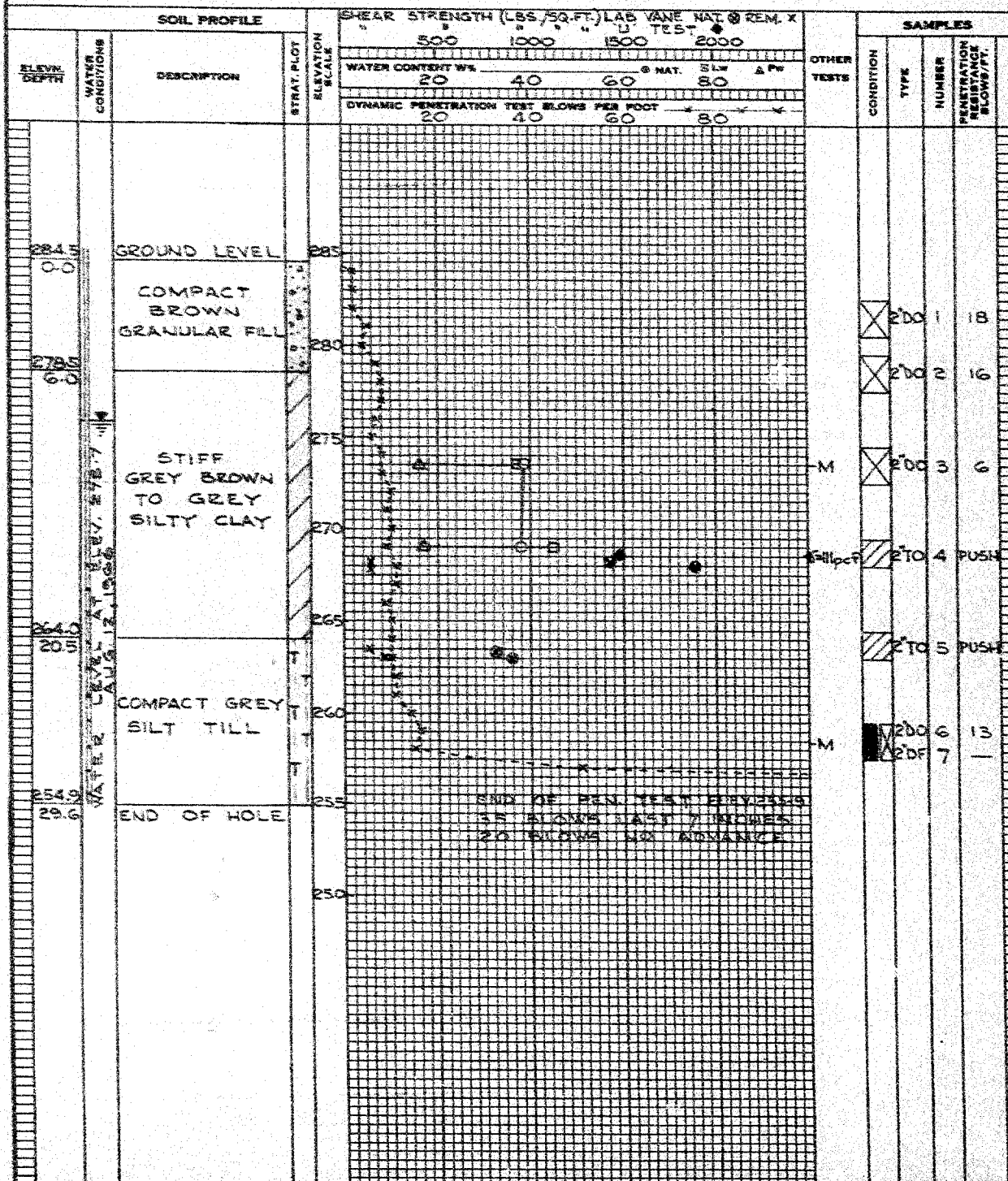
☒ DISTURBED
☐ FAIR
☐ GOOD
☐ BEST
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.D. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUCK SAMPLE
 F.S. - FOL 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 UNDRAINED
 Q - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7916 PEN. TEST 4 DATUM GEODETIC CASING ---
 BORING DATE AUG. 5, 1966 REPORT DATE AUG. 16, 1966 COMPILED BY J.W.A. CHECKED BY H.L.M.
 SAMPLER HAMMER WT. --- LBS. DROP --- INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

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 UNDRAINED
Q - TRIAXIAL UNDRAINED
S - TRIAXIAL DRAINED
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE				OTHER TESTS		SAMPLES			
ELEV. DEP'T	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT ELEVATION SCALE	WATER CONTENT W% RAT. BLW Δ PW	DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
285.1 0.0		GROUND LEVEL	290						
256.8 28.3		END OF PEN. TEST 28 BLOWS LAST 40 BLOWS NO ADV.	255						

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7916 BORING # 5 DATUM GEODETIC CASING BX
 BORING DATE AUG. 12, 1966 REPORT DATE AUG. 15, 1966 COMPILED BY J.W.A. CHECKED BY H.W.M.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

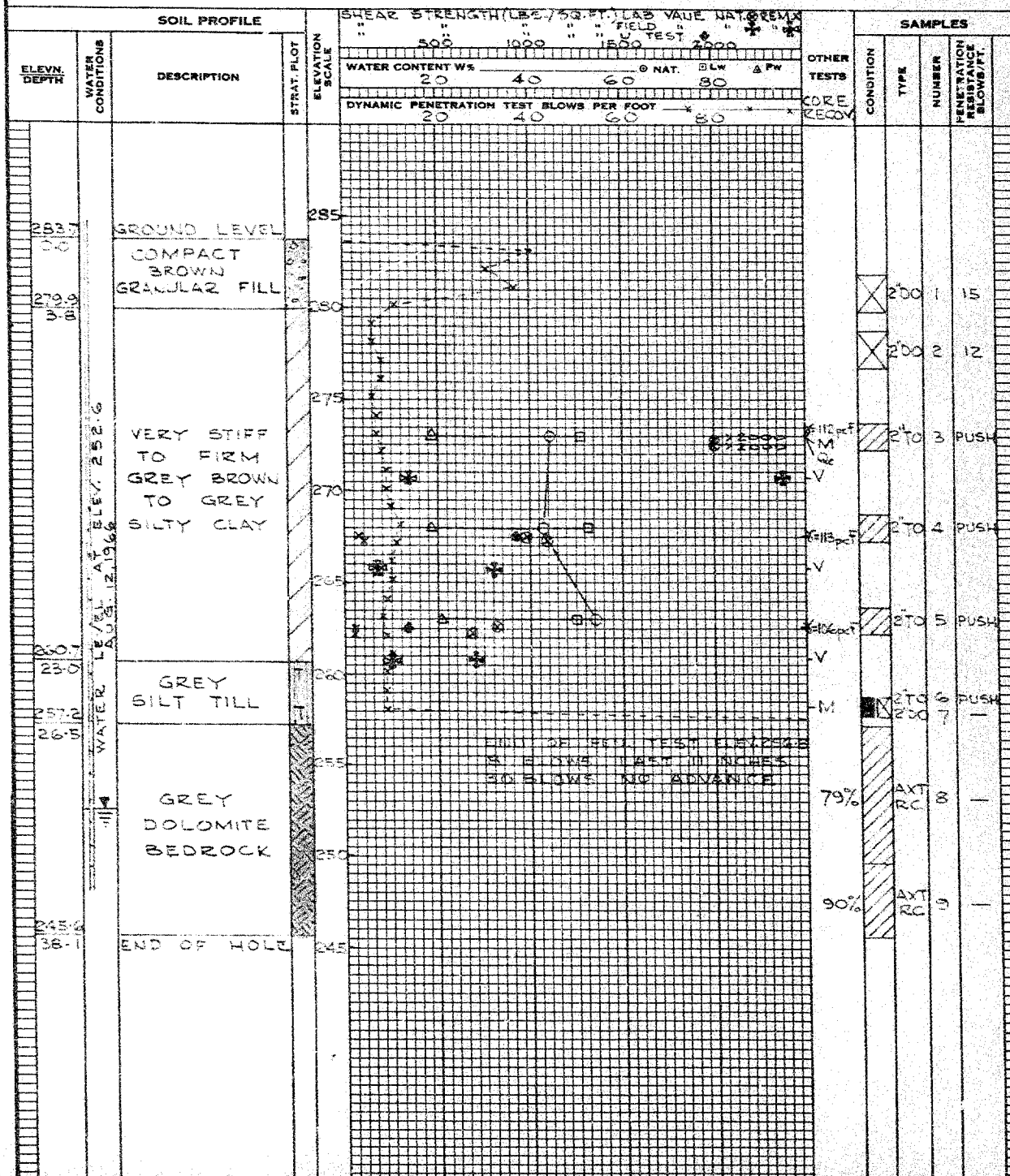
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
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ABBREVIATIONS

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GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7916 BORING # 6 DATUM GEODETIC CASING Bx
 BORING DATE AUG. 8, 1966 REPORT DATE AUG. 12, 1966 COMPILED BY J.W.A. CHECKED BY HLM
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

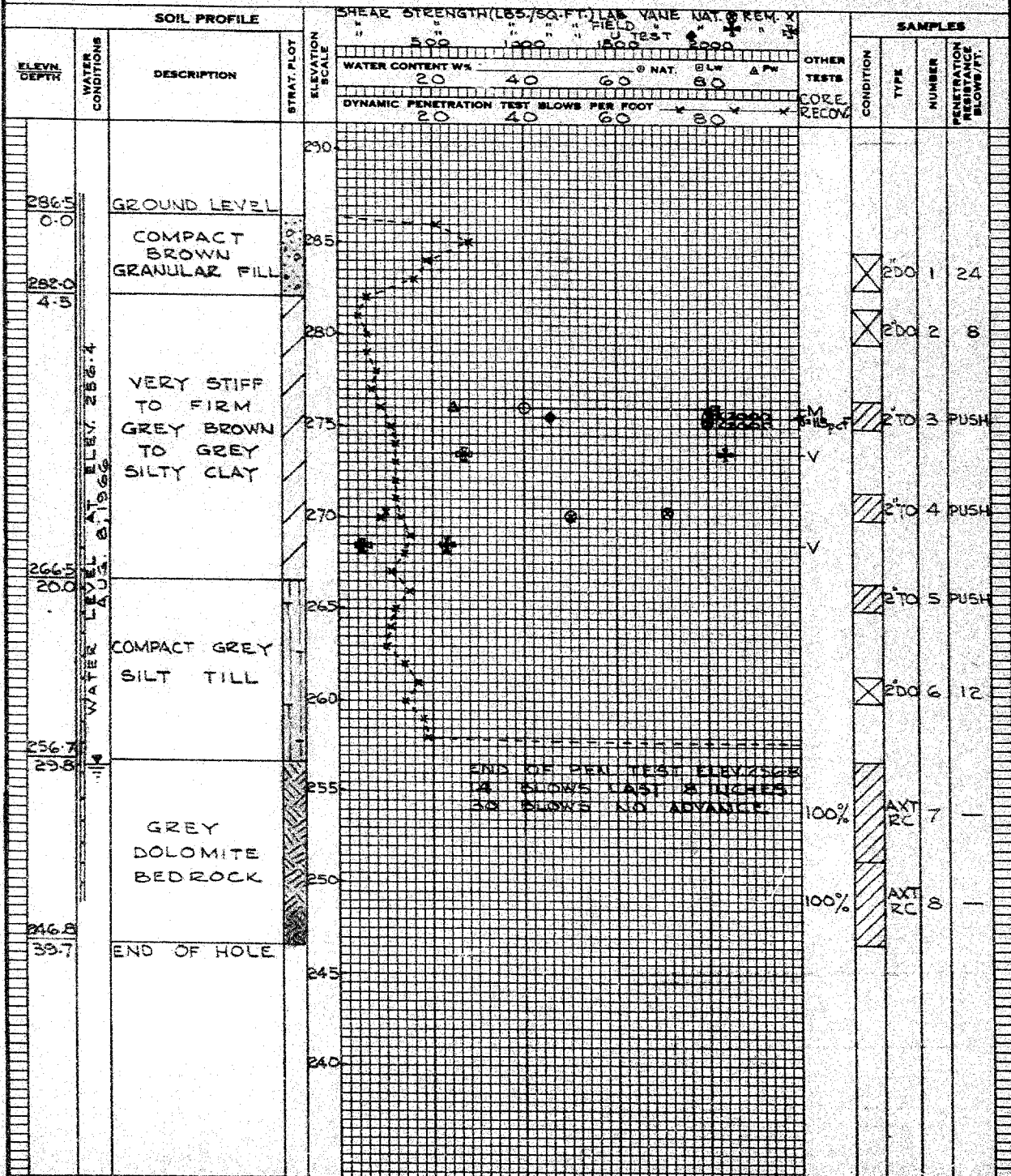
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

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

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
OC - TRIAXIAL CONSOLIDATED UNDRAINED
O - TRIAXIAL UNDRAINED
S - TRIAXIAL DRAINED

ABBREVIATIONS

W - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL



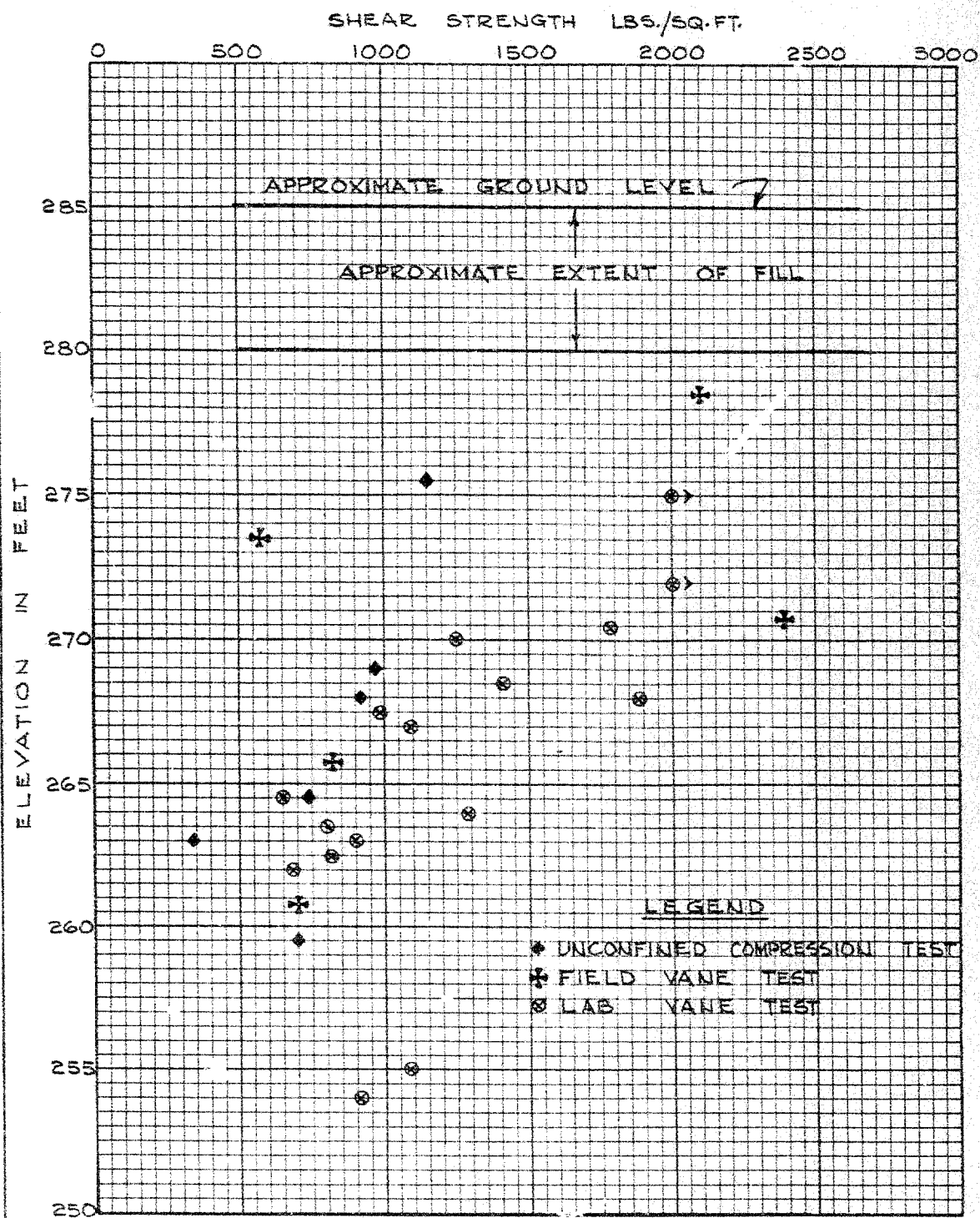
APPENDIX II

FIGURES - LABORATORY TESTING

GEOCON

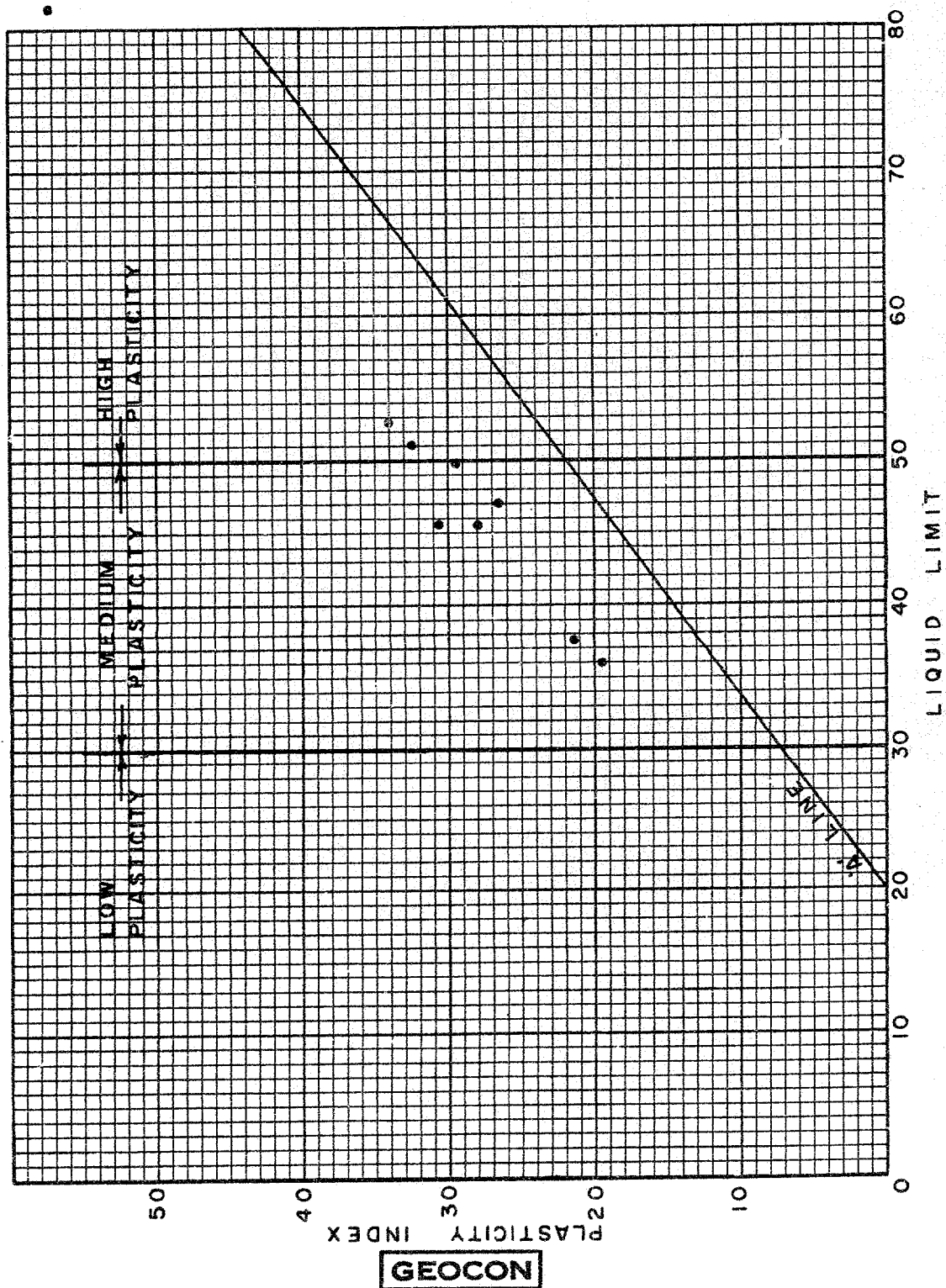
SHEAR STRENGTH vs ELEVATION

APPENDIX II
FIGURE 1
PROJECT T7916



PLASTICITY CHART

APPENDIX II
FIGURE 2
PROJECT T 7916



GRAIN SIZE DISTRIBUTION

SILTY CLAY

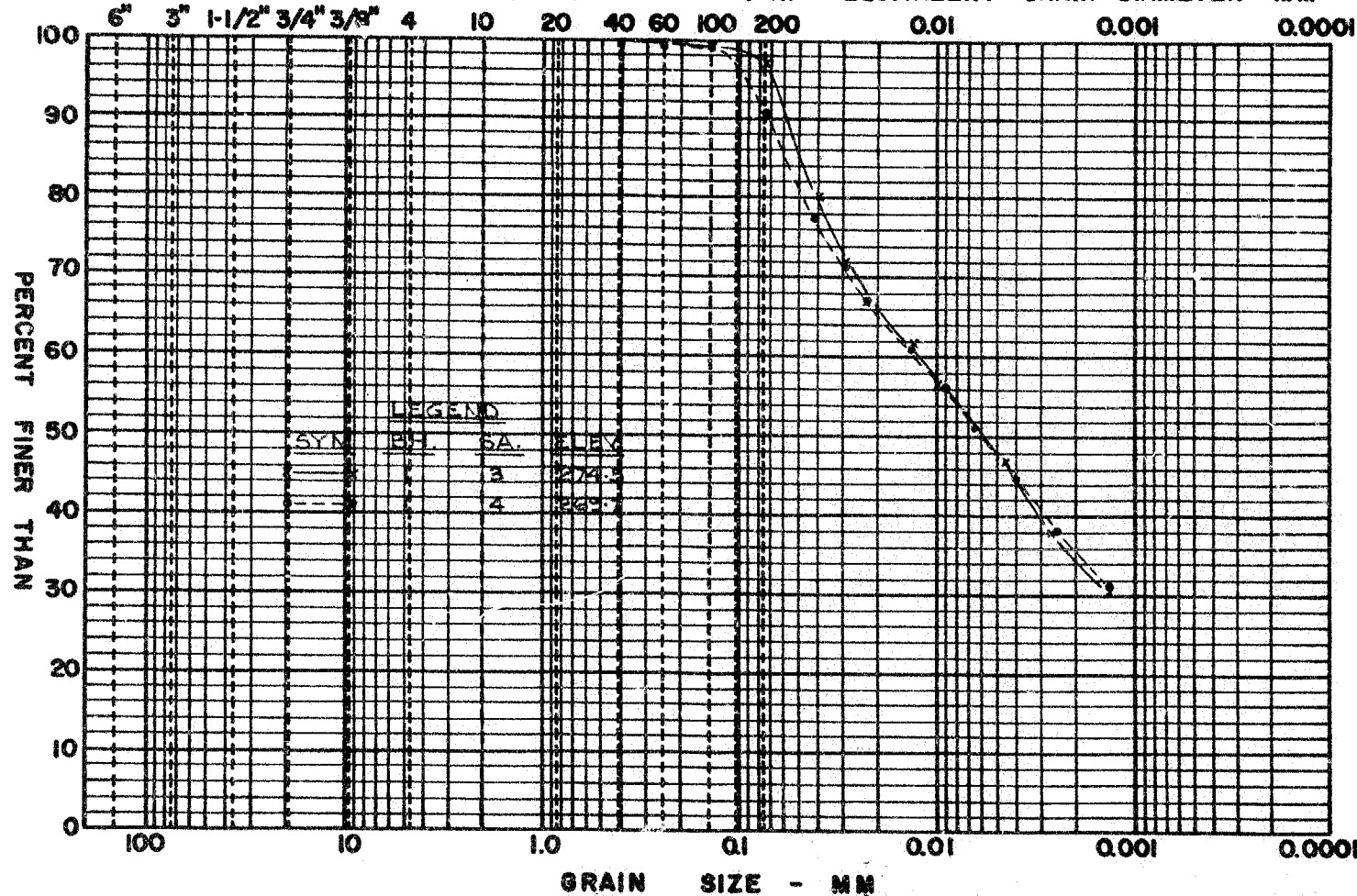
APPENDIX II

FIGURE 3

PROJECT T 7916

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

SIZE OF OPENING-INS. U.S.S. SIEVE SIZE-MESHES/IN. EQUIVALENT GRAIN DIAMETER - MM



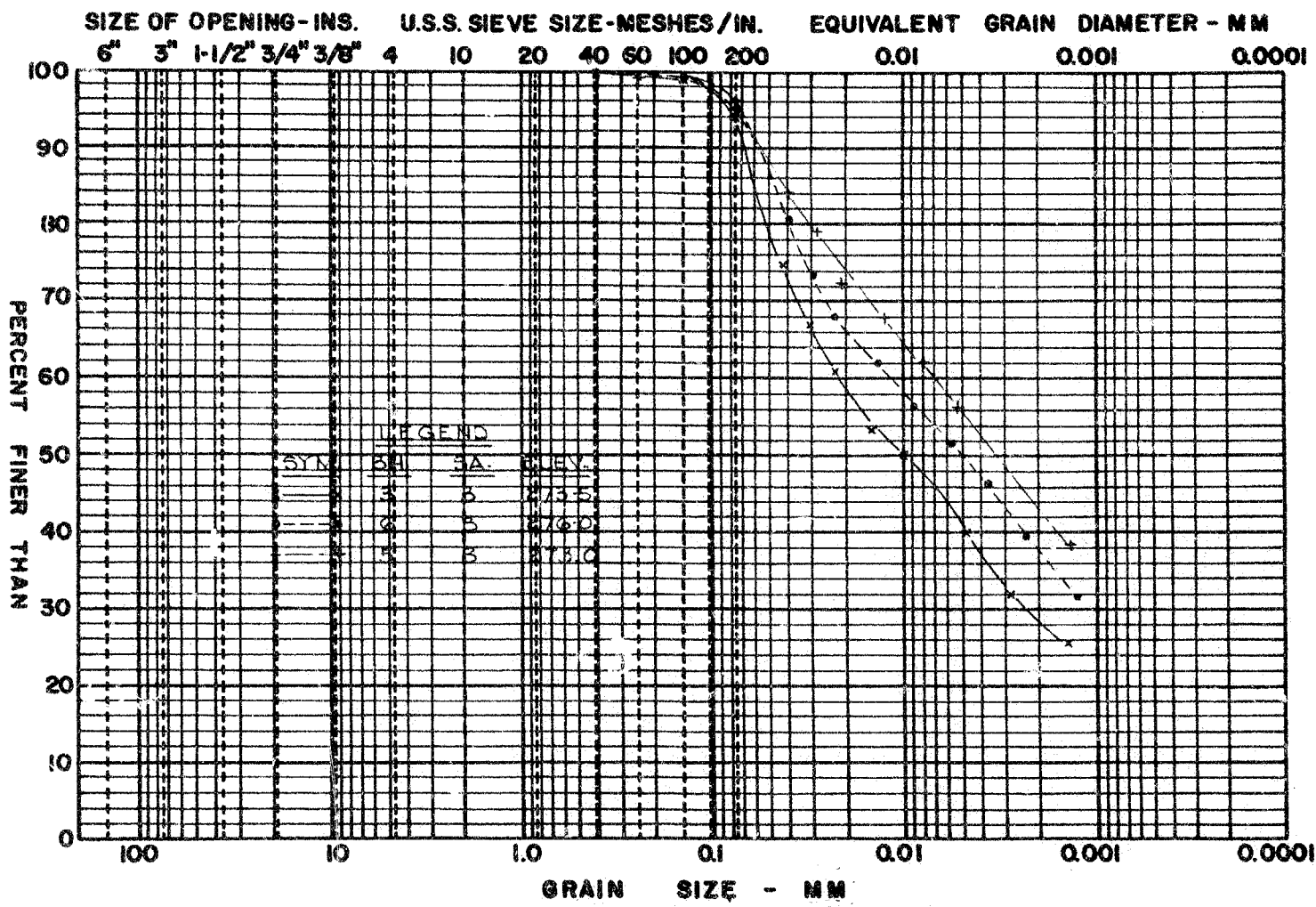
M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION

SILTY CLAY

APPENDIX II
FIGURE 4
PROJECT T 7916

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



M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

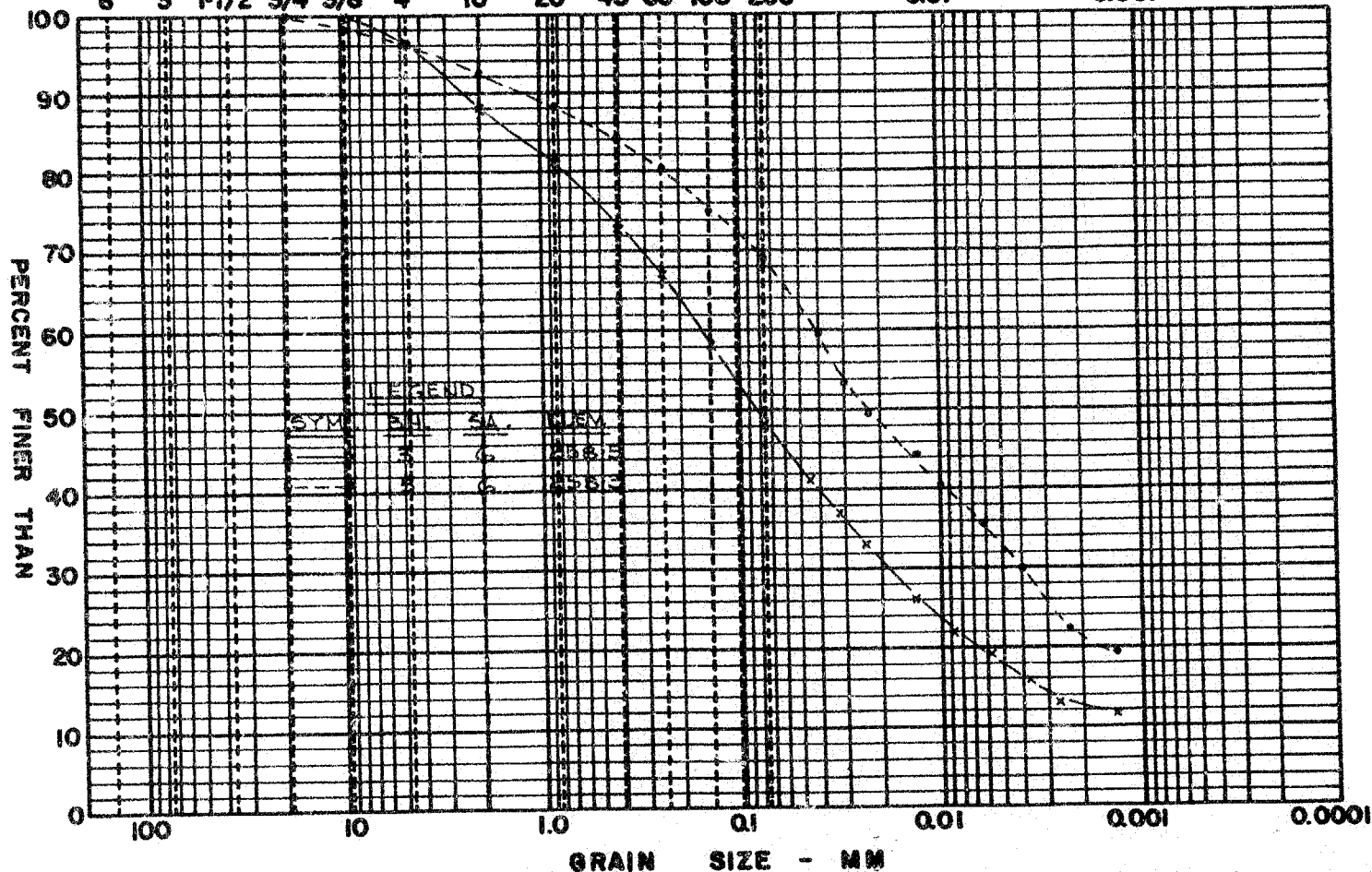
SILT TILL

APPENDIX II
FIGURE 5
PROJECT T 7916

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

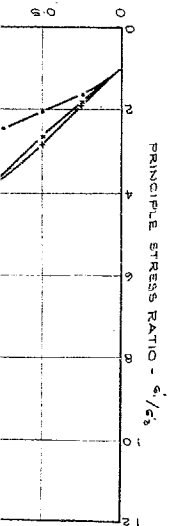
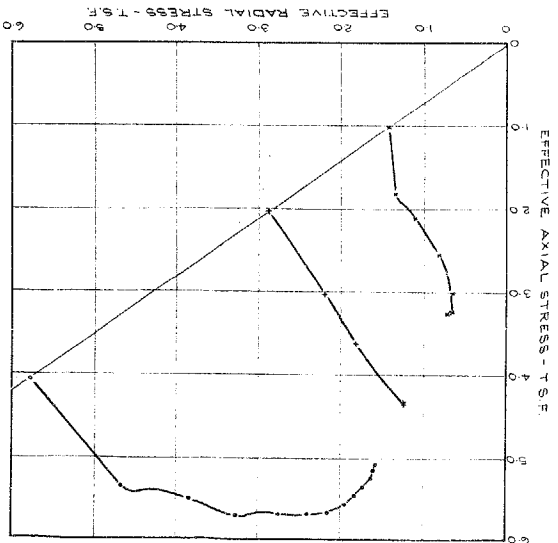
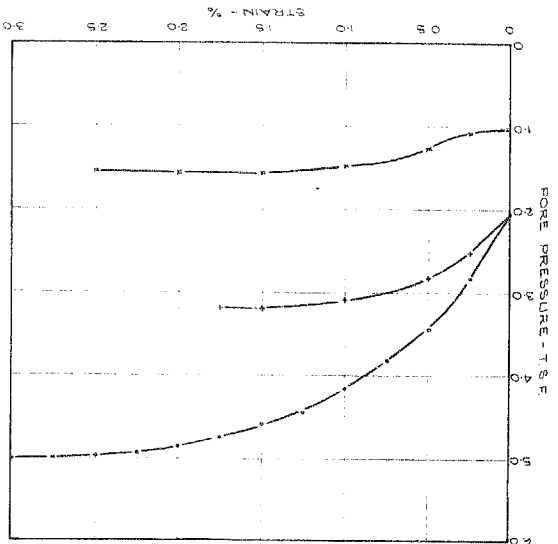
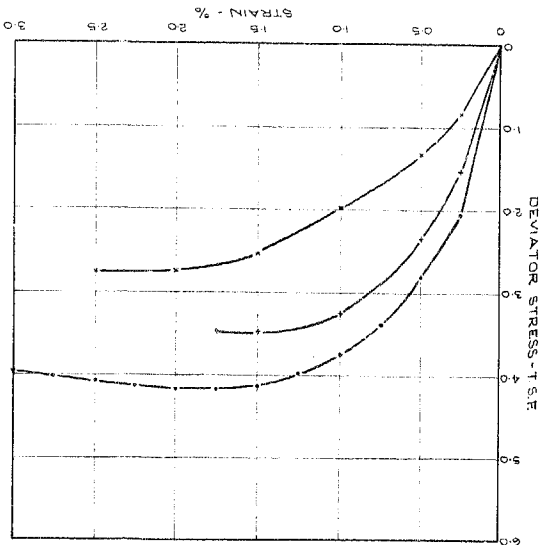
SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES / IN. EQUIVALENT GRAIN DIAMETER - MM

6" 3" 1-1/2" 3/4" 3/8" 4 10 20 40 60 100 200 0.01 0.001 0.0001



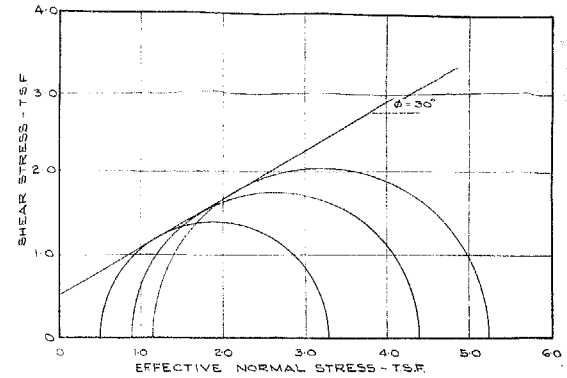
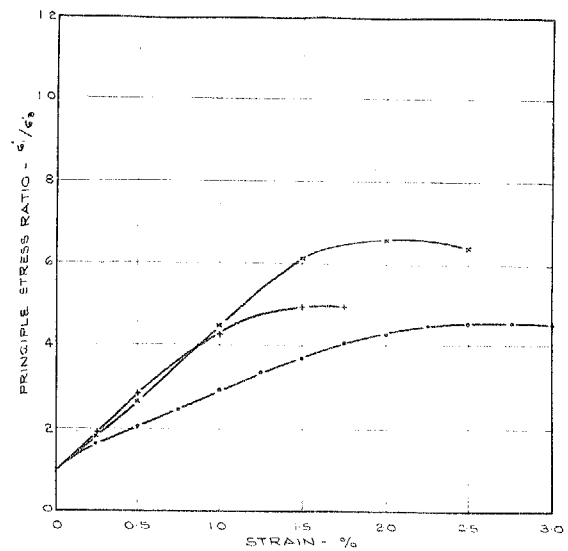
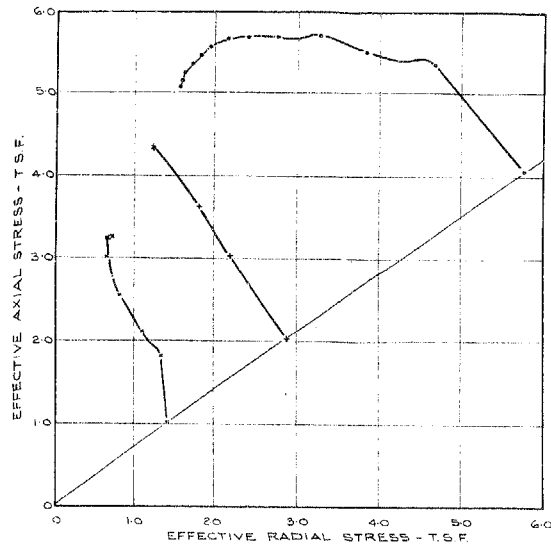
M.I.T. GRAIN SIZE SCALE

GEOCON



SYMBOL	STAGE No.	CONSOL. PRES. (T.S.F.)	γ - (P.C.F.)	WATER CONTENT
x	1	1.02	112	44.7
+	2	2.04	112	44.7
•	3	4.08	112	44.7

1. FILTER STRIPS - POROUS STONES, TOP & BOTTOM
2. RATE OF STRAIN - 2% PER HOUR
3. BACK PRESSURE - STAGE 1 = 1.02, STAGE 2 & 3 = 2.04
4. MULTIPLE STAGE TEST



Box. 401 & Keale St.,
Downsview, Ontario.

August 2, 1966

Materials and Testing Division

Geocor, Limited,
14 Main Road,
Kendale, Ontario.

Attention: Mr. D. Gates, P. Eng.

Re: Letter of Authority - Foundation Investigation -
M.T. 908-66 - C.B.C. Grade Separation,
Highway No. 7 - 15 - District No. 9 (Ottawa).

Dear Sir:

This is to authorize you to carry out the foundation investigation at the above site. The plans with the proposed footing locations, were given to your Mr. D. Gates, on July 29, 1966.

You are requested to submit eleven (11) copies of the report by September 1, 1966. Previous requirements as to preliminary borehole information and laboratory testing program should be followed.

In accordance with our terms of reference, you are to have a qualified soils engineer in charge of the field work at all times. Any deviation from this arrangement has to meet our prior approval.

Should you have any questions regarding this investigation, you can either call on this office, or on the Regional Bridge Location Engineer in Kingston, Mr. C. Scott - Telephone No. - 544-2220 (Area Code 613).

The mobilization costs for the drilling equipment will be paid from Ottawa to the site and back.

Since the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become a contract drawing, you are requested to prepare it in accordance with the C.B.C. Standards. To enable you to do this, we are supplying you with a sample drawing with all the necessary explanations, together with linen sheet for your drawing. You are also requested to provide us with a Cronaflex copy of the drawing.

Corcoran, Limited,
Attn: Mr. S. Bates, P. Eng.

- 2 -

August 2, 1966

Charges for the work will be in accordance with your
Schedule of Rates, dated July 8, 1966, and invoices to be addressed
to the attention of the undersigned.

We are attaching Purchase Order J 74221, covering the
purchase of any new material required for this work, in order that
you may use this as a basis for exemption from the Federal tax
for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

AMG/AMF
Attach.

Afterman
for S. Bates,
REGISTERED & TENTING ENGINEER

cc: Messrs. S. McConchie
E. S. Miller
C. Scott
C. B. Robertson
J. E. Graspier
Mrs. I. Steinberg
R. Jennings
A. Crowley
E. Heymans (2)
Foundations Office
Gen. Files (2)

Mr. J. E. Gruspier,
Regional Materials Engineer,
Regional Office,
Kingston, Ontario.

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

January 18, 1967

Your Memo -- Jan. 12/67

Attention: Mr. H. A. Meyer

W.P. 908-64 --
Hwy. #7 and #15,
C.N.R. Underpass,
District #9 (Ottawa).

908-64/66

Following your memo of January 12, 1967, we have reviewed the Foundation Report with regard to the feasibility of constructing the 4-ft. high detour immediately adjacent to a proposed 3:1 cut slope. Based on the stability analyses already carried out, it appears that this small height of fill will have very little effect on the overall stability.

KGS/adeF

cc: Foundations Office
Gen. Files

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

MEMORANDUM

To: Mr. A. G. Stermac
Principal Foundation Engineer
M. & T. Division, Downsview

FROM: M. & T. Division
Kingston

Attention: Mr. K. Selby

DATE: January 12, 1967

OUR FILE REF.

IN REPLY TO:

SUBJECT:

Re: Hwy. 7 and 15, W.P. 908-64
C.N.R. Underpass - Ottawa

Attached please find a plan indicating the general layout of the proposed highway detour and typical cross-sections of the proposed roadway excavation using 2:1 cut slopes. We have plotted the typical cross-sections on actual original groundline cross-sections, using 3:1 cut slopes as recommended in the foundation report and also the present proposal for the highway detour.

Please advise whether the detour is stable in this proposed location and if not what is the minimum safe distance between the detour centreline and the top of the cut slope.

H. A. Meyer
H. A. Meyer

for: J. E. Gruspier
Regional Materials Engineer

HAM:mgm

Attached

#66-F-212C

W.P. #908-64

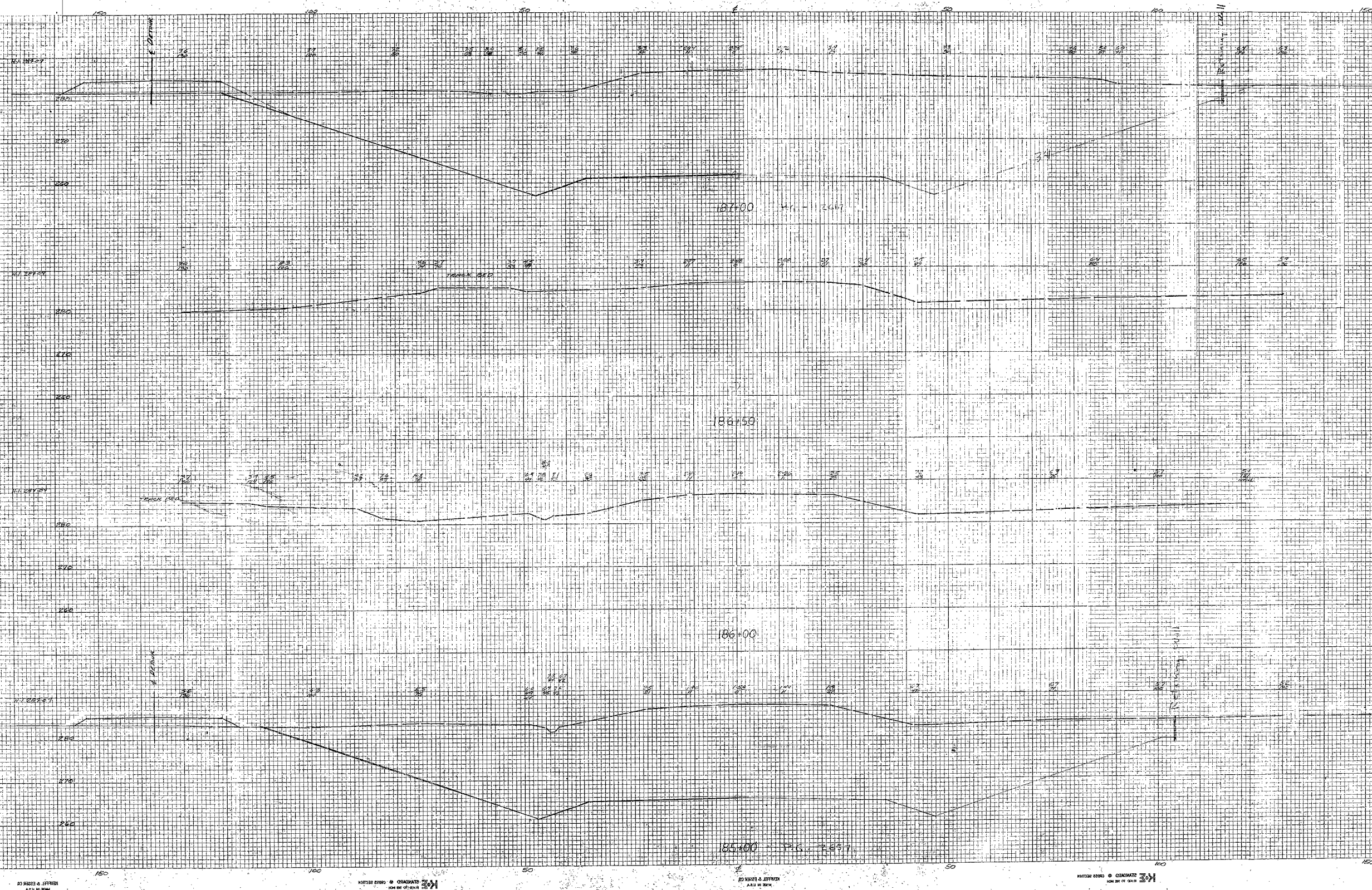
HWY # 7 & #15

C.N.R.

GRADE

SEPARATION

CHECKED BY	DATE
FORWARDED BY	DATE
CHECKED BY	DATE
CHECKED BY	DATE



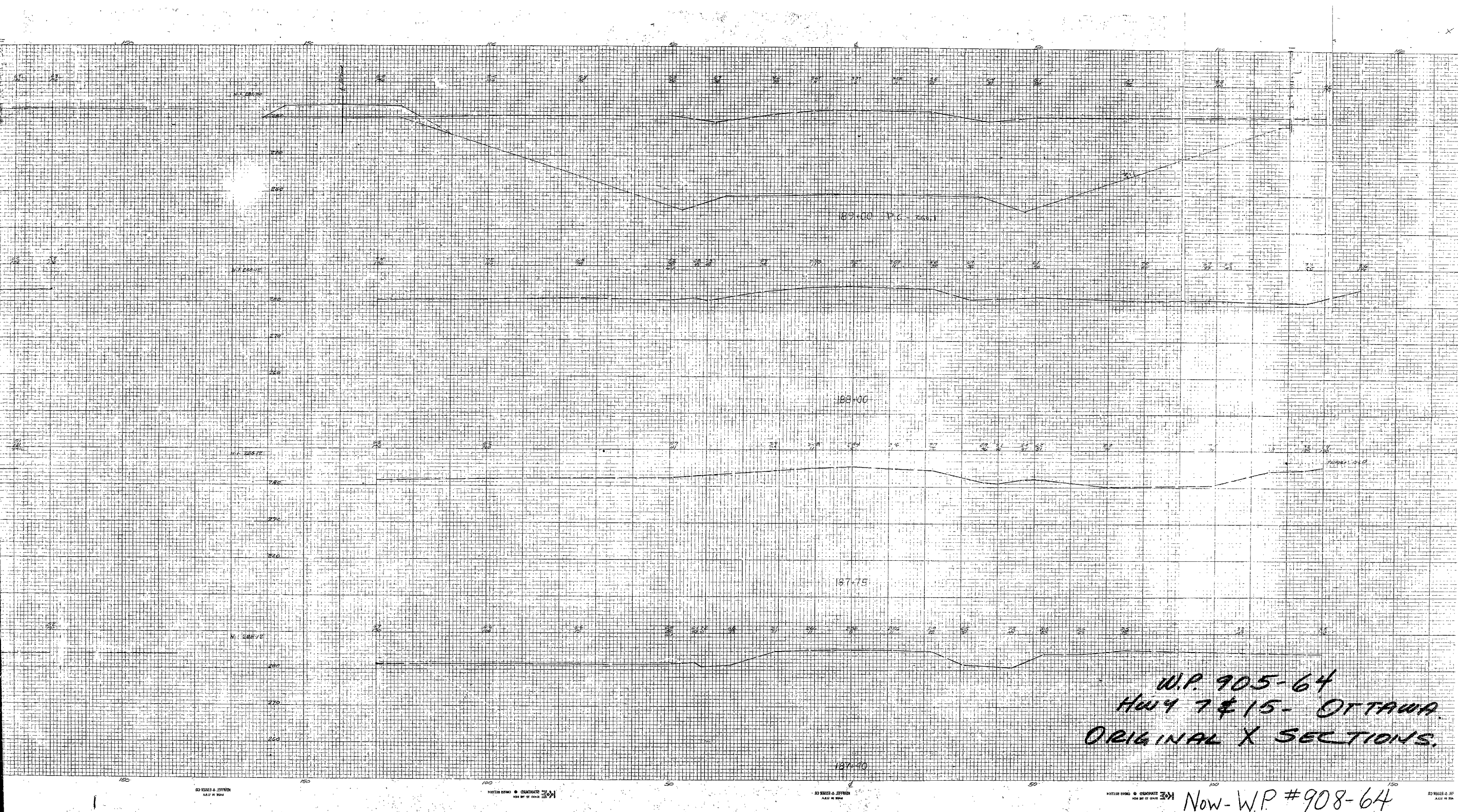
00 + 205 AFS OF 001181 AFS

DO NOT WRITE IN THESE SPACES

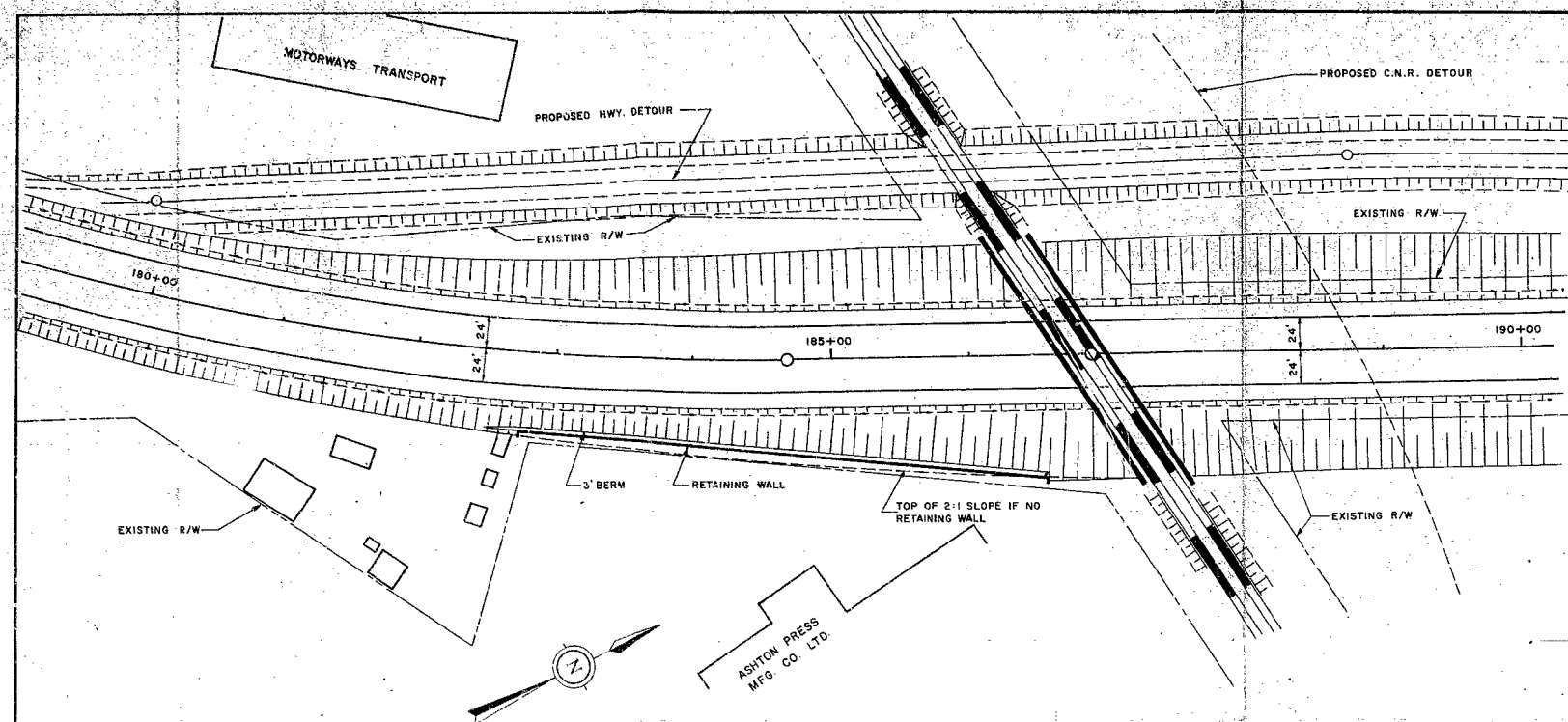
STANDARD & CROSS SECTION

MADE IN U.S.A.

FORM NO. OF ORDER 2-71

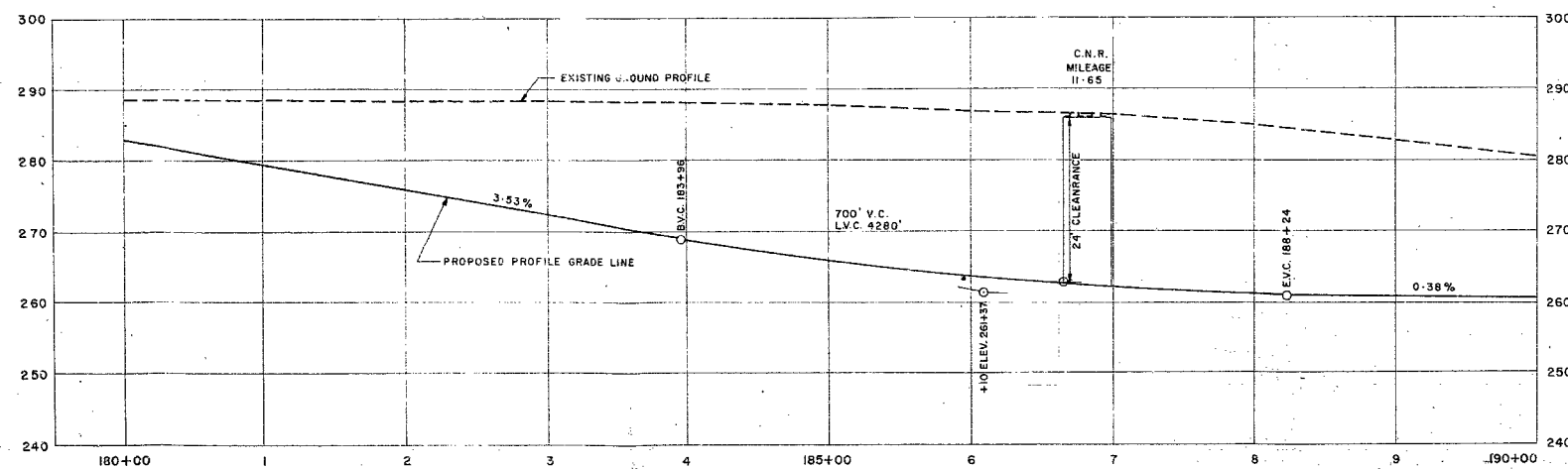


W.P. No. 905-64



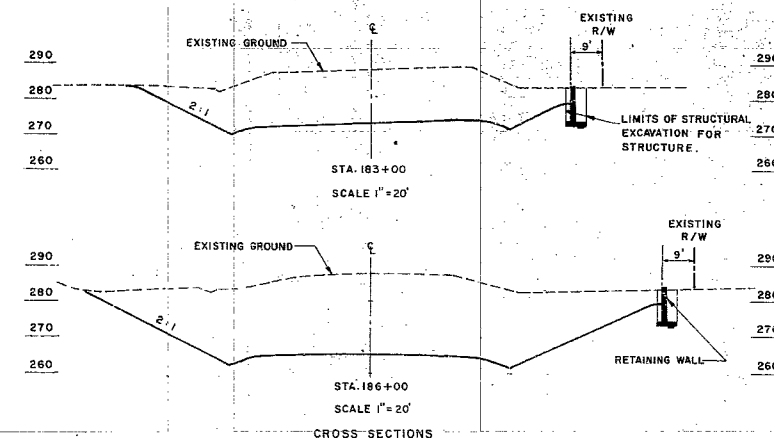
PLAN

SCALE: 1" = 50'



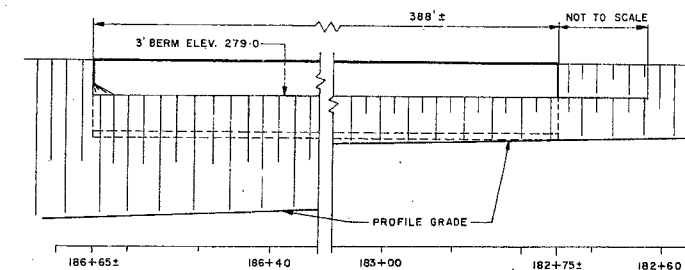
PROFILE

SCALE: VERTICAL 1" = 10'
HORIZONTAL 1" = 50'



CROSS SECTIONS

- NOTES:
1. ROADWAY WIDTH IS 70' ± 48' PAVEMENT, 8' SHOULDER, 3' ROUNDING. DD-202-B.
 2. FACE OF RETAINING WALL IS 9' FROM EXISTING R/W.
 3. ELEVATION OF TOP OF RETAINING WALL IS 284'-0".
 4. ELEVATION OF 3' BERM AT TOP OF SLOPE IS 279'-0".
 5. MAXIMUM SIDE SLOPE TO TOP OF BERM IS 2:1.



ELEVATION OF RETAINING WALL

SCALE 1" = 10'

W.P. # 908-64

DEPARTMENT OF HIGHWAYS
ONTARIO

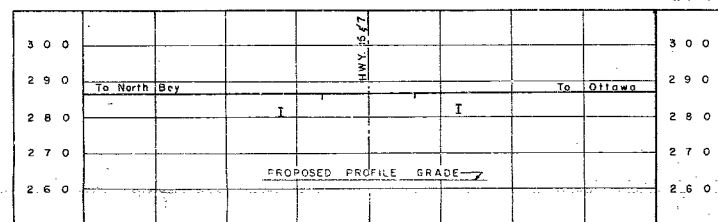
HWY. 7 & 15, OTTAWA

PROPOSED SUBWAY AT MILE 11-65
C.N.R. BEACHBURG SUBDIVISION

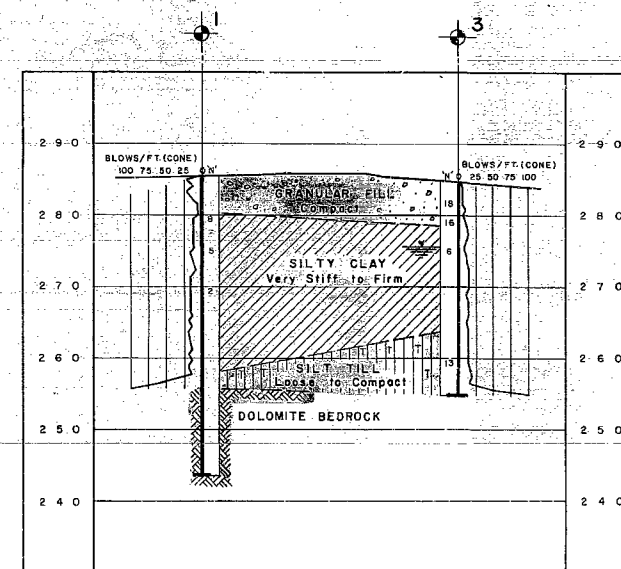
SCHEME 'A'

DE LEUW, CATHIER & CO. OF CANADA LIMITED
CONSULTING ENGINEERS
OTTAWA

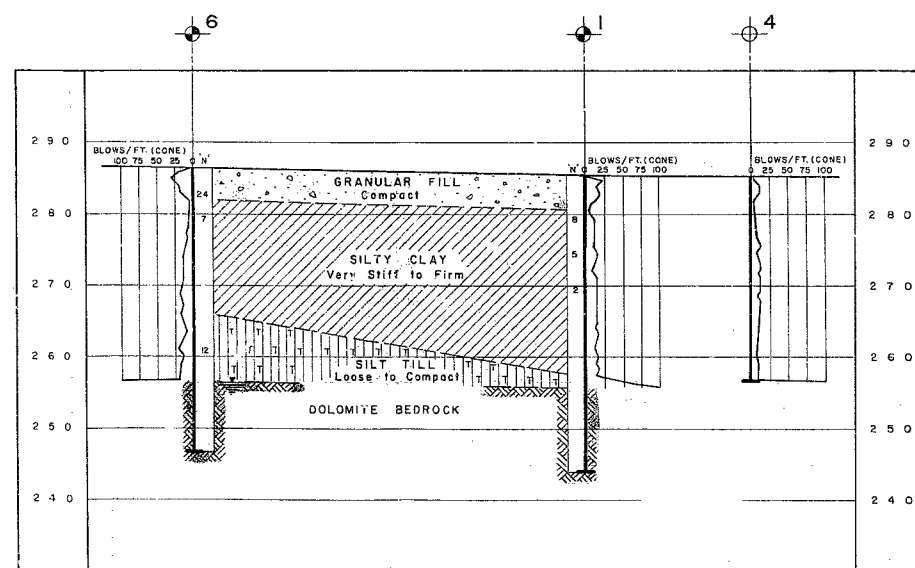
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Drawn: G.W.	Scale: AS SHOWN	DWG. No.
Checked: M.N.G.		0442-1



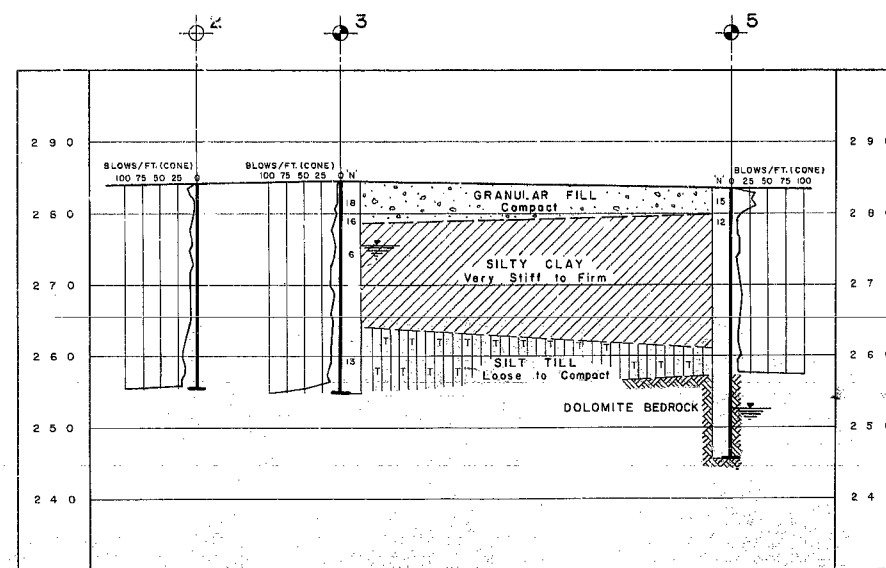
PROFILE C.N.R. SOUTH RAIL



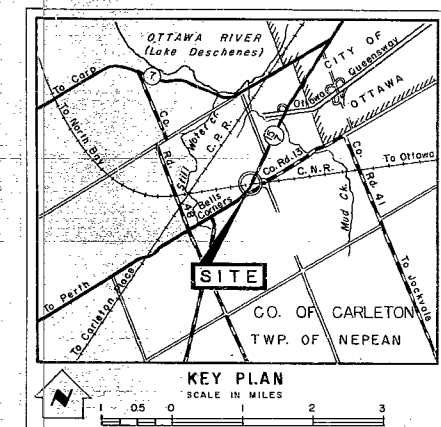
C - C







A - A



B-E



LEGEND

-  Bore Hole
 Cone Penetration Hole
 Bore & Cone Penetration Hole
 Water Levels established at time of field investigation. (Aug, 1966)

NO.	ELEVATION	STATION	OFFSET
1	285.4	186+82	3' R.T.
2	284.1	186+40	3' L.T.
3	284.5	186+80	3' L.T.
4	285.1	187+28	2' R.T.
5	283.7	187+90	2' L.T.
6	286.5	185+72	2' R.T.

B.M. 262:68 Geodetic Datum
N.W. in S. Root of 2nd Map. 45° LT. of 195+73

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVIEWS					
	DATE	BY	DESCRIPTION		
GEOCON LTD					
DEPARTMENT OF HIGHWAYS - TORONTO					
MATERIALS & TESTING DIVISION - FOUNDATION SECTION					
HIGHWAY 15 & 7 UNDERPASS					
KING'S HIGHWAY NO.		15 & 7		DIST. NO. 9	
CO. CARLETON					
TWP. NEPEAN		LOT 14		CON. 2 OTTAWA FRONT	
BORE HOLE LOCATIONS & SOIL STRATA					
SUBM'D H.L.M.		CHECKED D.B.O.		W.P. NO. 383-84	
DRAWN J.W.A.		CHECKED H.L.M.		JOB NO.	
DATE APPROVED 30/1966		SITE NO.		BRIDGE DRAWING NO.	
APPROVED <i>D.B.O.</i>		CONT. NO.		T 7916-1	

REF NO. E-4622-