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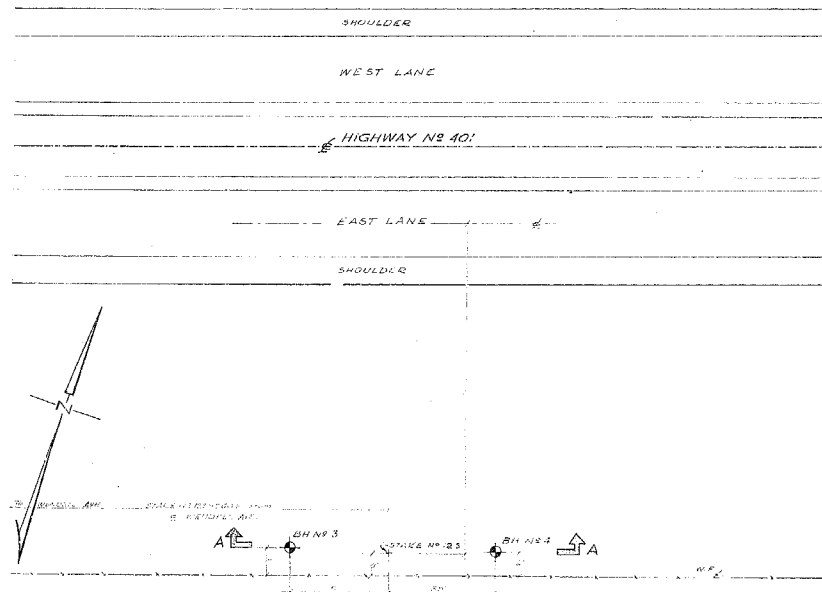
W.P. # 233-60

Hwy # 401 &

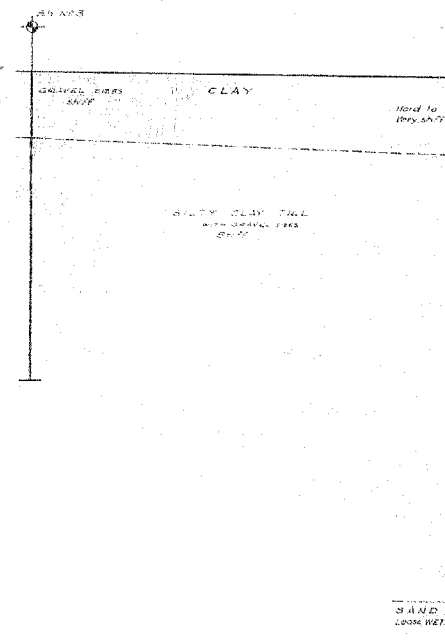
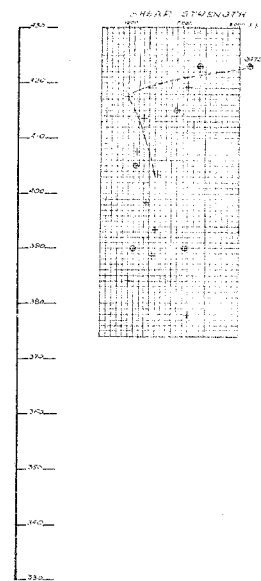
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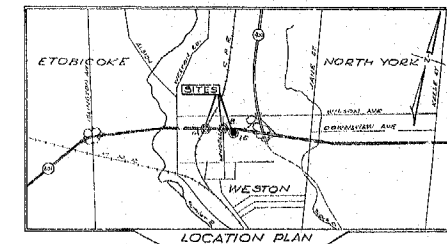
BRIDGES



BORE HOLE LOCATION PLAN
SCALE 1"=20'



SECTION A-A
SCALE 1"=10'



W.A.TROW & ASSOC. LTD.
FOUNDATION INVESTIGATION
EXIT STRUCTURE 125 HWY. 401.
PROJECT NO. 1940 WR NO. 228-60 DATE OCT. 1962 DWG. 1.C.

Materials and Research Division.

August 28, 1962.

William A. Trow & Associates,
1850 Jane Street,
Weston, Ontario.

Attention: Mr. W. A. Trow.

Re: W.P. 233-80, Hwy. #401, Hwy. #400 Interchange,
3 Structure Sites to West, District #6, Toronto.

Dear Sir:-

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles will be provided by the Bridge Design Consultants, De Leuw, Cather, who will also specify the extent of the investigation required.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of the completed foundation reports should be submitted to the Foundation Section prior to October 15, 1962.

Charges for the work performed will be in accordance with your Schedule of Rates, dated May 24, 1959, and invoice to be addressed to the attention of the undersigned.

WAT/raef

cc: Messrs. S. McCombie
C. K. Hunter
C. Fraser
T. J. Kovich
R. D. Smith (2)
Mrs. T. Tate
Foundations Office
Gen. Files. (2)

Yours very truly,

A. Rutka
A. Rutka,
MATERIALS & RESEARCH ENGINEER

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. E. McSporbie.

Mr. A. G. Stereac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

October 19, 1962.

FOUNDATION INVESTIGATION REPORT -
By William A. Trow & Assoc., Ltd.,
Three Bridges - Hwy. #401 Expansion
West of Hwy. #400, District #6,
W.F. 233-60.

Attached, we are forwarding to you the above-mentioned report submitted by William A. Trow & Associates, Ltd.

We are in agreement with the conclusions and recommendations contained in the consultant's report, and believe this information will prove adequate for your future design work.

Should there be any queries in connection with this project, please do not hesitate to contact our office.

KYL/edf
attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. G. Maxwell
G. H. Hunter
C. Fraser
T. J. Kovich
J. Roy
J. E. Graspier
B. H. Saint
P. Norman
A. Gatti

Foundations Office
Gen. Files

Kyl
K. V. Lo,
SUPERVISING FOUNDATION ENGR.
Per:

A. G. Stereac,
PRINCIPAL FOUNDATION ENGR.

62-F-241C 23-63-216
W.P. 233-60
WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.Sc., M.E.I.C., P.ENG.

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J 949

October 12, 1962.

Mr. A. Rutka,
Materials and Research Engineer,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ont.

Attention: Mr. A. G. Stermac, P. Eng.,

Re:

Foundation Conditions
Bridges at Station 12 S, Wendell Ave. and C.P.R. Crossing
Proposed Highway #401 Expansion

Dear Sirs:

The enclosed report contains the results of a study made to determine the foundation requirements for three bridge structures forming a portion of the enlargement of Highway 401, immediately to the west of the Highway 400 interchange.

In two instances, at the C.P.R. and Wendell Ave. crossings, the proposed construction involves a major expansion of existing facilities. Bridge 12 S will be a new structure forming a portion of the interchange complex proposed at the Highway 400 junction.

Four deep borings, at these bridge sites, revealed essentially similar subsoil stratigraphies. A surface mantle of desiccated lacustrine clay covers the area to a depth of approximately 12 feet, and this is underlain by stiff, generally lean clay till of the Wisconsin glaciation. This material extends at least to a depth of 100 feet. A buried valley, presumably a pre-glacial channel into bedrock, has been proven to a depth of about 300 feet in the vicinity of Hwy. 400. A deep seated stratum of sand, - outlined in this investigation, - seems to be draining into this valley and to the present Humber River to the west of Main St. According to laboratory measurements, the clay till exists in a heavily over-consolidated state.

On the basis of this field and associated laboratory study, it is concluded that no foundation difficulties will be experienced in the proposed construction. Maximum settlements in the order of 3 inches have been computed for the C.P.R. and Wendell Ave. crossings, and these estimates probably are quite conservative. Settlements of somewhat greater magnitude have been computed for bridge 12 S, which, according to present information, is heavily skewed. Although no major distress to any of the structures is anticipated as a result of settlement, it is recommended that all embankment fill should be installed before the bridge structures are built. In this way, at least 50 per cent of the computed movements, which may be **undesirable** in the construction, will be eliminated.

A safe net bearing value of 2 t.s.f. has been recommended for all bridge construction. In some instances, where the footings bear in or just above a localized pocket of relatively less stiff clay, the factor of safety, associated with this loading, may be less than three, temporarily, until the soil consolidates to a higher strength. Although this situation could be considered undesirable for a building having columns at 20 foot centres or less, it should not be a matter of concern for these wide span bridges. The main cause of settlement of the bridges is the weight of the approach fill, which will compress the soil regardless of the bearing stress used. The foregoing remarks apply only if the bridge additions are supported at the level of the existing footings. An increase in capacity or an improvement in the factor of safety can be obtained by supporting the bridge additions at higher levels in the desiccated surface crust. It is recommended that on site examinations of all footing beds be made in order to assure that all structures bear in competent soil.

We shall be pleased to discuss the factual results and opinions of this report if you consider this to be necessary, following your review of the contents.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH BRANCH
PARLIAMENT BUILDINGS, TORONTO, ONTARIO.

FOUNDATION CONDITIONS
BRIDGES AT STA. 12 S, WENDELL AVE. & C.P.R. CROSSING
PROPOSED HIGHWAY #401 EXPANSION

Project: J 949

William A. Trow and Associates Limited

October 12, 1962.

TABLE OF CONTENTS

Project	Page
C.P.R. Overhead - General	2
Subsoil	2
Ground Water	3
Foundations	4
Wendell Ave. Overpass - General	7
Subsoil	7
Ground Water	7
Foundations	8
Exit Structure 12 S - General	10
Subsoil	10
Ground Water	10
Foundations	11
Appendix I - Field Work and General Subsoil Laboratory Testing	(i) (ii)
Appendix II - Settlement Computations	(iv)

ENCLOSURES

Summary of Water Level Measurements, Holes 1 - 4	Table 1
Borehole Location Plan - C.P.R. Crossing	Dwg. 1A
Wendell Ave.	1B
Station 12 S	1C
Borehole Profiles, Holes 1 to 4	Dwgs. 2-5
Consolidation Test Results	6-7
Undrained Triaxial Test Results	8 - 10
Consolidated Undrained Test Results for Elastic Modulus Computations	11
Settlement Computations	12-14

FOUNDATION CONDITIONSTHREE BRIDGES ASSOCIATED WITH THE HIGHWAY NO. 401 EXPANSION
WEST OF HIGHWAY 400
W.P. 233 - 60PROJECT

The proposed scheme for traffic improvement along Hwy. 401 involves its expansion into 6 east-bound and 6 west-bound lanes of traffic, half of which will comprise collector systems paralleling the main highway.

The overall horizontal width of this new highway facility, including medians and shoulders will be approximately 238 feet, or 2.4 times the width of the existing traffic artery.

In order to accommodate this major expansion, additions must be made to the existing bridges over Wendell Ave. and the main line of the C.P.R. The top width of the approach fill must also be increased about 70 feet, both to the north and to the south of the present embankments. Retaining walls will be required in some locations where the fill would otherwise extend beyond the right of way limits.

The smooth flow of traffic to and from collector lanes and Hwys. 400 and 401 will be accomplished, where necessary, by the installation of additional bridges. One of these bridges, designated 12 S, will carry east-bound Hwy. 401 traffic, destined for Hwy. 400, over the east-bound traffic of its adjacent collector system.

The subsoil conditions and foundation requirements at these three bridge sites form the subject of this report.

In the interest of brevity, since the subsoil and foundation problems are essentially similar at all locations, the specific information required for the design of each structure will be presented in the following independent sections of the report. Other information of a general nature, applicable to all structures, will be presented in appropriate appendices.

C.P.R. OVERHEADGENERAL

The existing bridge over the double-track C.P.R. main line is a rigid frame skew structure approximately 115 feet wide. The highway pavement lies approximately 28 feet above the top of rail and the approach embankments rise to heights ranging from 17 to 24 feet above the original ground surface.

The ground slopes gently to the east northeast at this particular location, and therefore the embankment fill is highest, relatively speaking, at the northeast corner and lowest at the southwest corner. This situation will also apply when the additional fill for the collector lanes is eventually placed. In order to provide drainage along the southeast boundary of the highway, an estimated maximum thickness of 9 feet of soil has been excavated in this area.

There is definite evidence of differential settlement between the main bridge structure and the retaining walls adjoining it. These differential movements occur in the obtuse corner of the fill where the embankment weight is more concentrated. A maximum settlement of $1\frac{3}{4}$ inches, of the bridge with respect to the wall, is evident at the southwest corner, and a lesser differential movement of a $\frac{1}{2}$ inch can be seen at the northeast corner. The top of the retaining wall has moved out approximately 4 inches at this latter location. There is no visible differential movement at the acute southeast and northwest projections of the fill.

It is of interest to note that the elevations at the base of the guard rail of this bridge average about $1\frac{1}{2}$ inches lower than the levels specified on the 1951 design drawings. This difference, of course, may be fortuitous and not particularly indicative of settlement. The specified and present elevations of the structure and the railway track are shown on Dwg. 1(A).

SUBSOIL

The subsoil conditions in this entire area are reviewed in a following section of this report. They are described in detail in the log for borehole No. 1, made at the northeast corner of the bridge fill.

Briefly, the stratigraphy consists of approximately 10 feet of very stiff brown laminated or layered clay, underlain by a glacial clay deposit of Wisconsin origin which was proven to a depth of 100 feet. The former stratum possibly is a lacustrine deposit of the shallow Peel Ponding era or of some other temporary glacial pond in existence prior to the Lake Iroquois epoch and the associated final retreat of ice from Southern Ontario. The ground here is about 30 feet above the water level of the Lake Iroquois embayment which penetrated into the Jane Street - Wilson Avenue area.

The glacial clay has a variable plasticity with depth. In general, it consists of lean sandy clayey silt with gravel. However at many levels it was found to be interbedded with pockets of more plastic laminated clay. The locations of these more plastic pockets are evident from the results of moisture content and Atterberg limit measurements recorded on Dwg. 2.

A 10 foot stratum of silty sand with gravel was outlined below a depth of 65 feet in hole 1.

GROUND WATER

Originally, the boring at this bridge location was terminated at a depth of 50 feet. However, as information came available regarding the amount of additional fill to be applied over this area, it was decided to continue all borings at least to 100 feet. Consequently, another boring was made 10 feet to the west of the first hole and samples were taken below a depth of 50 feet. A piezometer was installed in this latter hole at a depth of 30 feet.

The water level records for both holes are shown on Table 1. The original 50 foot hole was terminated in impermeable clay till. It is noted that the water level in this hole rose very gradually almost to ground level. It is expected that the rise to this level signifies seepage of water from wet soil close to the ground surface. The land bounding the railway is wet and poorly drained. However, it is felt that the long term water table in the clay lies at a depth of about 14 feet, the level where the clay changes from brown to grey in colour.

This opinion is not supported by the piezometric measurements in the 100 foot boring, hole 1(A), made about 10 feet to the west of hole 1. This deep hole was quite wet during the augering process and, - when the piezometer was installed, - the water level was 19 feet below the surface. Before placing the piezometer at 30 feet, the space below was filled with crushed stone and cinders. The tamped bentonite seal was placed around the plastic tubing from $21\frac{1}{2}$ to $27\frac{1}{2}$ feet.

When the initial water level reading was attempted in the tubing, soon afterward, the piezometer was found to be dry. Water then was poured into the tube up to ground surface level, but it quickly sank below piezometer level. After a period of two weeks, the same procedure was repeated with the same result. This piezometer was found to be dry to a depth of 30 feet over a period of at least 17 days.

It is concluded from this observation that the water is draining into the sand stratum below the 65 foot level. According to Watt, this stratum continues to the Humber valley, a few hundred yards to the west.* If this reasoning is correct, the upper clay and till strata must support a water table condition which increases in a linear manner probably to a depth of about 40 feet and then decreases again to some lower value at the

* "Pleistocene Geology and Ground Water Resources of the Township of North York" - A.K. Watt, Ontario Department of Mines

sand stratum. Consequently, the effective overburden stress presently acting on these lower levels of clay till will be much greater than would apply if the water table increased in an uninterrupted linear manner to full depth.

FOUNDATIONS

The addition of three collector lanes of traffic on each side of the existing highway will require major bridge **additions** to the north and south of the existing structure, together with considerable fill ranging up to a maximum of approximately 24 feet above ground surface at the north-east corner.

Since the existing embankments appear to be quite stable, and the rigid frame structure seems to be in a sound condition, there obviously is no embankment stability problem associated with this bridge enlargement scheme. Considering that the lowest undrained shear strength of the clay is in the order of 1200 psf, - which value is believed to underestimate the clay strength, - this stable behaviour is to be expected. Consequently, the only information required for the design of the abutments and retaining wall foundations at this location, is the safe net bearing value of the soil and the estimated long term differential settlement following the construction of these highway additions.

The safe net bearing value to use in the design of the bridge or retaining wall footings will be determined by the undrained shear strength of the clay. Measurements of this soil property are indicated in the record of field vane and laboratory undrained triaxial tests shown in the log for hole 1. It is seen that the shear strength is high in the upper desiccated brown crustal clay and that this strength diminishes to a minimum value of approximately 1700 psf at a depth of 13 feet.

This low value has been arbitrarily interpreted from the results of vane tests above and below this level and from three undrained triaxial tests made on samples from hole 1 and hole 1 (A), taken at this level. The low result of 1200 psf for hole 1 is not considered to be representative, since the other samples showed much higher strength. When this soil was reconsolidated under a stress less than overburden, a shear strength in excess of 2000 psf was obtained, although the moisture content was essentially unaltered. A very sharp transition from hard brown to very stiff grey clay till was noted in the triaxial specimen from hole 1 (A). The higher vane test results recorded below 30 feet in the hole should be viewed with some reservation, since the presence of pebbles possibly caused the measurements to err on the high side.

The existing foundations for the bridge bear at approximate El 441.5 feet, or about 10 feet below the ground surface at the hole 1 location. It is reasonable to assume that the bridge additions also will be supported at this elevation, although a slightly higher bearing value is

desirable in order to obtain more benefit from the hardened crust. A higher bearing level should be feasible for the retaining wall footings although a 4 foot drop-off in ground level just to the north of the northeast boundary will require footings in this vicinity to be taken down at least to El 444 feet. The estimated average shear strength of the clay within the significant depth of footings below El 441.5 feet is in the order of 2000 psf.

The safe net bearing value to apply for the footing shape and foundation depth applicable here is given by the expression:

$$q = \frac{CN}{F}$$

where: C is the undrained shear strength = 2000 psf
N is a bearing capacity factor estimated to be equal to 6
F = 3 is the suggested factor of safety for this application

Solving this expression, the safe net bearing value, q, is determined as 2 tsf.

The settlement resulting from this pressure application will be small and unimportant. The major cause of settlement will be the approach fill added over the north and south slopes of the present embankments. In order to obtain some indication of the magnitude of this long term movement, settlement calculations were made at significant positions under the new bridge additions.

The results of these computations are presented in Dwg. 12-14. It is seen that the maximum long term settlement of the new structure ranges from 1.8 inches at the junction with the existing bridge to 3.3 inches at the north shoulder of the new fill. These values apply for the north-west side of the highway, where the fill loading is greatest. Lesser movement should be expected along the south side of the highway.

As with most consolidation predictions, these estimates probably err on the excessive side. This undoubtedly is the case at this site since the subsoil under the new additions has been processed to some extent by the existing embankment fill. In addition, since the new fill is of limited lateral extent, pressures from it will be spread out to a greater degree through the very stiff clay crust than would be the case for the much wider present embankments.

Attention is also drawn to the other qualifications to this analysis, outlined in the Appendix. In particular, it is noted that no allowance has been made for the spread of load through the existing embankment or for the weight of the new bridge structure. This latter weight will

result in a very small additional settlement, estimated to be much less than one inch, acting over the entire length of the additions. The spread out of load of fill, resting over the existing embankment slopes, will result in smaller settlements at the junction with the existing bridge.

In view of the approximations and uncertainties generally associated with settlement computations, the inclusion of all these additional factors into the analysis does not seem to be warranted. The maximum total and differential settlement of the bridge additions certainly should not be more than the values indicated above and, in all probability, the movements will be less. Since movements of these magnitudes can be accommodated by the single span additions, no purpose is served in performing a more rigorous analysis or in the computation of settlements at the other less heavily loaded corners of the structure.

However, although settlements should be smaller than the values indicated above, there is no doubt that some settlement will occur. The visible differential movement between abutment and wing walls of the existing structure attest to this view. Because of this movement there will be a slight tendency for the ground under the ends of the existing abutments to be depressed. Although this, theoretically, could introduce a cantilever condition into the ends of the existing bridge, the abutment frame probably is rigid enough to withstand this effect. The tendency to cantilever will produce a redistribution of load to the centre of the abutment which in turn will cause some slight additional settlement of the structure. The loss of support at the ends of the bridge should be resorted as a result. Consequently, the effect of the new construction on the existing bridge is considered to be unimportant.

The magnitudes of earth pressures exerted against the retaining walls, supporting this earth fill, will be determined in large part by the type and compacted condition of the fill and by the effective height of embankment at any particular location. It is assumed that provision will be made for rapid run-off of surface water along the top of the fill and for the disposal of seepage through granular drains behind the walls. Granular material, backed with well-compacted very stiff clay till probably will be used for the embankment extensions. From the long term view the earth pressure coefficient applicable for this soil condition is estimated conservatively to be $k = 1/3$. In view of the construction uncertainties involved in this phase of the work, a more detailed investigation of this earth pressure factor is not justified unless a considerable saving in the cost of the retaining wall can be achieved.

An earth pressure approaching the at rest condition will apply against the rigid abutment walls of the bridge addition. For granular backfill the earth pressure coefficient, k_0 , will have a value in the order of 0.5. With footings exerting a net bearing stress of 2 tsf. to the soil, there will be more than enough sliding resistance developed along the base of the footings to resist this force.

No ground water problems are envisaged when digging to the footing levels of the new construction.

WENDELL AVE. OVERPASSGENERAL

The existing bridge at this location also is a rigid frame structure having a width of approximately 100 feet. The height of embankment fill ranges from about 10 feet at the south-west corner to 19 feet at the north-east corner. The fill associated with the new construction also will reach heights of this order. The maximum thickness of fill will be along the north side of the existing embankments. The land in this area slopes gently to the east with a slight dip toward the north as well.

No visible evidences of settlement were noted in the existing structure. Measurements were taken of the present elevation of the bridge during this investigation. A difference of about 2 inches lower than those specified on the original design drawings of 1951 was noted. However, since the exact construction elevations may not conform to these recorded values, this observation does not have any particular significance.

SUBSOIL

The subsoil condition is essentially similar to the profile existing under the C.P.R. bridge to the west. The upper approximately 12 feet of soil consists of layered and more highly plastic lacustrine deposits and below this level is a deep deposit of lean clay till existing generally in a stiff condition. The shear strength of this clay till seems to be somewhat softer than the conditions noted at the C.P.R. bridge site, but the difference is unimportant.

A stratum of fine wet sand underlies the clay till below about 85 feet. Clay was encountered below it at approximately 9.9 feet depth.

GROUND WATER

Water level measurements were taken in the uncased boreholes over a period of almost one month. The results of these observations are recorded in Table 1. It is noted that the ground water eventually rose to a depth of 2 feet from the surface. This, however, is considered to be the result of water seepage from wet ground close to the surface.

For a considerable period, the water level remained about 52 feet below the surface in the borehole. Since the hole was open to the sand stratum below 85 feet, it is possible that this measurement is a

reflection of the water table in the sand which seems to dip to a deep valley in the bedrock farther to the east. An adjacent abandoned well, noted on Dwg. 1B, was found to be open and dry to a depth of 45 feet on October 9th, 1962. It also probably extended down to the sand stratum. The eventual rise of the water to a higher level probably could result from the plugging of hole 2 as clay particles slaked off the sides of the borehole.

The long term water level probably coincides closely to the depth where the clay changes from brown to grey in colour. This transition occurs about 12 feet below the surface.

The exact level of the ground water table is not of vital concern in this project, however, since the clay is heavily over-consolidated and it is too impermeable to be water bearing in the vicinity of excavations.

FOUNDATIONS

As in the report for the C.P.R. bridge, there does not seem to be any particular embankment stability problem at this site. The existing fill is stable, as should be expected in view of the strengths recorded in the log for borehole 2. Consequently, the only requirement for the design of the new construction at this crossing is the determination of safe bearing values and the estimate of settlements following embankment construction.

From design drawings, the footings of the existing structure are estimated to be at El. 435 feet, or about 9 feet below the ground surface at the hole 2 location. The footings of the bridge additions probably will be located at this level as well.

The undrained shear strength records, presented in the log for borehole 2, indicate a localized reduction in capacity at El. 435 feet. The shear strength at this depth reduces to a value of approximately 1450 p.s.f. However, at greater depths the strength increases to 1900-2000 p.s.f. The approximate average shear strength within the zone of influence of footings at El. 435 feet is about 1700 p.s.f.

According to the relationship indicated for the C.P.R. bridge, the safe net bearing value for a factor of safety of 3 will be in the order of 3400 p.s.f. This value is generally considered to be somewhat too low for bridge design, where a pressure of at least 4000 p.s.f. usually is desired. The factor of safety reduces approximately to 2.5 if a bearing stress of 4000 p.s.f. is used. A slight overstress of this magnitude should not be detrimental particularly since the main cause of settlement will be the addition of fill on each side of the existing highway.

The installation of footings 2 feet lower or higher than this elevation should overcome this difficulty with weak clay. However, it should be appreciated that other weak zones may be encountered as the footing beds are opened up and therefore the stipulation of a specific bearing level is not warranted at this stage. The apparently sound condition of the existing bridge attests to the general competence of the subsoil in this area. If it is desired to remove any doubts regarding bearing conditions at all footing locations, additional shallow borings must be made.

According to the computations in Dwg. 12 -14, the estimated settlements under the north additions to the highway will reach values ranging from one inch at the north boundary of the existing bridge, to $3\frac{1}{4}$ inches at the north shoulder of the new fill. The same approximations made for the C.P.R. analysis apply here as well. It is felt that the actual settlements will be smaller than these values. The settlements along the south side of the highway will be smaller still.

These estimates allow for the effect of the fill applied on the opposite side of the roadway. Since Wendell Ave. is only about 40 feet wide, and there is no skew to the bridge, the loading from the new fill becomes effective under the opposite side of the road at a shallower level than is the case for the C.P.R. bridge. However, since the fill heights are the same, essentially, this transfer of stress at depth merely produces a general subsidence which will not be detrimental either to the existing or proposed construction.

The retaining walls, if any are required, will be very small at this site. Consequently, the earth pressures exerted against them will not be a serious design consideration. For the reasons given for the C.P.R. crossing, an earth pressure coefficient, $k = 1/3$, is recommended wherever embankment fill presses against retaining walls. As before, the earth pressure against the abutments will be in the order of $k_0 = 0.5$.

EXIT STRUCTURE 12.SGENERAL

This bridge will serve to transfer traffic from Highway 401 to Highway 400. It will overpass the south collector lines carrying east bound traffic out to Highway 401. It is understood that it will be a three span, sharply-skewed structure. Its exact location has not been finalized at the time of this study.

Two borings, designated holes 3 and 4, were put down at the location shown in Dwg. 10 for this bridge. The ground level here corresponds closely to the surface of the highway. Between the highway and this bridge site, a shallow drainage ditch, about 6 feet deep, has been excavated as part of the original Highway 401 construction. The estimated height of fill required for this new bridge is 20 feet.

SUBSIL

Borehole 3, at this bridge site, was terminated at a depth of 52 feet; borehole 4 was continued to 102 feet. The subsurface stratigraphy is similar to the conditions described for the C.P.R. and Wendell Ave. crossings, immediately to the west, although clay till appears to be less stiff, particularly between depths of 10 and 25 feet.

Consolidation tests were performed on samples of more plastic clay from depths of 15 feet in hole 4 and 40 feet in hole 3. The results of these tests have been used for settlement calculations at all bridge sites and therefore they are considered to be conservative for the C.P.R. and Wendell Ave. projects. These results are recorded on Dwg. 6 and 7. They indicate that the soil is heavily overconsolidated and therefore that the embankment loadings will merely involve a recompression of the soil. The recompression curves, although determined by small incremental loadings, probably are not entirely representative of in situ conditions, since it was necessary to patch the samples to some extent when small pebbles were removed. A less compressible condition undoubtedly applies.

GROUND WATER

Ground water observations were made in uncased borehole 4, which was taken to a depth of 102 feet, and in the piezometer installed at 30 feet in hole 3. Hole 4 terminated in the same sand stratum noted in hole 2, and which possibly extends to hole 1 at the C.P.R. bridge. The same dry condition was noted in this hole for a considerable period of time.

Eventually the deeper level of the hole must have become plugged by clay sloughing from the sides of the hole, since the water level in the hole eventually began to rise.

The stabilized water level, recorded in the piezometer in hole 3, was 26 feet. This hole, of course, was terminated in impermeable clay at 50 feet. Considering the flat poorly drained character of the land, this level appears to be low. It is felt that the water table must correspond to the depth where the soil changes from brown to grey in colour, which is about 10 feet below the surface. As at the other test locations, the deep-seated sand stratum is considered to be draining this clay to some extent and therefore the hydrostatic gradient below the water table will not show an uninterrupted linear increase with depth.

FOUNDATIONS

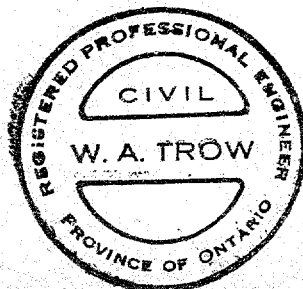
As at the other bridge locations, no embankment stability problem appears to exist at this bridge crossing. The lowest undrained shear strength of the clay is 1000 p.s.f., which value is more than sufficient to support the weight of 20 feet of fill safely.

The footings for the multi-span bridge can be founded in the upper clay crust material at just sufficient depth for frost protection. According to the undrained shear strength results recorded in Dwg. 4 and 5, the safe net bearing value for footings founded at a depth of 5 feet, or Elev. 424 ft., is 8000 p.s.f. Since the shear strength decreases sharply with depth, particularly in the hole 3 location, it is important that the footings be maintained at this level. Provided that the embankment fill is well compacted and sloped toward the collector road, there should be no objection to the support of the abutment footings directly on the fill. These footings will settle under the weight of the fill regardless of of the support level.

According to calculations made at the heavily loaded obtuse corner of the skewed embankment fill, the maximum settlement under 20 feet of fill is determined to be 5.8 inches, - 45% of which is estimated to be of an elastic nature. The computed total settlements at the acute corner, and at the adjacent pier location, were determined to be 4.4 and 1.9 inches, respectively.

As stated previously, these computations are considered to represent overestimates of the magnitude of movements. Even with these movements, the differential settlement across the structure is not too severe, particularly if a simple span is used between the abutment and first pier. By placing the fill in advance of bridge construction at least 50% of this total and differential settlement will be eliminated.

WAT/lt
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W. Trow
William A. Trow (P. Eng.)

APPENDIX IFIELD WORK AND GENERAL SUBSOIL

A total of 4 borings were made for this project at the locations shown on Dwg. 1A to 1C. Three of these borings were taken to 100 feet and a fourth, hole 3 at Bridge 12 S, was terminated at 50 feet. Continuous flight auger equipment was used to make the borings and consequently they were uncased to full depth.

Samples, recovered either with a 2-inch O.D. split spoon or with 2 and 3 inch shelly tubes, were obtained at five foot intervals of depth. Below a depth of approximately 70 feet, the sampling intervals were increased to 10 feet and, in the case of hole 1, samples were recovered from the auger only. Field vane measurements were made just below each sample interval, whenever the soil was found to be soft enough for the performance of this test.

Water level observations were recorded in each hole during and after completion of each boring. Piezometers were installed in holes 1 and 3 in order to permit long term observations. The measurements in these installations were found to be not in accord with those anticipated, having consideration for the plastic and saturated condition of the soil and the generally unsatisfactory drainage conditions in the entire area. It is concluded that a deep-seated stratum of sand is draining the overlying clay deposits. Drainage of the sand either is in a westerly direction toward the Humber River, or it is directed toward a deeply incised channel into bedrock located farther to the east under Hwy. 400.

The elevations of all borings were related to bench marks No. T-201 and T-200, located on the Weston Rd. crossing of Hwy. 401 and on the Hwy. 400 crossing of Hwy. 401, respectively.

Essentially similar subsoil conditions were encountered in each of the borings despite the fact that there is a difference of 22 feet between the C.P.R. crossing and the lower land in the vicinity of Bridge 12 S, and the general Hwy. 400-401 interchange complex. The top approximately 12 feet of soil comprise lacustrine stratified or laminated clay deposits which have been desiccated to a very stiff condition and to a brown colour. This clay probably was deposited in some glacial lake or pond prior to the emergence of Glacial Lake Iroquois about 10,000 to 12,000 years ago. Some pebbles, possibly the result of ice rafting, are present in the clay. The shoreline of Lake Iroquois has been established at Elev. 421 feet in the Jane St. - Wilson Ave. area. This is approximately 8 feet lower than the ground surface at Bridge 12 S.

The undrained shear strength and plasticity characteristics of this upper clay are indicated in Dwg. 2 to 5, the borehole logs for these three bridge sites. The estimated shear strength profile with depth at each bridge location is presented in Dwg. 1A to 1C, as part of the subsoil stratigraphy information.

The soil mass, underlying this lacustrine covering, consists of Wisconsin clay till material generally of low plasticity. This soil type continues at least to 102 feet, the maximum depth of the borings and was interrupted only by a deep-seated layer of sand which could be continuous and which probably drains to a buried valley in the bedrock located under and just to the east of this highway location. According to scanty records for this area, the valley in the bedrock extends down to Elev. 150 feet, which is almost 100 feet below the level of Lake Ontario.

The till contains numerous sand and gravel sizes and is characterized by a very low plasticity, low moisture content and high unit weight. It is very similar to the lean clay tills found in the height of land which rises to the east of Jane St. and Black Creek. It is interbedded with pockets of more plastic clay which contains very few gravel sizes and which is slightly laminated or stratified.

LABORATORY TESTING

Several routine laboratory tests, such as moisture content, density, Atterberg Limits and undrained shear strength, were performed on representative samples of the lacustrine and till clays in order to assist in the appraisal of the existing strength and compressibility of these soils. The undrained triaxial tests were performed on clay which was too stiff for field vane tests, or on samples of more gravelly clay which would provide unreliable field vane strength measurements. The estimated shear strength profiles shown on Dwg. 1, were prepared, keeping the limitations of both tests in mind. These interpreted strength relationships were used in the determination of the safe net bearing value of the soil for each structure.

In order to obtain an indication of the settlement to be anticipated at the three bridge locations, consolidation tests were performed on two samples of more plastic clay obtained from holes 3 and 4 at Bridge 12 S. Sample tubes 2-7/8 inches inside diameter were used to obtain these consolidation specimens. Unfortunately, the large samples recovered at depths of 30 to 40 feet in hole 4 were found to contain numerous gravel sizes and after considerable time was spent in an effort to recover a representative specimen, these tubes were finally abandoned. Consequently, one test was made on a 2 1/2 inch specimen, taken at approximately 14 feet in hole 4 and the other was made on a 1.9 inch sample, recovered from a depth of 40 feet in hole 3. Both materials consist of more cohesive soil relatively free of stones, although some patching of stone voids on the sides and bearing surfaces of the specimens had to be carried out during the trimming process.

Even though the samples would be very slightly disturbed, because of these patching operations, it was decided to apply load in very small increments in order to define the pre-consolidation range of the soil as accurately as possible. Above the estimated preconsolidation range, load was applied in normal increments. By these procedures, it is felt that the virgin curve and the recompression characteristics of the clay were defined in a reasonably satisfactory manner.

The results of these tests are shown on Dwg. 6 and 7. Little difference can be detected between the results for the 1.9 and the 2.5 inch diameter sample. It is noted that the clay is heavily overconsolidated and consequently that all embankment loads will merely produce a slight recompression of the soil. Modulus of compressibility values equal to .006 and .003 sq.ft./kip are computed for these tests. These values have been arbitrarily reduced in the settlement calculations of Dwg. 12 to 14 in order to account for the small disturbance during sample preparation. Values of 0.004 sq.ft./kip above 20 feet and 0.003 sq.ft./kip at greater depths have been assigned to the soil.

In order to obtain some indication of elastic settlement, three samples of clay till from various depths were isotropically consolidated in a triaxial chamber under a cell pressure having a magnitude slightly less than the appropriate overburden pressure. They, then, were subjected to undrained triaxial compression in which load, well below the anticipated failure value, was applied and removed in two cycles before the samples were taken to failure. The initial straight line relationships between stress and strain in the second and third cycles were used to compute the elastic modulus of the soil. In one instance this method was used also in order to remove the assumed disturbance from a sample and thereby to obtain a more representative indication of the in situ undrained shear strength. This work was carried out on a sample from 14 feet in hole 1. The original test on this Shelby tube sample produced a stress strain relationship and an ultimate strength well below the anticipated capacity. By this additional processing, a more representative shear strength was obtained even though the moisture contents of all samples were essentially the same.

A record of these tests is presented on Dwg. 11.

APPENDIX IISETTLEMENT COMPUTATIONS

The following assumptions, allowances and approximations were made in the estimation of the magnitude of settlement to be expected at the three subject highway bridges:

A - Conservative Factors

1) The calculations were based upon the results of consolidation tests performed on samples from more plastic material in holes 3 and 4. Although some slight allowance was made for possible disturbance of these samples, the assumed compressibility values may be high for the less plastic levels of holes 3 and 4, and for the consolidated conditions prevailing in the vicinity of the Wendell Ave. and the C.P.R. bridges.

2) No allowance was made for a spread-out of load through the upper desiccated clay crust present at all locations. This spread out effect of this crust would be most marked in the Wendell Ave. and C.P.R. bridge locations since the base width in contact with the original ground is not very great and therefore a greater spread of load should be expected. In addition, no allowance has been made for a spread of load of the new fill through the existing embankments at these sites. Allowance for this factor must certainly reduce the settlement estimates for the locations immediately adjacent to the existing bridges.

3) A Boussinesq mode of pressure distribution into the soil has been assumed. It is thought by many investigators that the pressures fan out to a greater degree than is assumed in the Boussinesq equations. The computations of stress at various depth were computed using the graphical Newmark Method. Allowance was made for the presence of embankment fill on the opposite side of the road or railway in most of these computations.

B - Factors Neglected

1) Compression of soil only to a depth of 100 feet was considered. Soil pressures of considerable magnitude still prevail at and below this depth. Consequently, additional elastic and consolidation settlement should be expected from these deep seated levels. However, at a depth of 100 feet the stresses from the two opposite embankments have essentially overlapped and blended and therefore any additional settlement merely takes the form of a general subsidence of the entire roadway.

2) The settlement produced by the weight of the bridge structure has been ignored. Using the net bearing pressures recommended in the report and considering the rigid nature of the abutment footings, the additional settlement from the bridge weight should be much less than 1 inch and it should be uniform over the entire length of each footing.

After weighing all of these qualifications, and having regard for the

results of recent records of embankment fill settlement, it is concluded that the computed results of this study will represent an overestimate of actual settlements.

The summarized settlement calculations for the most heavily loaded sections of each bridge site are presented on Dwgs. 12 to 14.

No attempt has been made to estimate the duration of settlements following this proposed embankment construction. Since the subsoil is heavily overconsolidated, the rate of settlement should be much more rapid than the results of the consolidation tests would suggest.

TABLE NO. 1

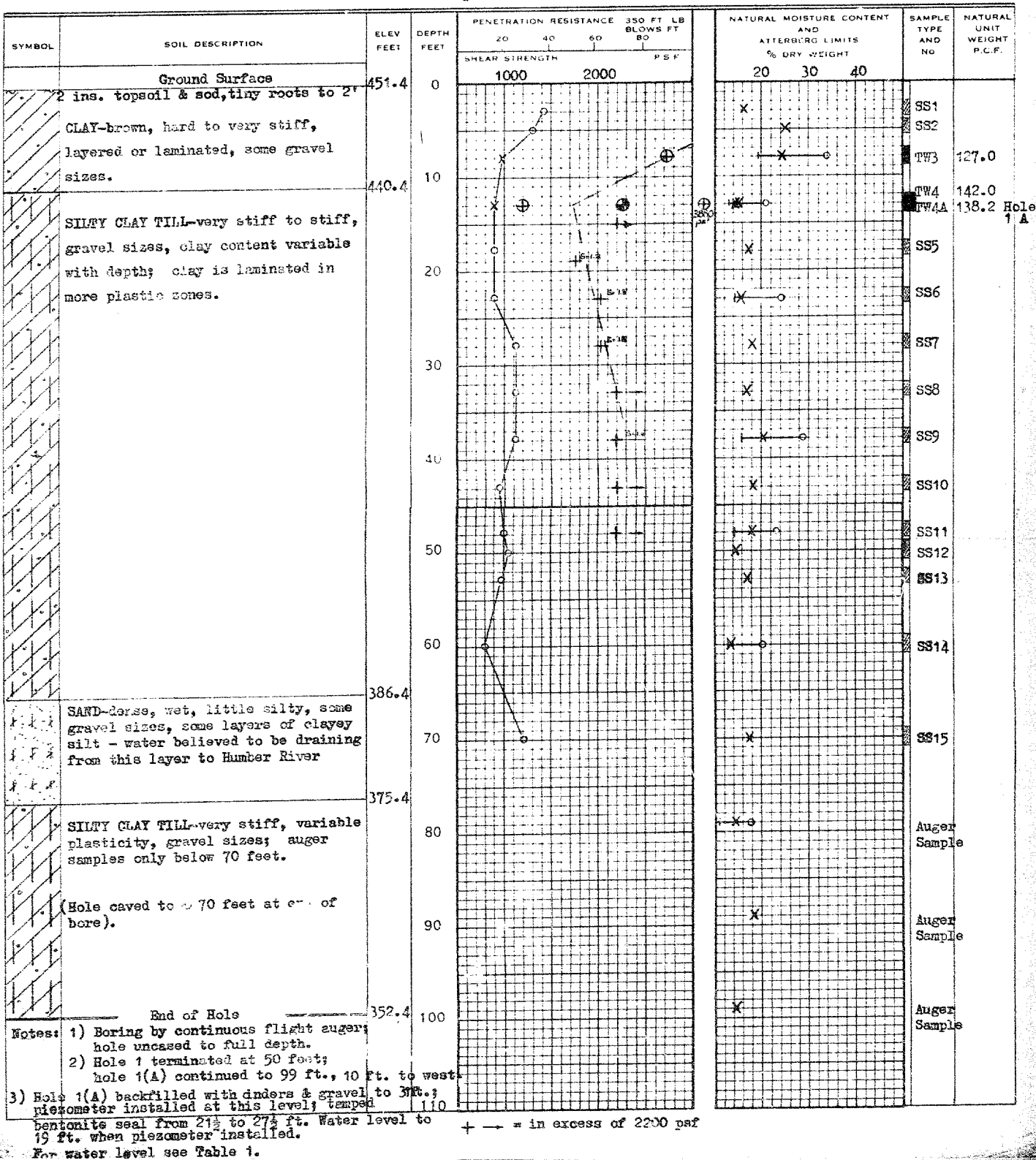
SUMMARY OF WATER LEVEL MEASUREMENTS--HOLES 1-4

Date	Hole 1 Depth: 52 ft.	Hole 1A 99 ft. (Piez. at 30')	Hole 2 99 ft.	Hole 3 49 ft. (Piez. at 49')	Hole 4 100 ft.
Sept.					
12	Open and dry to 52 ft. on completion.				
13	do		Wet sand at 85 ft.		
14	Blocked and dry at 40 ft.		W.L. about 50 ft. at end of augering.	Hole dry after augering to 49 ft.	
17				Water poured into piezometer for 2 hrs. after installation. W.L. after 2 hrs. = 24.7 ft.	
18	Wet at 32.5		W.L. = 52.5 ft.	Water poured in intermittently all day. Final WL = 12.7.	
19				21 ft.	W.L. 75 ft. on completion.
20		Tube wet at 12' after augering to this level.		Water poured intermittently to 1 pm. WL at 1 pm = 12 ft. " 5 pm = 19.5 ft.	
21	Wet at 28 ft.	Hole at 42'. W.L. overnight at 5.1 ft. Hole caved at 70' when augering deeper. Water at 19' at end of bore. Piez. dry after installation.	W.L. 52.5 ft.		
28		Piezometer dry	Hole dry and blocked at 41 ft.	24.1 ft.	Dry to 50 ft.
Oct.					
4	Wet at 5.9 ft.	Piez. dry; filled with water, but dry in 8 min.	Water lev. - 17.3'	24.7 ft.	Wet blockage at 26 ft.
9	Wet at 1.8 ft. Open to 19 ft.	Piezometer dry.	W.L. at 2 ft. Hole open to 23'.	26.0 ft.	Wet blockage at 25 ft.

BORERHOLE NO. 1
PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
LOCATION C.P.R. West of Wendell Avenue
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 451.4 ft.
DATUM B.M. No. T201 Weston Road Bridge over Hw. 401
= El 431.255

PENETRATION RESISTANCE
2 O.D. SPLIT TUBE
2 I.D. SHELBY TUBE
2 DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S₁)
Consolidated cyclic
undrained triaxial test

NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX
X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2 O.D. SPLIT TUBE
2 I.D. SHELBY TUBE
3 O.D. SHELBY TUBE



BOREHOLE NO. 2
 PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 444.4 ft.
 DATUM See Hole 1.

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—
 2" I.D. SHELBY TUBE —x—x—x—
 2" DIA. CONE —+—+—+—

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
 UNCONFINED COMPRESSION ⊗
 VANE TEST AND SENSITIVITY (S) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

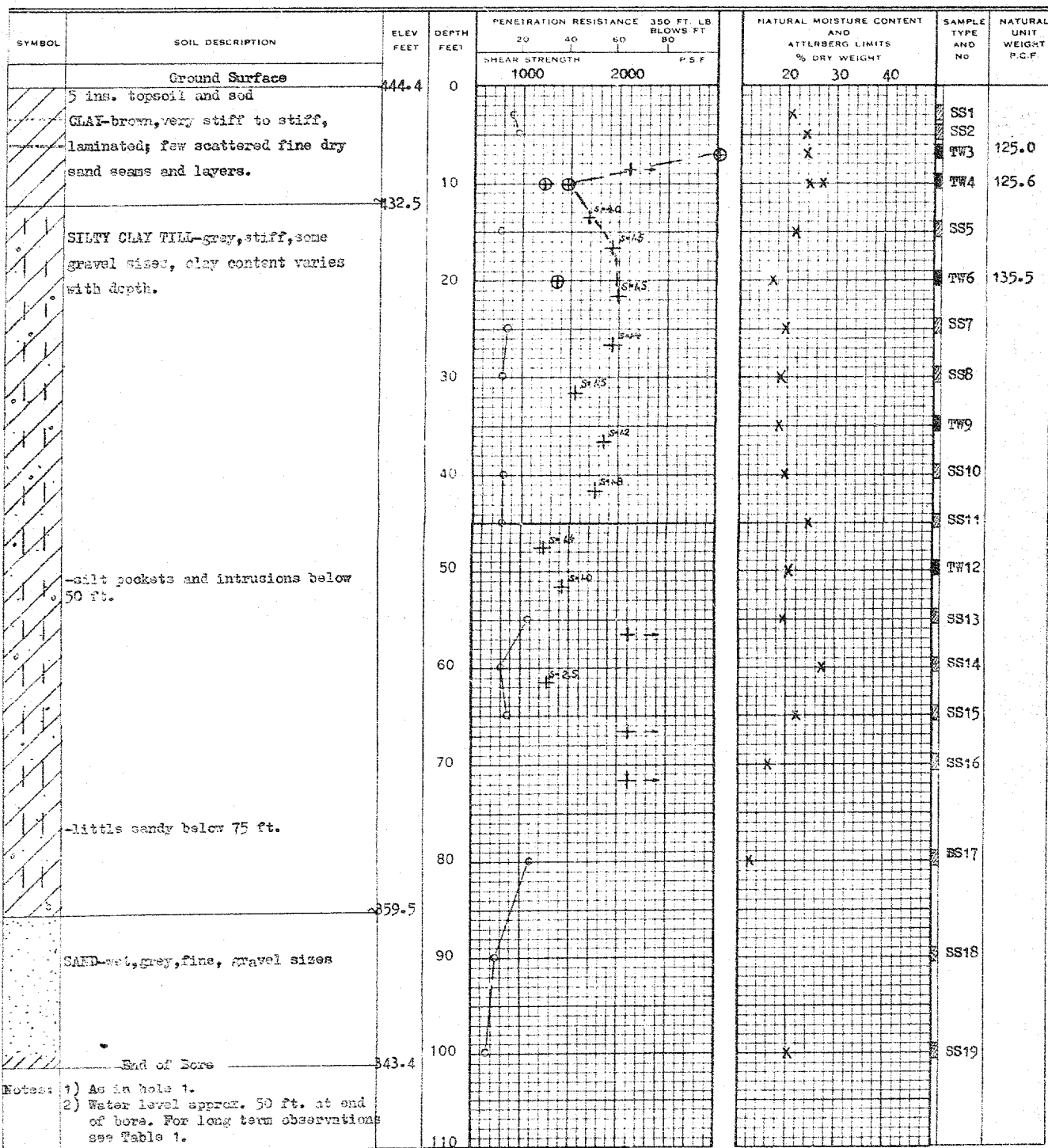
ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT —+—

SAMPLE TYPE

2" O.D. SPLIT TUBE —○—
 2" I.D. SHELBY TUBE —x—
 3" O.D. SHELBY TUBE —+—



LEGEND

BOREHOLE NO. 3
 PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue, Bridge 125
 HOLE LOCATION See Eng. 1.
 HOLE ELEVATION 429.9 ft.
 DATUM See Hole 1.

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE
 2" I.D. SHELBY TUBE
 2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL
 AT OVERBURDEN PRESSURE
 UNCONFINED COMPRESSION
 VANE TEST AND SENSITIVITY

Consolidated, cyclic,
 undrained triaxial test.

NATURAL MOISTURE CONTENT

AND LIQUIDITY INDEX

ATTEBERG LIMITS

LIQUID LIMIT

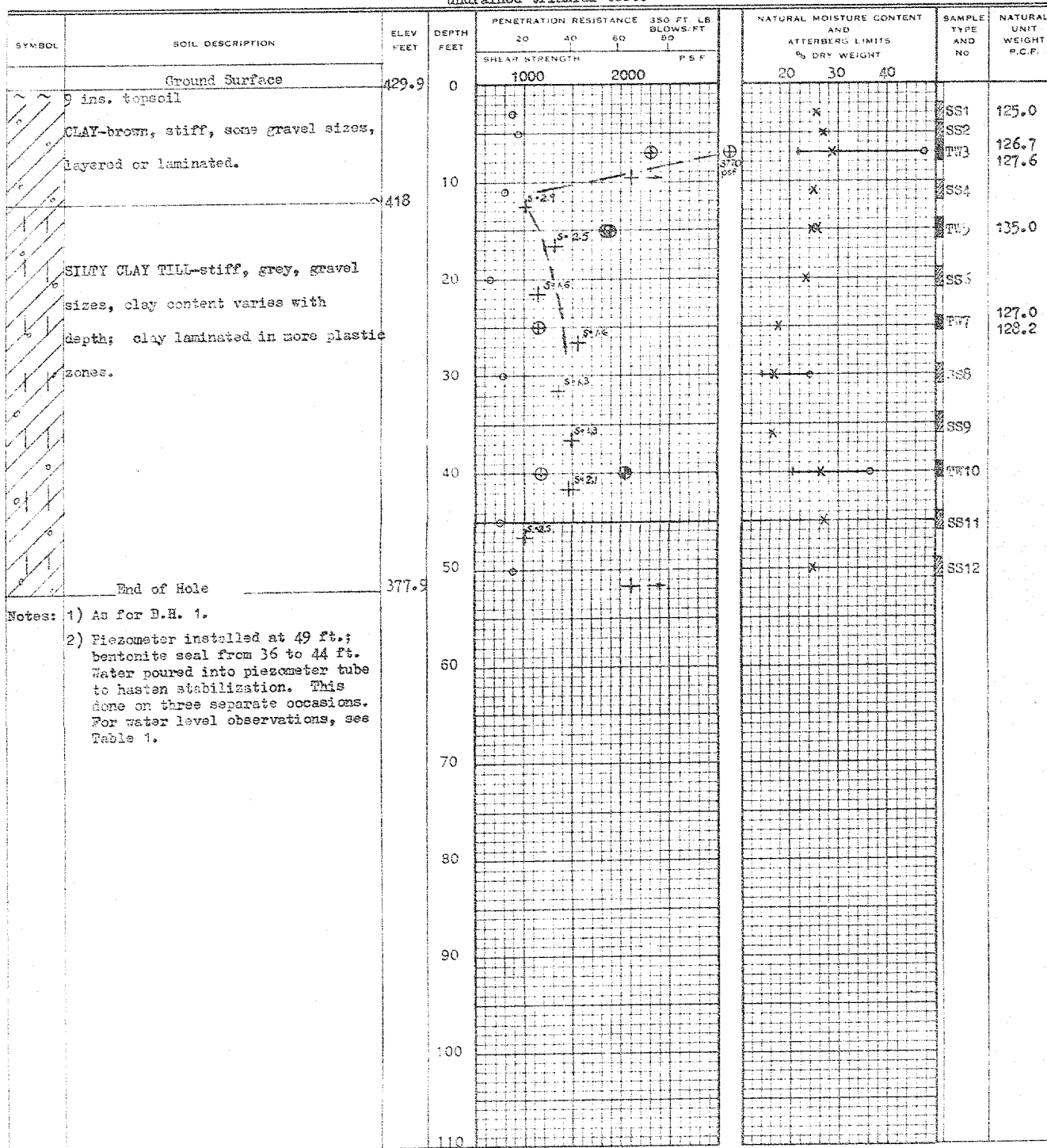
PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE



LEGEND

BOREHOLE NO. 4
PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
LOCATION C.P.R. West of Wandell Avenue
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 429.0 ft.
DATUM See Hole 1.

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE

UNCONFINED COMPRESSION

VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

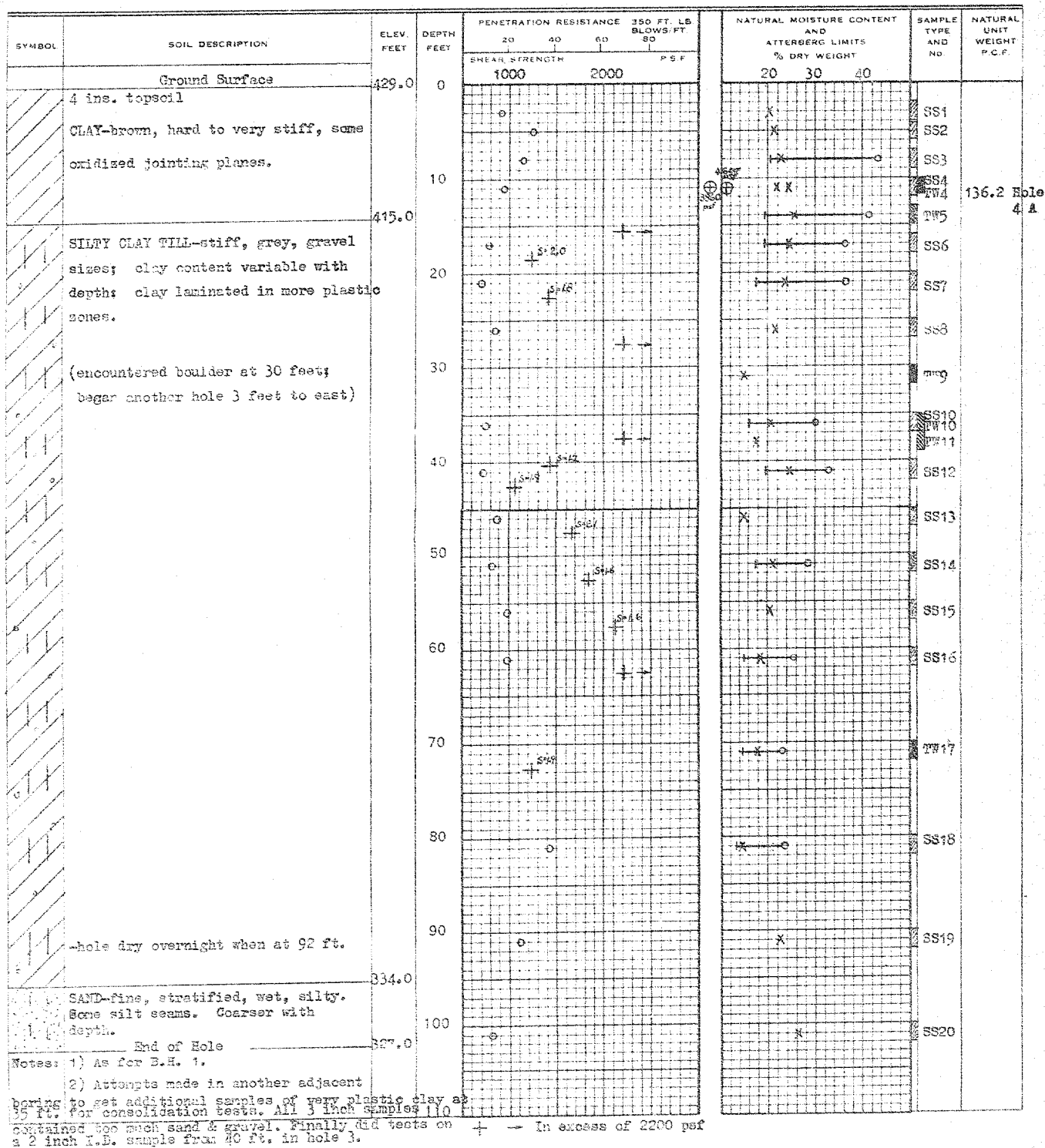
PLASTIC LIMIT

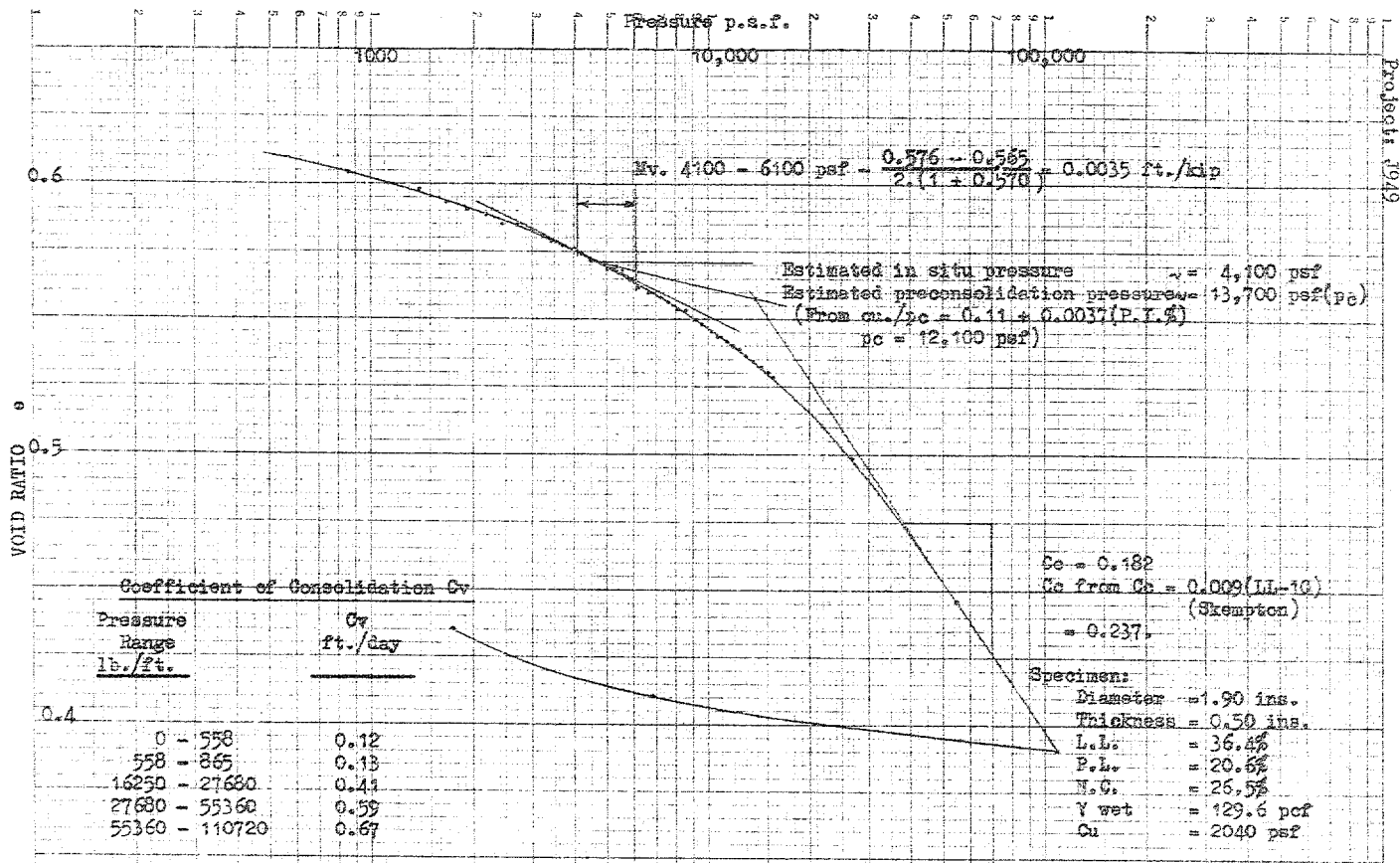
SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

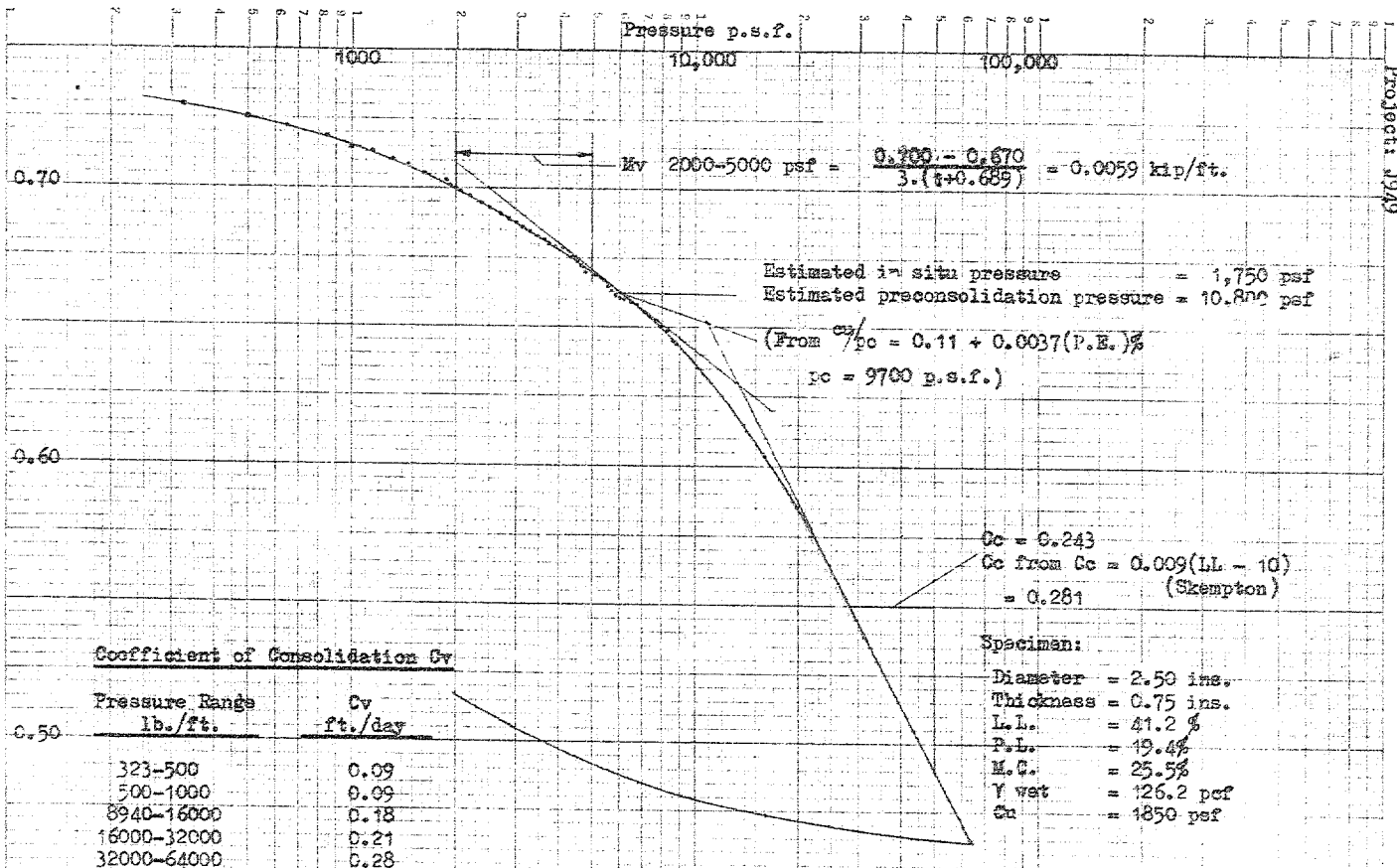
3" O.D. SHELBY TUBE





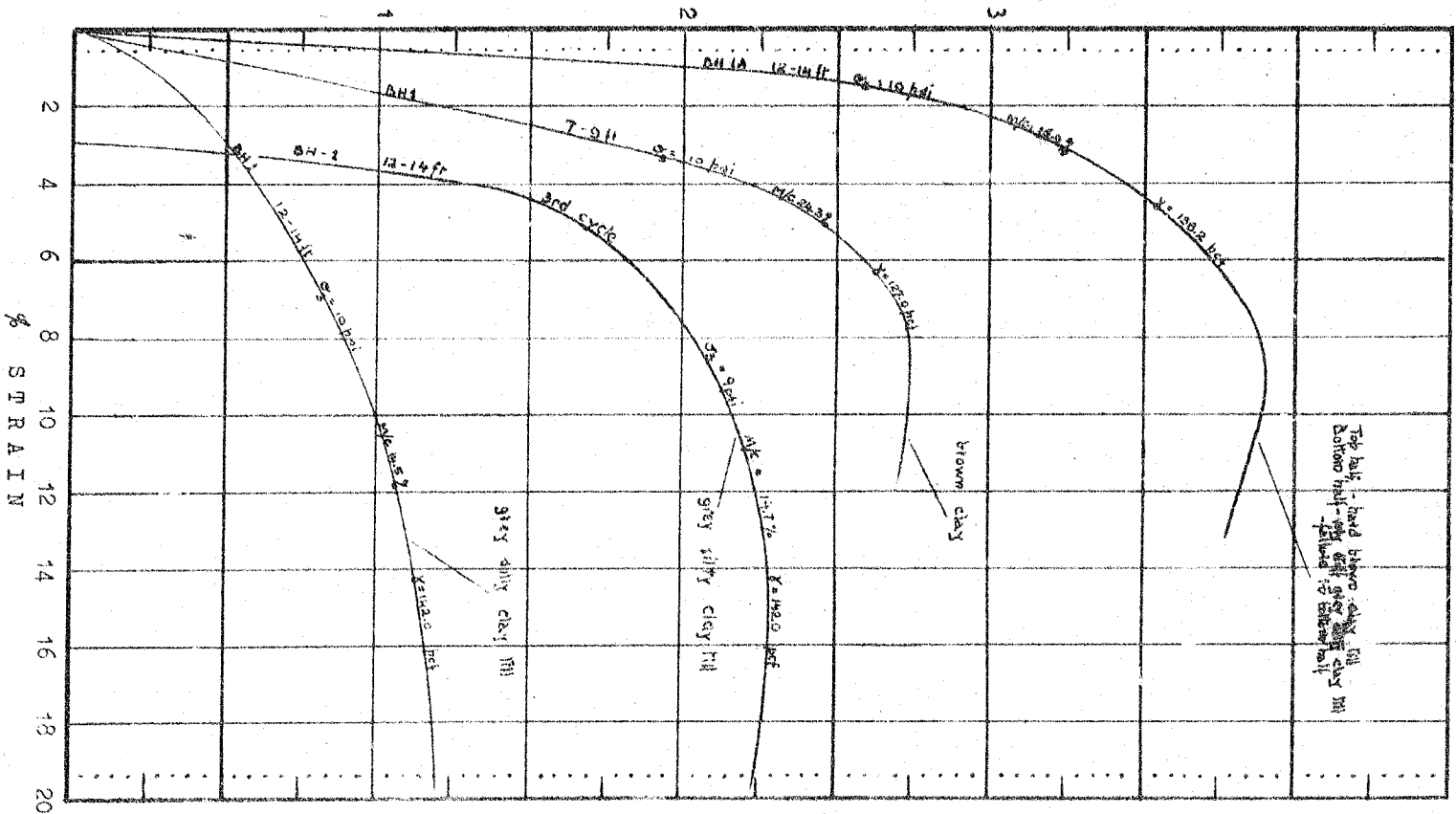
CONSOLIDATION TEST RESULTS - HOLE 3, DEPTH 39'-0" - 40'-8"

VOID RATIO e



CONSOLIDATION TEST RESULT - HOLE 4, DEPTH 13 - 15 FT.

SHEAR STRESS ksf

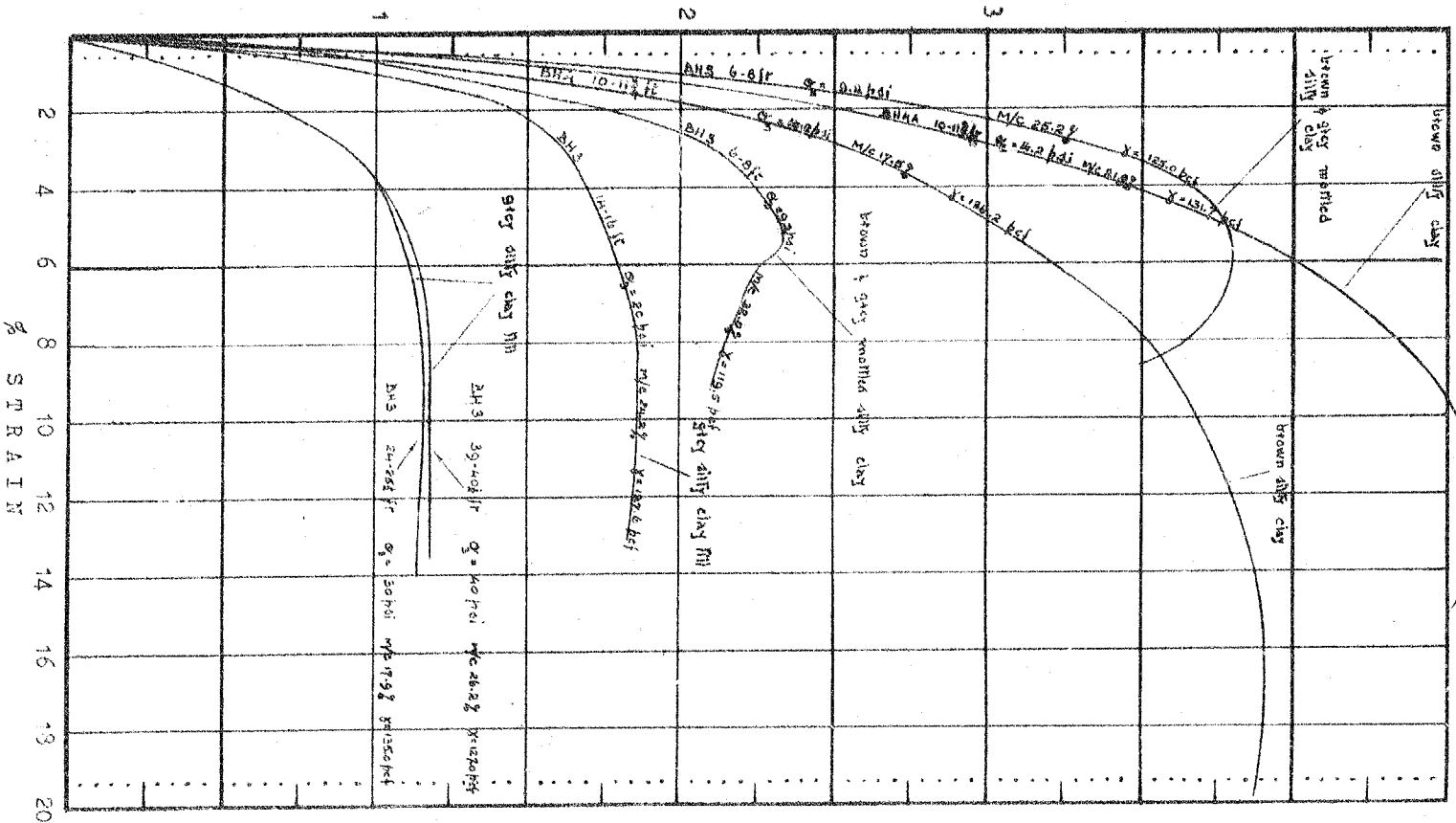


UNDRAINED TRIAXIAL TEST RESULTS, HOLE 1, C.P.R. CROSSINO

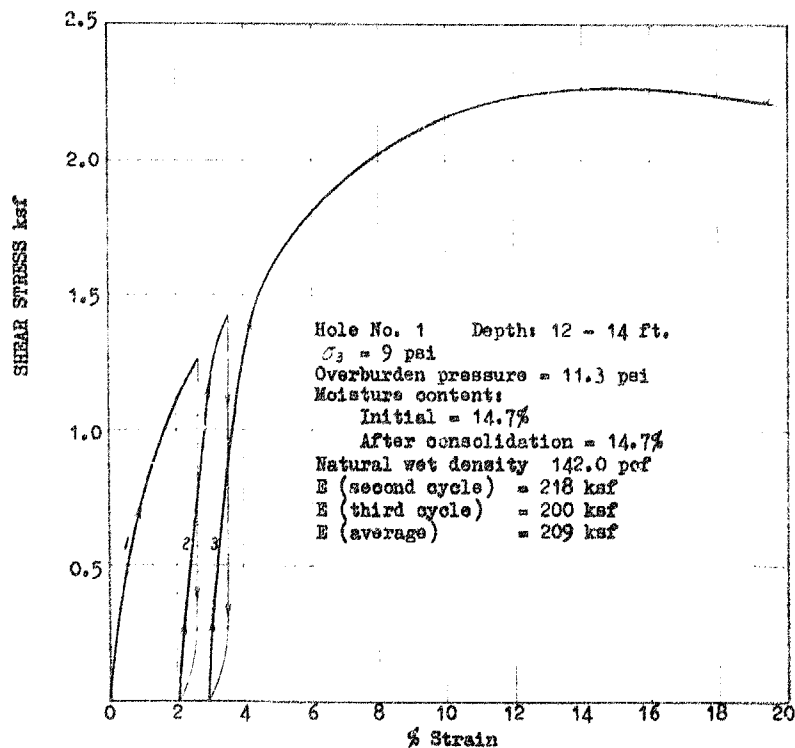
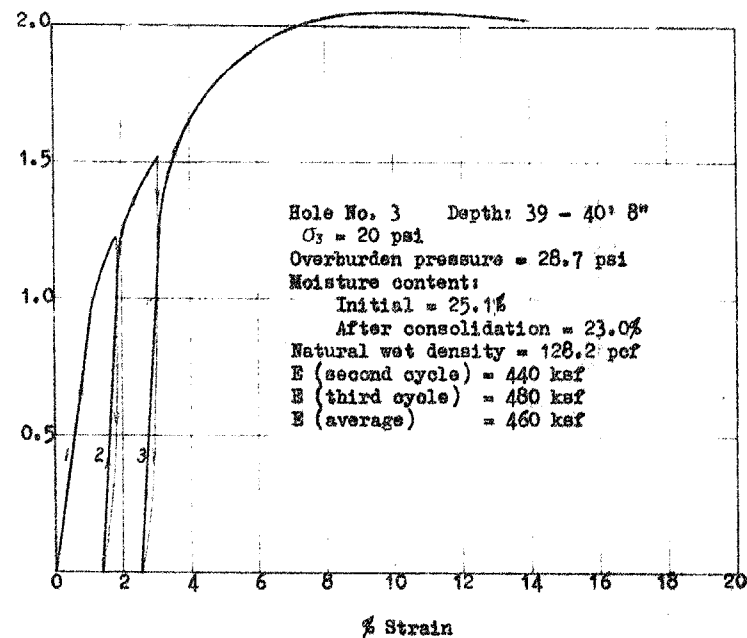
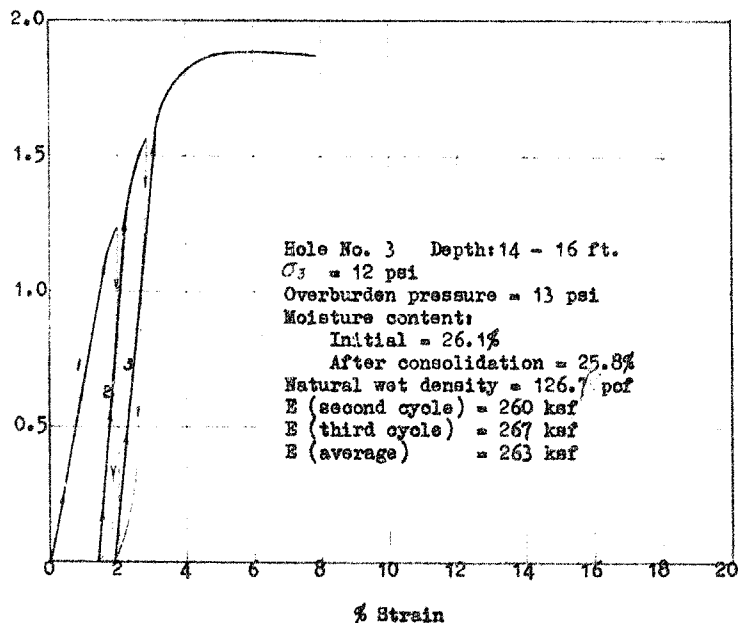


STRAIN

1



WILLIAM A. TROY AND ASSOCIATES



$$E = \frac{2(\Delta \text{ shear stress})}{\Delta \text{ strain}} \quad \text{over straight line portions of reloading curves}$$

CONSOLIDATED-QUICK UNDRAINED TRIAXIAL TESTS
 WITH CYCLIC LOADING

Depth Ft.	Assumed Soil Properties	Applied Stress From Fill Only ksf	Settlement			
			Under Point a		Under Point b	
			Si. ins.	Sc. ins.	Si. ins.	Si. ins.

10	Embankment weight $\gamma = 125$ pcf		Surface crust assumed incompressible			
20	Elastic modulus $E = 300$ ksf		0.11	0.16	0.47	0.51
30	Modulus of compressibility $M_v = 0.004$ sq.ft./kip above 26 feet depth		0.18	0.19	0.36	0.39
40	$M_v = 0.003$ sq.ft./kip below 26 feet depth					
50						
60	Immediate settlement = S_i $= 0.5 \cdot \frac{\Delta P \cdot \Delta h}{E}$		0.20	0.22	0.30	0.32
70	Consolidation settlement = S_c $= \sum \mu \cdot \Delta P \cdot \Delta h \cdot M_v$					
80	where $\mu = 0.6$ (overconsolidated soil).		0.21	0.12 (Stratum incl. sand layer).	0.27	0.14 (Stratum incl. sand layer).
90	Note: An allowance has been made for the influence of the fill on the N.E. abutment.					
100			0.21	0.22	0.26	0.28
106			0.91	0.91	1.66	1.64

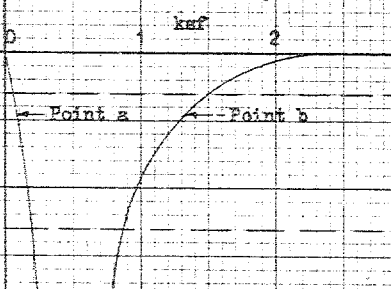
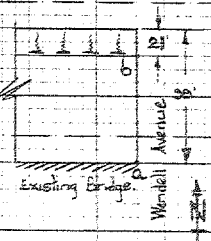
Total Settlements:		Point a = 1.82 ins.		Point b = 3.30 ins.	
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Project: 1949

DWG. 4

Project: 1949

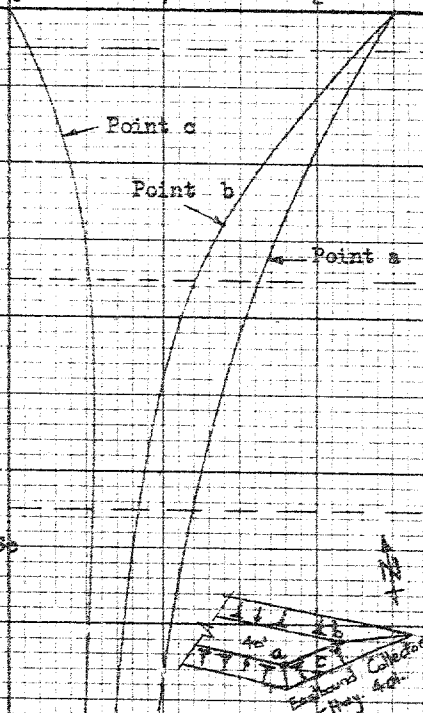
Page 12

Depth Ft.	Assumed Soil Properties	Applied Stress From Fill Only ksf	Settlement			
			Under Point a		Under Point b	
			Si. ins.	So. ins.	Si. ins.	So. ins.
			Surface crust assumed incompressible			
10	Embankment weight $\gamma = 125 \text{ pcf}$					
20	Elastic modulus $E = 300 \text{ ksf}$		0.05	0.08	0.41	0.60
30	Modulus of compressibility $M_v = 0.004 \text{ sq.ft./kip}$ above 26 feet depth		0.10	0.10	0.33	0.35
40	$M_v = 0.003 \text{ sq.ft./kip}$ below 26 feet depth					
50						
60	Immediate settlement = S_i		0.13	0.14	0.29	0.31
70	$= \sum 0.5 \frac{\Delta p \Delta h}{E}$					
80	Consolidation settlement = S_c		0.15	0.16	0.28	0.30
90	where $\mu = 0.6$ (overconsolidated soil).					
100	Note: An allowance has been made for the influence of the fill on the N.E. abutment.		0.16	0.05 (Stratum incl. sand layer)	0.27	0.09 (Stratum incl. sand layer)
106						
Totals:			0.59	0.53	1.58	1.65
Total Settlements:			Point a = 1.42 ins.		Point b = 3.23 ins.	

Project 1949

Dwg. 13

IMMEDIATE AND CONSOLIDATION SETTLEMENT COMPUTATIONS - WENDELL AVENUE BRIDGE, N.W. ABUTMENT

Depth Ft.	Assumed Soil Properties	Applied Stress From Fill Only ksf	Settlement					
			Under Point a		Under Point b		Under Point c	
			Si. ins.	So. ins.	Si. ins.	So. ins.	Si. ins.	So. ins.
Surface crust assumed incompressible								
10	Embankment weight $\gamma = 125 \text{ pcf}$							
20	Elastic modulus $E = 300 \text{ ksf}$		1.16	1.67	0.92	1.32	0.26	0.37
30	Modulus of compressibility $M_v = 0.004 \text{ sq.ft./kip}$ above 35 feet depth							
40	$M_v = 0.003 \text{ sq.ft./kip}$ below 35 feet depth							
50			0.83	0.89	0.60	0.65	0.31	0.34
60	Immediate settlement = S_i $= \sum 0.5 \frac{\Delta p \cdot \Delta h}{E}$							
70	Consolidation settlement = S_c $= \sum \mu \cdot \Delta p \cdot \Delta h \cdot M_v$							
80	where $\mu = 0.6$ (overconsolidated soil).		0.62	0.68	0.44	0.48	0.31	0.33
90	Note: An allowance has been made for the influence of the fill on the E. abutment.							
100		Totals:	2.61	3.24	1.96	2.45	0.88	1.04
Total Settlements:			Point a:	3.95	Point b:	4.41	Point c:	1.92

Project: 1949

Page 14