

BA 716

58-F-271C

PROPOSAL
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
FOR
SITE INVESTIGATION AND ENGINEERING STUDY
PROPOSED PIC RIVER BRIDGE
MARATHON ONTARIO

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March 26th, 1958.

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Rexdale, Ontario,
March 26th, 1958.

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Department of Highways, Ontario,
Parliament Buildings,
Toronto, Ontario.

Attention: Mr. F.C. Brownridge, P.Eng.,
Materials and Research Engineer.

Re: Site Investigation and Engineering Study,
Proposed Pic River Bridge,
Marathon, Ontario.

Dear Sirs:

Further to the recent meeting between your Mr. Brownridge and the writer, we herewith present, as requested, our proposal to carry out an engineering study and additional site investigation work for the proposed Pic River Bridge.

HISTORY OF PREVIOUS WORK

It is understood that the proposed bridge will be a three-span steel structure, the centre span being approximately 270 feet in length. As presently designed, the piers and abutments will be supported on friction piles. It is further understood that two separate site investigations have been previously carried out during the past two years by other Soil Consultants. It was determined that the site is covered by loose sand overlying about 40 feet of varved clay, then a varved silt stratum, the complete thickness of which was not determined.

The design of the piled foundations and the construction procedures to be adopted have not yet been finalized. To make a final decision on the effective pile lengths required and the overall stability of the foundations, further foundation studies have been considered necessary.

The scope of these studies is detailed below:

1. THE FOUNDATION PROBLEM

No definite decision has yet been made covering the general foundation engineering problem, which we feel falls under three headings:



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- (a) The design of the piled foundations including the length and type of piles required, the spacing of piles, the allowable pile loads and the expected settlement of pile groups and the construction procedures during pile driving.
- (b) The overall stability of the piers and abutments with particular reference to earth pressure behind the abutments, slope failure under the abutments and piers and surface creep of the trimmed banks at the site.
- (c) The stability of earth cuts for the road approaches at the ends of the proposed bridge.

2. FURTHER FIELD INVESTIGATION REQUIRED

In general further field investigation would include the following:

- (a) Exploratory Borings - purpose, to obtain good undisturbed samples of the varved clay and silt strata for stability studies, settlement analyses and an accurate observation of the groundwater conditions. At least one borehole would be carried down to a sufficient depth to determine, if possible, the complete thickness of the underlying varved silt stratum and possible artesian water conditions at depth.
- (b) In situ Permeability Tests - purpose, to determine the degree of drainage which may be expected from wellpoints for construction drainage or wells installed for relief of any hydrostatic pressures which may be encountered.
- (c) Observation of Natural Slopes - purpose, to investigate the degree of groundwater drainage in natural slopes and any previous slope failures due to instability or surface creep.

3. LABORATORY STUDIES

Detailed laboratory tests would be carried out to determine the variation of water content, shear strength and consolidation characteristics with depth. Quick triaxial and consolidated quick triaxial tests and consolidation tests would be made on the undisturbed samples obtained. Slow drained triaxial tests would be further carried out to determine the effective angle of internal friction and true cohesion of the clay and silt samples. Laboratory permeability tests would also be made to study the relation of vertical permeability to horizontal permeability as determined from the in situ tests.

4. ENGINEERING STUDIES

Upon completion of the field work and laboratory testing, a complete engineering study would then be carried out to investigate all the points covered under Clause 1 and our detailed recommendations for foundation design and the construction procedures to be adopted would be presented in an engineering report.

5. SERVICES TO BE RENDERED

With respect to all this work, we propose to act as your Soil Consultants working under your general direction; and assuming full responsibility for the execution of the work.

We suggest that four detailed exploratory boreholes in 4 inch size be put down to penetrate the lower varved silt stratum for a depth of at least 20 feet. These boreholes would be strategically located to cover the revised position of the bridge abutments and to obtain adequate undisturbed samples at one of the pier locations. We propose to carry out this phase of the field work using a Swedish Fall Sampler for complete recovery of undisturbed continuous samples of the subsoil.

In addition, at least one deep cased borehole would be put down in 2 inch size to carry out in situ falling head permeability tests and to determine the complete depth of the varved silt stratum. We propose to put down this hole using a standard skid-mounted machine drillrig.

The field work would be supervised throughout by an experienced and competent soils engineer.

Laboratory testing of samples would be carried out in the Toronto Soil Mechanics laboratory of Gecon Ltd.

6. COMPENSATION

Our charges would be in accordance with our Schedule of Rates given in Appendix I of this proposal.

We include under Appendix II of this proposal, a summary of the Estimated Total Costs of the various parts of this investigation.

7. COMMENCEMENT AND COMPLETION

We are prepared to commence this work on April 1st, 1958. We estimate that the field work would take approximately two weeks to complete and that our final report would be submitted about six weeks following completion of the field work.

8. TERMS OF PAYMENT

Progress invoices will be submitted monthly for the full amount of field and office work completed less a retention of 5% and payment will be made within 15 days of receipt of our invoice. Payment in full including all retentions will be made within 30 days of completion of our work. If the work is commenced and completed in the same month, a final invoice will be submitted at the end of that month.

9. ACCEPTANCE

Your acceptance of this proposal shall constitute a binding contract between us.

Yours very truly,

GEOCON LTD



V. Milligan, P. Eng.,
District Engineer.

VM/dw
Attach.
S-6657

ACCEPTED

.....

DATE.....195..

APPENDIX I
SCHEDULE OF RATES

SCHEDULE OF RATES

1. For the provision of engineering services as required.

Soils Engineer on report writing, computations and field supervision.....\$ 60.00 per day
Junior Engineer, Technician or Draftsman.....\$ 40.00 per day

When the services of the Chief Engineer and District Engineer are necessary to check or review the work, the fees for their services will be as follows:

Chief Engineer.....\$100.00 per day
District Engineer.....\$ 80.00 per day

2. For the transportation of all equipment, tools and supplies to and from our Toronto yard and the site, including initial loading and final unloading at our yard, travelling time, expenses, contingencies and also including the expenses of shipping Foil samples by C.P.R. Express to Toronto and a supervisory site visit.

LUMP SUM.....\$1,570.00

3. For the provision and operation of a standard skid-mounted machine drillrig with all necessary tools and experienced three-man crew, including expendable stores, living and daily travelling expenses and other site costs.

For each 8-hour day of operation.....\$200.00

4. For the provision and operation of a Swedish Foil Sampling Apparatus with all necessary tools and experienced three-man crew, including expendable stores, living and daily travelling expenses and other site costs.

For each 8-hour day of operation.....\$210.00

5. For soil testing carried out in our laboratory.

(See attached Schedule of Rates for Laboratory Tests)

NOTES

1. Under Item 1 the daily fees for field supervision cover for a calendar working day. The daily fees for office engineering services cover for a 7-hour day. Where only part of a day is worked or in excess of 7 hours in any one day, the fees will be pre-rated on an hourly basis.
2. Items 3 and 4 are based on an 8-hour day, 40-hour week.
3. If overtime is required under Item 3, the rate for each overtime hour will be.....\$25.00.
4. If overtime is required under Item 4, the rate for each overtime hour will be.....\$27.00.

Department of Highways, Ontario,
Site Investigation and Engineering Study,
Proposed Pic River Bridge,
Marathon, Ontario.

SCHEDULE OF RATES FOR LABORATORY TESTS

	<u>UNIT RATE</u>
EXTRUSION AND IDENTIFICATION:	
Extrusion of tube sample, labelling, and storing.	\$ 2.50
Visual identification and description.	2.50
CONSISTENCY LIMITS:	
Natural water content.	1.50
Plasticity index, liquid and plastic limits.	7.50
Shrinkage limit.	4.00
GRAIN SIZE ANALYSIS:	
Distribution curve included.	
a) Sieve.	5.00
b) Hydrometer.	15.00
c) Sieve and hydrometer.	20.00
STRENGTH AND COMPRESSIBILITY TESTS:	
Unconfined compression:	
a) Maximum stress only reported.	7.50
b) Stress-strain curve included.	10.00
Triaxial compression; stress-strain curve included:	
a) Quick.	15.00
b) Consolidated quick.	25.00
c) Slow.	30.00
Consolidation:	
a) Loading, initial cycle; pressure-void ratio curve included.	50.00
b) Rebound and reloading, per cycle.	10.00
UNIT WEIGHT:	
a) Sample of geometrical shape.	2.50
b) Sample of irregular shape.	7.50
PERMEABILITY:	
a) Falling head.	50.00
b) Constant head.	30.00
COMPACTION TEST:	
Moisture-density curve included.	50.00
RELATIVE DENSITY:	
Cohesionless soils.	10.00
SPECIFIC GRAVITY.	5.00

APPENDIX II

SUMMARY OF ESTIMATED TOTAL COST

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SUMMARY OF ESTIMATED TOTAL COST

Based on the investigation programme outlined, we estimate the total cost of this work would be as follows:

- | | |
|--|-------------|
| 1. Field Work Costs..... | \$ 4,300.00 |
| 2. Laboratory Testing..... | \$ 1,700.00 |
| 3. Engineering Costs..... | \$ 3,200.00 |
| 4. For Consultation Services, if required..... | \$ 500.00 |

TOTAL COST	\$ 9,700.00
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Department of Highways, Ontario,
Site Investigation and Engineering Study,
Proposed Pic River Bridge,
Marathon, Ontario.

PROJECT S6657

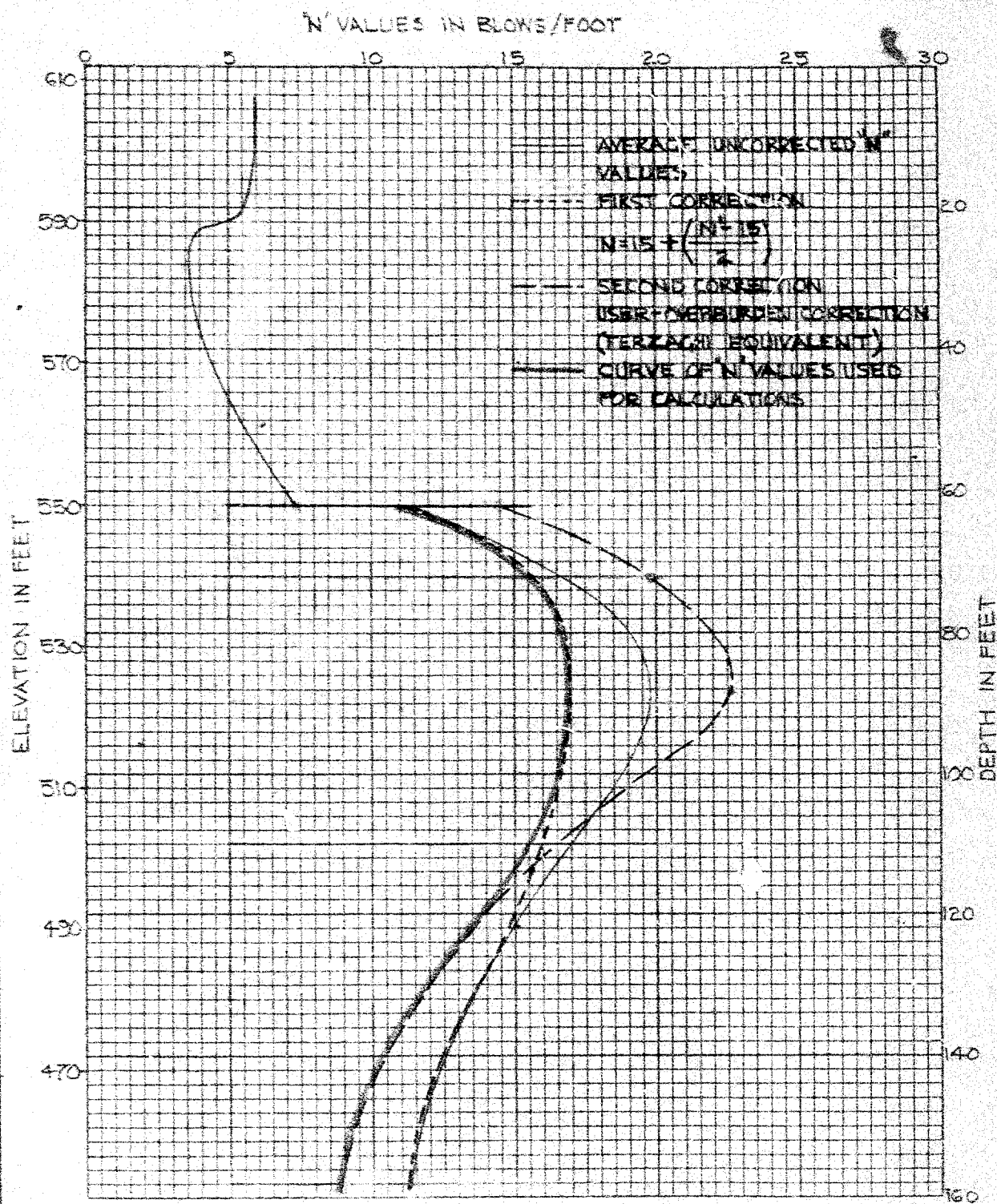
PROPOSED BIG PIC RIVER BRIDGE

CALCULATIONS AND PRELIMINARY RECOMMENDATIONS

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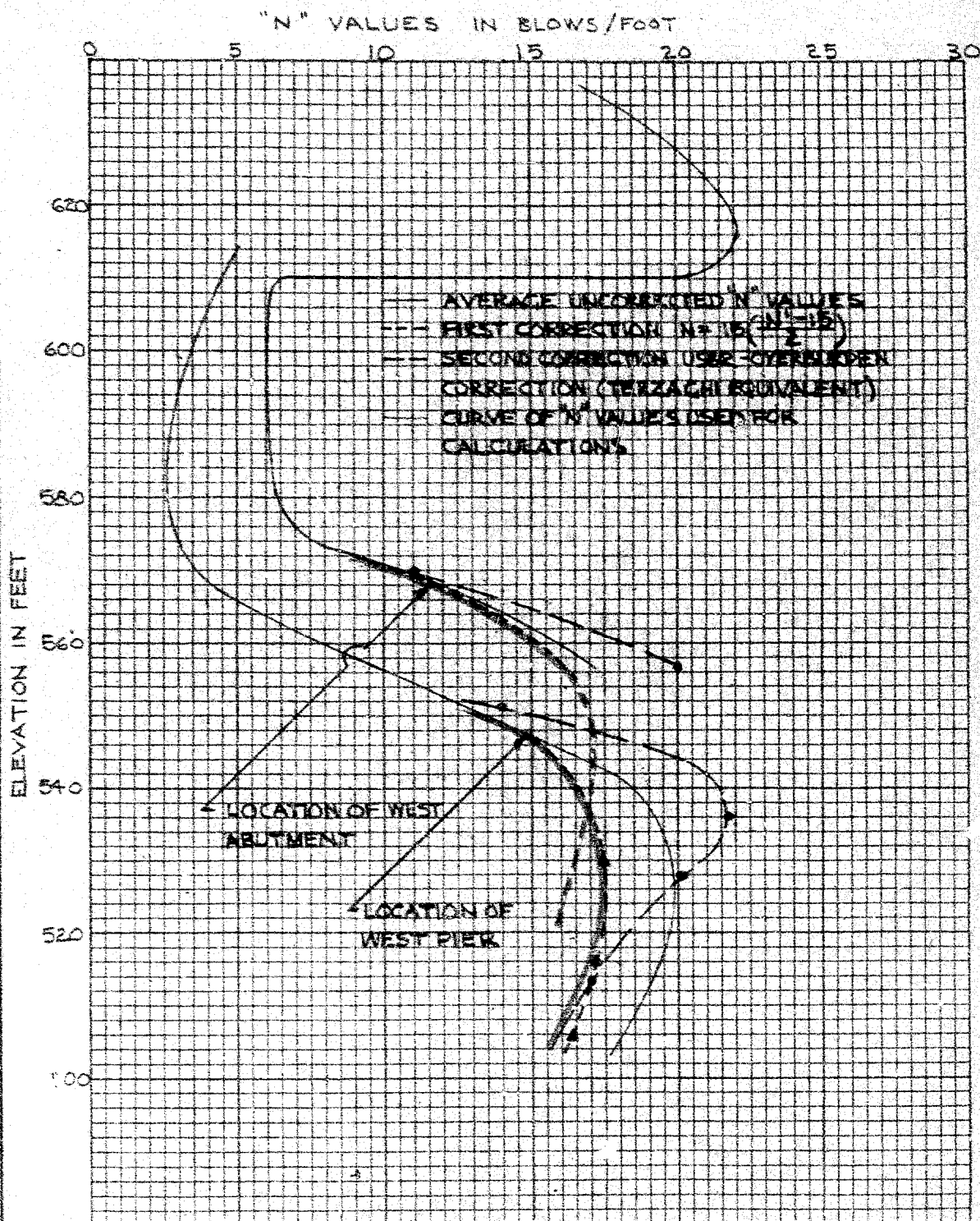
"N" VALUES VS DEPTH (EAST PIER)

APPENDIX
FIGURE 1
PROJECT 56657



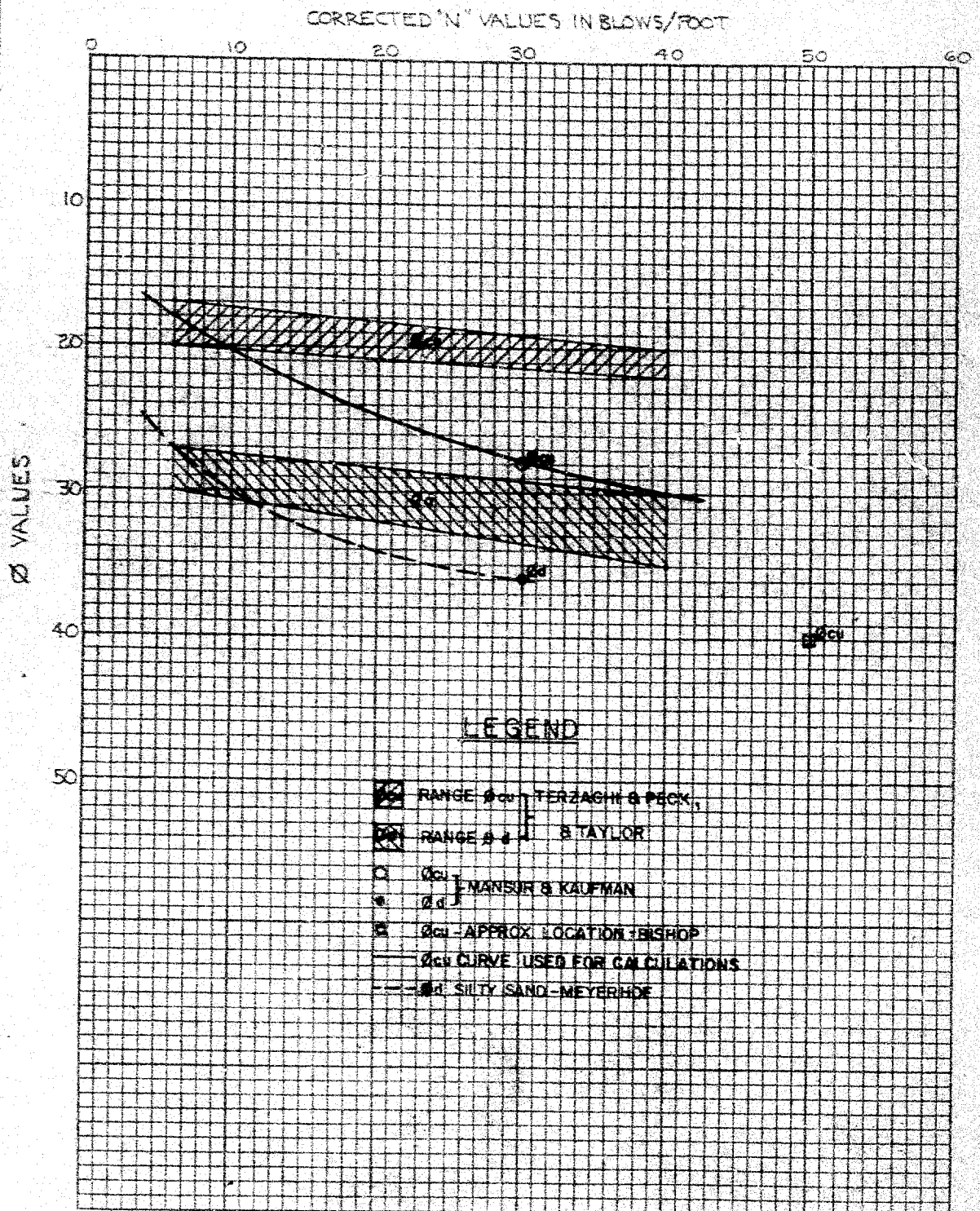
'N' VALUES VS DEPTH (WEST PIER AND WEST ABUTMENT)

APPENDIX
FIGURE 2
PROJECT S 6657



CORRELATION OF ϕ VALUES WITH CORRECTED N VALUES FOR SILTY SOILS

APPENDIX
FIGURE 3
PROJECT SC657

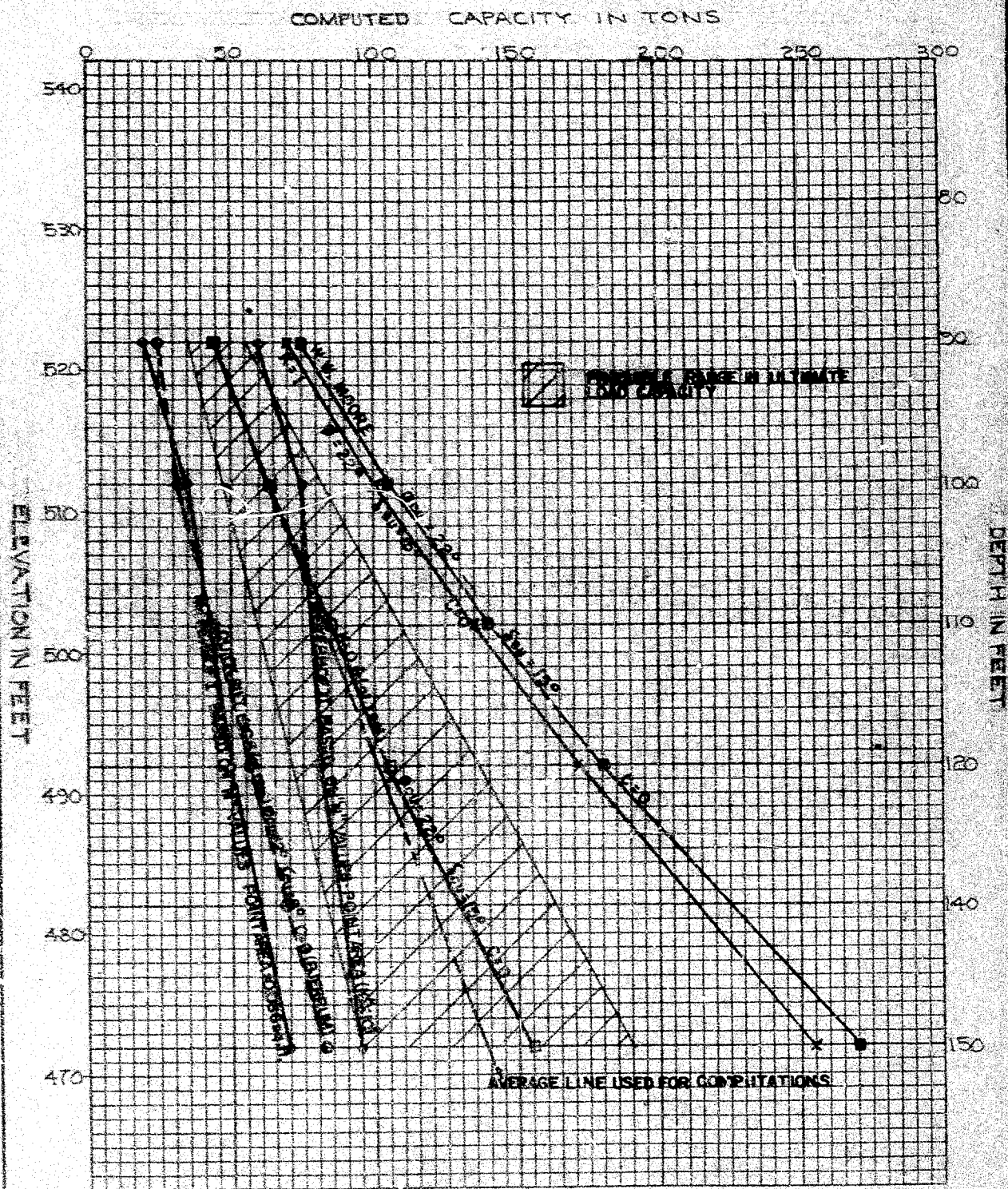


COMPUTED CAPACITY VS DEPTH

EFFECT OF UPPER 62 FEET NEGLECTED

SINGLE STEEL H PILE (10 BP 42)

APPENDIX
FIGURE 4
PROJECT S 6657



COMPUTED CAPACITY VS DEPTH

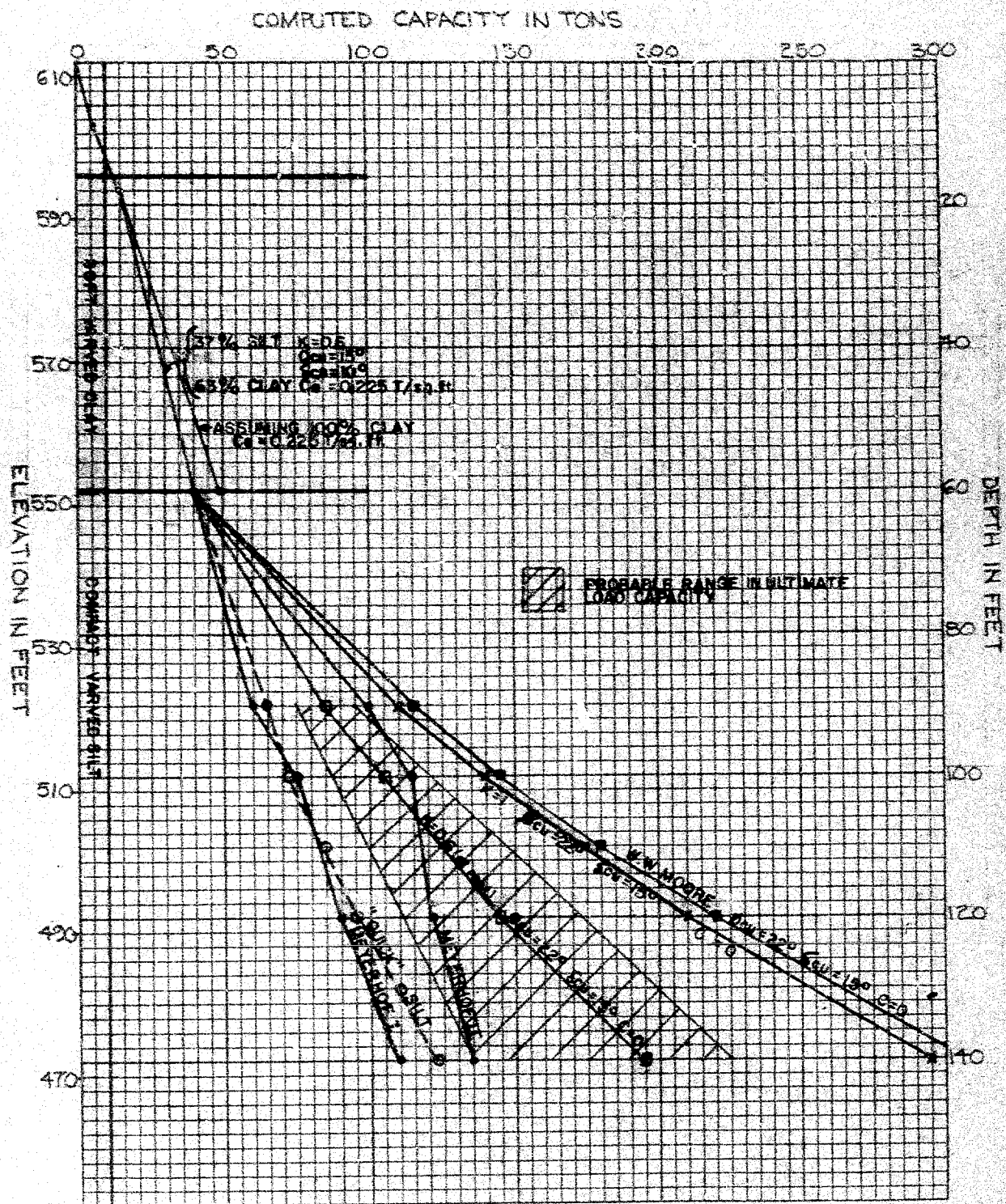
SINGLE STEEL H PILE

(10 BP 42)

APPENDIX

FIGURE 5

PROJECT 56657



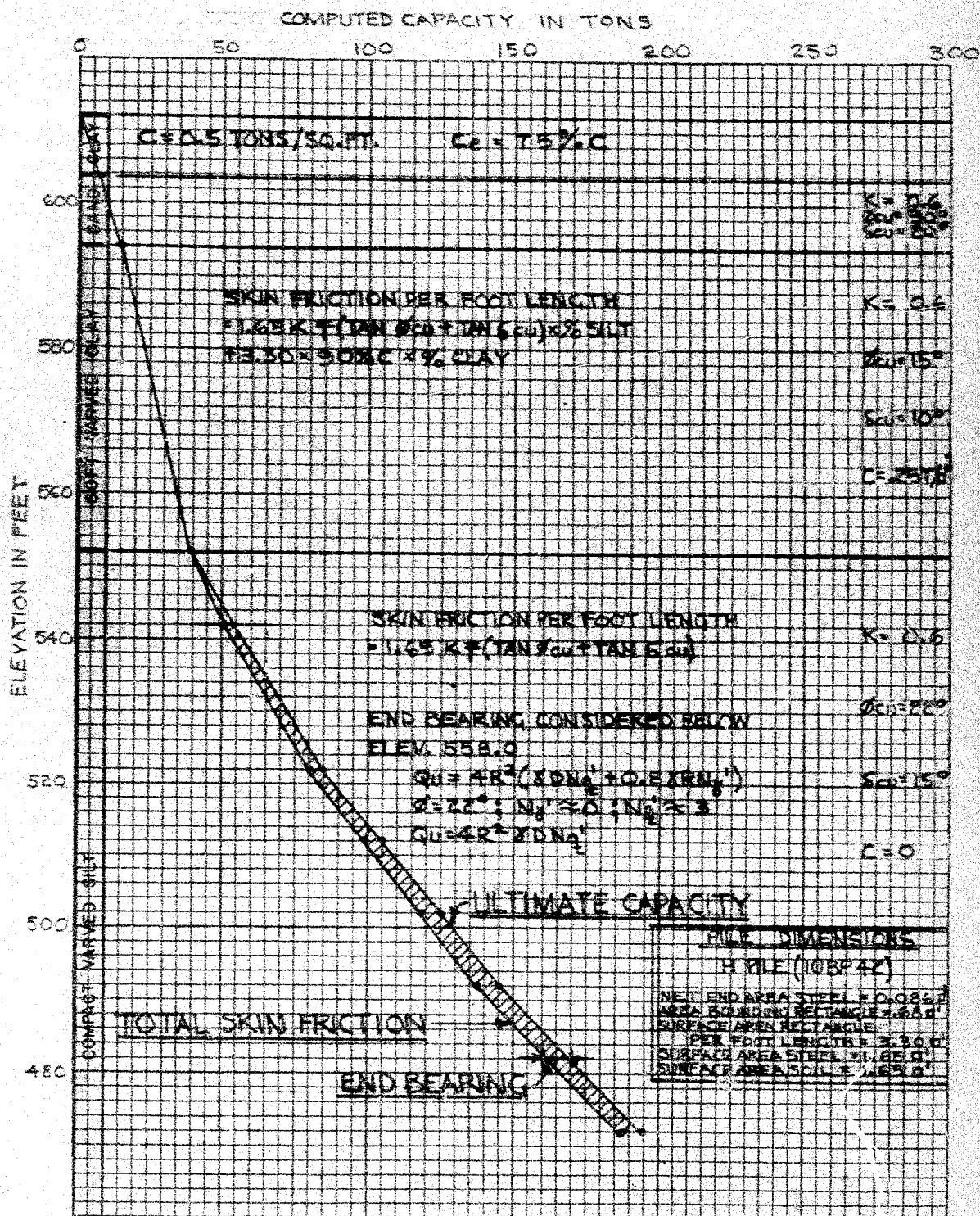
COMPUTATION OF ULTIMATE CAPACITY

SINGLE STEEL H PILE (10 BP 42)

APPENDIX

FIGURE 6

PROJECT S665T



DISCUSSION
PRELIMINARY FOUNDATION RECOMMENDATIONS
PROPOSED BIG FIC RIVER BRIDGE

1. Piled Foundations

- a) Type of pile
- b) Length
- c) Spacing
- d) Design load including group action
- e) Total settlement
- f) Driving
- g) Load tests

2. Stability

- a) Permanent stability
- b) Stability during construction
- c) Drainage and/or relief wells
- d) Possible piezometric observations

3. Specifications

COMPUTATION OF ULTIMATE CAPACITY

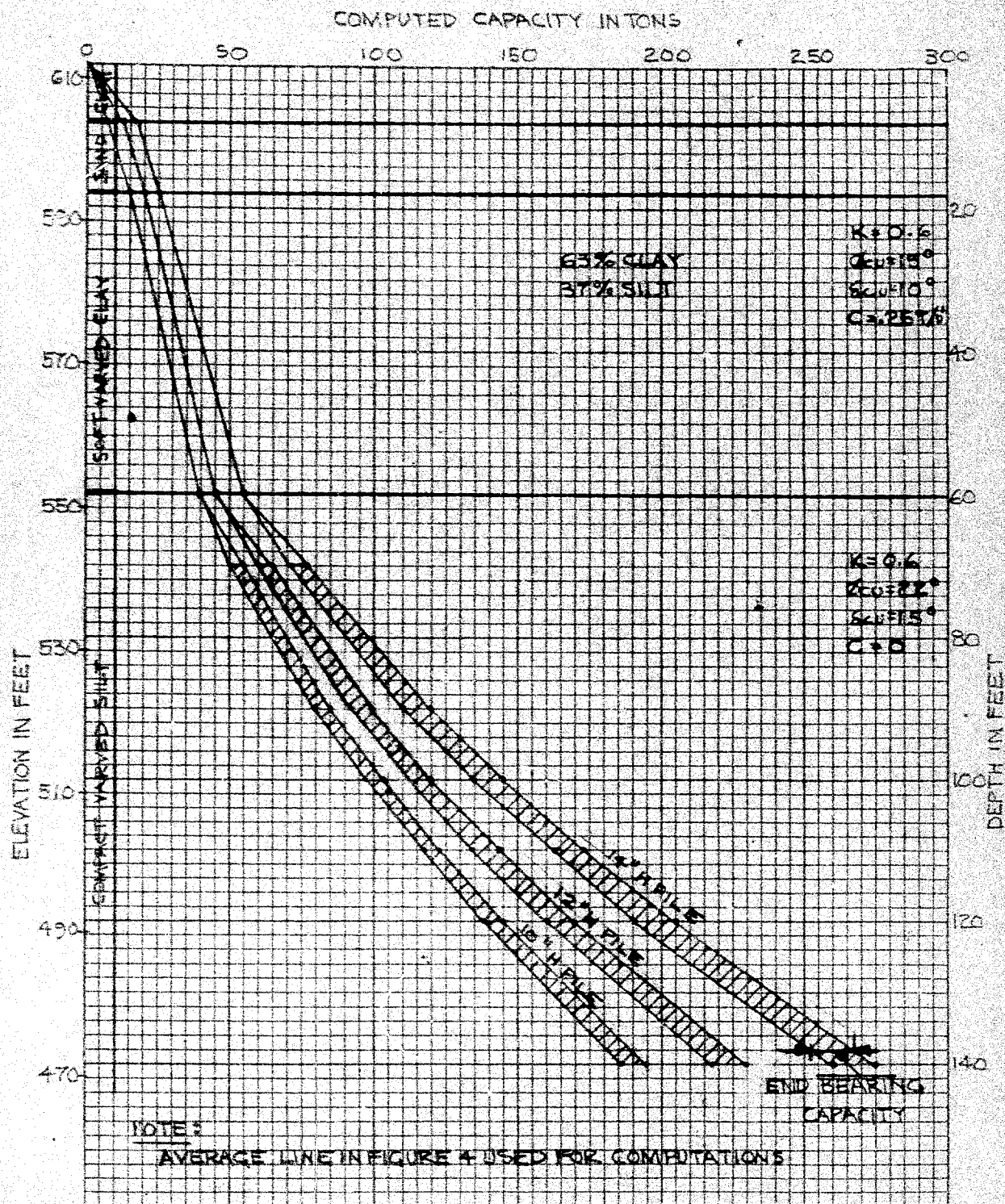
10, 12, & 14 INCH STEEL H PILE

APPENDIX

FIGURE

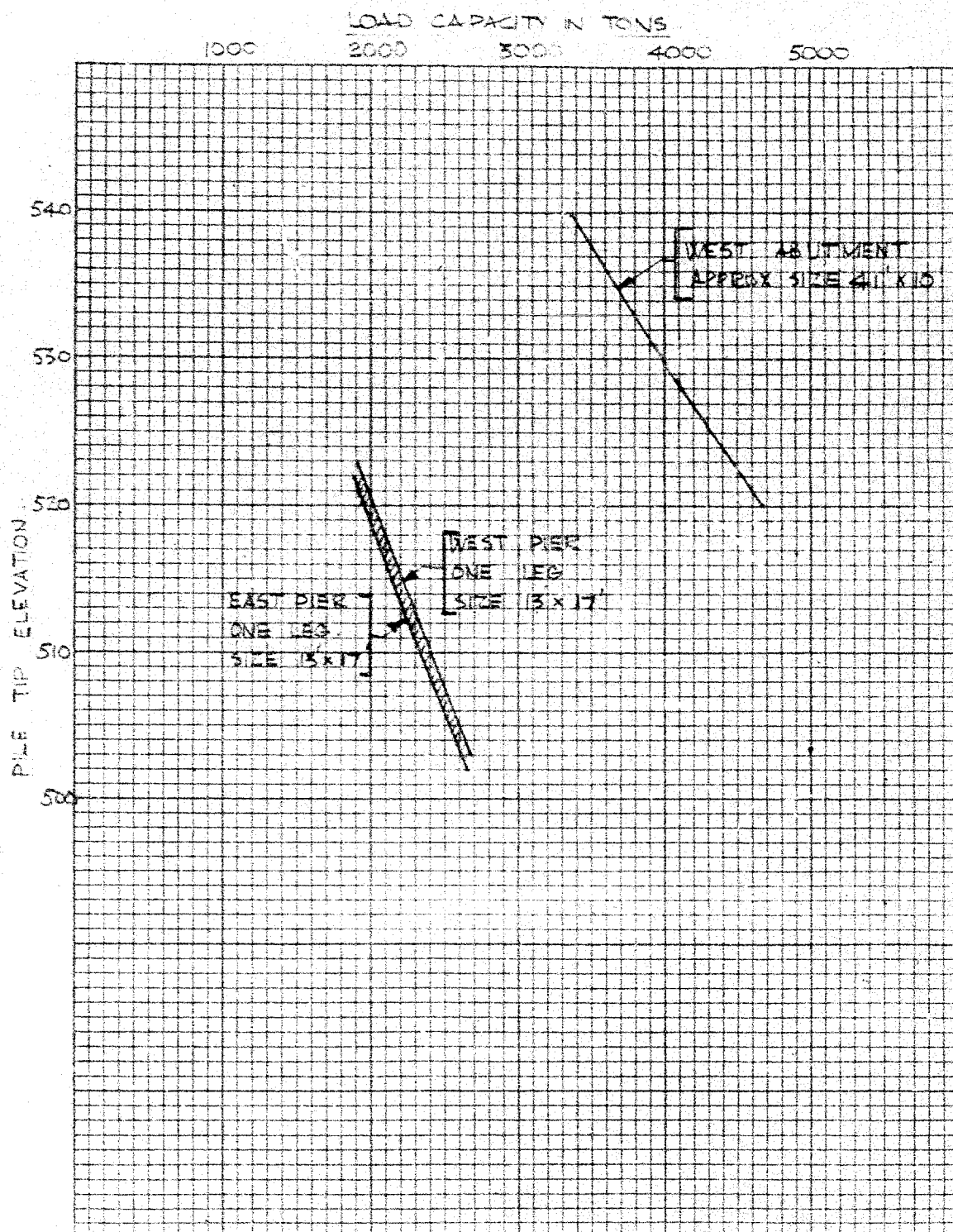
7

PROJECT S6657



COMPUTED ULTIMATE CAPACITY PILE GROUPS

APPENDIX
FIGURE 8
PROJECT S6657



NOTE : PILE LENGTHS 90 TO 110 FEET.

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SETTLEMENT VS N_o. OF FILES IN CLUSTER

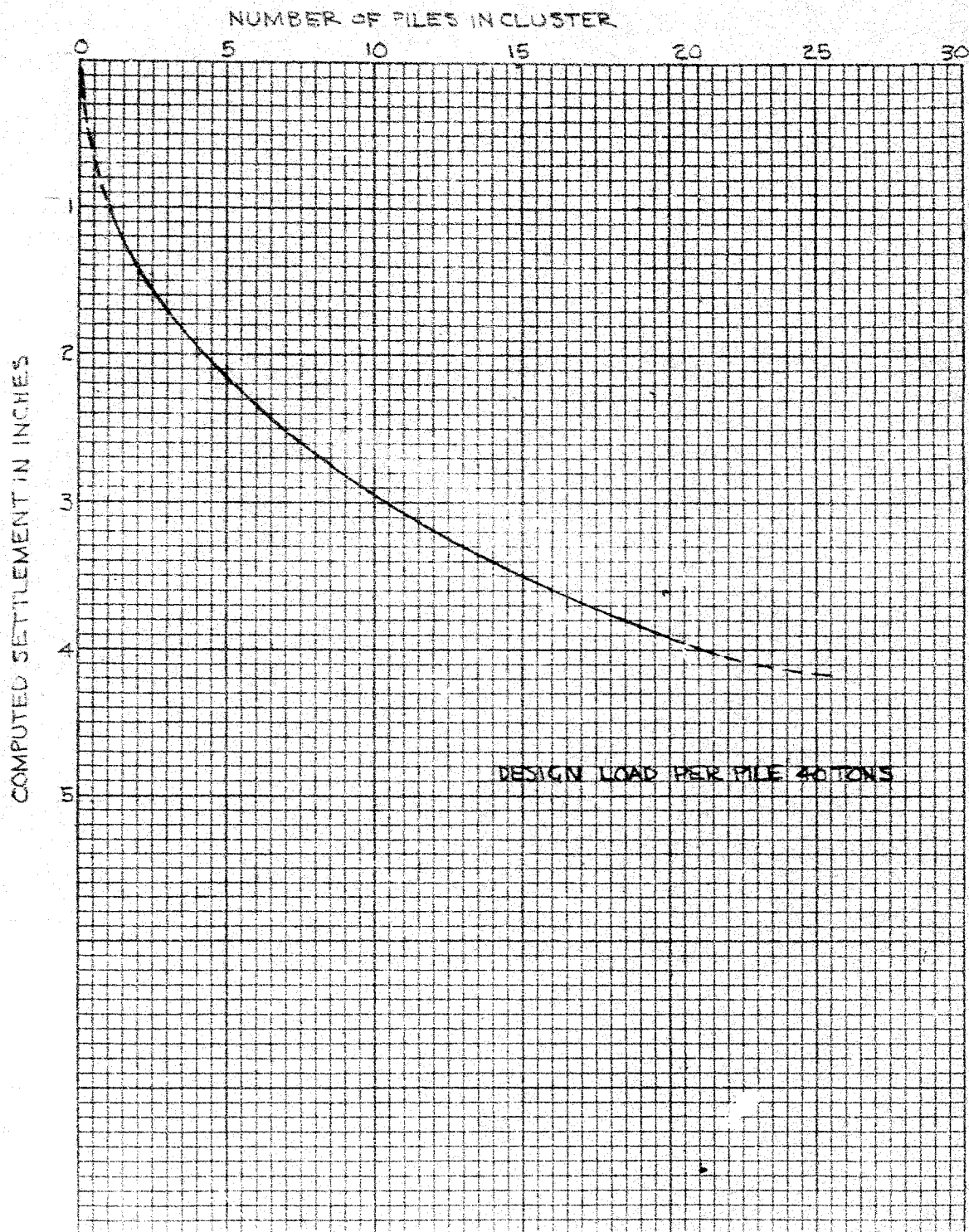
SPACING - FOUR FEET CENTRE TO CENTRE

APPENDIX

FIGURE

3

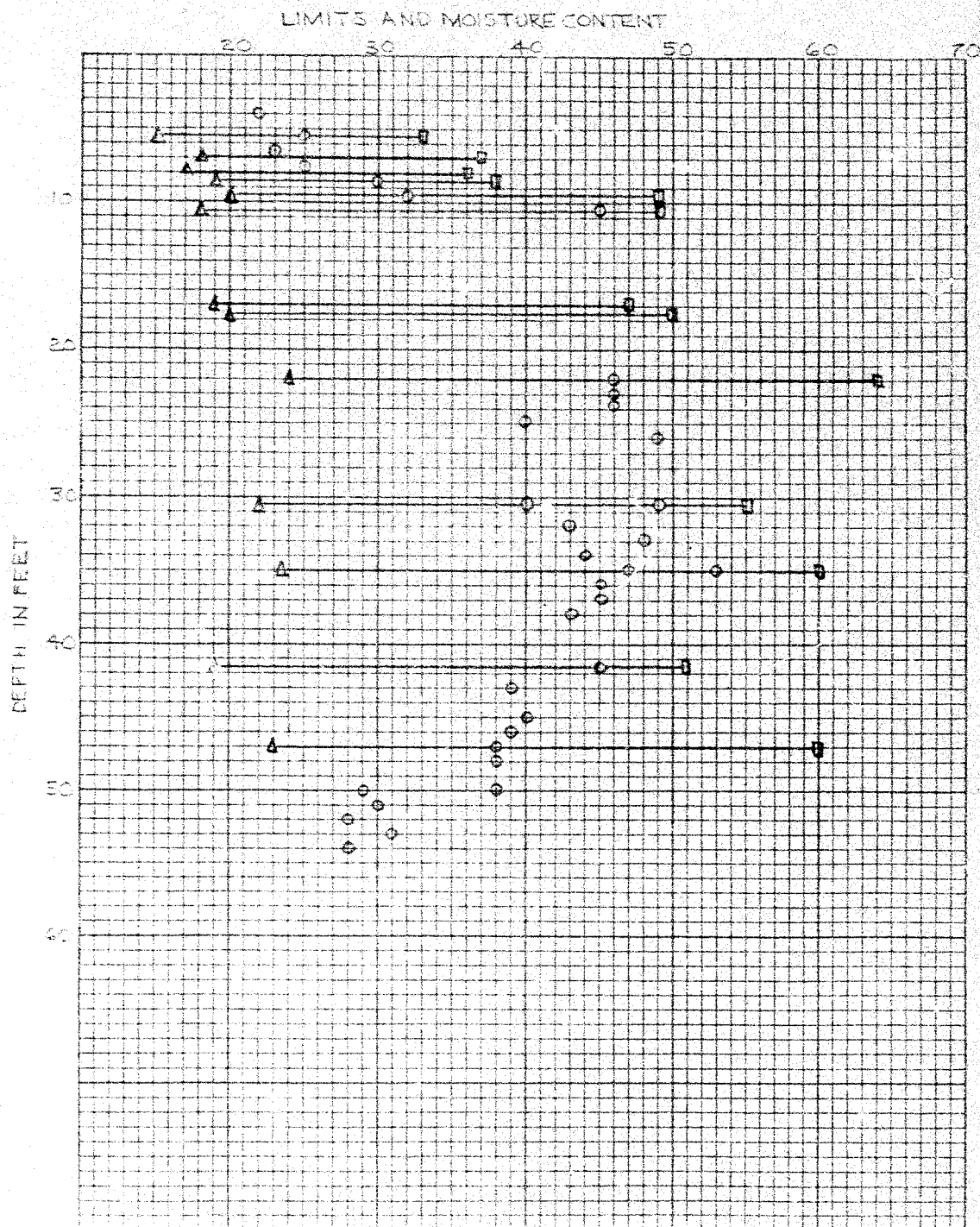
PROJECT S6657



LABORATORY RESULTS
TYPICAL FOIL SAMPLE BORESOLE

AT TERBURG LIMITS & MOISTURE CONTENT VS DEPTH BOREHOLE # 8

APPENDIX
FIGURE
PROJECT 56657



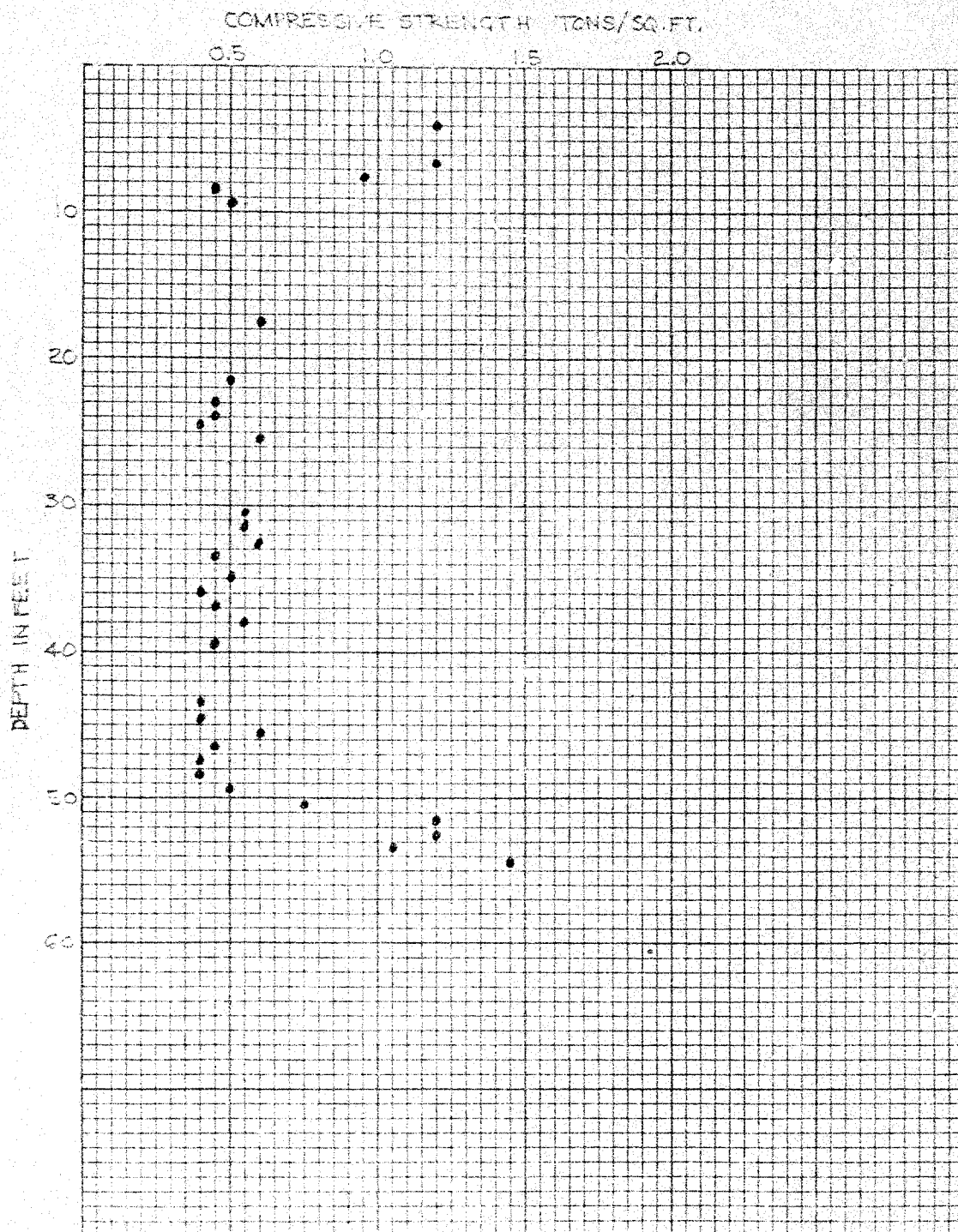
COMPRESSIVE STRENGTH VS DEPTH

BOREHOLE #8

APPENDIX

FIGURE

PROJECT 56657



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Rexdale, Ontario,
September 19th, 1958.

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TEL. CH. 5810

Department of Highways, Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A. M. Tove, P. Eng.,
Bridge Engineer.

Re Soil Investigation and Engineering Study,
Proposed Big Pic River Bridge,
Marathon, Ontario.

Dear Sirs:

This letter accompanies our detailed report covering the above soil investigation and engineering study.

We find that there is a deep stratum of compact silt at the site, which controls foundation design. The actual soil conditions encountered are described in detail in the report.

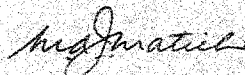
Based on the results of our study, it is recommended that a non-displacement type pile foundation be used for the bridge, and specifically that 110 feet long 12 BP 53 lb. steel "H" piles be used at an initial design load of 40 tons. Check pile loading tests accompanied by piezometer observations for control are recommended.

Recommended drainage and stabilizing measures for approach slopes and river banks, together with construction procedures are given in the report.

We believe that our report gives all the foundation information necessary for design of the bridge to be finalised. If we can be of further assistance, please do not hesitate to give us a call.

Yours very truly,

GEOCON LTD



M. A. J. Matich, P. Eng.,
Chief Engineer.

MMJM/cor
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SS657
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND ENGINEERING STUDY
PROPOSED BIG PIC RIVER BRIDGE
HIGHWAY 17
MARATHON ONTARIO

Distribution:

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INDEX

	<u>Page</u>
Introduction	1
Summarized Soil Conditions	1
Discussion	1
Conclusions and Recommendations	11
Personnel	13
Appendix I	
Procedure	
Site and Geology	
Soil Conditions	
Water Conditions	
Office Reports on Soil Exploration	
Appendix II	
Figures - Laboratory Testing	
Appendix III	
Figures - Ultimate Pile Capacity	
Appendix IV	
Photographs	
Appendix V	
Piezometers	
Appendix VI	
Pile Loading Tests	
Appendix VII	
Corrosion	

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INTRODUCTION

Gecon Ltd has been retained by the Department of Highways, Ontario by proposal dated March 26th, 1958, to carry out a soil investigation at the site of the proposed Big Pic River Bridge on Highway 17 near Marathon, Ontario. The object of the investigation was to determine in detail the soil conditions at the site, to review the present foundation analysis and design and to give specific recommendations on foundation design and construction procedures for the proposed bridge.

SUMMARIZED SOIL CONDITIONS

Along the centre line of the proposed bridge, the site is covered by 2 to 3 feet of loose to compact sand and gravel fill, underlain by a stratum of light grey and brown clay and silt, up to 24 feet in thickness which is generally of firm to stiff consistency, except in the lower ranges where it is soft. This stratum is underlain by compact medium sand at the west bank and by both compact medium sand and loose sandy silt at the east bank of the river. The sand and silt deposits varying in thickness from 5 to 15 feet are underlain by about 30 to 40 feet of soft to firm grey varved silty clay which grades into a compact varved silt which in turn grades to silt. The varved silt to silt stratum is about 200 feet in thickness at the east bank and was found to overlie a stratum of fine grey silty sand of undetermined thickness. The sand is probably underlain by volcanic bedrock.

The side slopes of the approach cuts generally consist of nearly horizontal layers of clay and silt with some shallow sand layers.

DISCUSSION

1. General

The proposed bridge will carry Highway 17 over the Big Pic River about 325 feet north of the existing Bailey Bridge and about 8 miles east of Marathon, Ontario.

1. General (continued)

It is understood that the proposed bridge will be a three-span cantilever type steel truss structure. The centre suspended span will be 135 feet long and the end cantilever sections each 225 feet long. The general arrangement of the bridge as presently proposed is shown on Procter & Redfern drawing D-3712-1. The abutments will be reinforced concrete and the piers of steel bent construction. Fixed anchorages will be provided at both abutments and the suspended span will have a pin bearing at the west end and a rocker bearing expansion joint at the east end of the span. Longitudinal movements of the structure will be taken up by the flexible pier members which are provided with rocker bearings at the top and bottom.

It has been initially proposed to found the piers and abutments on friction piles. To allow for possible settlement of the piers and abutments provision has further been made in the structural design to allow for jacking and shimming of the bearing plates to a maximum vertical distance of about 4 inches.

2. Piled Foundations

In our initial engineering studies both shallow and deep spread footing foundations for the proposed structure were examined. However, due to the high compressibility and low shearing strength of the upper strata at the site, it is considered that a piled foundation, as planned, is the only type of foundation feasible.

From examination of the stability of the present river banks adjacent to the proposed pier locations it is considered that remoulding and possible liquefaction of the laminated clay and silt strata should be kept to a minimum. Consequently it is recommended that a non-displacement type pile be used for the foundation. In view of this and the probable length of pile required to mobilize effective skin friction in the lower compact silt stratum an H pile section was used for preliminary design computations.

2. Piled Foundations (continued)

The principal soil strata at the site consist of about 40 feet of soft to firm varved clay underlain by about 200 feet of compact silt which in the upper 20 feet is distinctly varved. The clay content in the varved silt becomes negligible at about a general elevation of 550 across the site. Computations of ultimate pile capacity were based upon the measured laboratory shear strength of the upper clay stratum and on the estimated relative density of the silt strata. These computations utilizing methods proved by past experience serve to establish the initial design load per pile; however it is recommended that the design load be checked by pile load tests prior to final construction.

The shear strength of the clay, as obtained from slow drained tri-axial tests, was approximately 400 pounds per square foot and the mean angle of shearing resistance was 25 degrees. Measured penetration resistance or "N" values obtained in the silt strata were correlated with relative density and typical angles of shearing resistance as shown graphically on Figure 3, Appendix III. Laboratory results on similar silts and results of research by other Authorities, noted on the Figure, were used in this correlation. Figures 1 and 2, Appendix III, show plots of "N" values as a function of elevation. The "N" values obtained were corrected both for soil type and for overburden effects. The effective "N" values thus obtained after correction were used in the computations.

Initially, ultimate pile capacity computations were carried out using a 10 HP 42 lb. H pile. The results of the computations together with the assumptions and methods of computation used are given on Figures 4, 5 and 6 of Appendix III. Figure 4 shows the computed ultimate bearing capacities, neglecting the varved clay and overlying deposits. Figure 5 includes the skin friction effect of the overlying varved clay. As shown on Figure 6, Appendix III, the values of effective skin friction were computed on the basis of the earth pressure coefficient, K_0 , the angle of shearing resistance ϕ cu

2. Piled Foundations (continued)

from consolidated undrained triaxial tests, and the angle of wall friction δ cu. In the computations δ cu was taken as approximately equal to $2/3$ of ϕ cu.

The curve shown for a "quick silt" condition in Figures 4 and 5 is considered to represent the minimum pile capacity. Computations show that although the porosity of the silt is high, about 40 percent, it is still below the critical value of porosity which is normally in the range of 45 to 50 percent. Based on both the corrected and uncorrected standard penetration resistance or "N" values the relative density of the silt is estimated to be of the order of 0.6. Experience shows that silts or fine sands are not likely to liquefy unless the relative density is below about 0.4 to 0.5. Consequently it is considered that the assumption of a "quick silt" condition is conservative. The maximum pile capacity has been computed assuming K_0 equal to 1.0. Recent studies have shown that K_0 may reach a maximum value of 1.5 in silty soils under a pile settlement of about one thousandth of the pile length; however, at the site investigated, as the effective overburden pressures may be reduced by the artesian condition encountered in the silt stratum, it is considered that the computed capacity taking K_0 equal to 1.0 represents the probable maximum condition.

Further computations were carried out using the empirical formulae for the ultimate capacity of friction piles in fine granular soils, derived by Meyerhof. These are also plotted on Figures 4 and 5. Examination of the above methods of analysis indicates that the mean pile capacity may be represented in the present case by assuming K_0 equal to 0.6. The probable average pile capacity has been computed on this assumption and is shown on Figure 6, Appendix III. For reasons of comparison the computed ultimate capacities of 12 and 14 inch "H" piles together with the 10 inch "H" pile of Figure 6 are given in Figure 7, Appendix III.

Computations of individual pile capacity were next applied to the ultimate capacity of a pile group. The capacity of the pile group as a unit

2. Piled Foundations (continued)

was assumed equivalent to that of a deep footing of the same outside dimensions and depth of penetration under the same applied load. It was computed that no group reduction factor need be applied if the spacing of piles, centre to centre, was at least 4 times their maximum side dimension, and if the outside piles in each group be driven on a slight batter which would increase the group perimeter with depth. Check computations of the pile arrangement proposed on Frocter & Redfern drawing B-3712-1 for a pile spacing, centre to centre, of 4 times the side dimension and the outside piles on a batter of 1 in 10 show that the ultimate group capacity is a simple function of the number of piles multiplied by the computed individual pile capacity.

Based on the computed pile capacities versus length as shown on Figure 7, Appendix III, and considering the rapid decrease in the relative density of the silt stratum below about elevation 520 on the west bank of the river and elevation 500 on the east bank, it is recommended that a pile length of 110 feet be used for foundations. Since the change in relative density with depth of the silt has been observed to occur at an approximately constant depth below ground surface at the site, it is considered that a pile length of 110 feet will be necessary for both pier and abutment foundations on both sides of the river. For a pile length of the order of 110 feet, transverse vibration of the pile during driving may induce local liquefaction of the silt surrounding the pile. At the same time, difficulties may be encountered in maintaining a relatively flexible pile truly vertical during driving. Consequently, to reduce excessive flexure and possible liquefaction, it is recommended that a 12 HP 53 lb. "H" pile be used in preference to the 10 HP 42 lb. "H" pile as proposed. It may be noted that for an increase of about 30 percent in initial cost of the piling an increase of almost 100 percent in pile rigidity will be obtained. It is further recommended in order to minimize transverse pile vibration that the piles be driven with a heavy hammer comparable to a Vulcan No. 0.

2. Piled Foundations (continued)

From Figure 7, Appendix III, the computed ultimate capacity of a 12 inch pile, for a pile length of 110 feet is about 140 tons. Applying a normal factor of safety of 3, it is recommended that a 40 ton pile load be used for design.

It is recommended that the computed pile capacity be checked in the field by a pile loading test at each abutment and pier location. These loading tests should be carried out prior to the final piling programme. It is recommended that 3 test piles be driven at each pier and abutment location at the minimum spacing specified above. The centre pile in each group of 3 should then be load tested. The method of loading the test pile as given on Procter & Redfern drawing J465 is considered suitable and a suggested load test procedure is detailed in Appendix VI.

Settlement computations were carried out for different pile groups, with piles spaced at 4 feet centres under the design load of 40 tons per pile. The results of the computations are given on Figure 8 where the computed settlement of the group is plotted as a function of the number of piles in the group. For an expected maximum of 30 piles in a group, the computed total settlement is about 4 inches. It is estimated that the greater part of this settlement will take place during construction.

Pile Corrosion

It is understood that, in the present bridge design, allowance has been made to protect the upper 18 feet of steel piling, where used, from corrosion effects. The protection will consist of a concrete cover approximately 18 inches in section. Details of the protective measure and the method of formation are shown on Procter & Redfern drawing D3712-2.

Laboratory and field tests have shown that corrosion will take place to varying degrees along the complete embedded length of a steel "H" pile in

2. Piled Foundations (continued)Corrosion (continued)

clayey soils. This is discussed in more detail in Appendix VII. It is concluded from the above and the results of our laboratory tests that the specified protective measure will only give a partial protection against corrosion. At the same time, the driving of a steel box section around each "H" pile in pier and abutment groups in order to form the proposed protective measure could induce local liquefaction of the upper strata at the site. It is considered that, in view of the low structural pile stresses induced under the recommended pile load of 40 tons, adequate protection against corrosion within the economical life of the bridge structure will be ensured by the adequate sectional area of a 12 HP 53 lb. "H" pile. If this pile is used, no protective measures are considered necessary.

3. Stabilitya. Approach Cuts

During the period of the field investigation, the approach cuts were only partially excavated. The determination of the soil stratigraphy in exposed excavations was complicated by local flow-outs of sand and silt which partially covered the excavation. A sketch of the observed soil stratigraphy is shown on Drawing S6657-2. The slopes were generally found to be composed of sandy silt and varved silt with occasional shallow sand layers. Analyses of slope stability were based on the measured shear strength of the clay where it occurs at the bridge site, in slow drained triaxial tests. Based on the results, it was computed that slopes up to 20 feet in overall height would have an adequate factor of safety against both a spreading type failure and a circular slide, provided that the side slopes were at least $3\frac{1}{2}$ horizontal to 1 vertical. This was confirmed by observation of natural slopes in the area which were noted to be generally stable at a slope angle of 16 degrees.

3. Stability (continued)a. Approach Cuts (continued)

Consequently it is recommended that all slopes be trimmed to at least 4 to 1. If the total height of the bank is greater than 20 feet, a berm 25 feet wide should be provided as indicated on Drawing S6657-3.

In the sequence of layers of clay, coarse silt and sand in the approach slopes, the coarser layers are commonly water-bearing during a large part of the year. The water seeping out of these layers is leading to sloughing; frost action during the winter will cause further deterioration of the slope. It is therefore recommended that at least 2 feet of clean granular borrow be placed on all slopes to minimize sloughing. Adequate toe drains should be provided at the base of all cuts and backfilled with graded filter material to prevent clogging as shown on Drawing S6657-3. A suggested grading to meet the filter requirements is given on Figure 9 of Appendix III.

b. Abutments

At the time of the investigation, the sand and gravel fill covering the approach cuts was saturated and water carrying. In order to avoid excess water during construction and to avoid surface erosion, it is recommended that drainage be provided on the land side of each abutment. A suitable interceptor drain would be provided by laying a 12 inch diameter perforated pipe in trench excavation, carried down about 1 foot into the oxidized clay and silt stratum on the west bank and into the loose sand stratum on the east bank. The drain should be placed perpendicular to the centre line of the approach cuts and back-filled with graded filter material. It is recommended that this drain be connected to the proposed 24 inch diameter longitudinal sewers about 200 feet distant on each side of the bridge centre line and shown on Procter & Redfern drawing B3712-1. A suggested grading to meet the filter requirements is given on Figure 9 of Appendix III.

3. Stability (continued)b. Abutments (continued)

Stability computations show that the long term factor of safety against general shear failure of the abutments and approach embankments following construction is adequate.

Analysis of settlement of the approach embankments to the abutments showed that because the applied net load on the varved clay stratum due to the weight of the embankment will not exceed the maximum past overburden pressure, settlement due to consolidation of the varved clay stratum will be small. It is however recommended that, to allow for the expected settlement of the friction piles under load and consequent minor horizontal translation of the abutment, the clearance between the front face of the abutment ballast wall and the centre line of the abutment anchorage pins be increased from 1 foot 6 inches to 2 feet.

c. Piers and River Banks

As discussed above, it is recommended that drainage measures be provided at the land side of each pier similar to the details given under "Abutments". In this case, the interceptor drains should be placed one foot into the oxidized clay stratum on each side of the river.

The stability of the river banks and piers was examined in detail. The east bank of the river, because of the depth of soft varved clay encountered at this location was found to be the more critical from a stability point of view. Both long term stability and stability during construction were considered. Computations based on effective stress analyses using strength parameters obtained for the clay from slow drained triaxial compression tests, showed that the factor of safety of the east river bank for long term stability was greater than 2. For computation purposes, level of the river was assumed at the lowest recorded elevation of 602.

3. Stability (continued)c. Piers and River Banks (continued)

In analyzing the stability during construction, both sliding wedge and circular arc type failures were considered. Remoulding of the clay during pile driving was assumed to take place between lines a-a and b-b on Drawing S6657-3. The most critical slip circle obtained by calculation is shown on the drawing, and had a factor of safety of about 1.4. However, as the sandy silt and sand strata are loose and saturated, pile driving may cause excessive build up of pore pressure in the silt layers and lead to a spreading type failure of the river bank during construction. In order to check this possible pore pressure build up and to determine if the installation of relief wells will be necessary for stability, it is recommended that at each pier location, piezometers be installed prior to the driving of the specified test piles. Suitable piezometers are described in Appendix V. It is further recommended that the piezometers be installed in sets of three on the land side of the test group. It is suggested that the piezometers be placed in the pattern and to the depths shown on Drawing S6657-4. A continuous record of piezometric levels should be kept during the driving of the test pile group and during subsequent loading of the test pile. The records obtained from the piezometric observations will determine if relief wells or wellpoints will be necessary to reduce excessive pore pressure during driving of the pier pile groups and construction of the piers.

To safeguard against possible river bank erosion and as an additional stability measure during construction, it is also recommended that the existing river bank on each side of the river be trimmed to a slope not steeper than 2 to 1 and covered by a coarse sand and gravel blanket 3 feet in thickness and protected by heavy random rip rap. Stones greater than 50 pounds weight are suggested for rip rap purposes. The blanket and rip rap should extend from river bottom to about 10 feet back from the top of the bank as shown on Drawing S6657-3.

4. Construction Procedure

Based on the results of this study, the following foundation construction procedure is recommended.

- i. Trim and blanket the approach cut slopes in accordance with Drawing S6657-3.
- ii. Provide site drainage by installing toe drains along the approach cut slopes, connecting with transverse drains on the land side of piers and abutments.
- iii. Place granular blankets and rip rap cover at the river banks as shown on Drawing S6657-3.
- iv. Install piezometers at the pier locations as shown on Drawing S6657-4.
- v. Drive test and anchor piles at the pier and abutment locations.
- vi. Load test piles and, at the pier locations, observe the piezometers.
- vii. Install wellpoints or relief wells depending on the results of the control observations in the piezometers.
- viii. Drive the pile groups, preferably abutment groups before pier groups, and working outwards from the centre of each group.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by 2 to 8 feet of loose to compact sand and gravel fill underlain by up to 24 feet of soft to stiff clay and silt, followed by deposits of medium sand and sandy silt. These deposits are underlain by about 30 to 40 feet of soft to firm varved clay which grades into a very deep stratum of compact varved silt to silt.

2. The water level in the river during the time of the investigation was at elevation 692. Groundwater level was generally at or close to ground surface.

3. Based on the results of this study, it is recommended that a non-displacement type pile foundation be used for the bridge and specifically, that 110 feet long 12 BP 53 lb. steel "H" piles be used.

4. Computations of bearing capacity of single steel "H" piles have been carried out and group action has been considered. The procedures have been discussed in the report and the results are summarized on the Figures of Appendix III.

5. Based on computations, a design load of 40 tons is recommended for use in preliminary design.

6. It is recommended that check pile load tests be carried out at each of the abutment and pier locations prior to construction to confirm the design load.

7. A maximum settlement of 4 inches has been computed for a pile group comprising 20 piles under the design load of 40 tons. It is considered that the greater part of this settlement will occur during construction.

8. A heavy hammer, comparable to the Vulcan No. 0, is recommended for pile driving in order to minimize possible disturbance to the soil during driving.

9. Analyses of the river bank stability, both during and after construction have been carried out and are discussed in the report.

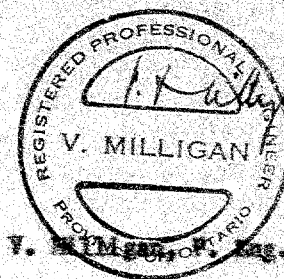
10. It is recommended that the river banks be protected with a weighted filter and rip-rapped as discussed in the report.

11. To determine whether relief walls will be necessary adjacent to the pier pile groups, it is recommended that piezometers be installed at the pier locations prior to test piling, as discussed in the report.

12. Measures to stabilise the slopes of the approach cuts are given in the report.

PERSONNEL

The field work was supervised by Messrs. A. Prior and R. Chevalier. The report was written by Mr. A. Prior and Mr. V. Milligan, assisted by Mr. R. M. Quigley and reviewed by Mr. M. A. J. Matich.



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APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

PROCEDURE

The field work was commenced on April 9th, 1935 and completed on May 2nd, 1938. At 6 locations continuous undisturbed samples were taken to a maximum depth of 65 feet using a Swedish Foil Sampler. An additional four boreholes were put down using a skid-mounted machine drillrig.

The location of the boreholes together with the inferred soil stratigraphy are given on Drawing S6657-1 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. A sketch of the soil stratigraphy in the exposed slopes of the approach cuts is given on Drawing S6657-2.

The laboratory testing of soil samples was carried out in the Soil Mechanics Laboratory of Gecon Ltd in Toronto. The results of the tests are plotted on the Office Reports and on the Figures in Appendix II. The samples remaining after testing will be stored until March 15th, 1939.

Elevations are referred to Geodetic Datum and were supplied by the Department of Highways.

Appendix III contains graphs and figures relating to pile bearing capacity computations.

Appendix IV shows photographs of typical soil samples.

Appendix V details the installation and measurements of piezometers.

Appendix VI describes in detail the loading and measuring procedures of test piles.

Appendix VII gives general comments and tests relating to the corrosion of steel piling in the soil encountered at the site.

SITE AND GEOLOGY

The proposed bridge will replace the existing Bailey bridge where Highway 17 crosses the Big Pic River, about 8 miles east of Marathon, Ontario.

From available geological information, it is known that the area of investigation is covered by a thick glacial deposit of varved clays and silts overlying volcanic bedrock of Keewatin age.

SOIL CONDITIONS

Since the original ground level has been changed by cutting back the river banks along the proposed centre line of the bridge in the fall of 1957, the soil conditions as existing at the time of the field investigation will be discussed under separate headings as follows:

A. Soil Conditions Along Centre Line

Loose to Compact Sand and Gravel Fill

The cuts along the centre line of the proposed bridge have been covered by fill in the fall of 1957. The fill consists of sand and gravel in all grain sizes and contains about 10 percent silt sizes as indicated by the grain size curve of Figure 1 of Appendix II which shows the grain size distribution of a typical sample of the fill.

The thickness of the sand and gravel fill varied from 2 to 5 feet at the east bank to 5 to 8 feet at the west bank of the river.

During the time of the field investigation, the fill was usually in a very loose to loose state in the upper 2 feet of the stratum, due to surface drainage of the adjoining hill slopes through the fill. In several cases, the upper foot of the fill was in a quick condition. The fill in the lower ranges of the stratum was generally loose to compact as indicated by the results of standard penetration tests with "N" values ranging from 7 to 24 blows per foot.

A. Soil Conditions Along Centre Line (continued)Soft to Stiff Light Gray and Brown Clay and Silt

Beneath the fill along the centre line and at the surface near the sides of the excavation is a stratum of light gray and brown clay and silt. The thickness of the stratum beneath the fill ranged from 1 to 6 feet at the east bank to 3 to 24 feet at the west bank of the river. The maximum thickness encountered was 24 feet in borehole 3. The structure of the stratum is very indistinct, but in general, the stratum consists of layers of clay, about 1 inch in thickness, separated by about 6 inches of very silty clay to silt. The layers, where noticeable are usually inclined at an angle between 10 and 20 degrees to the horizontal. The upper portion of the stratum is mainly light gray-brown to brown in colour due to oxidization of organic material. Oxidized fossil-like remains of leaves, cones, roots and twigs were encountered. The remaining non-oxidized portion of the stratum is light gray in colour.

The upper 2 feet of the stratum in boreholes 1, 2, 3 and 7 adjacent to the river, where excavation has been minimal, have a peat content of about 30 percent of the total dry weight. Fragments of shell were occasionally encountered in the stratum. In boreholes 1 and 3 about 6 and 12 inches of grey brown medium to coarse sand were encountered at 4 and 7 feet below ground level respectively.

The results of laboratory testing carried out on samples of the stratum are summarized in the following table:

	<u>Min.</u>	<u>Average</u>	<u>Max.</u>
Unconfined Compressive Strength (tons per square foot)	0.27	0.45	1.22
Moisture Content (percent)	17	30	53
Wet Unit Weight (pounds per cubic foot)	105	116	130
Liquid Limit	29	36	53
Plasticity Index	7	18	33

A. Soil Conditions Along Centre Line (continued)Soft to Stiff Light Gray and Brown Clay and Silt (continued)

The compressive strength of the stratum is plotted as a function of elevation on Figure 2 of Appendix II. The upper portion of the stratum has been desiccated and the compressive strength is comparatively high. The remainder of the stratum is normally loaded and the compressive strength after decreasing to a minimum value at about elevation 607, gradually increases with increasing depth. Typical stress-strain curves obtained from unconfined compression tests on samples of the stratum are given on Figure 3 of Appendix II.

Depending on the silt and clay content of the different layers in the stratum, the moisture content varies between 17 and 53 percent. Similarly, from the results of Atterberg limit tests, the plasticity of the clay varies from very low to very high.

Loose Gray Very Sandy Silt

At the east bank, underlying the clay and silt is a stratum of grey sandy silt with a maximum thickness of 10 feet encountered in borehole 2, diminishing in thickness in an eastward direction. The silt is very sandy as indicated by a typical grain size distribution curve shown on Figure 1 of Appendix II. Organic matter was found dispersed throughout the stratum. Plate I in Appendix IV shows a photograph of the sandy silt in borehole 5 between a depth of 21 feet and 24 feet, 8 inches. The photograph also shows the cross bedded structure and the organic matter recognizable as numerous black spots and streaks.

Standard penetration resistance or "N" values obtained on samples of the stratum gave values of 2 and 8 blows per foot, indicating the relative density of the stratum to be loose.

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

A. Soil Conditions Along Centre Line (continued)Loose Gray Very Sandy Silt (continued)

The permeability of the silty sand as determined by falling head field permeability tests is about 5×10^{-5} centimeters per second.

A wet unit weight of 124 pounds per cubic foot at a corresponding natural moisture content of 17 percent was obtained.

Compact Gray and Brown Medium Sand

Underlying the clay and silt on the west bank in all the borings and on the east bank in borehole 4 and underlying the sandy silt in borehole 6 is a stratum of gray and brown medium uniform sand. The thickness of the stratum at the west bank ranges from 5 feet in borehole 7 to about 16 feet in borehole 9 and probably more in boreholes 1 and 3. At the east bank the maximum thickness encountered was about 12 feet in borehole 4, diminishing in thickness in a westerly direction. A typical grain size distribution curve of the sand is given on Figure 1 of Appendix II.

In an undisturbed sample of the sand in borehole 8, it was found that the sand was stratified at an angle of about 25 degrees to the horizontal. Within the sand in this borehole, two clay layers, about 1 inch in thickness were encountered, inclined at the same angle.

Standard penetration resistance or "N" values obtained on samples of the stratum ranged from 7 to 24 blows per foot with a general value of 18 blows per foot, indicating that the sand is generally compact.

The permeability of the stratum is estimated at about 10^{-2} to 10^{-3} centimeters per second.

Unit weight determinations gave an average wet unit weight of 119 pounds per cubic foot, the natural moisture content of a sample of the sand containing a clay layer was 25 percent.

A. Soil Conditions Along Contra Lias (continued)Soft to Firm Gray Varved Silty Clay

Beneath the medium sand at the west bank and beneath the sandy silt and medium sand at the east bank, is a layer of gray varved silty clay, about 30 to 40 feet in thickness. The varves are composed of dark gray, usually brittle, clay layers, about 1 to $1\frac{1}{2}$ inches in thickness and light gray silt layers, about $1/4$ to $1/2$ inch in thickness. Occasionally thin sand seams were encountered in the upper portion of the clay stratum. With increasing depth, the clay layers become thinner and the silt layers become thicker. Plate 1 in Appendix IV shows a photograph of a partially dried sample of the varved clay. The varves are usually inclined at angles varying between 5 and 25 degrees to the horizontal. Evidence of local failures was encountered in many samples of the stratum. Plate 1 shows failures having a failure plane angle of between 41 and 47 degrees to the horizontal. The same photograph also shows a local flow-out failure at a depth of about 27.5 feet. Plate 2 shows an enlargement of a similar failure observed in a different sample of the stratum. This photograph also shows the usually brittle structure of the clay. It further shows the fine laminations which become visible during desiccation of the sample.

The results of laboratory testing carried out on samples of the stratum are summarized in the following table:

	<u>Min.</u>	<u>Average</u>	<u>Max.</u>	<u>Unit</u>
Unconfined compressive strength	0.3	1.5	1.5	tons/sq.ft.
Axial strain at failure	2	8	18	percent
Effective shear stress	-	0.2	-	tons/sq.ft.
Effective angle of shearing resistance	-	25	-	degrees
Compression index	0.4	0.6	0.8	-
Sensitivity	-	3.5	-	-
Wet Unit weight	108	111	130	pounds/cu.ft.
Moisture content of varve	23	44	53	percent
Moisture content of clay in varve	14	50	65	percent

A. Soil Conditions Along Centre Line (continued)Soft to Firm Grey Varved Silty Clay (continued)

(continued)	<u>Min.</u>	<u>Average</u>	<u>Max.</u>	<u>Unit</u>
Moisture content of silt in varve	14	23	27	percent
Liquid limit	47	58	55	-
Plasticity index	28	36	42	-
Porosity of varve	-	45	-	percent
Porosity of clay in varve	-	59	-	percent
Porosity of silt in varve	-	38	-	percent
Void ratio of varve	-	0.8	-	-
Void ratio of clay in varve	-	1.4	-	-
Void ratio of silt in varve	-	0.6	-	-
pH value	8.67	8.1	8.24	-

Figure 4 of Appendix II shows a plot of unconfined compressive strength as a function of elevation. Typical stress-strain curves obtained from the unconfined compression tests are given on Figure 5. Figure 6 shows the Mohr stress circles obtained from slow drained tri-axial tests. The dash lines on the figure give the range of effective shear stresses and effective angles of shearing resistance which may be obtained from the figure. The solid line on the figure is taken as the mean envelope and the effective shear stress of 0.2 tons per square foot and the effective angle of shearing resistance of 25 degrees obtained from this line were used for design purposes. Figures 7 and 8 show the results of 2 consolidation tests on samples of the varved clay. The field virgin curves were obtained using the Schmertmann method and are shown on the figures. Figure 9 shows plots of moisture contents, wet unit weights and Atterberg limits as a function of elevation.

Compact Light Grey Varved Silt to Silt

With increasing depth, the clay content in the varved clay decreases gradually. At a depth of between 30 and 40 feet below the surface of the

A. Soil Conditions Along Centre Line (continued)Compact Light Grey Varved Silt to Silt (continued)

varved clay stratum, the silt and clay content become approximately equal and below this depth, the silt becomes predominant and the clay content decreases rapidly. The transition generally occurs at about elevation 565. Below this elevation the clay layers are between 1/8 and 1/2 inch in thickness and between 1 and 6 inches apart. Below about elevation 510, clay is practically non-existent. With further increase in depth, the silt becomes occasionally sandy and thin layers of fine sand were irregularly encountered.

Standard penetration tests carried out in this investigation and in a previous investigation by others, gave "N" values ranging between about 7 and 20 blows per foot with a general value of about 15 blows per foot, indicating that the stratum is generally compact. Figures 1 and 2 of Appendix III give plots of "N" values versus elevation at the east and west bank respectively. Both curves show a marked increase in penetration resistance at about elevation 550 and a general decrease again between elevation 530 and 520. This coincides with the occurrence of a hydrostatic head which was first observed when the casing was washed out to about elevation 542 in borehole 2. The hydrostatic head observed was about 18 inches above ground level. Below elevation 542 variable hydrostatic heads were observed with an average height of about 3 to 5 feet above ground level.

Laboratory testing on samples of the upper clayey portion of the stratum gave an average unconfined compressive strength of about 1.2 tons per square foot. The average wet unit weight was about 122 pounds per cubic foot at an average natural moisture content of about 28 percent. The average porosity obtained was about 38 percent at a corresponding void ratio of 0.6.

A. Soil Conditions Along Centre Line (continued)Grey Fine Silty Sand

In borehole 2, below elevation 362 a stratum of grey fine silty sand was encountered. The stratum was sampled by washing and density determinations were not carried out. Hydrostatic heads of between 10 and 20 feet above existing ground level were observed in the stratum, but it was found that excess hydrostatic pressure dissipated rapidly, sometimes in a matter of minutes. The borehole was terminated in this stratum at a depth of 300 feet below ground surface.

B. Soil Conditions of Hill Slopes

The slope exposures along the cuts as left after excavation in the fall of 1957 at the east and west banks have been examined and the observed soil conditions are summarized below.

The top stratum of the slopes at the east and west banks of the river and on the north and south sides of the cuts consists of light grey brown sandy silt. The silt is generally underlain by a layer of light brown fine and medium sand. Evidence of the presence of grey clay between the silt and sand was encountered in places. Beneath the sand is a stratum of varved silt varying in colour from light grey to grey brown and brown, depending on degree of oxidation. The "light grey and brown clay and silt" described under A is part of this stratum.

WATER CONDITIONS

At the time of the investigation the water level in the river was at about elevation 602. The groundwater level in the approach cuts which were in the drainage area of the surrounding hill slopes was generally at or within a foot from ground surface. The groundwater level in borehole 8 was at about 8 feet below ground surface.

Evidence of an excess hydrostatic head was encountered with increasing depth of the boreholes below about elevation 550. The apparent decrease in relative density with increasing hydrostatic head is discussed under the general stratum "Compact Light Grey Varved Silt to Silt".

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".

GEOCON

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56657 BORING # APPEX #1 DATUM GEODETIC CASING FOIL
 BORING DATE APRIL 10, 1958 REPORT DATE MAY 8, 1958 COMPILED BY MAX E. A. CHECKED BY P
 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

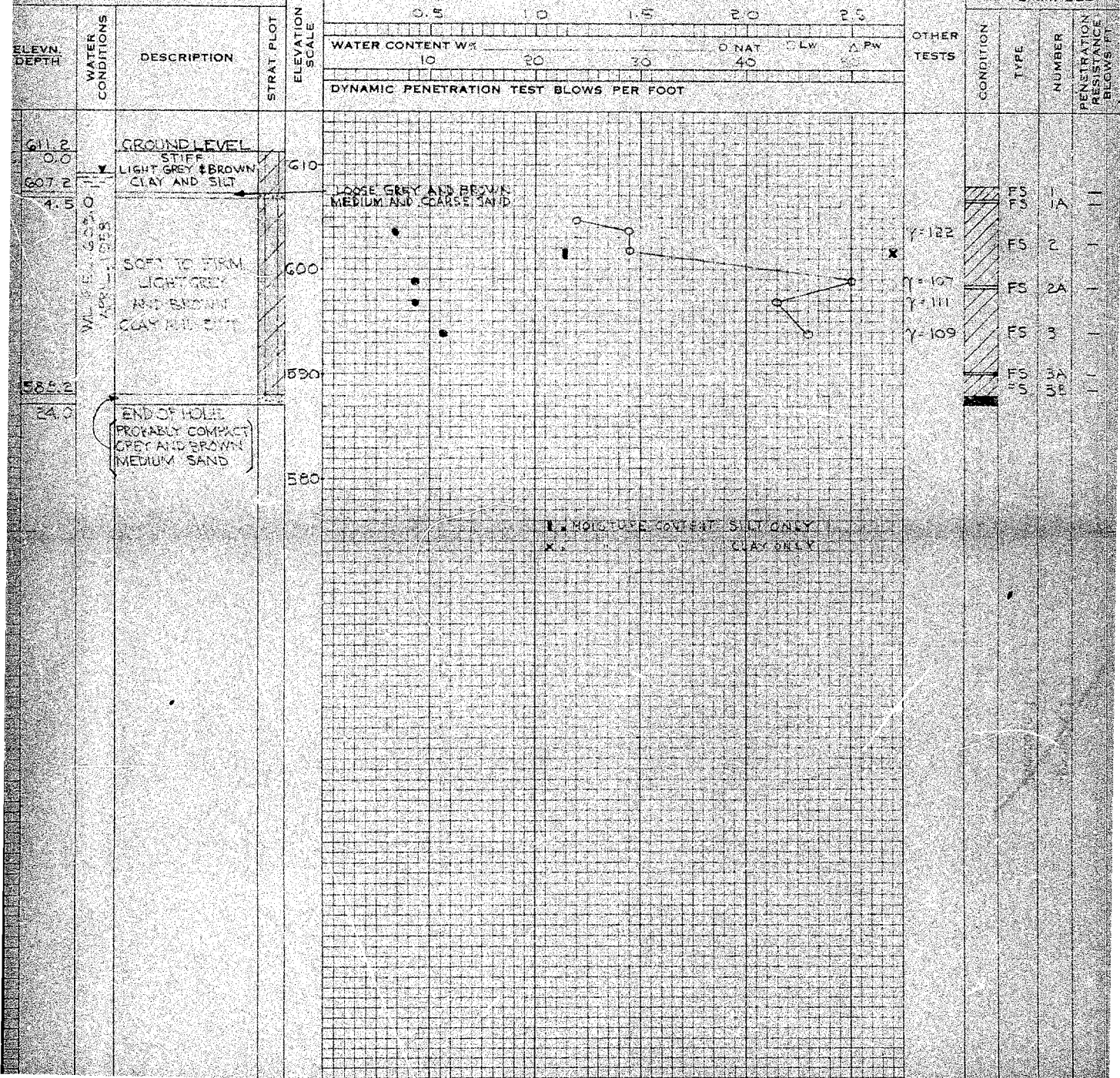
ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED 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

COMPRESSION STRENGTH - TONS/SQ. FT. • UNCONFINED

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56657 BORING # 2 DATUM GEODETIC CASING HX, BX, AX
 BORING DATE APRIL 11, 1958 REPORT DATE APRIL 28, 1958 COMPILED BY J.A. CHECKED BY PC
 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
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 S - TRIAXIAL SLOW
 1 - W.T. 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
611.8 0.0 608.8 3.0 603.3 8.0 593.8 18.0 561.8 50.0	WL & EL 610.2 APRIL, 1958	GROUND LEVEL LOOSE SAND AND GRAVEL FILL FIRM TO STIFF LIGHT GREY & BROWN CLAY AND SILT LOOSE GREY VERY SANDY SILT SOFT TO FIRM GREY VARVED SILTY CLAY		610 600 590 580 570 560 550 540 530 520

WATER CONTENT W% _____ O NAT. ☐ LW ☐ PW ☐

DYNAMIC PENETRATION TEST BLOWS PER FOOT _____

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	CS	1	1
	2'50	1A	9
	3'50 SF	2	12
	2'SF	4	8
	SF	5	4
	3'50	6	4
	2'50	7	3
	3'50	8	5
	2'50	9	5
	3'50	10	9
	50	11	8
	2'50	12	7
	3'50	13	12
	2'50	14	17
	50	15	15
	50	16	16
	50	17	28
	50	18	21
	W.S.	19	1
	2'50	20	21
	50	21	1

HYDROSTATIC HEAD FIRST OBSERVED @ EL 542.0 WITH HEAD TO EL. 613.1
 BELOW EL. 542.0 HYDROSTATIC HEAD'S VARYING BETWEEN EL. 613.0 AND 617.0

HYDROSTATIC HEAD FIRST OF
BELOWEL 542.0 HYDROSTAT

COMPACT
LIGHT GREY
VARVED SILT
TO SILT
WITH DEPTH

530

520

510

500

490

480

470

460

450

440

430

420

410

SO	16	16
SO	17	28
SO	18	20
W.S.	19	—
2 nd SO	20	25
SO	21	18
SO	22	20
SO	23	19
SO	24	16
2 nd SO	25	1
DO	26	14
DO	27	17
DO	28	15
DO	29	14
DO	30	13
W.S.	31	—
DO	32	—
W.S.	33	—
2 nd DO	34	—
W.S.	35	—
W.S.	36	—
W.S.	37	—
W.S.	38	—
W.S.	39	—
W.S.	40	—

361.8
250.0

MAXIMUM HYDROSTATIC HEAD OBSERVED IN
SILTY SAND STRATUM AT EL. 427.0

GREY FINE
SILTY SAND

311.8
300.0

END OF HOLE

400
390
380
370
360
350
340
330
320
310
300

W.S.	40	—
W.S.	41	—
W.S.	42	—
W.S.	43	—
W.S.	44	—
W.S.	45	—
W.S.	46	—
W.S.	47	—
W.S.	48	—
W.S.	49	—

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APP. #1

CONTRACT 56657 BORING # 3 DATUM GEODETIC CASING FOIL
 BORING DATE APRIL 16, 1958 REPORT DATE MAY 8, 1958 COMPILED BY M.W. & J.A. CHECKED BY A
 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

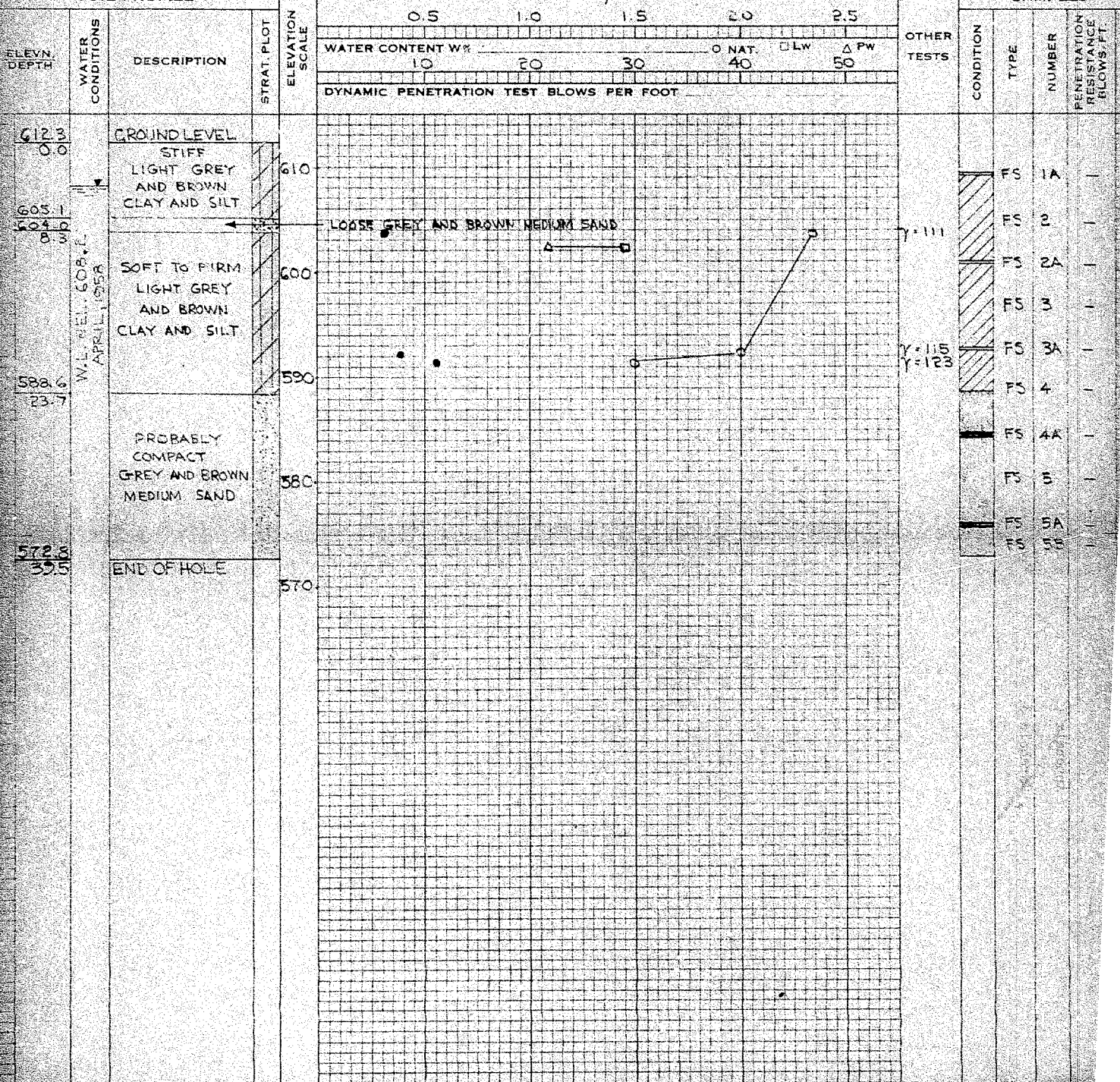
ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED 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

COMPRESSIVE STRENGTH - TONS / SQ. FT. • UNCONFINED

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPX. #1

CONTRACT 56657 BORING # 4 DATUM GEODETIC CASING FO
 BORING DATE APRIL 18, 1953 REPORT DATE MAY 2, 1953 COMPILED BY W.M. L.A. CHECKED BY A
 SAMPLER HAMMER WT. _____ LBS. DROP _____ 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

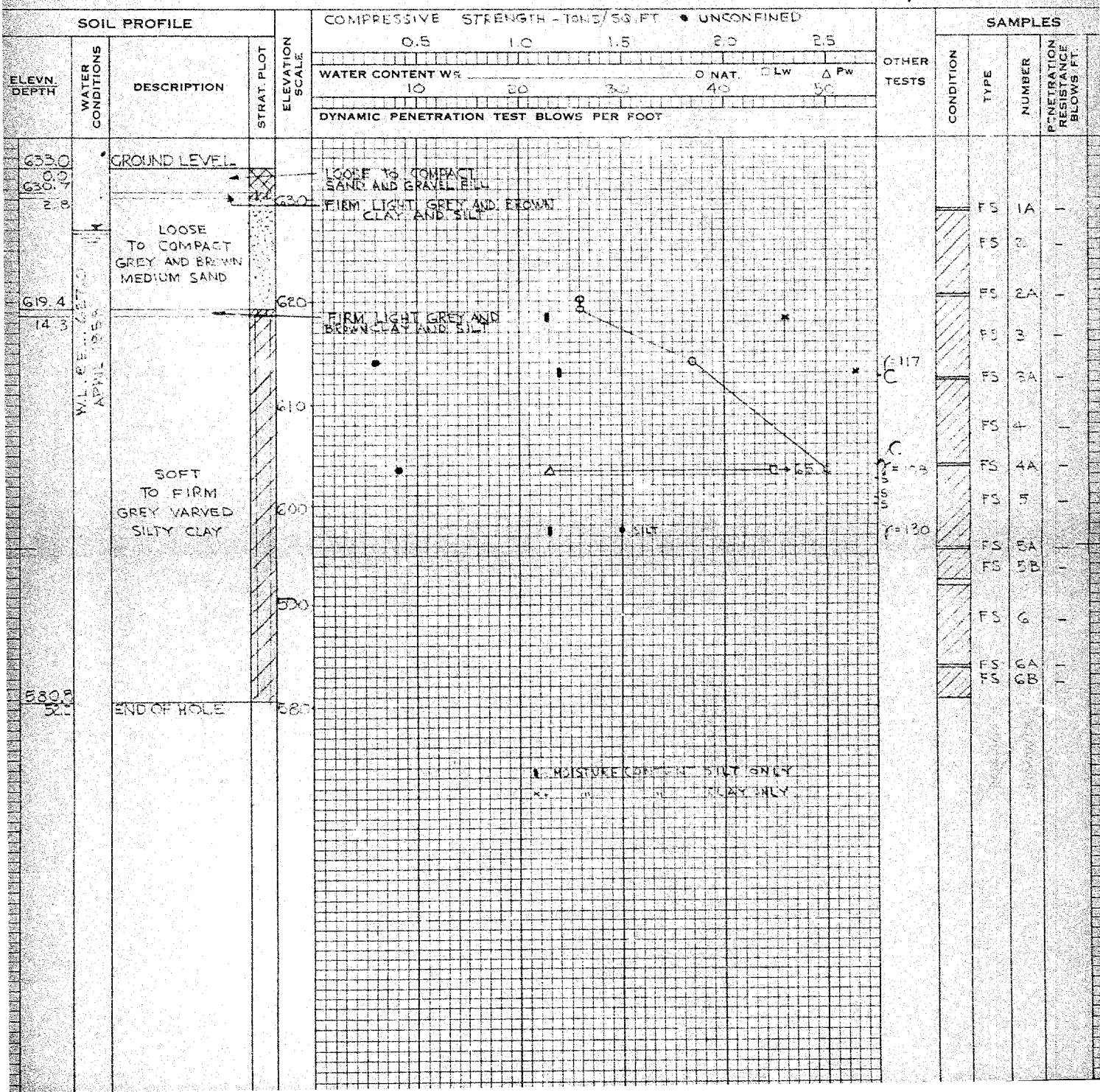
F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

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 56657 BORING # APPEX #1 DATUM GEODETIC CASING FOIL
 BORING DATE APRIL 19, 1958 REPORT DATE MAY 2, 1958 COMPILED BY MMW: JJA CHECKED BY AK
 SAMPLER HAMMER WT. LBS. DROP 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
 W.L. - WATER LEVEL IN CASING
 WT. - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
611.8 0.0		GROUND LEVEL		210
589.9 21.9	W.L. 589.9 DATE 4/19/58	START OF SAMPLING		600
587.2 24.6		LOOSE GREY VERY SANDY SILT		590
561.8 50.0		SOFT TO FIRM GREY VARVED SILTY CLAY		580
546.1 65.7		COMPACT LIGHT GREY VARVED SILT		570
		END OF HOLE		560
				550
				540

WATER CONTENT W% _____

○ NAT. □ LW △ PW

DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	FS	1	—
	FS	1A	—
	FS	2	—
	FS	2A	—
	FS	3	—
	FS	3A	—
	FS	4	—
	FS	4A	—
	FS	4B	—
	FS	5	—
	FS	5A	—
	FS	6	—
	FS	6A	—
	FS	6B	—

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPEX #1

CONTRACT 56687 BORING # 6 DATUM GEODETIC CASING FOIL
 BORING DATE APRIL 22, 1958 REPORT DATE MAY 2, 1958 COMPILED BY MW. F. A. CHECKED BY
 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

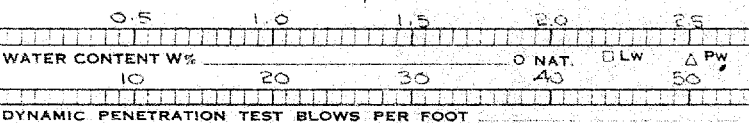
V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 R - REMOVED

γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION

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

SOIL PROFILE

COMPRESSIVE STRENGTH - TONS/SQ. FT. • UNCONFINED

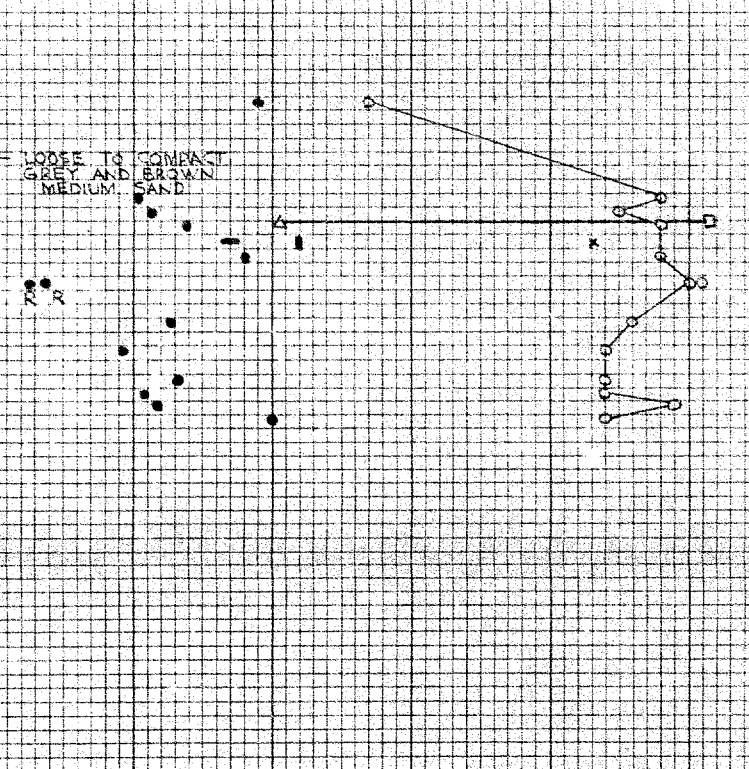


SAMPLES

OTHER TESTS

CONDITION TYPE NUMBER PENETRATION RESISTANCE BLOWS/FT.

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
620.9 0.0		GROUND LEVEL		620
614.4 6.5		FIRM LIGHT GREY AND BROWN CLAY AND SILT		
611.4 9.5		LOOSE GREY VERY SANDY SILT		
611.1				
	WL 610.5 AT 10.0			
		SOFT TO FIRM GREY VARVED SILTY CLAY		
568.9 52.0		END OF HOLE		570



γ = 125
 γ = 124
 γ = 123
 γ = 109
 γ = 112
 γ = 112
 γ = 110
 γ = 108
 γ = 108
 γ = 111
 γ = 111
 γ = 110
 γ = 111
 γ = 108
 γ = 111

FS	1A	-
FS	2	-
FS	2A	-
FS	3	-
FS	3A	-
FS	4	-
FS	4A	-
FS	5	-
FS	5A	-
FS	5B	-
FS	6	-
FS	6A	-
FS	6B	-

MOISTURE CONTENT SILT ONLY
 CLAY ONLY
 SAND TEST

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 56657 BORING # T DATUM GEODETIC CASING HX
 BORING DATE APRIL 22, 1958 REPORT DATE MAY 9, 1958 COMPILED BY W. J. A. CHECKED BY 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. - 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

SOIL PROFILE

COMPRESSIVE STRENGTH - TONS/SQ. FT. • UNCONFINED

0.5 1.0 1.5 2.0 2.5

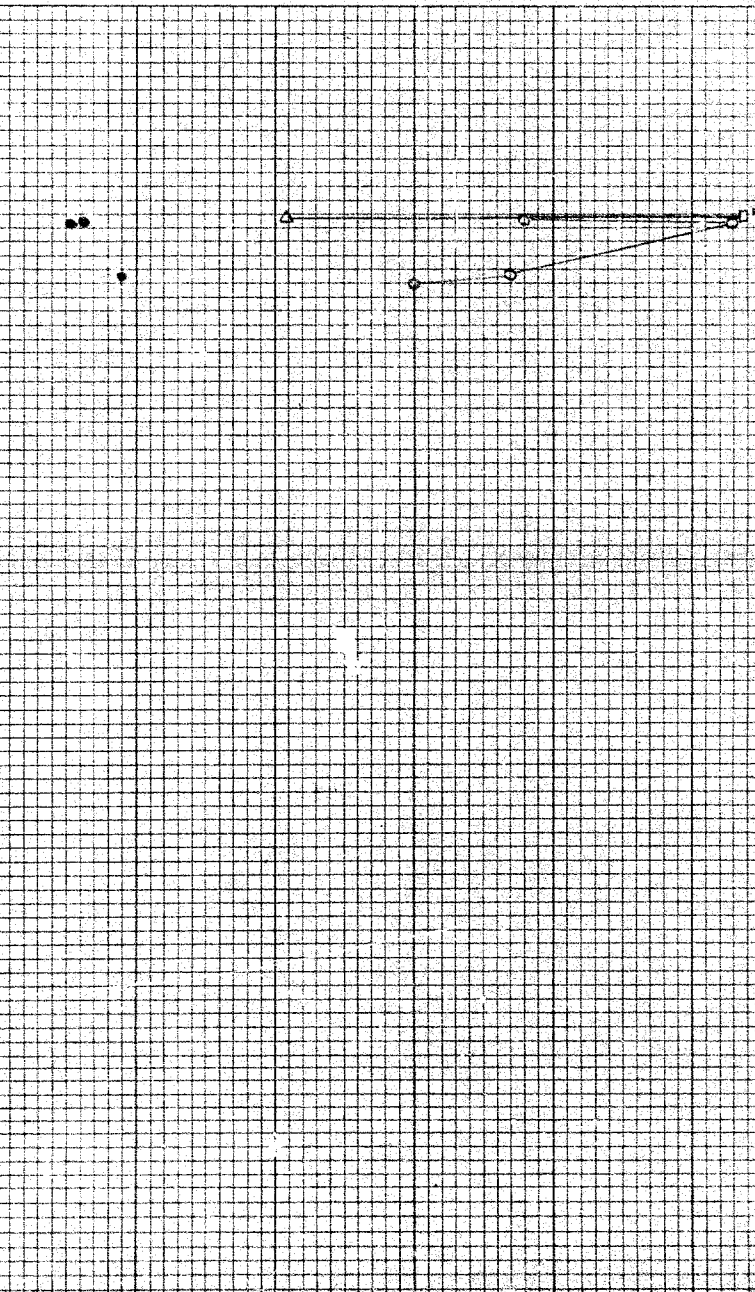
WATER CONTENT W% 10 20 30 40 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.

OTHER TESTS



CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2'DO	1	7
	3'SO	2	4
	3'SO	3	5
	2'DO	4	7
	3'SO	5	5
	SO	6	6
	SO	7	5
	SO	8	7
	SO	9	10
	SO	10	9
	SO	11	9
	SO	12	12
	SO	13	20

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX #1

CONTRACT 56057 BORING # 3 DATUM GEODETIC CASING FOIL
 BORING DATE APRIL 26, 1958 REPORT DATE MAY 9, 1958 COMPILED BY J.A. CHECKED BY
 SAMPLER HAMMER WT. LBS. DROP 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

SOIL PROFILE

COMPRESSIVE STRENGTH - TONS/SG. FT. • UNCONFINED

0.5 1.0 1.5 2.0 2.5

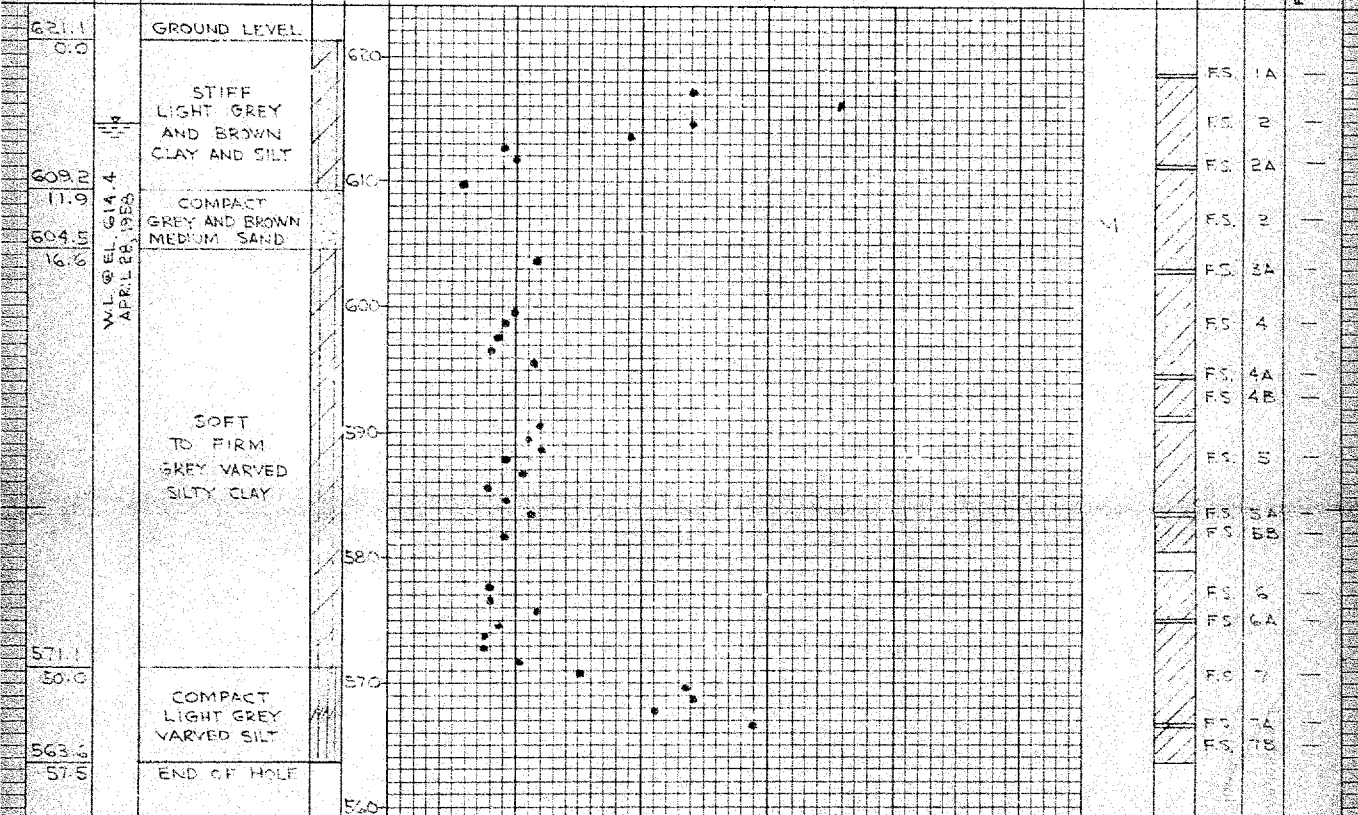
WATER CONTENT W% 0 NAT. DLW Δ Pw

10 20 30 40 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION
 RESISTANCE
 BLOWS/FT.



NOTE:

FOR ADDITIONAL LAB TESTING RESULTS
 SEE FIGURE 9, APPENDIX II

APPENDIX II

FIGURES - LABORATORY TESTING

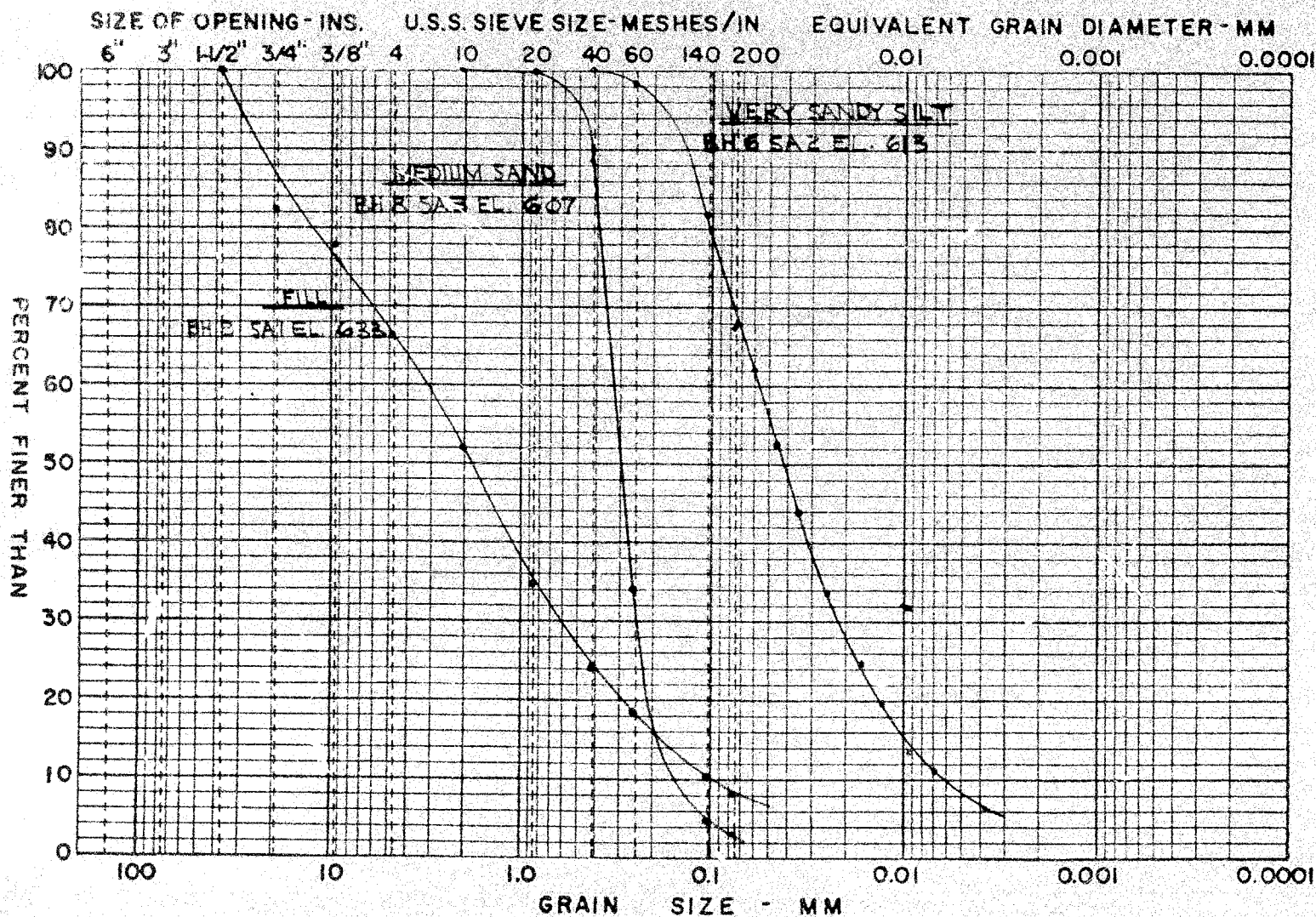
GRAIN SIZE DISTRIBUTION

APPENDIX II

FIGURE 1

PROJECT S6657

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



GEOCON

UNCONFINED COMPRESSIVE STRENGTH VS ELEVATION

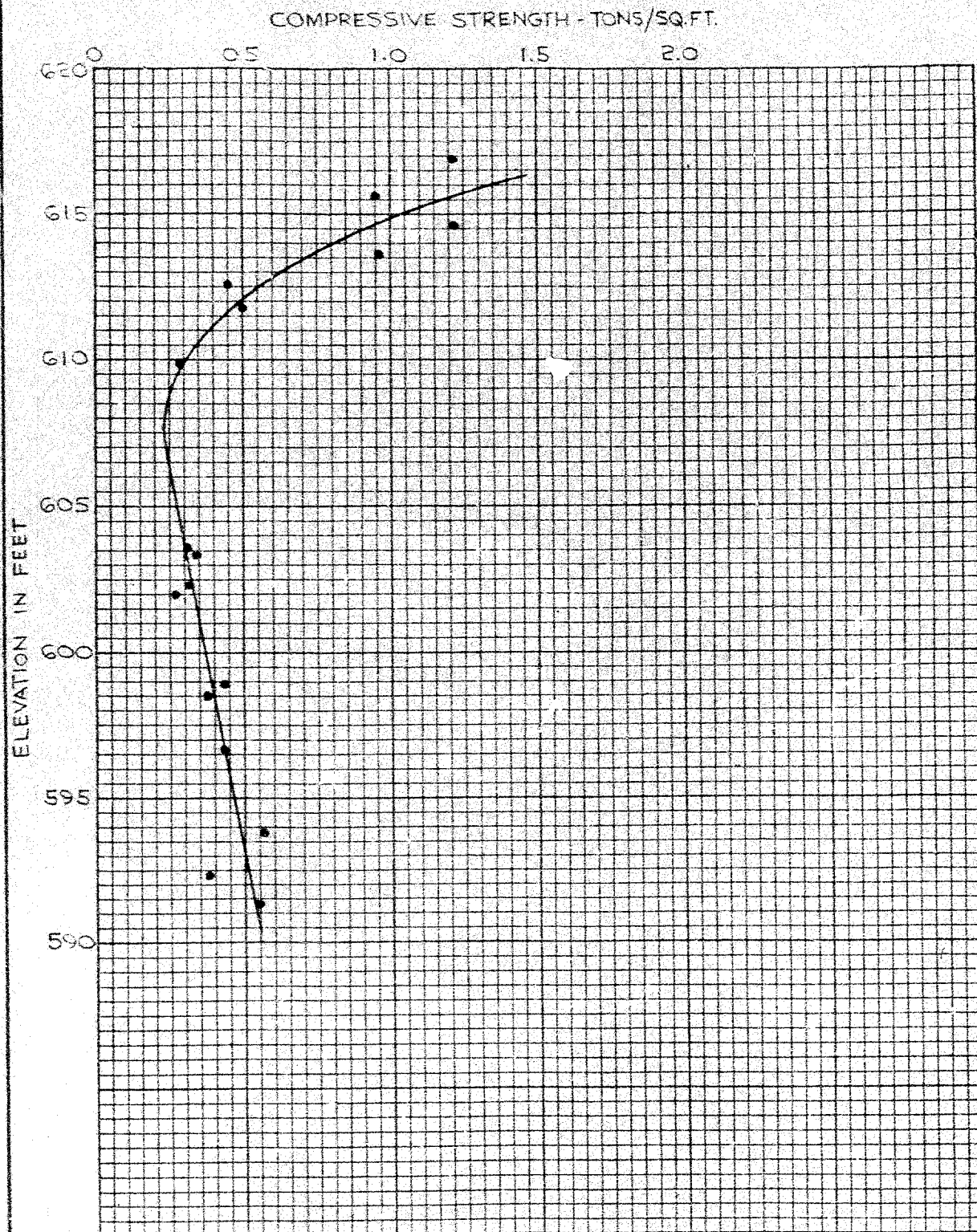
LIGHT GREY AND BROWN

CLAY AND SILT

APPENDIX II

FIGURE 2

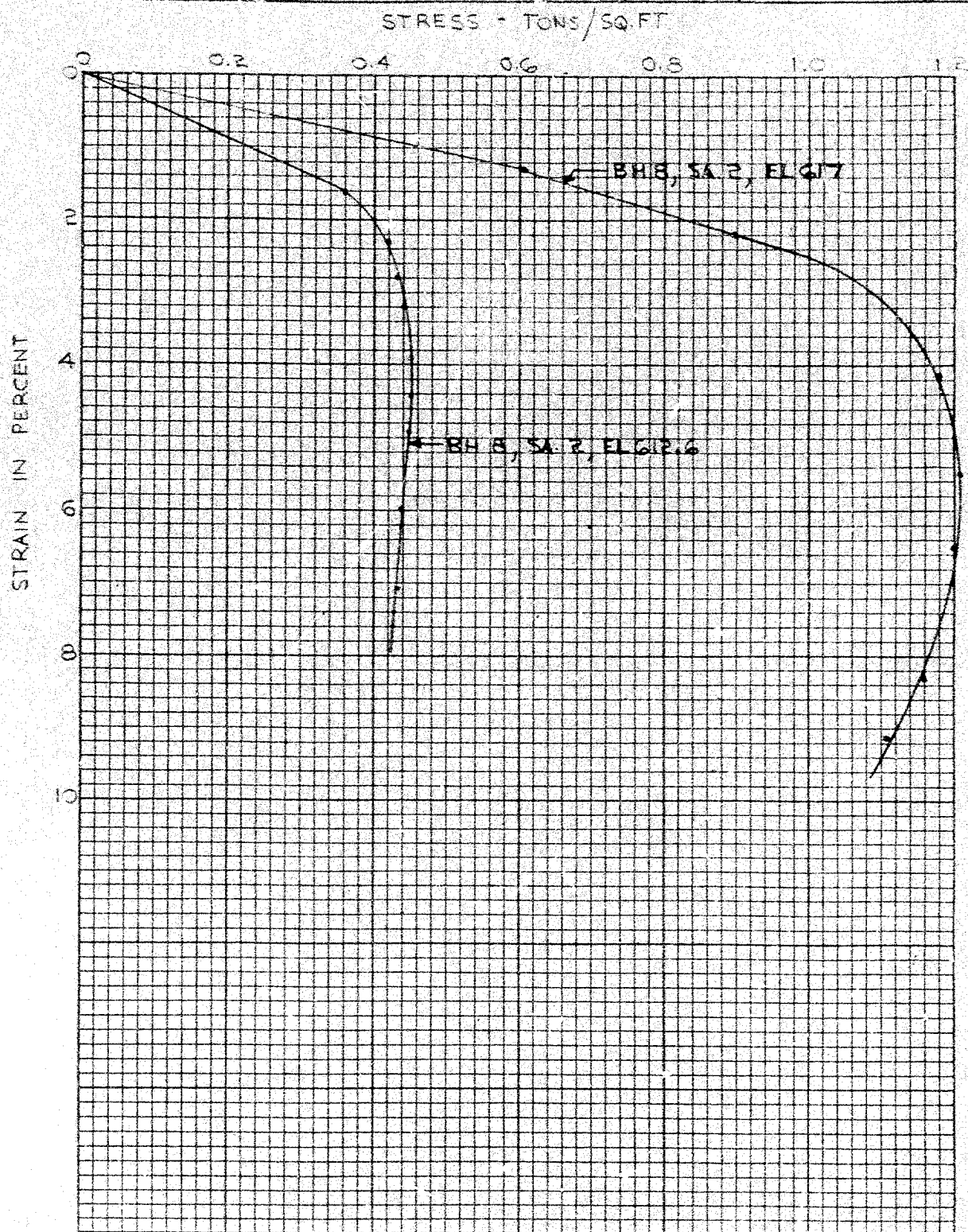
PROJECT SG657



GEOCON

UNCONFINED COMPRESSION TESTS
LIGHT GREY AND BROWN CLAY AND SILT
TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 3
PROJECT S6657



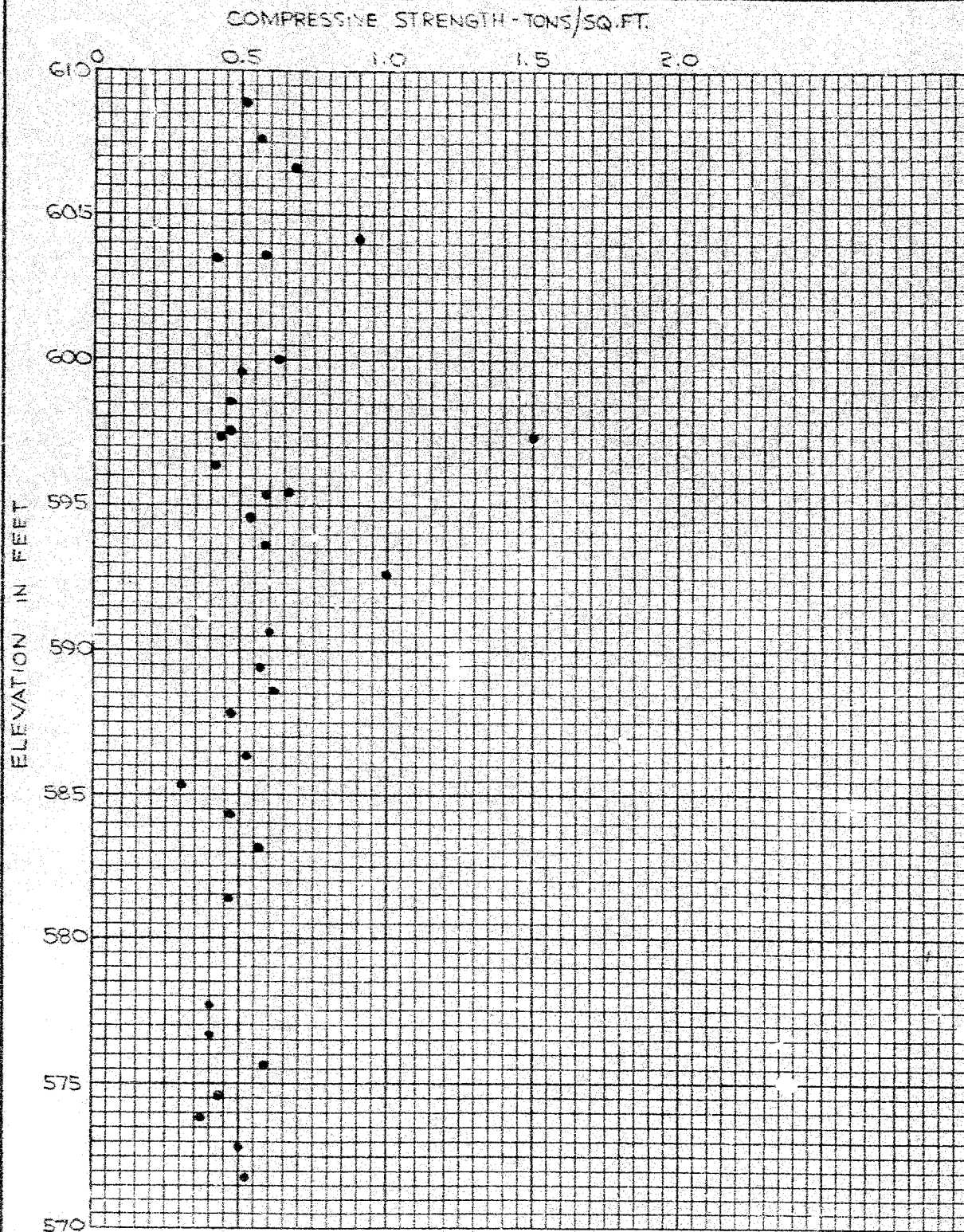
UNCONFINED COMPRESSIVE STRENGTH VS ELEVATION

GREY VARVED CLAY

APPENDIX II

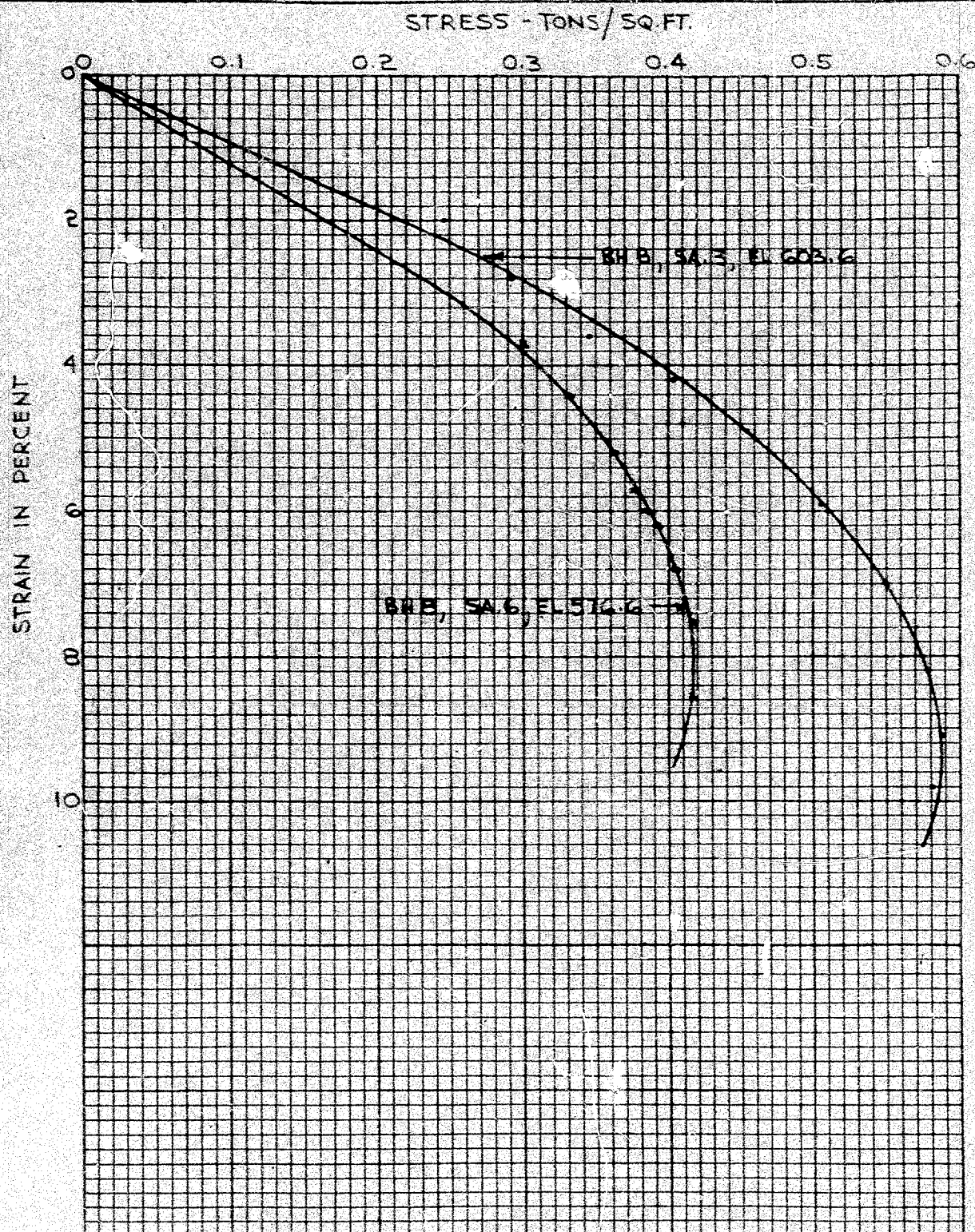
FIGURE 4

PROJECT S6657



UNCONFINED COMPRESSION TESTS
GREY VARVED CLAY
TYPICAL STRESS-STRAIN CURVES

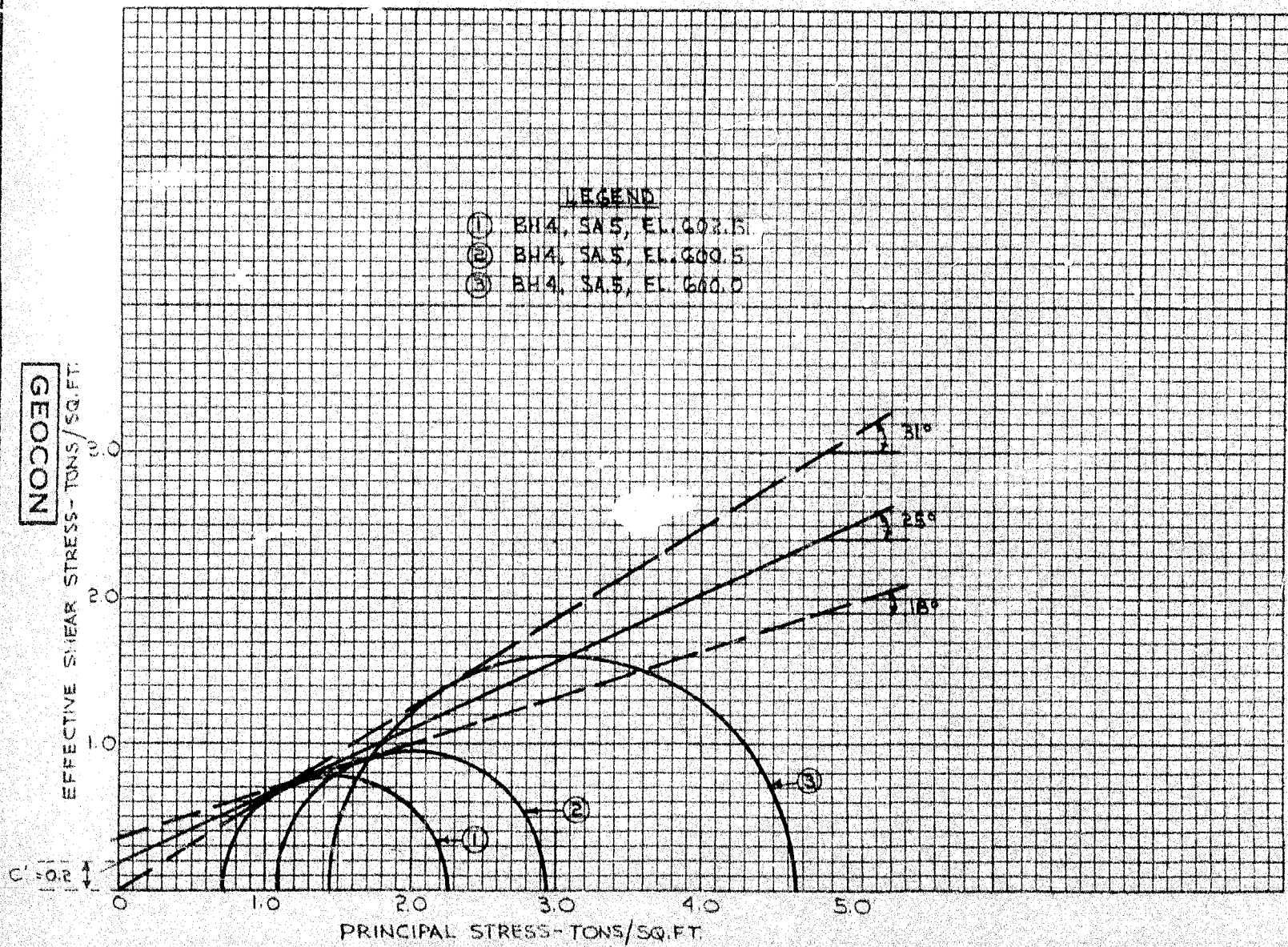
APPENDIX I
FIGURE 5
PROJECT S6657



GEOCON

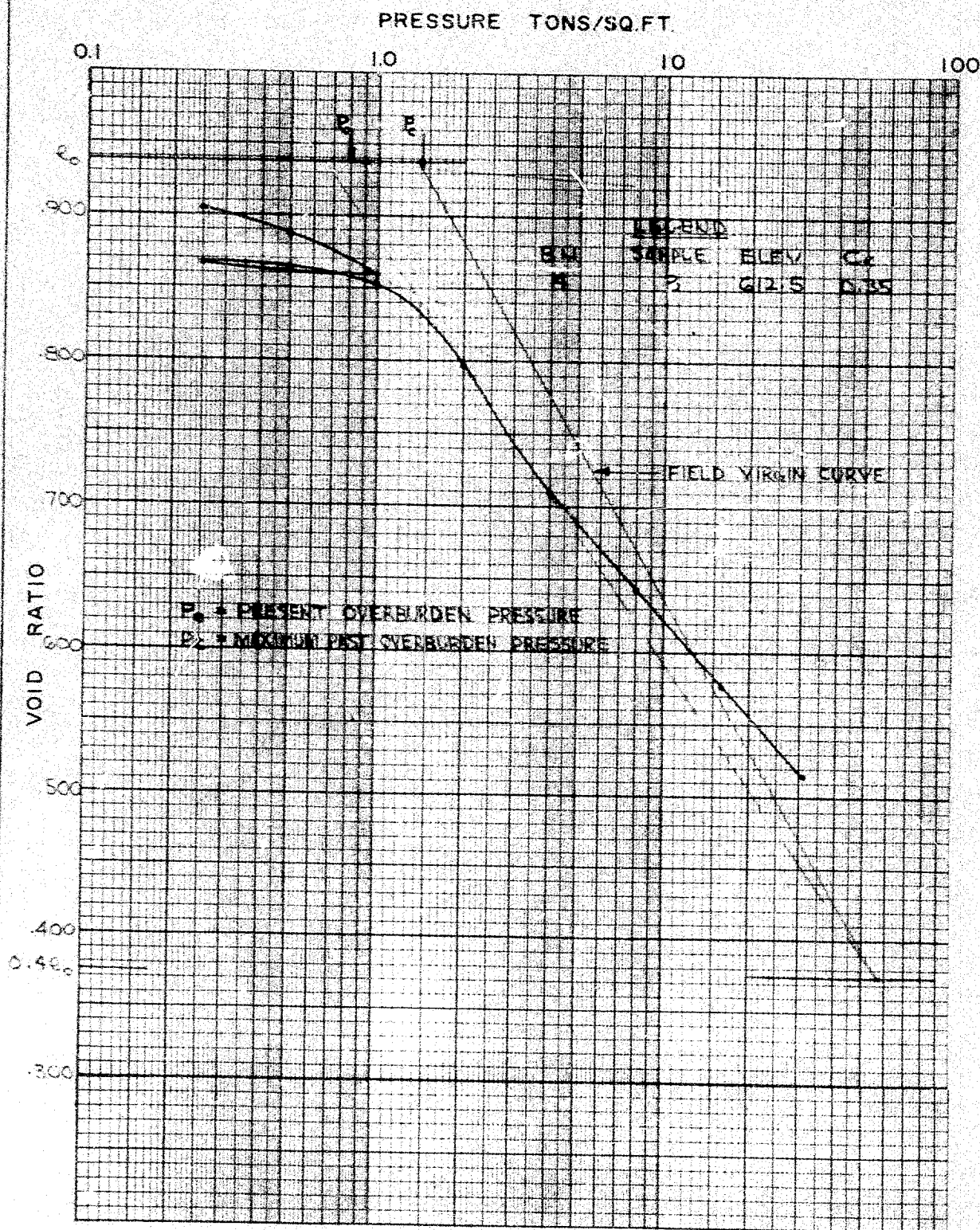
MOHR'S CIRCLES SLOW DRAINED TRIAXIAL TESTS GREY VARVED CLAY

APPENDIX II
FIGURE 6
PROJECT S6657



VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

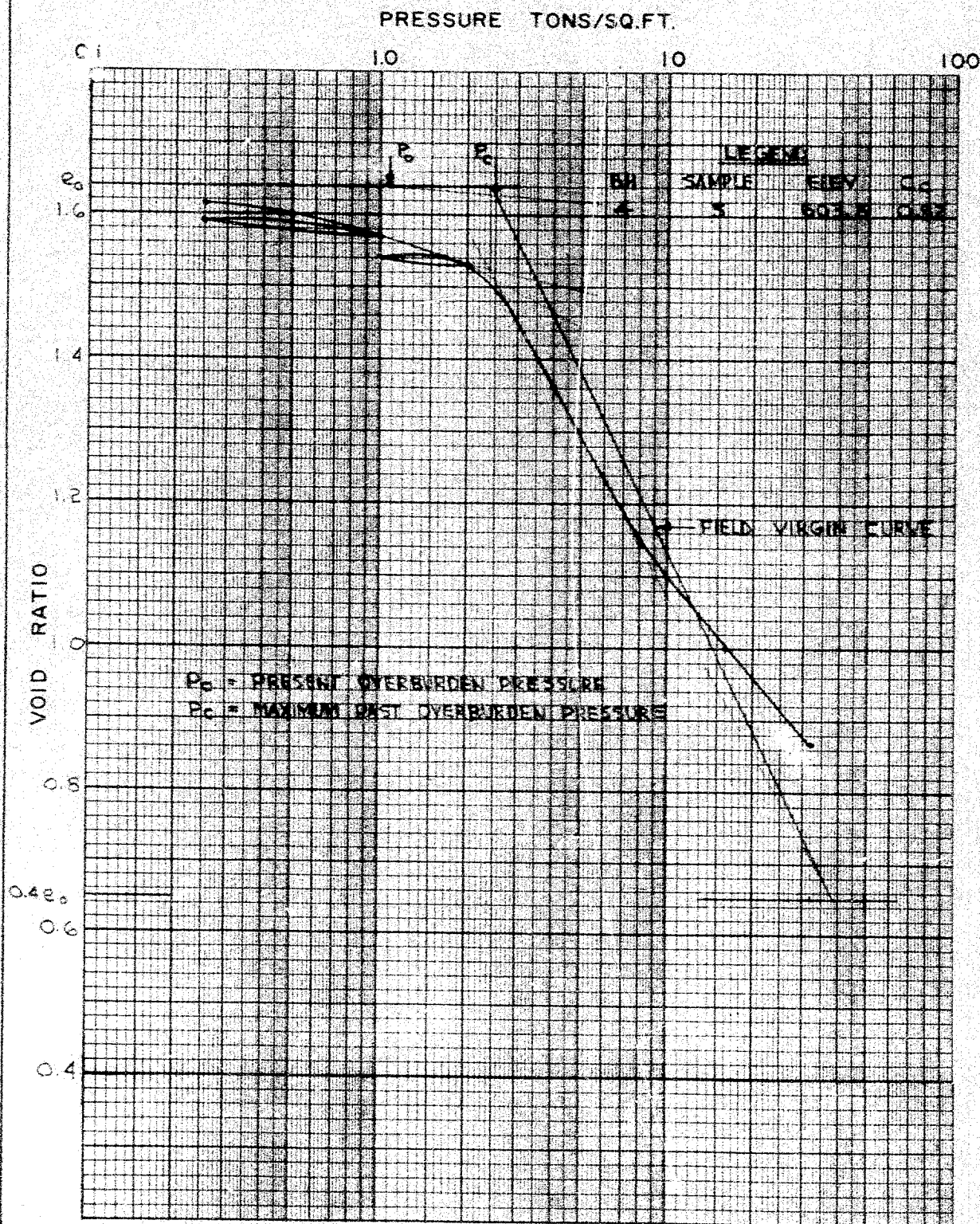
APPENDIX ☐
FIGURE 7
PROJECT S6657



GEOCON

VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

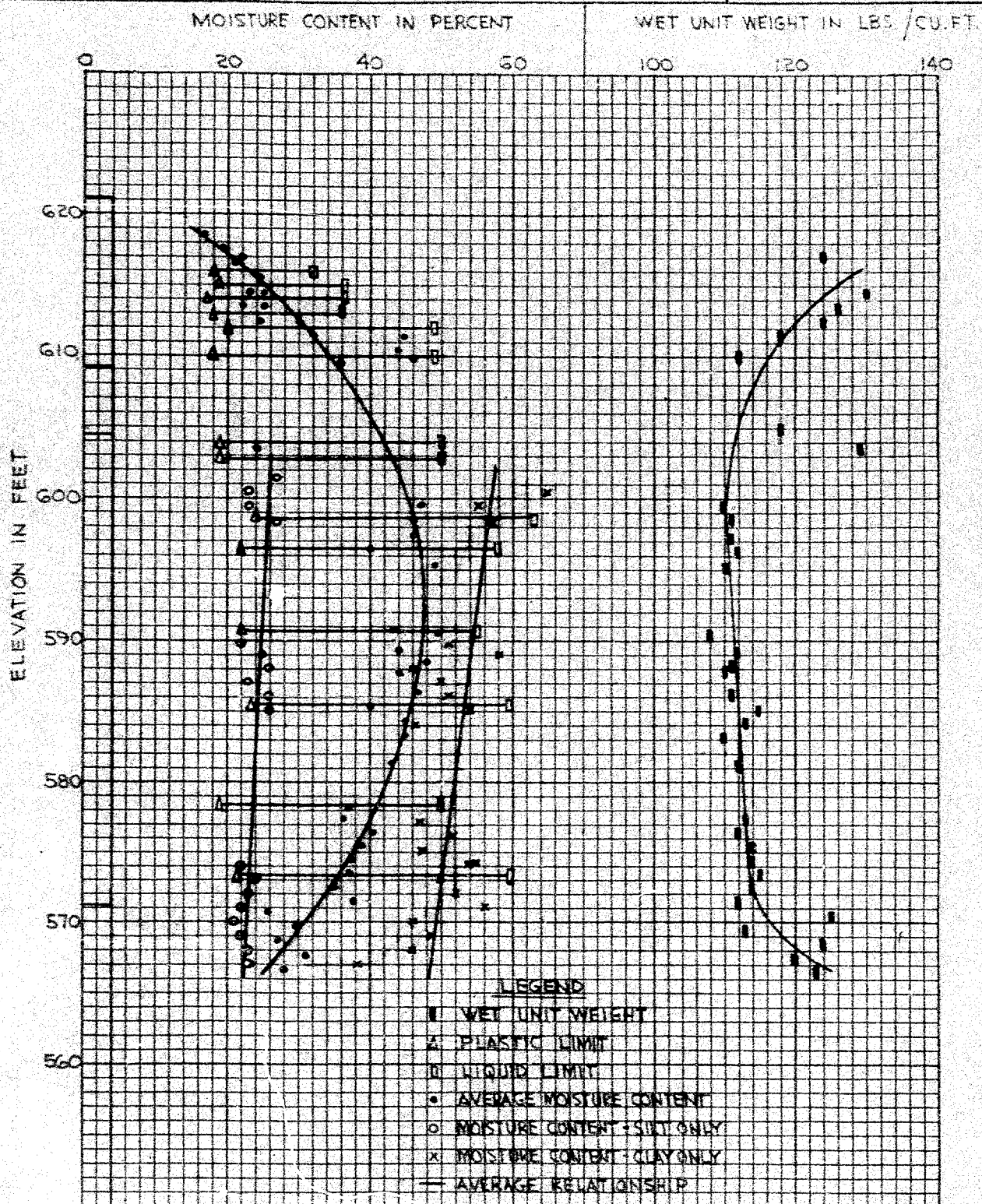
APPENDIX II
FIGURE 8
PROJECT S6657



GEOCON

MOISTURE CONTENTS, ATTERBURG LIMITS, UNIT WEIGHTS VS ELEVATION BOREHOLE 8

APPENDIX II
FIGURE 9
PROJECT S6657

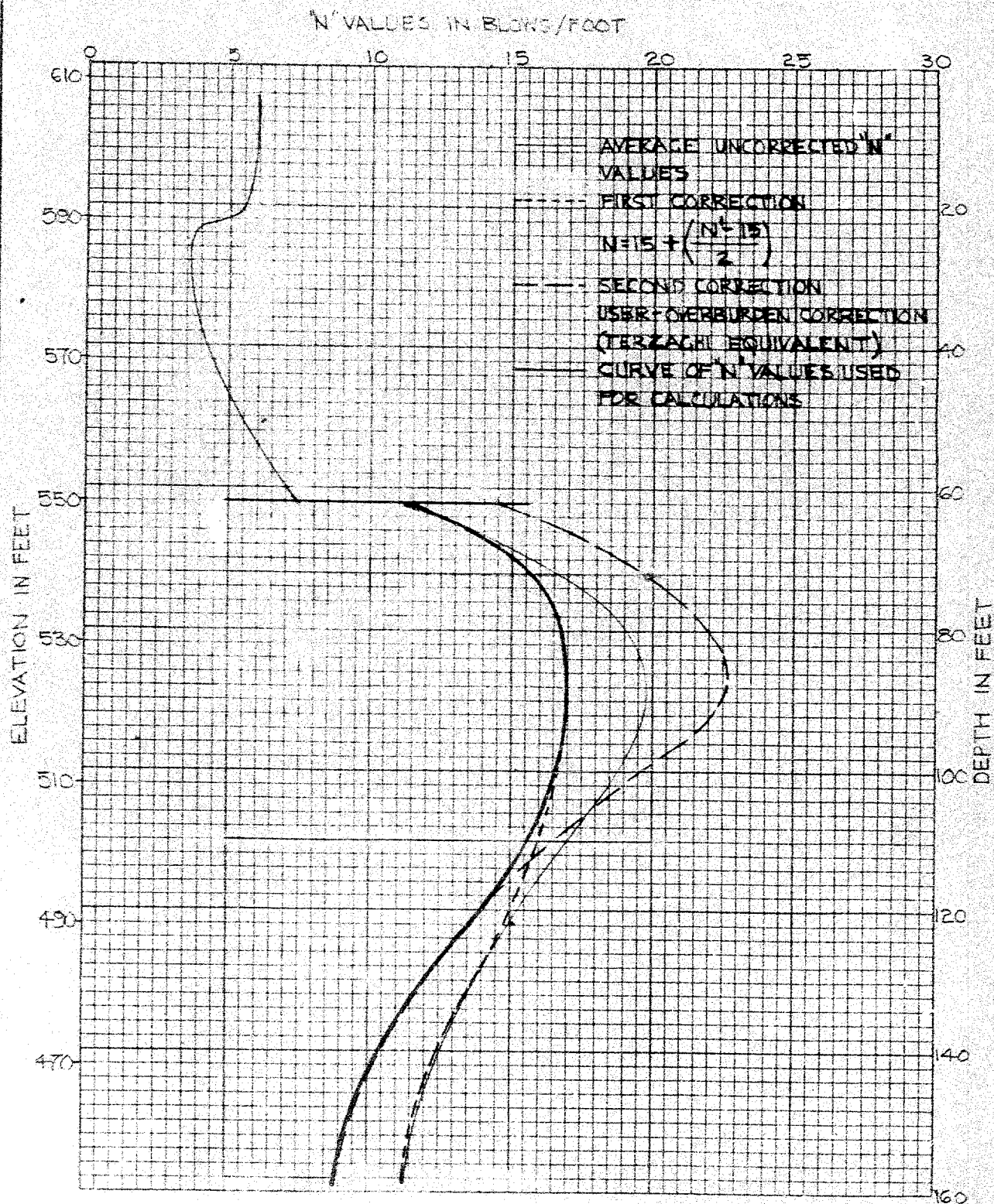


APPENDIX III

FIGORAS - ULTIMATE FILE CAPACITY

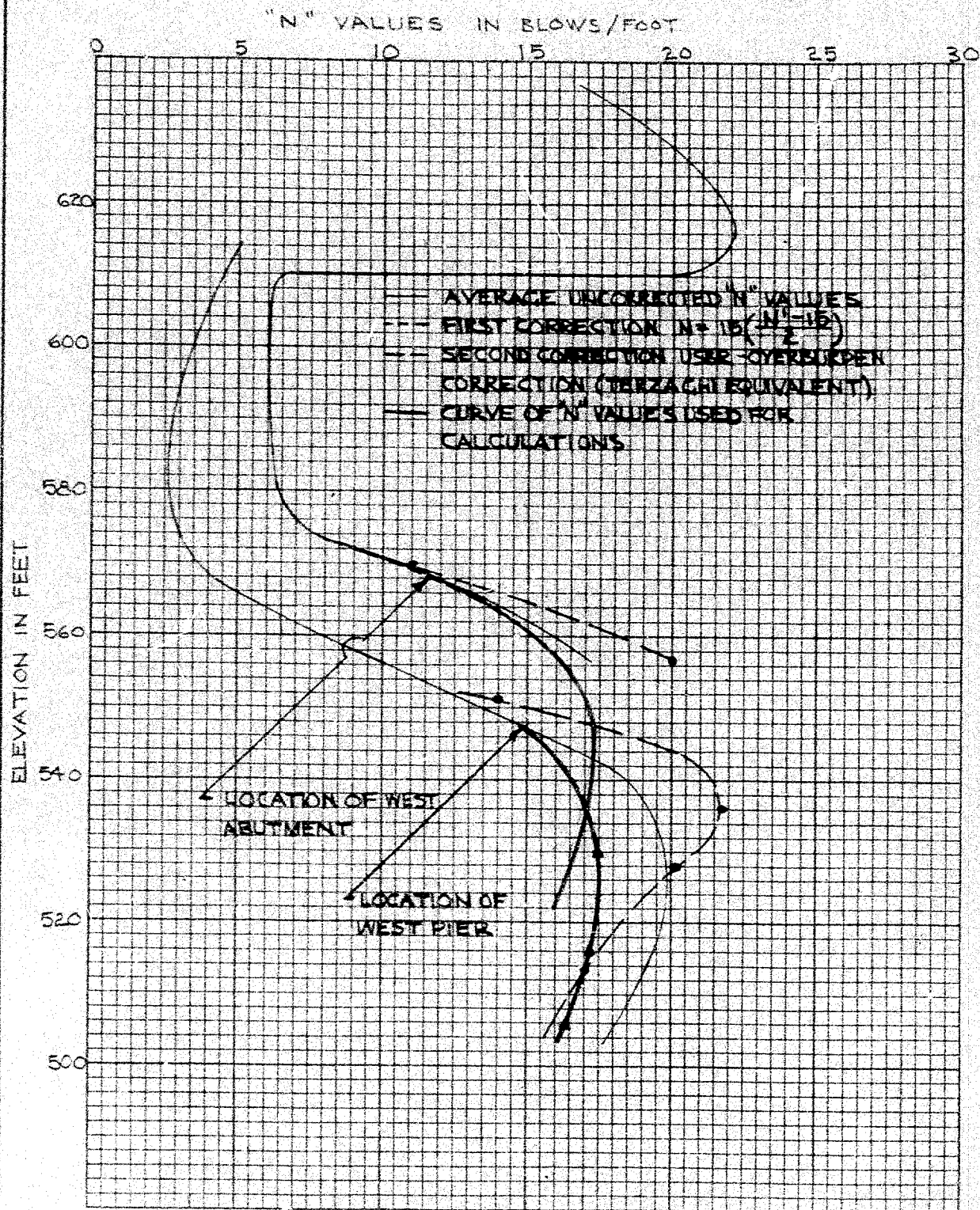
"N" VALUES VS DEPTH (EAST PIER)

APPENDIX III
FIGURE 1
PROJECT 56657



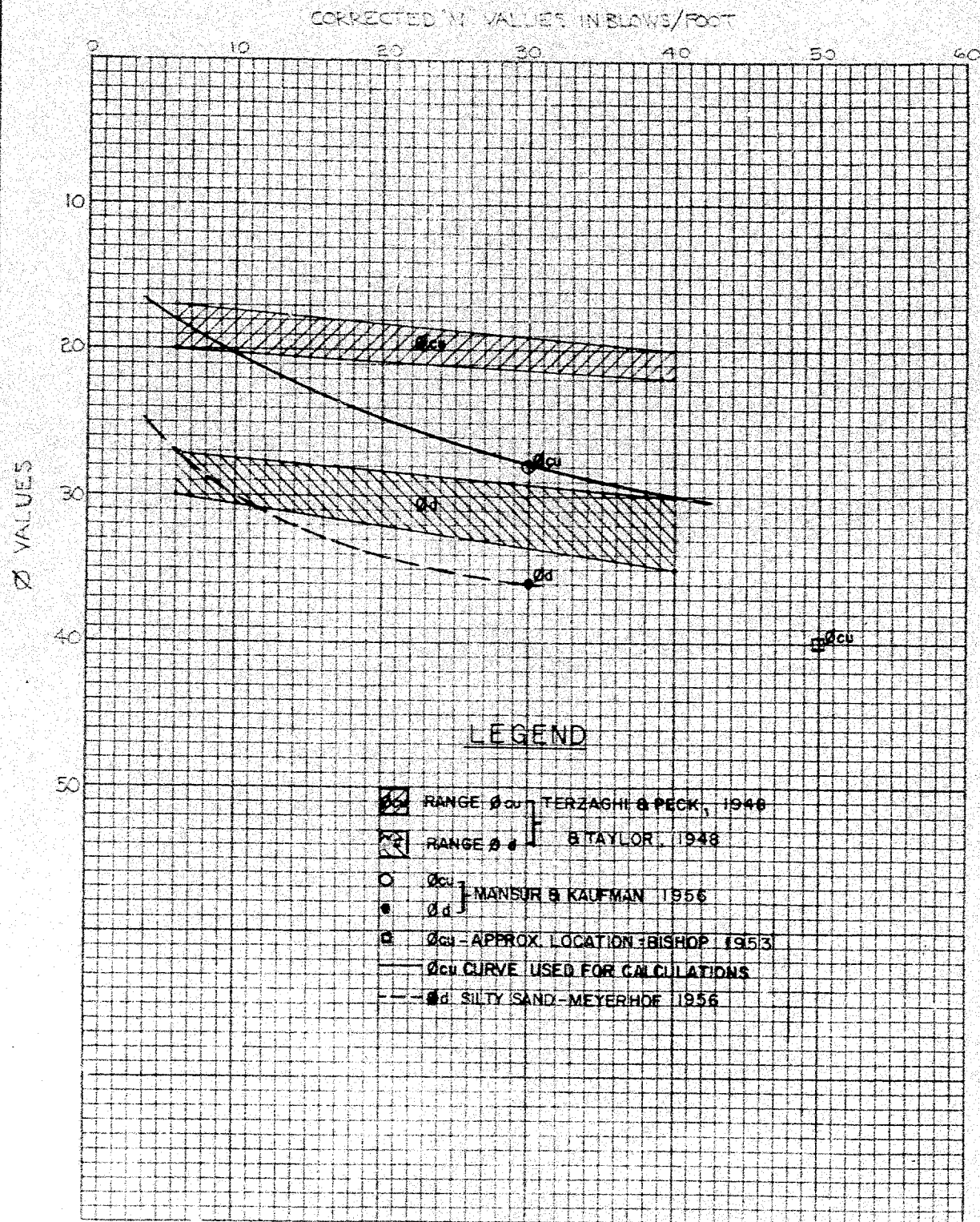
'N' VALUES VS DEPTH (WEST PIER AND WEST ABUTMENT)

APPENDIX III
FIGURE 2
PROJECT S 6657



CORRELATION OF ϕ VALUES WITH CORRECTED "N" VALUES FOR SILTY SOILS

APPENDIX III
FIGURE 3
PROJECT SG657

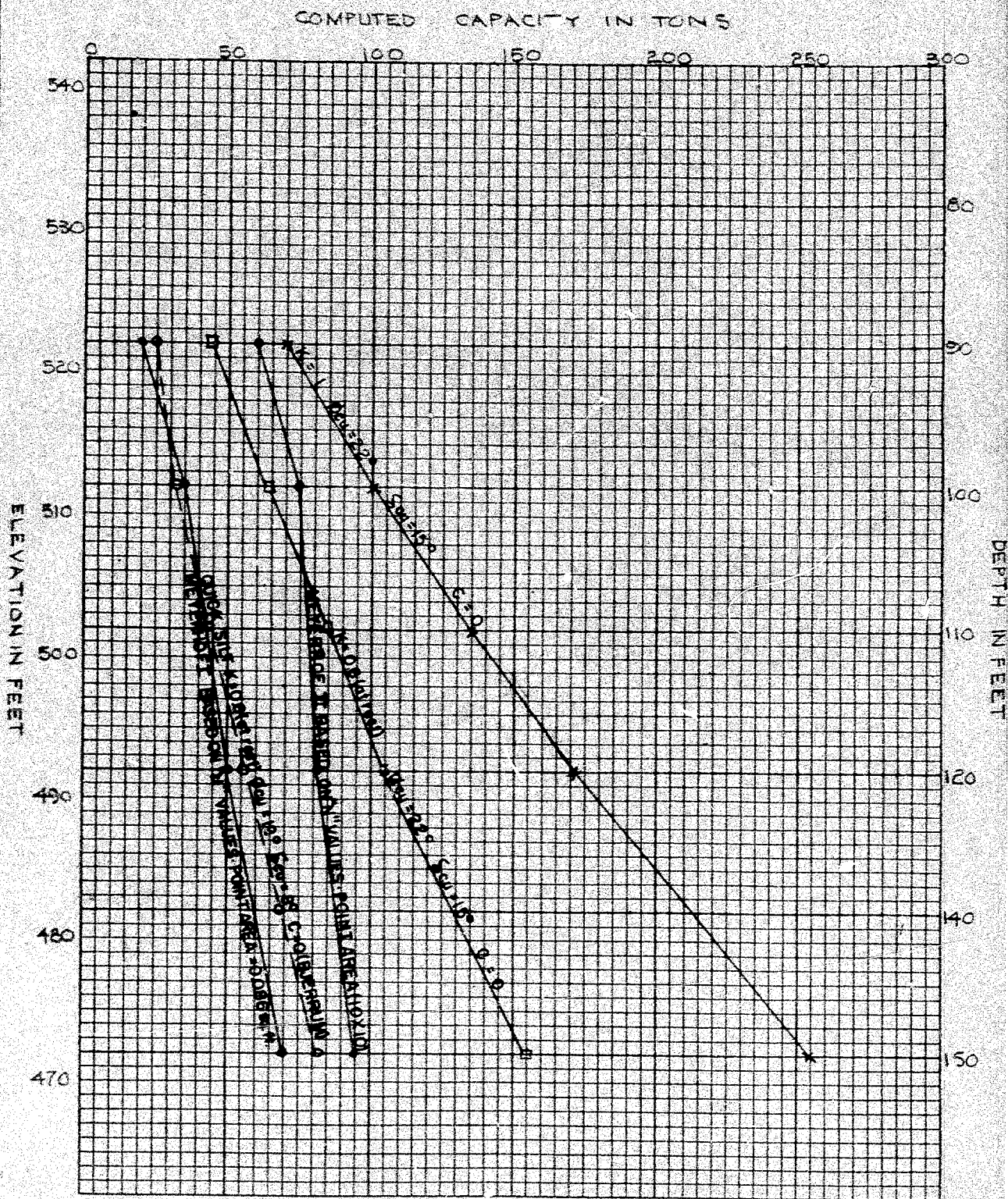


COMPUTED CAPACITY VS DEPTH

EFFECT OF UPPER 62 FEET NEGLECTED

SINGLE STEEL H PILE (10 BP 42)

APPENDIX III
FIGURE 4
PROJECT 56657



GEOCON

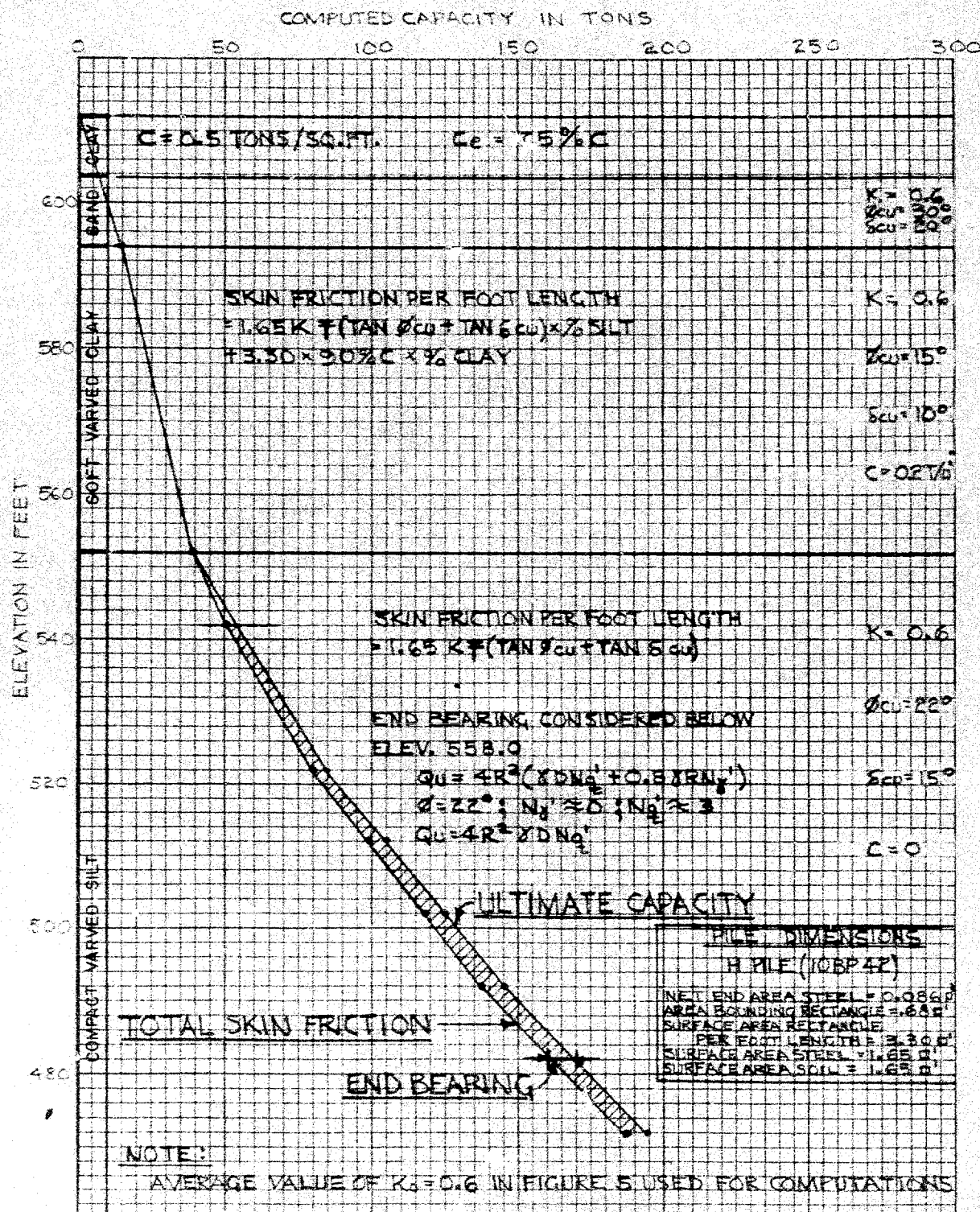
APPENDIX III
FIGURE 5
PROJECT S6657



COMPUTATION OF ULTIMATE CAPACITY

SINGLE STEEL H PILE (10 BP 42)

APPENDIX III
FIGURE 6
PROJECT S6657



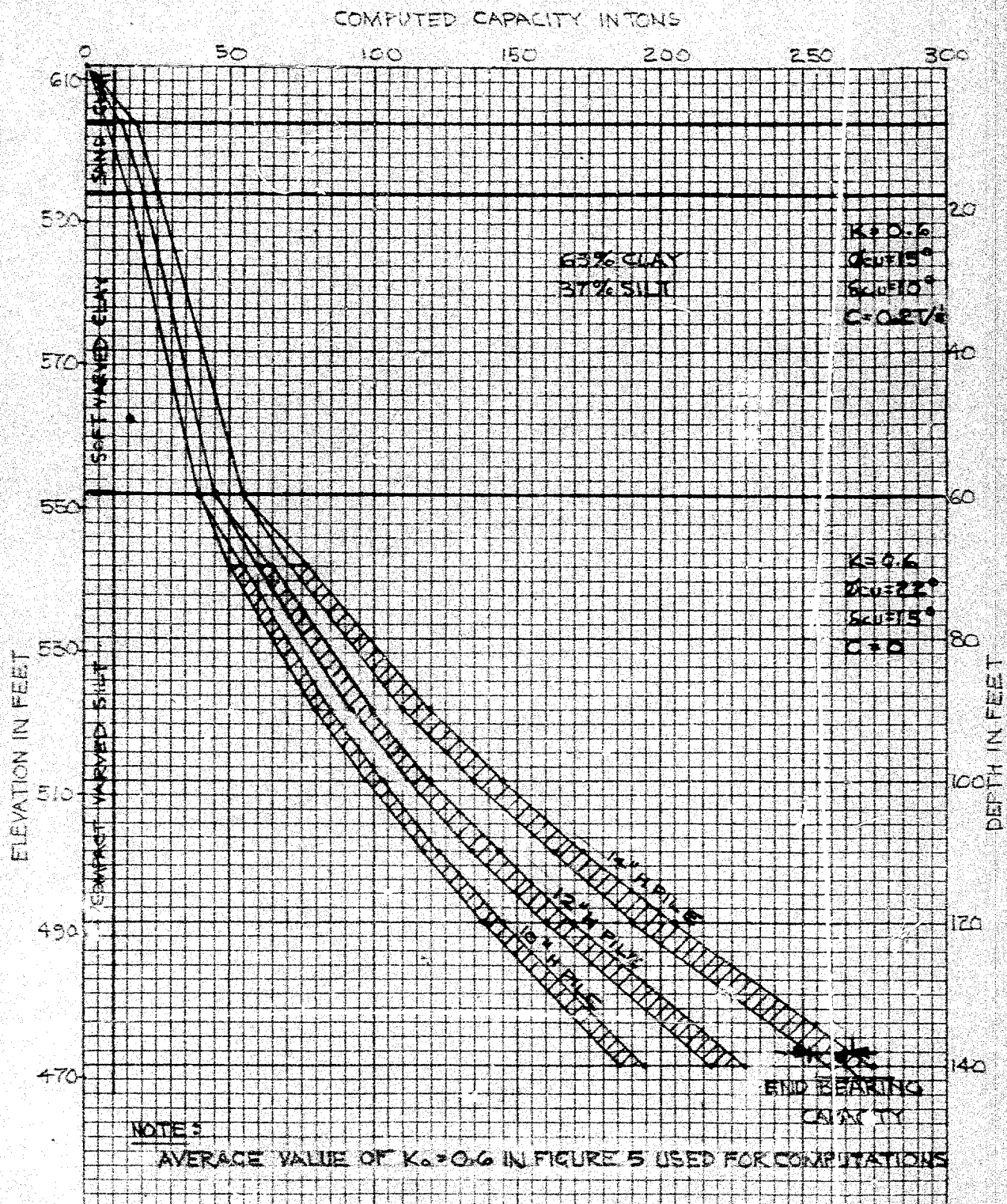
COMPUTATION OF ULTIMATE CAPACITY

10, 12, & 14 INCH STEEL H-PILE

APPENDIX III

FIGURE 7

PROJECT S6657



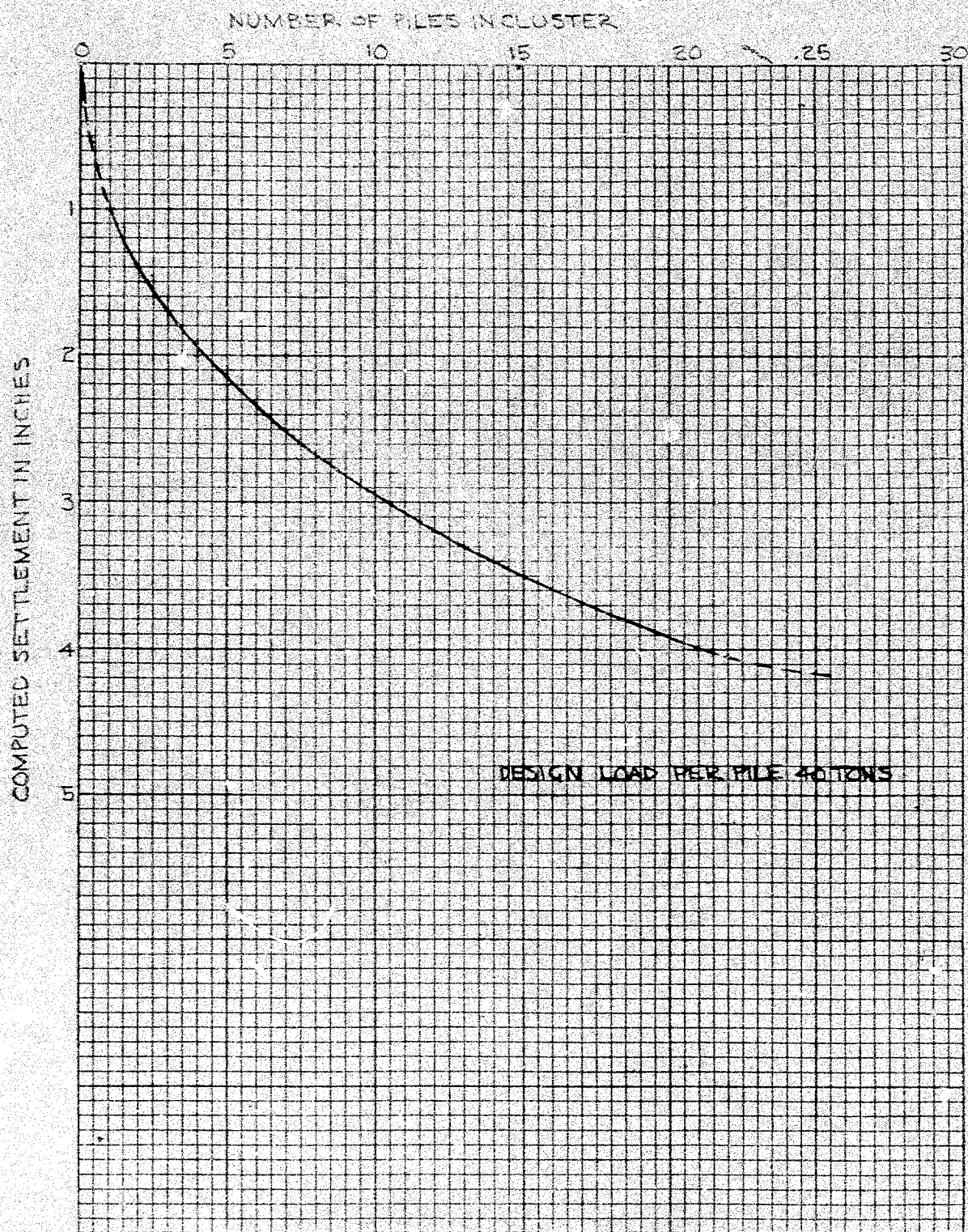
SETTLEMENT VS No. OF PILES IN CLUSTER

APPENDIX III

SPACING - FOUR FEET CENTRE TO CENTRE

FIGURE 8

PROJECT 56657



GRAIN SIZE DISTRIBUTION

APPENDIX III

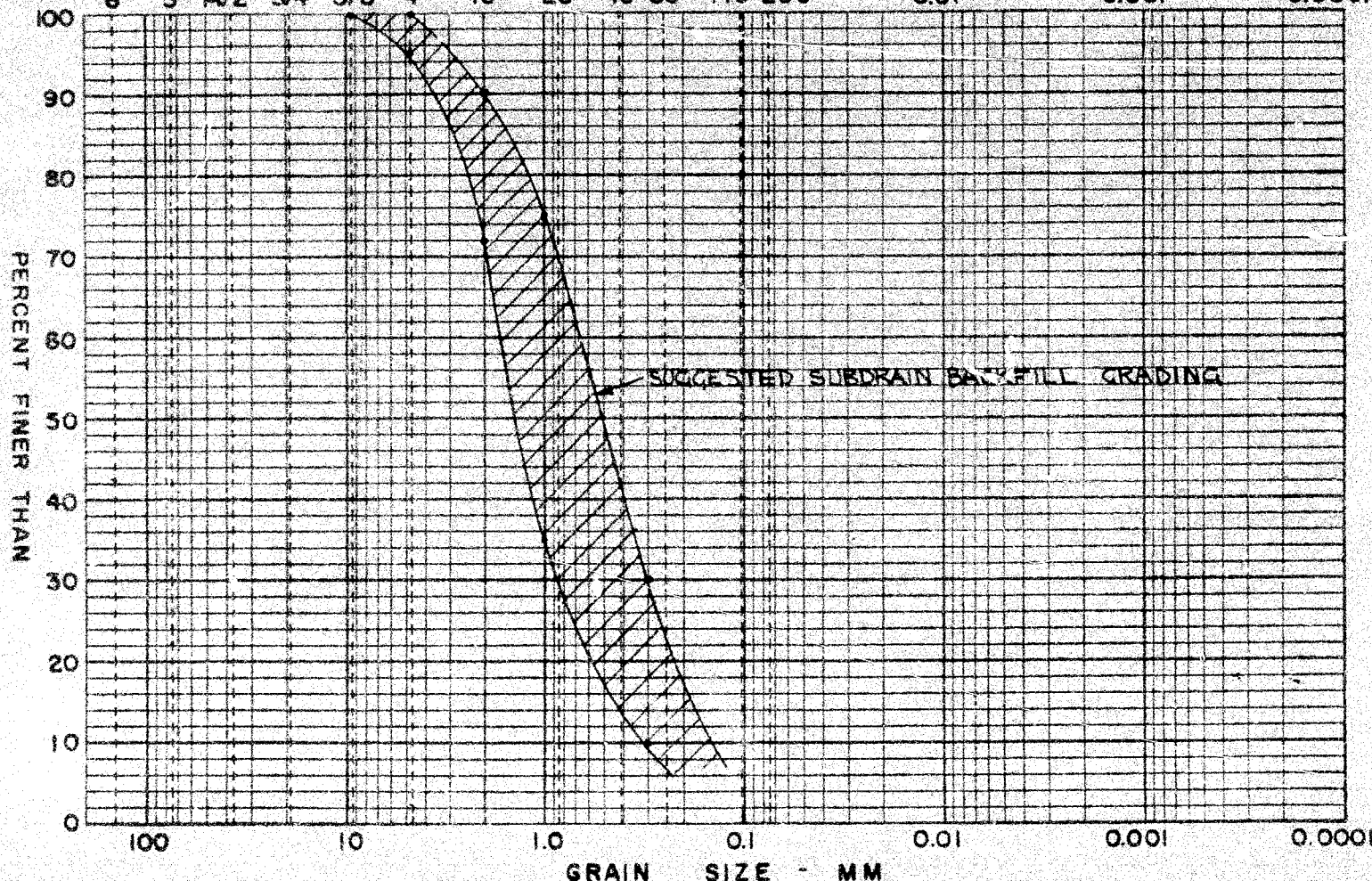
FIGURE 2

PROJECT S6657

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 60 140 200 0.01 0.001 0.0001



GEOCON

APPENDIX IV

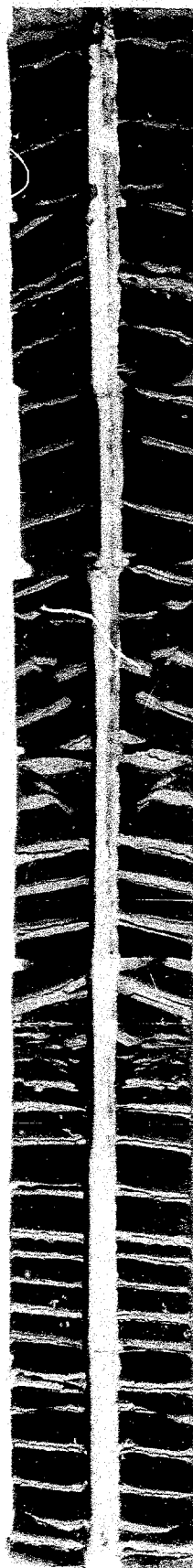
PHOTOGRAPHS



EL. 591.2

Very Sandy
Silt

EL. 587.2



EL. 587.2

Varved Clay

EL. 583.0

—EL.596.9



Flow Structure
in Varved Clay

—EL.596.3

APPENDIX V

PIEZOMETERS

INTRODUCTION

Three piezometers should be installed on the land side of each test pile group at the proposed pier locations. It is suggested that the piezometers be installed at a line parallel to the line formed by test and anchor piles (See Appendix VI). It is recommended that the first piezometer be installed as close to the proposed test pile location as practically possible. The remaining two piezometers should be about 7 to 10 feet distant from the test pile locations. The piezometers should be installed at the approximate locations and to the depths shown on Drawing S6657-4.

MATERIALS

The following materials are required for the installation of one complete piezometer:

- 20 feet (approximately) of 2 inch diameter casing
- 1 Casagrande type porous pot, 24 inches long with outside and inside diameter of $1\frac{1}{2}$ and 1 inch respectively
- 20 feet (approximately) of $\frac{1}{2}$ inch diameter plastic tubing
- 1 Neoprene rubber tube coupling, 4 inches long with outside and inside diameter of 1 and $3/8$ inch respectively for use between porous pot and plastic tubing
- 1 No. 5 rubber stopper to plug lower end of porous pot
- 50 pounds of packing sand graded between No. 20 and No. 35 sieve sizes, for backfilling piezometer hole
- 10 pounds of Bentonite for sealing of piezometer hole
- 1 tubular type steel tamping hammer filling inside casing and around plastic tubing, to centre plastic tubing and to tamp Bentonite seals
- 1 Bourdon type pressure gauge sensitive between 0 and 10 pounds per square inch, to record increase in pore pressure

A two inch diameter hole should be advanced to the elevation planned for the bottom of the permeable space. The bottom section of the casing should be at least 10 feet long and should not be provided with a drive shoe or coupling. The casing should be washed out completely, but washing ahead of the casing should not be allowed. The wash water is then entirely replaced by clean water of the same dielectric constant as the groundwater. The casing should then be pulled up two feet and simultaneously saturated packing sand should be poured in, while the casing is kept full of water, to fill the bottom 2 feet of the open hole. The packing sand should not be poured in faster than the casing is raised. The volume of sand required should be computed and closely controlled.

The porous pot, attached to the plastic tubing, is then immersed a few feet below the top of the casing and a vacuum is applied to the other end of the tubing until the system is completely filled with water. Care should be taken that no air bubbles are entrapped. The porous pot is then lowered to the bottom of the hole while maintaining enough vacuum to assure that a small amount of water will flow out of the tubing during lowering.

With the porous pot resting on the sand in the bottom of the hole, the casing is then pulled up another two feet and saturated packing sand is poured in to fill the space around the pot. The casing is then pulled an additional feet while the open hole is backfilled with more saturated packing sand. Next, enough saturated packing sand is poured in to fill the bottom 3 feet of the casing. This sand is then tamped by 10 blows of the tubular hammer with a drop of about 6 inches.

Following this, Bentonite, which has been prepared in a putty-like state and formed into balls of about $\frac{1}{2}$ inch in diameter, is used to seal the permeable space. The Bentonite balls are dropped through the water to the bottom of the casing. Five 3 inch thick layers, each one well tamped should be provided. To prevent the Bentonite from sticking to the hammer, a $\frac{3}{4}$ inch thick layer of $\frac{3}{8}$ inch diameter rounded pebbles should be dropped on each of

the Bentonite layers. The hammer is then lowered on to the pebbles and twenty blows are applied by raising the hammer about 6 inches and allowing it to drop freely. This process is repeated until the 5 layers of Bentonite are in place. A plug of packing sand about two feet in length is then added and tamped into place. As an additional precaution against leakage another seal of five 3 inch layers of Bentonite seals should be placed on top of the sand in the manner described above. The second Bentonite seal should then be covered with several feet of sand and the remainder of the hole may be left open.

MEASUREMENTS

Following this procedure the plastic tubing should be completely filled with water making sure that no air bubbles are entrapped. The Bourdon gauge should then be attached to the plastic tubing and mounted on a rigid stand. Care should be taken that while attaching the gauge, the water filled tubing is held at precisely the same elevation as where it will finally be mounted.

Alternatively, the plastic tubing need not be attached to a pressure gauge. It should then be taken about 15 feet above ground level and adequately supported by a wooden stand. Measurement of pore pressures are then taken by simply observing the water level in the transparent plastic tubing against a graduated scale.

APPENDIX VI

PILE LOADING TESTS

INTRODUCTION

One pile should be load tested in each of the pier and abutment pile groups. The method of loading by jacking against two tension piles as detailed on Proctor & Radford Drawing No. J465 is recommended. It is suggested that the test and anchor piles be part of the proposed pile groups.

METHOD OF MEASUREMENTS

Measurement of pile settlement may be carried out in two ways, each acting as a check on the other. Coarse measurement of settlement may be achieved by the use of a level measuring from a fixed bench mark remote from the test pile group to a bracket welded to the test-pile. Precise measurement of pile settlement may be obtained by using two piano wires, one on either side of the test-pile, tightened by turnbuckles to rigid stakes driven at least 20 feet away on each side of the test pile. The piano wires may rest against finely graduated steel scales attached to the test pile and readings can be taken using a light and magnifying glass. At each period of settlement, both level readings and steel scale readings from the piano wires should be taken.

TESTING PROCEDURE

The test load will be applied by means of a hydraulic jack anchored against a rigid member between the tension piles. It is suggested that the hydraulic jack be calibrated before loading is commenced. Since other piles are being used as anchors, some upward movement of these piles may take place. Consequently, constant attention must be given to the jack throughout the test to maintain the desired stage of loading.

Loading of the test pile should not be commenced until at least 14 days after the pile has been driven. A test load of about 80 tons should be applied in stages and no further increase of load should be added until the settlement of the previous load is observed to have practically ceased. With any given load kept constant, readings of settlement should be taken at intervals of 15 minutes for the first hour and then every hour. These readings should be

plotted on a curve showing settlement as a function of time and from the trend of the curve, it will be found possible to judge when movement has virtually ceased. As a general indication, a limiting rate of movement of 1/50 inch in 6 hours may be used as control. When equilibrium is reached, a further increment of load may be applied and the foregoing procedure repeated until the final test load is reached. A multiple graph showing load versus time, load versus settlement and settlement versus time can be plotted when equilibrium is reached.

It is recommended that the first applied load be about 25 tons, amounting to about 1/3 of the total test load. Subsequent increments of load may be of the order of 10 tons except for the last 4 increments, which should be of the order of 5 tons. Once the final test load has been applied and initial settlement recorded, it is recommended that this load be maintained on the pile for at least 48 hours. If settlement at the end of the 48 hours has virtually ceased to be measurable, then the test load may be removed. It is recommended that the load be removed in successive stages of about 25 tons while settlement is being recorded in the prescribed manner.

APPENDIX VII

CORROSION

In general, corrosion of a body of steel surrounded by a given soil will depend on resistivity of that soil, its moisture content and the degree of aeration.

To determine these properties in the varved clay stratum, laboratory testing was carried out over an 8 feet long typical sample from this stratum, taken at a depth from 29 to 37 feet below ground surface.

The average resistivity obtained was about 3900 ohms per centimeter, indicating that the clay is a fair conductor and, from a resistivity point of view, conducive to corrosion.

The average moisture content obtained was about 35 percent in the silt layers, 53 percent in the clay layers, and about 47 percent in the combination of silt and clay layers over the whole sample.

The average porosity obtained was about 38 percent in the silt layers, about 59 percent in the clay layers, and about 46 percent in the combination of silt and clay layers over the whole sample, indicating a rather poor aeration.

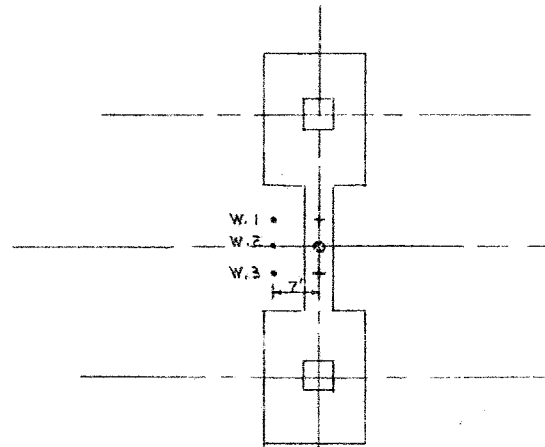
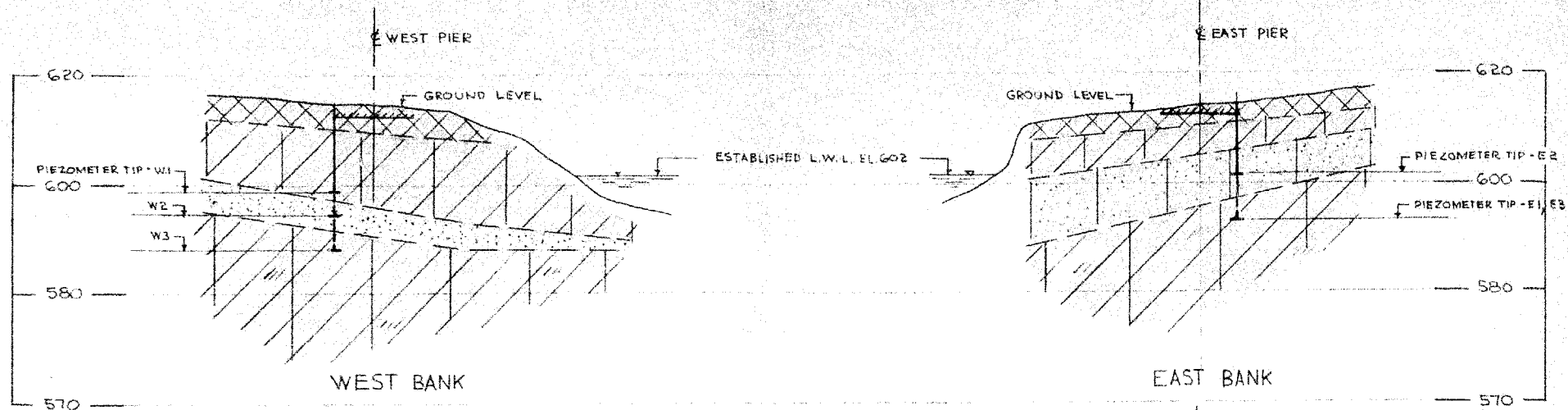
The effect of aeration is explained in the following extract from the U.S. National Bureau of Standard Circular C450, on "Underground Corrosion".

"In well aerated soils the rate of pitting of ferrous metals is initially very high because of the abundant oxygen supply at the cathodic areas. However, oxidation and consequent precipitation of the corrosion products in close contact with the anodic areas cause a marked reduction in the rate of corrosion with the result the ultimate depth of pitting is relatively slight. On the other hand, in poorly aerated soils, the rate of pitting although low because of deficiency of oxygen at the cathodic areas is relatively unchanged with time because the corrosion products in the reduced condition are precipitated at points remote from the anodic area. Thus, the depths of deepest pits after a long period are usually considerably greater in the poorly aerated soils than in the well aerated soils".

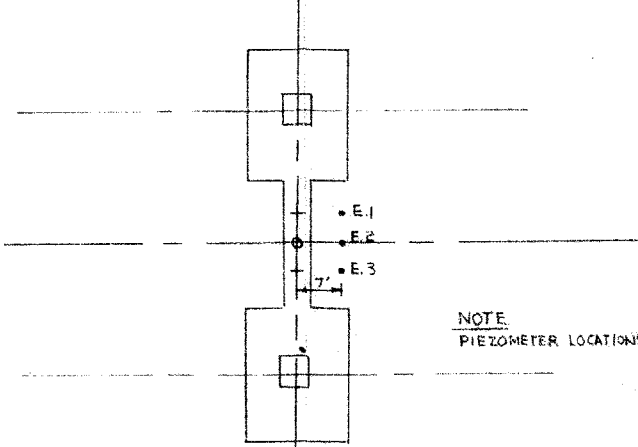
A small scale accelerated corrosion test was carried out on the varved clay samples by passing an electric current between two steel electrodes placed in the clay. The electrical circuit so induced, was allowed to operate for 18 hours. At the end of this period, the positive electrode was coated with a rust film.

The pH value of the varved clay as obtained from the average results of 2 determinations was about 8. This places the soil in a neutral zone from the viewpoint of corrosion.

In conclusion, it appears from the tests carried out and the nature of the soil that there will be a tendency towards corrosion of steel along the complete length of the pile.



FOUNDATION PLAN - WEST PIER



FOUNDATION PLAN - EAST PIER

NOTE
PIEZOMETER LOCATIONS APPROXIMATE

STRATIGRAPHY

- LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL
- SOFT TO STIFF LIGHT GREY AND BROWN CLAY AND SILT
- COMPACT GREY AND BROWN MEDIUM SAND
- LOOSE GREY VERY SANDY SILT
- SOFT TO FIRM GREY VARVED SILTY CLAY

LEGEND

- + ANCHOR PILE LOCATION
- ⊕ TEST LOAD PILE
- PIEZOMETER

REFERENCE	
DWG. NO.	DESCRIPTION

GEOCON LTD

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO

SUGGESTED PIEZOMETER LOCATIONS
EAST AND WEST RIVER BANKS

DATE SEPT. 21, 1958 SCALE 1" = 20'-0"

MADE J.A. CHKD. Y.M. APPD. Y.M. No. 56657-4

BA 527-D

PROCTOR & REDFERN

Founded by E. A. James, 1912

CIVIL AND CONSULTING ENGINEERS

11 JORDAN STREET
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EDWARD M. PROCTOR
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R. G. TREDGETT
G. U. PROCTOR

TELEPHONES

Head Office - EM 3-5374
Scarboro Office AM 7-4641
Etobicoke Office BE 3-6251

December 31, 1957.

Mr. A. M. Toye,
Bridge Engineer,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario.



Re: Pic River Bridge
W.P. 150-56

Our E.O. 569

Dear Sir:

We enclose herewith, three copies of letters dated December 23, 1957, and December 27, 1957, from Stone and Webster Engineering Corporation Limited, referring to the Pic River Bridge.

Since our meeting in Ottawa with the Department of Public Works we have re-examined the design of this bridge and discussed the conditions with Mr. R. D. Chellis of Stone and Webster. We feel that fixing the bridge at the abutments is the better design at this location and we recommend this method.

We discussed the rocker bearing expansion joint of the suspended span with Mr. Chellis and you will note he has suggested in his letter of December 27, 1957, that a pintle be incorporated in the design of the rocker and that a larger drain hole be used. We are also re-examining the design of this bearing assembly with the idea of replacing the welded joints with rivetted connections and will report to you further on this in the next week. Since this assembly is part of the structural steel contract a revision to this bearing will not affect the piling contract.

In our discussions with you we have pointed out the importance of very close field supervision of this structure during construction and particularly during driving of

Mr. A. M. Toye

Page 2.

December 31, 1957.

the piles. The banks of the river should be watched at all times during piling and if any signs of slippage or excessive water flow in a soil strata is observed, drainage wells should be installed. Copies of the Stone and Webster reports should be forwarded to the field engineer for his information and guidance.

We regret the delay in preparing this letter, however, the suggestions made by the Federal Department of Public Works were based on sound judgement and before submitting our report we required time to check the basic design and to confer with Mr. Chellis. We hope that this delay has not prevented the calling of piling tenders for this winter's construction.

Yours very truly,

PROCTOR & REDFERN

A handwritten signature in dark ink, appearing to read 'A. E. Read', written over the typed name.

A. E. Read

AER/ct.

Encls.

STONE & WEBSTER ENGINEERING CORPORATION

COPY

Proctor & Redfern,
11 Jordan Street,
Toronto 1, Ontario.

December 27, 1957



Proctor & Redfern,
11 Jordan Street,
Toronto 1, Ontario.

J.O.No.CJ-220(9850)

Attention Mr. E. M. Proctor

Dear Sirs:

INVESTIGATION OF PROPOSED FOUNDATION DESIGN FOR
PIC RIVER BRIDGE

Thank you for your letter of December 20, 1957 revising information on the suspension span bearings and amplifying surface drainage plans.

We agree with you that rocker bearings are less likely than roller nests to give trouble from icing, that a larger drain hole would be better and that pintles to guard against lateral movement of the rockers would be desirable.

We note that there will be a system of drainage from approach ditches, perforated drains and 24 in. diameter drain pipes extending to the river to lead the water in the top few feet well away from the piers. The hydrostatic pressure to which we referred in our recent letter was that originating in a confined stratum at a depth of about 85 ft, and which would act independently of the surface water. If its head exceeds El. 615 and the pressure is unrelieved, we fear that the factor of safety against plane sliding of the bank might be unduly reduced due to buoyancy, regardless of the excellence of the surface drainages. Methods of detecting such possible pressure and of relieving it were discussed in our recent letter. In our letter of April 1, 1957, we suggested the use of horizontally inclined or vertical wells if the banks showed signs of weeping, and that such work be done before the jarring from pile driving. The greatest danger of slides in these banks is from moisture, due to the thin, alternating layers of clay and silt which facilitate the travel of water in the silty layers to lubricate sliding surfaces in the clay. The banks should be stable if adequately drained of water from the various sources.

STONE & WEBSTER ENGINEERING CORPORATION

P&R

2.

December 27, 1957

We note that batters of unmarked amount are shown for the interior piles in the piers in Section "E-B", but that the plan view does not indicate the directions of batters for any interior piles, including corner and end piles.

We trust that these comments will be of use to you. Please let us know if we can be of further assistance.

Yours very truly,

R. D. Chellis,
Structural Engineer.

Copy to Proctor & Redfern-4-AM

COPY

C
O
P
Y

STONE & WEBSTER ENGINEERING CORPORATION

49 Federal Street, Boston 7, Mass.

December 23rd, 1957.

J.O. No.CJ-220(9850)

Mr. E. M. Proctor,
Proctor & Redfern,
11 Jordan Street,
Toronto 1, Ontario, Canada.

Dear Mr. Proctor:

INVESTIGATION OF PROPOSED FOUNDATION DESIGN
FOR PIC RIVER BRIDGE

In your letter of December 11, 1957, you asked for our opinion on the matter of fixing the Pic River bridge spans at the abutments, as shown on your drawing, compared with fixing them at the piers as suggested by the Federal Department of Public Works.

We prefer anchorages located at the abutments, based upon present information, for the following reasons. The river banks are high and relatively unstable, as indicated by the report of not uncommon slippages along the course. The soil below the top strata generally consists of varved clay and silt. We understand from you that the promontories on each bank were placed in 1930, with the intent of building a bridge at this site. We assume that the added material may be represented by the fine to medium ^{loose} sands reported as the inclined top strata.

Conditions are so variable that assignment of numerical soil shearing values and establishing the exact locations of the weakest slip circles or planes of sliding are impracticable. In the case of slip circle failure tendencies, removal of bank weights would increase the factors of safety by decreasing the overturning moments and shear stresses, whereas on plane surfaces, removal of this load would decrease the factors of safety by decreasing the available shearing friction resistance. However, straightening the contours to show possible ground surfaces as they might have been before the promontory fills were added indicates that the proposed surface on the west bank would still be higher, and therefore having higher factors of safety against sliding friction, than the bank prior to 1930, and that the flatter east bank conditions would probably be no worse than before that date.

Free running water was reported in boring holes nearest the river after they reached depths of about 85 ft. The top hydrostatic head was not reported and the "running water" reported is not indicative of the pressure head because low permeability may have restricted the volume.

If hydrostatic pressures had been reported from higher underground elevations, we might have expected the possibilities of slips on flat planes to be increased.

The type of tendency to failure and locations of possible failure surfaces bear upon the choice of anchorage points. The slight tendency of the bridge to creep with temperature changes because of being set on an 0.5 per cent incline also affects the selection of anchorage locations. Removal of soil as indicated should improve the factors of safety appreciably, over those existing since placing the 1930 fills, against slip circle failures; and while it may decrease the factors of safety against plane sliding friction existing since 1930, these factors should be as good or better than those prior to that date. While the cuts shown at the abutments introduce slip circle stresses where none existed before, these will be of relatively low magnitude. Anchorages at the abutments would have batter piles and passive earth pressure values for resistance to accumulative creep forces, situated well back of most likely slip circles. Creep acting against fixed connections at piers would tend to add horizontal forces toward the river and increase any tendency to plane sliding.

Further light might be cast upon the matter by observing the old bank slips for evidence as to characters of slides and by putting down a pipe to tap the hydrostatic pressure stratum, which pipe can be extended high enough above ground to observe the height to which the water will rise. If the head does not extend above El. 615, we doubt if a plane slip failure would be likely, but if it is higher consideration should be given to the installation of drainage wells, locating a pair about 50 ft. apart and 15 ft. behind each pier. Such wells might be formed by driving a 12 in. diam casing, placing a 4 to 6 in. diam slotted wood pipe, filling the casing with sand and pulling the casing. Drains should be located below frost line and extended into the river.

If we can be of any further assistance, please call on us. We should be pleased to hear of conditions encountered, and of the progress of the design and construction.

With best wishes for the Christmas Season and the New Year,

Yours very truly,

"R. D. Chellis",
Structural Engineer.

BA 527 C

Refile

STONE & WEBSTER ENGINEERING CORPORATION



49 FEDERAL STREET, BOSTON 7, MASSACHUSETTS

NEW YORK
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SEATTLE

April 11, 1957



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Proctor, Redfern & Laughlin,
11 Jordan Street,
Toronto 1, Ontario.

J.O.No.CJ=220(9850)

Attention Mr. E. M. Proctor

Dear Sirs:

INVESTIGATION OF PROPOSED FOUNDATION DESIGN FOR
PIC RIVER BRIDGE

We are pleased to hear from your letter of April 4, 1957 that the copies of our report letter on the Pic River Bridge foundations reached you safely. In reply to your question regarding pile carrying capacity, you are correct in assuming that we believed that this should be determined by load tests. We indicated tentative pile lengths on our study drawing based upon our impression of the shearing values of the various strata, a working load of 30 tons per pile, a factor of safety of 2.0, and no group reduction factor because of spreads of piles on batters sufficient to avoid any considerable overlaps in bulbs of pressure in the lowest 40 ft assumed as comprising the location of load carrying strata.

We note your question as to whether a 10 per cent increase in load, resulting from the lengthening of span which we recommended, should cause an increase in the number of piles or whether the battering of the piles would obviate the need of using the former group reduction coefficient of 0.85, by acting as a compensating factor. Although we used certain pile friction values in our investigations, these are hypothetical and we would not expect them to be within an accuracy range of 10 per cent. Theoretically, since the total load will be increased, either the load per pile must be increased and the piles lengthened by 10 per cent of 40 ft, which would be 4 ft, or the number of piles must be increased by 10 per cent. Since we considered the piles as battered when computing their values, we did not use a group reduction coefficient, and therefore can not use such a value as a compensating factor to apply against the increased total load. The only method of determining the

true pile capacities is by load tests, and because of the approximate character of preliminary designs, we suggest awaiting such results before changing the number of piles. It is possible that no increase either in number of piles or lengths will be necessary. If some increase in total supporting capacity is found desirable, it would be more economical to increase the lengths of the piles than to add piles. The fiber stress in the piles would not be excessive if the pile loads are increased 10 per cent.

Our study indicated, in the cross section, suggested batters in the longitudinal direction. Load carrying capacities would be improved and settlements decreased if piles in north and south rows were to be battered north and south, and possibly if corner piles were battered diagonally. This will depend upon the type of driving rig used.

We are glad to have been of service to you in this matter.

Yours very truly,

R. D. Chellis
R. D. Chellis, *c*
Structural Engineer.

ES-56-1
P-56-1
Reg. No. A.

PROCTOR, REDFERN & LAUGHLIN

Founded by E. A. James, 1912

CIVIL AND CONSULTING ENGINEERS

11 JORDAN STREET
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EDWARD M. PROCTOR
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BA 5271
[Signature]
TELEPHONES
Head Office - EM 3-5374
Scarboro Office AM 7-4641
Erbicoke Office BE 3-6251

April 15th, 1957.

Mr. A.M. Toye,
Bridge Engineer,
Department of Highways,
Parliament Buildings,
Queen's Park,
Toronto, Ontario.

Attention: Mr. Bruce Davis

Re: Bridge over Big Pic River
Highway No. 17.
Our Project No. E.O. 569.

Dear Sir: -

It has been brought to our attention by E.M. Peto & Associates that a discrepancy appears to exist in the chainage of the piers and abutments for the above structure.

We refer you to the soil site plan prepared by E.M. Peto & Associates in their original soil report dated May 4th, 1956, copies of which you have and to our preliminary drawing No. SK1, two copies of which we enclose herewith. We draw your attention to the fact that the centre to centre span of the piers on drawing No. SK-1 is 225'-0". Our latest layout proposal is to increase this span to 270'-0". Drawing No. SK-1 is used only at this time to illustrate the discrepancy in chainage.

The location of the centre line of piers on drawing No. SK-1 is about 2 to 3 feet beyond the high water limits. The chainage of the east pier, based on chainages given by the Department of Highways. Contour and Profile Plan is 3202 + 90 and for the west pier, chainage 3200 + 65.

On the E.M. Peto & Associates soil site plan, the chainage shown for the centre line of the east pier is 3202 + 72 and for the west pier, chainage 3200 + 47. These chainages were given to E.M. Peto & Associates by the local Department of Highways Engineer.

April 15th, 1957.

The discrepancy of approximately 18 feet in chainage should be clarified at this time in order that no possible error be made in locating the bridge and in order that material quantities for excavation be accurately calculated. We request that you investigate this matter and advise us of the correct chainage at the site with reference to the stream and the contours.

We also request that you provide us with the following additional information which we require to determine the slope cuts and drainage for the approach embankments.

- (a) The location of the Bailey Bridge with reference to the location of the proposed bridge.
- (b) Contours of the existing ground to a 1 inch = 20 feet scale extending on the north side of the centre line of the proposed bridge to a distance of 125 feet and on the south side to the south limit of the Bailey Bridge approach. The contour and profile plan which you previously sent to us extends to only 75 feet on both sides of the bridge centre line.

Yours very truly,

PROCTOR, REDFERN & LAUGHLIN

CSU/sm
Encl.

C.S. Ufnal.



STONE & WEBSTER ENGINEERING CORPORATION



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April 1, 1957



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CONSULTING
ENGINEERING

Proctor, Redfern & Laughlin,
11 Jordan Street,
Toronto 1, Ontario.

J.O.No.CJ-220(9850)

Attention Mr. E. M. Proctor

Dear Sirs:

INVESTIGATION OF PROPOSED FOUNDATION DESIGN FOR PIC RIVER BRIDGE

In accordance with your oral authorization of March 8, 1957, confirmed by your letter of March 18, we have investigated the foundation design for the Pic River Bridge, based upon the following drawings which you handed to us:

- Drawing No. E-3060-1 - Sh. 1 of 3 - Proposed Bridge Over the Pic River, Department of Highways, dated March 4, 1955
- Project No. E.O.569 - Preliminary Bridge Layout - Proposed Bridge Over the Pic River - Proctor, Redfern & Laughlin
- Project No. E.O.569 - Proposed Details for Pic River Bridge - Proctor, Redfern & Laughlin

You also furnished us with single copies of the first report of E. M. Peto & Associates Ltd. dated May 4, 1956, and their supplementary report, undated. These reports contained drawings showing surface contours, locations of borings and boring logs. Numbers of standard penetration test spoon blows were shown, and locations and properties of disturbed samples and of undisturbed Shelby tube samples were given. We note, however, that all of the Shelby tube samples were taken in the material upon which reliance can not be placed for permanent load carrying capacity of piles, and that no Shelby tube samples were taken in the load supporting strata, assumed as those having 13 or more spoon blows per foot.

No soil samples have been available for our inspection, and we have based our analyses upon the soil data, descriptions, and test results contained in these two reports.

Three prints are enclosed of our Study SK-32457 - Suggested Foundation Arrangement - Pic River Bridge, which we have prepared to indicate suggested changes in pier and abutment locations and bank cuts.

GENERAL LOCATION OF BRIDGE AND SURFACE CONTOURS

The two promontories on opposite sides of the river lend themselves to a minimum span for the bridge. Rather than excavate the ground on each bank to form right of way cuts, we suggest that all surfaces of these promontories be removed to a little below center line elevations, following cut gradients and pitching slightly away from the rights of way. Exceptions must be made where the ground does not fall off and embankments would be created. Such embankments should be constructed with flat slopes starting up well away from the cut. Paved drainage ditches should be provided to catch road surface drainage and to prevent runoff from seeping into the soil. Lead-off ditches are suggested to remove from the vicinity of the bridge as much drainage from more distant points as possible. Cut spoil should be dumped far enough away from the site so it can not add to the dangers of river bank slides or right of way embankment slides.

EFFECT OF PROPOSED CUTS AND PIER LOCATIONS

We have made comparative stability analyses of the west bank by several methods. Soil shear values obtained by both E. M. Peto and us indicate that they are in a range so high that little, if any, factor of safety may be present. We believe that the best approach to this problem is to increase the factor of safety as much as practicable before starting pile-driving operations. While it would be very difficult, and probably impossible, to determine theoretically the weakest possible surfaces of slip failure in these varved or heterogeneous materials, for comparative purposes we studied relative values of selected possible failure lines under present conditions and under conditions proposed after making the cuts shown on your drawing. The average slope, for the west bank, shown on the Department of Highways drawing, is approximately 22 deg. The cut shown for the west bank, on the drawings submitted to us, should reduce the tendency of the bank to slip, possibly by 25%, which should provide some addition to whatever factor of safety now exists.

The average slope shown for the east bank, on the Department of Highways drawing, is approximately 16 deg, and your drawing has already reduced this slope. We have indicated a further modification to provide a terrace for the pile driving rig and pile cap forms. Conditions appear, by inspection, to be slightly less severe at the east bank.

The soil at the river edge banks is indicated as very loose, silty sand, and the drawings indicate slopes of 35 to 45 deg. These banks do not appear stable, and might well slide locally when subjected to nearby pile driving vibration and weight of the rig. We suggest moving each pier approximately 22.5 ft farther away from the river than shown on your drawing, increasing the distance between piers from 225 ft to 270 ft. This will keep the pile-driving rig load more safely away from the edges of the banks, particularly as it may be necessary to bring the rigs out onto the lines of the pile groups in order to drive batter piles in several directions. In such case, the pile rig load might add from 5 to 10% to the stress on a possible local failure surface. Relocation of the piers will permit omission of the sheeting and rock-fill around the footings, including the proposed fenders. We have shown terraces at El. 611.5 to provide level areas for the pile driving rig and to serve as earth forms for the pile caps. We suggest carrying these cuts approximately 50 ft each side beyond the cap and making gradual transitions to existing slopes. The benches shown on our study at El. 611.5 are above reported high water El. 609.4.

The net result of pier relocations and additional cuts suggested on our study should be an increase in the factors of safety which would be available to withstand the temporary effects of driving.

RELOCATION OF ABUTMENTS

If the distance between piers is increased, it doubtless will be necessary to move each abutment back at least one panel, or 22.5 ft. This should not have a detrimental effect upon bank stability; instead, it should improve conditions slightly.

PILES

We suggest that pier pile cutoffs be lowered from El. 615 to 612 and that pile lengths be reduced accordingly. The pile caps could then be poured directly on the terraces which we have

suggested at El. 611.5. The proper type of pile for this site, in our opinion, is the one that will cause the least jarring or displacement of the soil. We concur in your choice of 10 in. BP 42 lb steel piles. These piles should require the least driving energy and the fewest blows of any type. Displacement pipe piles would require much harder driving and cause displacement of the soil, thus disturbing the equilibrium of a bank much more than would the H-pile. We do not believe that it would be practicable to use open-end pipe piles, because of the reported hydrostatic pressure originating in the strata at lower levels.

Based upon the number of spoon blows, shearing and cohesion values contained in the E. M. Peto reports, and our experience and that of certain authorities, we studied the lengths which we would recommend for piles under the piers by the application of suitable cohesion values. For permanent load-carrying purposes at the east pier, we selected the 40 ft of soil having 13 to 20 blow resistance, located between El. 552 and El. 512. This indicated a pile length of 103 ft below your original cut-off elevation of 615. This length would become 100 ft if the cap is lowered as we recommend.

For the west pier, the numbers of spoon blows in the 14 to 21 blow range occur at practically the same elevations as similar blows at the east pier, and we suggest that the piles be of the same length at all piers.

At the east abutment, your bridge drawing shows pile cut-off at El. 630. If 40 ft embedment in load-carrying material of similar type to that under the piers is used, the pile lengths would be 127.5 ft and the tip elevation would be 502.5. We recommend the use of 127 ft piles at the east abutment.

At the west abutment, your bridge drawing shows pile cut-off at El. 632.6. If 40 ft penetration in similar material is used, the pile lengths would be 126.8 ft and the tip elevations would be 505.8. We, therefore, suggest the use of 127 ft long piles also at the west abutment.

Except for load test piles which should be vertical, we recommend that piles on close centers be battered, so that the bulbs of pressure would not overlap to any significant extent where avoidable. This would distribute the load over a much larger area of the strata at and below the pile tips and thus tend to reduce settlements, and would permit less reduction in bearing values due to group action.

We recommend that all piles in any one group be driven to the same tip elevation.

We recommend encasing the top sections of the pier piles in concrete down to El. 595. Methods of encasement which we have used successfully are shown on pages 366 to 370 of PILE FOUNDATIONS by the writer.

PILE DRIVING

The driving rig should be kept as light as possible, but should be able to handle a drop hammer or single-acting hammer large enough to drive with a minimum number of blows. The rig should also be able to handle pile sections 50 to 60 ft long, and to drive on batters.

We recommend the use of a drop hammer or a Vulcan No. 0 hammer operated by either steam or compressed air. It would be desirable to have the heavier but fewer blows, at longer intervals, than would be obtained by the use of a Vulcan No. 1 hammer or by double acting or differential acting hammers. The lighter hammers might require the use of jetting to reach the desired depths, which we consider objectionable.

CONSTRUCTION PROCEDURE

We concur with your suggestion that it would be safer to wait until autumn before driving piles, to allow the wet banks to drain as much as possible and thus increase the stability. It is possible that artificial drainage might be helpful, or even necessary, but we are unable to predict this. If the banks show signs of horizontal weeping, this might be an indication that wells or horizontal drains should be installed. Vertical wells could be installed at fairly frequent intervals, particularly on the steeper west bank, and by the use of continuous pumping, the stability of the bank might be improved. If wells were to be installed, it would be preferable to have them in use some time before the start of pile driving. If they were not provided prior to pile driving, it might be necessary to install them if driving produced signs of hydrostatic head at the bank surface.

We recommend that all earth cuts be made before placing the pile driving rig on the banks.

It would be preferable to drive center piles first in a group, to avoid the soil tightness which might occur in the center if center piles were to be driven last.

Splices should be avoided during the last 40 ft of driving, because possible setup of surrounding soil might make the restart of driving difficult.

We suggest that continuous pile driving records be kept so that uniformity of underground conditions may be observed and special features reported. A set of three report forms which we use is attached for reference, to indicate the various items of information of which records may be useful.

Application of the Canadian National Building Code pile driving formula is suggested, but piles should nevertheless be of the lengths determined by the pile friction values which we have used, unless modifications are found permissible after study of load test results. The correction for batter pile reductions is shown on pages 40 and 41 of PILE FOUNDATIONS. The use of a pile driving formula on this project can only serve as a guide to uniformity of conditions and identification of strata, and no reliance should be placed upon it for load carrying capacity. This is particularly the case because of the hydrostatic pressure present and the fact that the results of a blow causing vibration will probably be considerably different from the results obtained under static load.

We believe that it would be preferable to drive pier piles before abutment piles since pier piles pass through less thickness of soft soils above the bearing strata and should provide better load test information. Otherwise, we would have a slight preference for driving abutment piles first, to minimize any tendencies for bank movements to affect the pier piles.

LOAD TESTS

We believe that a load test on the first pile to be driven probably is the only way of determining pile carrying capacity at this site, because of possible hydrostatic pressure with water flows along the pile during driving, and uncertainties as to its continuance even if not appearing at the surface because of flow through sand or silt seams. Such conditions might result in a quicksand effect during the vibration of driving, and temporarily reduce resistance to penetration. A load test, which would avoid the vibration from driving, may show adequate setup. For these tests, one or more vertical piles should be driven at each river pier. The tests should be made at the river pier instead of at the abutment, because of the greater ease of observing hydrostatic conditions at this location and the shorter depth of material to penetrate above the load-carrying strata. Redriving, after the soil has been given opportunity to close in around the pile, can be tried before the load test, to provide information to guide driving of other piles, but this would not obviate the need of the initial load test.

By using a cantilever test rig on the test pile, with two uplift anchor piles, a convenient light load consisting of a small water tank or stack of rail fish plates could be used, or jack pressures might be provided. These methods of testing might avoid possible dangers from tilting or settling of a direct full load test platform of material, and will facilitate cyclic testing to enable the elastic and plastic soil deformations to be separated. One of these schemes, using the load of removable fish plates, was employed at Marathon. These types of load tests are shown on pages 400 to 407 of PILE FOUNDATIONS.

BRIDGE SUPPORTS

We believe that use of the flexible post members above the pier caps, as shown on your bridge drawings, is desirable. We would not provide any diagonal members from these posts to adjacent truss panel points.

We agree with you that lateral anchorage should be provided at the points where the trusses rest on the abutments.

SETTLEMENT

Some settlements should be expected, and the bridge construction should be designed to permit them without causing secondary stresses. Jacks or shims could be provided for future use.

INDICATIONS OF POSSIBLE DAMAGE OR DANGER

We suggest that a close watch be kept for indications of incipient sliding. We suggest setting markers for locations and elevations at various points on the banks, referred back to immovable markers on land well out of zones of possible effects from rig loading, pile driving, or dumped materials.

We can not be sure that there will not be a plane of weakness on some horizontal, inclined, or circular surface, which may permit failures triggered by the rig weight and driving vibrations. We consider that the various precautions shown on your drawings and suggested by us will materially increase the factors of safety which might otherwise be present, and will provide a reasonable basis of design. We are sure that you realize that there is some calculated risk involved in construction at this location. In our opinion, this risk exists principally during construction.

8.

We trust that the information contained in this letter will assist you in preparing your final designs for the substructure and superstructure of this bridge. Please feel free to call upon us if any of these points require further clarification.

Yours very truly,

R. D. Chellis

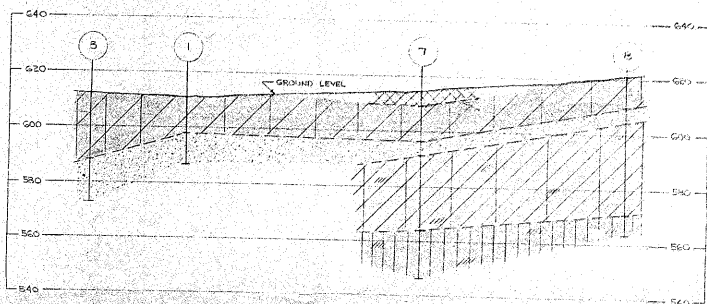
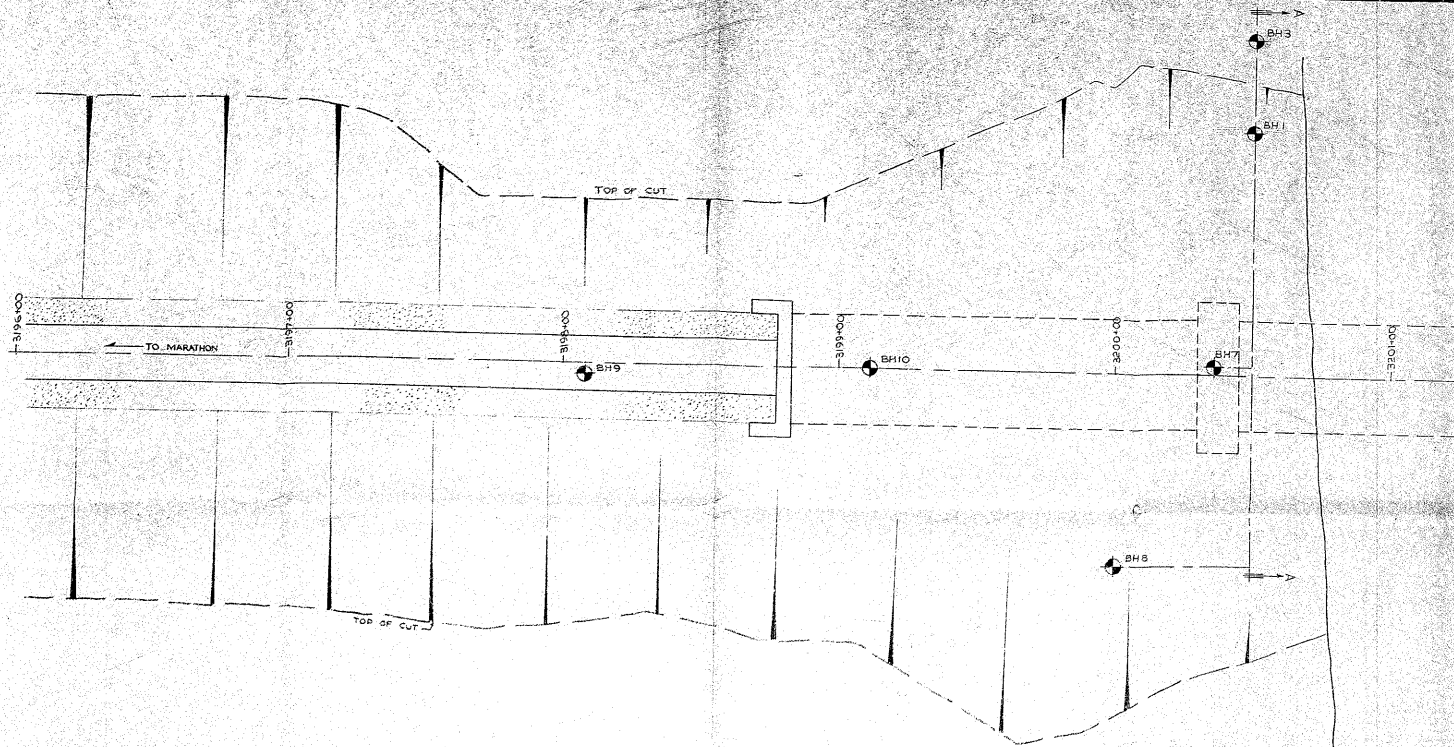
R. D. Chellis,
Structural Engineer.

Enclosures.

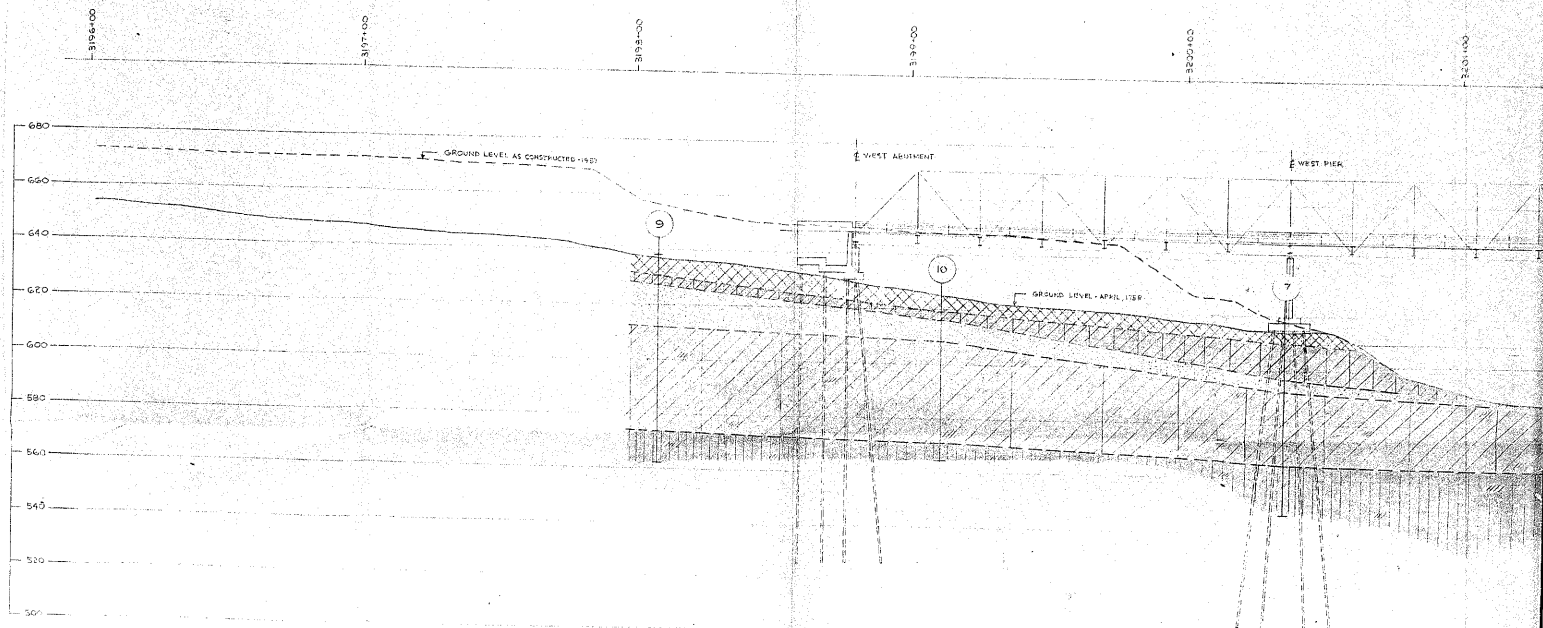
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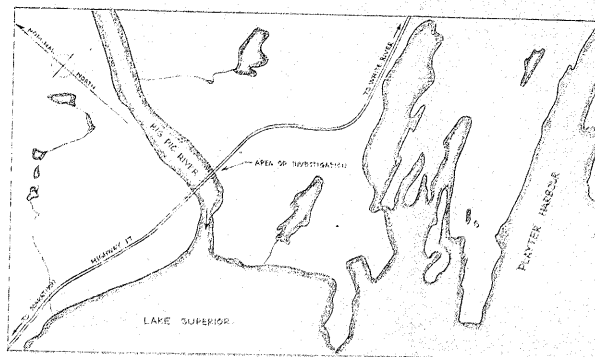
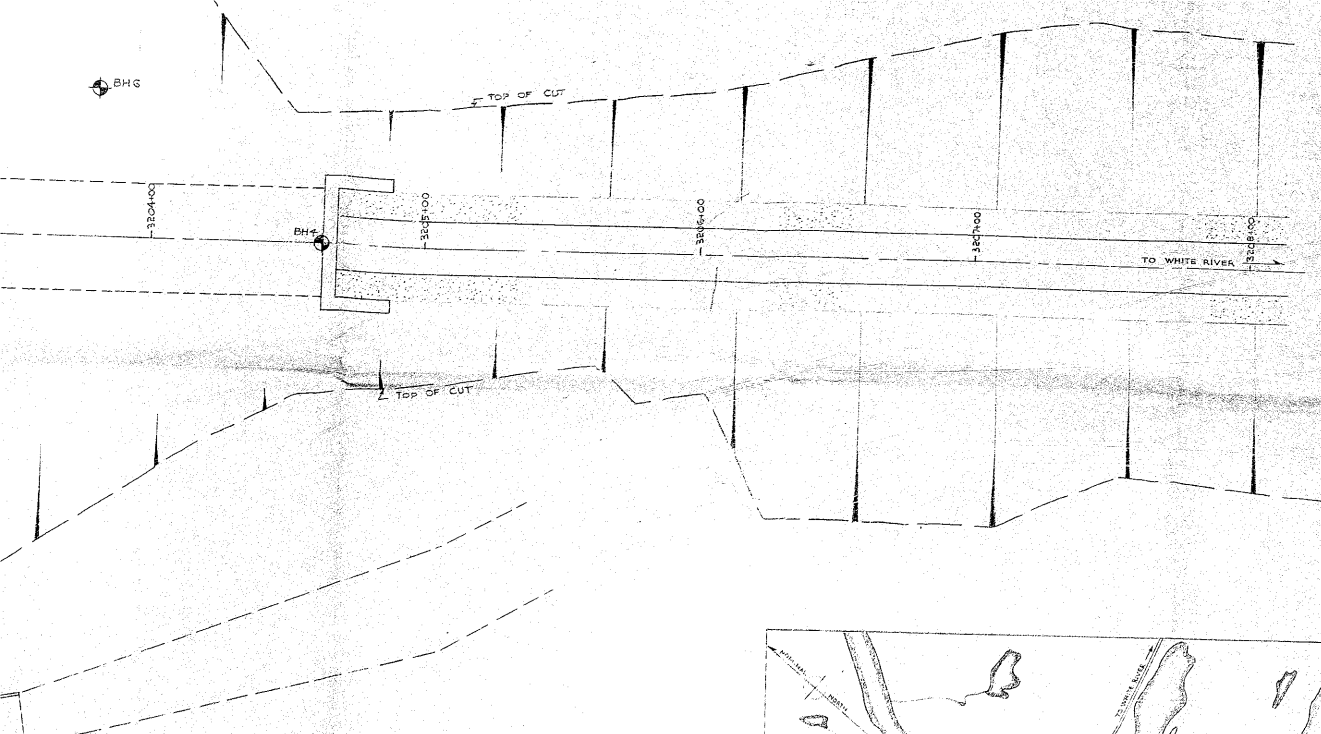
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BIG PIC RIVER

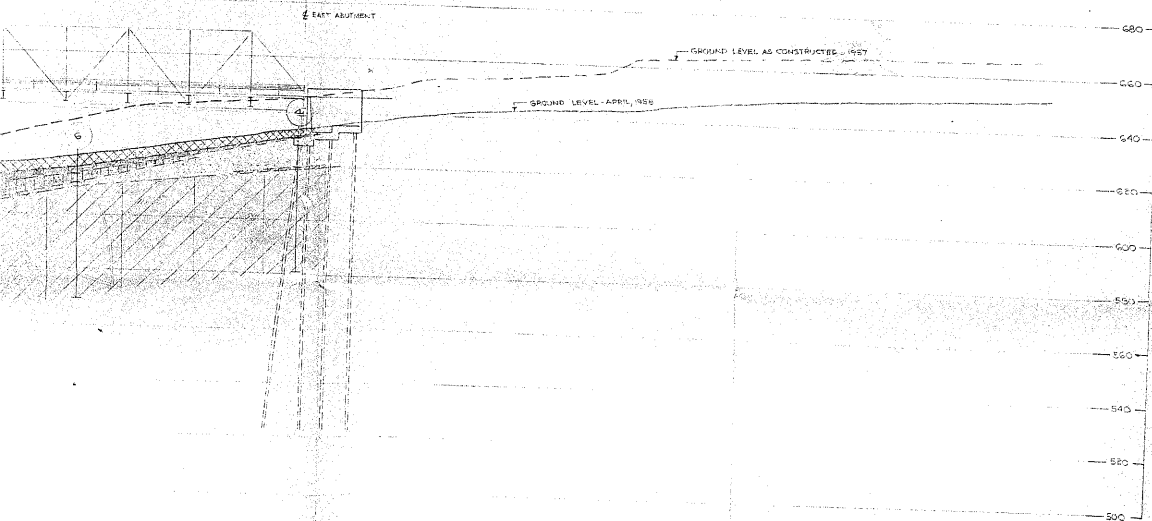


SECTION A-A





KEY PLAN
SCALE: 1" = 1/4 MILE



STRATIGRAPHY

- LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL
- SOFT TO STIFF LIGHT GREY AND BROWN CLAY AND SILT
- COMPACT GREY AND BROWN MEDIUM SAND
- LOOSE GREY, VERY SANDY SILT
- SOFT TO MEDIUM GREY VARVED SILTY CLAY
- COMPACT LIGHT GREY, VARVED SILT
- FINE GREY, SILTY SAND

LEGEND

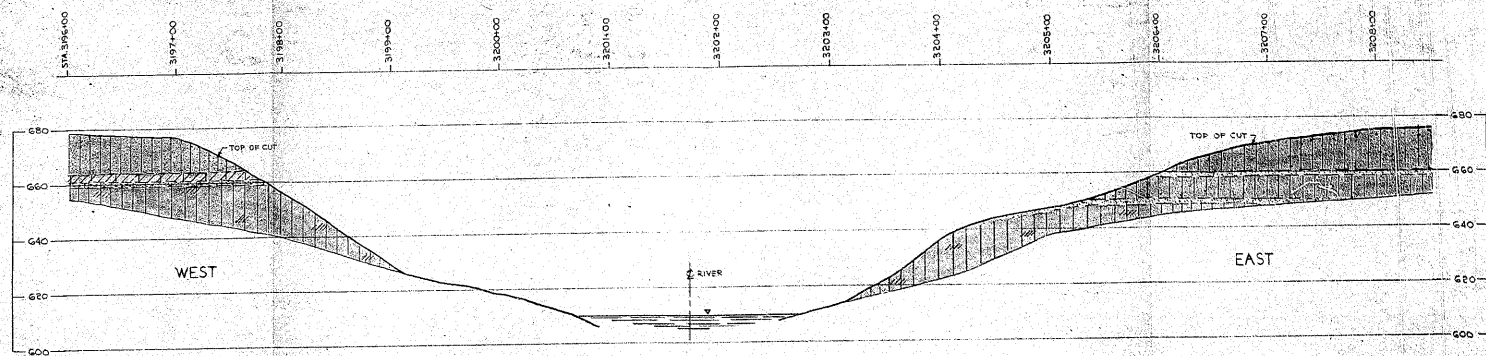
- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION

NOTES

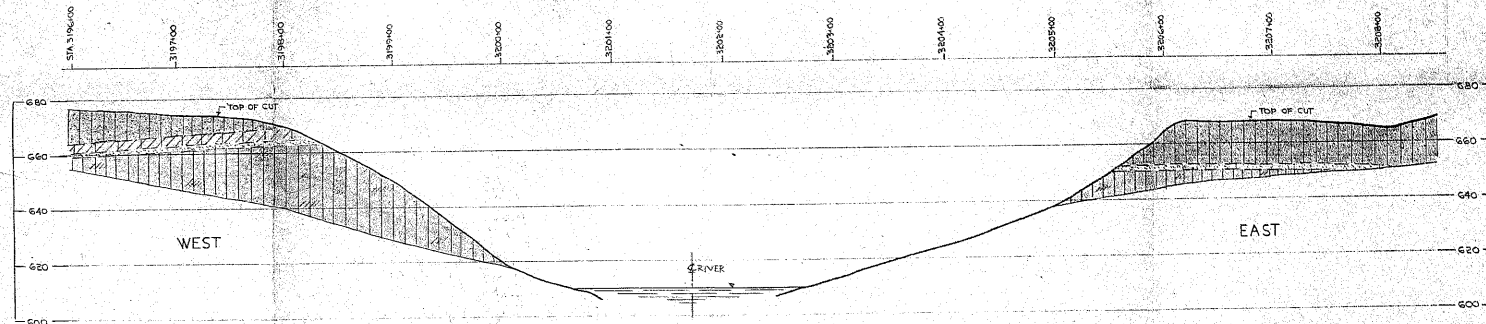
- (1) BOREHOLES NO. 1, 3, 4, 5, 6, AND 8 CARRIED OUT BY SWEDISH FOR SAMPLES
- (2) ALL CHAINAGE REFERRED TO PROCTOR & REDFERN DRAWING AN D-571-E-1

REFERENCE		DEPARTMENT OF HIGHWAYS, ONTARIO		TORONTO	
		PROPOSED BIG PIC RIVER BRIDGE		ONTARIO	
		BORING PLAN AND SOIL STRATIGRAPHY		ONTARIO	
		DATE AUGUST 20, 1958 SCALE 1" = 1/4 MILE		GEOCON LTD	
		No. S6657-1			

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE NO. 1 AND THE SOIL STRATIGRAPHY OBTAINED AT BOREHOLE NO. 1 HAS BEEN REFERRED FROM BOREHOLE NO. 1 TO ALL OTHER BOREHOLES.



NORTH SLOPES

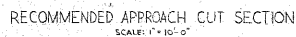


SOUTH SLOPES

LEGEND

- LIGHT GREY AND BROWN SANDY SILT
- GREY SILTY CLAY
- GREY AND BROWN SAND
- LIGHT GREY VARVED SILT

REVISIONS		REFERENCE		REFERENCE		DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO PROPOSED BIG PIC RIVER BRIDGE MARATHON ONTARIO SLOPES OF CUT - SKETCH OF SOIL STRATIGRAPHY	GEOCON LTD DATE AUGUST 1977 SCALE 1" = 20'-0" No. S 6657-2
DATE	DESCRIPTION	DATE	DESCRIPTION	DATE	DESCRIPTION		



SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THE PRESENT.