

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

DATE: December 4, 1968

OUR FILE REF.

IN REPLY TO DEC 11 1968

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
The Crossing of the C.N.R. Tracks
And Proposed East Side Highway
(Near Forkes Road)
Twp. of Humberstone, Co. of Welland
District No. 4 (Hamilton)
W.J. 68-F-73 -- W.P. 60-68-02

Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/MdeF

Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
H. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh

Foundations Files ✓
Gen. Files

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

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FOUNDATION INVESTIGATION REPORT

For

The Crossing of the C.N.R. Tracks
And Proposed East Side Highway
(Near Forkes Road)

Twp. of Humberstone, Co. of Welland
District No. 4 (Hamilton)

W.J. 68-F-73 -- W.P. 60-68-03

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the crossing of the C.N.R. tracks and the proposed East Side Highway in the Twp. of Humberstone, Co. of Welland. The request was contained in a memo from the Bridge Office (Mr. F. I. Hewson, Senior Bridge Liaison Engineer), dated September 23, 1968.

Subsequently, a foundation investigation was carried out at the proposed site to determine the subsoil and ground-water conditions.

This report contains the results of the investigation, together with recommendations pertaining to the foundations of the proposed structure, as well as the stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located some 200 ft. east of the intersection of Forkes Rd. and Kleinsmith Rd., approximately 2 miles east of Welland Junction. At this location the C.N.R. tracks, which run parallel to Forkes Rd., are about 100 ft. to the north. The tracks are elevated about 4 ft. above the surrounding ground level on a 25-ft. wide embankment. Forkes Rd. is a two-lane, paved County road; the profile grade of this road is about 1 to 2 ft. above the surrounding terrain. Shallow ditches run along both sides of Forkes Rd. as well as the C.N.R. embankment.

cont'd. /2 ...

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

The surrounding area is generally flat-lying; the surficial drainage is very poor. The land to the north of the site is used for farming purposes, while the land to the south, at the present time, is abandoned.

Physiographically, the site is situated in the region known as the Haldimand Clay Plain. In this area the subsoil consists of extensive, mainly glacial-lacustrine deposits, laid down in glacial Lake Warren during the Wisconsin age. These deposits are composed of stratified silts and clays, and are generally underlain by a basal glacial till sheet, which in turn, is followed by dolomitic limestone or shale bedrock. The bedrock is of the Salina formation of the Silurian period.

3. FIELD AND LABORATORY WORK:

A total of four sampled boreholes, three of which were accompanied by dynamic cone penetration tests, were carried out. B.H.'s #1, 2 and 3 were advanced to bedrock using a Penn drill employing power auger techniques. In B.H.'s #1 and 2, a diamond drill rig was set up over the pre-augered hole and bedrock was proven by BX size rock core samples. B.H. #4 was put down by the diamond drill rig, which was adapted for soil sampling purposes.

Samples, of the cohesive portion of the overburden, were recovered at required depths, where possible, in 2" and 3" I.D. Shelby tubes, which were pushed either manually or hydraulically into the soil. Elsewhere, samples were obtained in a 2" O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. Field vane tests were carried out to determine the undrained shear strength of the cohesive stratum.

The groundwater level conditions across the site were determined by installing sealed piezometers in two of the boreholes.

3. FIELD AND LABORATORY WORK: (cont'd.) ...

This information was supplemented by recording the groundwater level in the open holes at the remaining boring locations.

The locations and elevations of all the borings were surveyed in the field by personnel from the Central Region Engineering Surveys Section. This information is shown on Dwg. 68-F-73A, together with the estimated stratigraphical profile across the site. All elevations are referred to a Geodetic datum.

All samples were visually examined and identified in the field and subsequently in the laboratory. Following this inspection, laboratory tests were carried out on selected representative samples to determine the physical properties of the subsoil, namely:

- Bulk Densities
- Natural Moisture Contents
- Atterberg Limits
- Grain-Size Distributions
- Undrained Shear Strengths
- Consolidation Characteristics

On completion of these tests, the various soil samples were classified as to type and consistency in accordance with the Unified Soil Classification System (Oct. 1963).

The results of the laboratory testing are plotted on the Record of Borelog sheets and summarized in Appendix I of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a firm to hard silty clay to clay with traces of sand and gravel; this deposit is about 73 to 82 feet thick. This stratum is underlain by a 1 to 7 ft. thick deposit of hard (or very dense) glacial

cont'd. /4 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

till, followed by dolomite bedrock.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying borehole sheets.

The stratigraphical profile across the site, inferred from this data, is shown on Drawing 68-F-73A.

From ground surface downwards, the various soil types encountered are described as follows:

4.2) Silty Clay to Clay with Traces of Sand and Gravel:

In general, the surficial cover across the site is composed of up to 6 inches of clayey topsoil. In B.H. #3, put down on the edge of Forkes Rd., a 4.5 ft. surficial layer of a very stiff clayey silt roadway fill was encountered.

Immediately below the fill or topsoil is the predominant overburden stratum across the site, composed of a reddish-brown (with occasional grey seams) silty clay to clay with traces of sand. Gravel sizes were scattered at random throughout the deposit. The overall thickness of the silty clay to clay stratum ranges from 73 to 82 feet. The upper 15 to 20 feet of the stratum is characteristically brown in colour; it is considered that this zone has been desiccated. In all the borings occasional grey silt partings and seams, varying from a fraction of an inch to up to 3 inches in thickness, were encountered throughout. Random pockets of gypsum crystals were observed in many of these seams above elevation 554. In B.H. #2 thin sand seams up to 1/4" thick were observed below elev. 510. Grain-size distribution curves, carried out on samples of the stratum, are appended to this report.

cont'd. /5 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Silty Clay to Clay with Traces of Sand and Gravel:

(cont'd.) ...

The physical properties of the overall stratum, as determined by field and laboratory testing, are summarized on Figure 1; a brief resumé follows:

| | <u>Desiccated Crust</u> <u>Range (Average)</u> | <u>Lower Zone</u> <u>Range (Average)</u> |
|--|---|---|
| Liquid Limit (W_L) (%) | 39 - 64 (51) | 32 - 64 (49) |
| Plastic Limit (W_P) (%) | 21 - 29 (24) | 15 - 30 (23) |
| Natural Moisture Content (W) (%) | 25 - 33 (27) | 24 - 49 (37) |
| Bulk Density (γ) (p.c.f.) | 114 - 127 (122) | 108 - 127 (120) |
| Initial Void Ratio (e_0) | - | 0.68 - 1.09 |
| Compression Index (C_c) | - | 0.28 - 0.50 |
| Undrained Shear Strength (C_u) (p.s.f.) | | |
| i) Field Vanes | >2,000 | 1,300 - >2,000 |
| ii) Lab. Vanes | 1,200 - 1,700 | 1,100 - 1,700 |
| iii) Lab. Testing | 1,500 - 2,650 | 900 - 1,600 |
| Sensitivity | 2 - 7 | 2 - 5 |
| 'N' Values (Blows/ft.) | 13 - 76 | 7 - 19 |

The Atterberg Limit tests, summarized above, are also plotted on the Plasticity Chart, Fig. #2. These results indicate that the stratum is inorganic and of intermediate to high plasticity. The liquidity index of the upper desiccated zone is typically between 0.1 to 0.3, while the index of the remaining portion of the stratum is generally between 0.2 to 0.6.

cont'd. /6 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Silty Clay to Clay with Traces of Sand and Gravel:

(cont'd.) ...

The consistency of the overall stratum, as determined from the undrained shear strength testing, varies from hard to very stiff in the upper 20 to 25 feet (desiccated zone), decreasing to very stiff to stiff with depth.

The consolidation characteristics of the stratum were determined by carrying out three laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure Plots, on Figure #5. The results of this testing indicate that the main body of the clay is preconsolidated by about 2 to 4 t.s.f. in excess of the existing overburden pressure. It is estimated that the upper 20 to 25 feet of the stratum (desiccated crust) is preconsolidated by something in excess of 5 t.s.f.

4.3) Clayey Silt with Sand and Gravel (Glacial Till):

This heterogeneous, but generally cohesive deposit, was encountered immediately below the silty clay to clay stratum between elevations 504 and 500. The thickness of the glacial till varies from 1 to 6 ft. In general, the deposit is composed of a brown to grey clayey silt with sand and gravel. In B.H. #1, however, the deposit is basically non-cohesive, being composed of a sand with gravel and fragments of bedrock. In B.H. #2, a layer (6") of white gypsum was encountered just above the bedrock. A grain-size distribution curve carried out on a representative sample of the deposit is shown on Figure #4 in the Appendix of this report.

The Atterberg Limit tests, carried out on representative samples of the glacial till, are shown on the Plasticity Chart, Figure #2. These results indicate that the liquid limit and plastic limit are, on the average, about 18% and 11%, respectively. The corresponding natural water content is generally at or below the plastic limit. Based on these results, it is estimated that the matrix of the glacial till is of low plasticity.

cont'd. /? ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Clayey Silt with Sand and Gravel (Glacial Till):
(cont'd.) ...

The standard penetration resistance or 'N' values vary from 29 to well over 100 blows per foot, indicating that the consistency of the cohesive deposit is very stiff to hard.

4.4) Dolomite Bedrock:

Bedrock was established in B.H.'s #1 and 2 by obtaining 5 ft. of BX size rock core. In the other borings bedrock was inferred to exist at the location where the split-spoon sampler met practical refusal. The depth at which bedrock was encountered ranged from elevation 497 to 500 - i.e., some 79 to 85 ft. below existing ground surface.

The bedrock is generally composed of a grey dolomite with numerous gypsum lenses throughout. In B.H. #1 the bedrock was interbedded with dark grey calcareous shale, while in B.H. #2 a white gypsum bed, some 2 ft. thick, was encountered just below the bedrock surface. The bedrock is generally sound; occasional horizontal fractures are present, however, in the upper few feet.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out during the period of the investigation in 1) sealed piezometers installed in boreholes #1 and 2, and 11) the open holes at the remaining boring locations. These observations are recorded on the Borelog sheets and summarized on Drawings 68-F-73A. The results of the measurements indicate that, at the time of the investigation, the piezometric groundwater level within the glacial till deposit ranged from elevation 554 to 558 - i.e., some 25 feet below ground level. The groundwater level within the overlying silty clay to clay stratum ranged from elevation 576 to 579 - i.e., some 3 to 5 feet below ground level.

Previous subsurface investigations in the immediate vicinity indicated that an artesian pressure existed in the lower

5. GROUNDWATER CONDITIONS: (cont'd.) ...

glacial till deposit and the upper fractured zone of the bedrock. As discussed in the foregoing paragraph, this is not now the case at this site, since the piezometric groundwater level in the basal till deposit is at a much lower level than that in the overlying cohesive stratum. This change in condition is probably caused by the excavation for the realigned Welland Canal presently underway at a location some 2 miles to the west. At this excavation site, a dewatering scheme is being employed to lower the piezometric groundwater level within the confined aquifer composed of the glacial till and upper zone of the bedrock. The effects of the dewatering in the vicinity were observed at three farm wells, located adjacent to the site; these wells extend into the glacial till. Prior to dewatering, the water level in the wells was at or slightly below ground surface; however, once the dewatering was put into effect it was lowered below the intake elevation of the pump.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct an overhead structure to carry the proposed East Side Hwy. over the Canadian National Railway's track. Tentative proposals call for a three-span (40'-40'-40') structure with approach fills having a maximum height of about 30 ft. above surrounding ground level.

Subsoil at the site consists generally of an extensive stratum of silty clay to clay with traces of sand and gravel, followed by a relatively thin glacial till deposit, composed primarily of clayey silt with sand and gravel. The overburden is underlain, at a depth of 79 to 85 feet below existing ground surface, by sound dolomite bedrock.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Foundations:

6.2.1) Pier Foundations:

The upper very stiff to hard zone of the cohesive stratum (desiccated crust) is a competent foundation subsoil. Based on this, it is recommended that the piers be founded on spread footings founded in this upper zone. A minimum soil cover of 4 feet should be provided to satisfy the frost protection requirements in the area. Spread footings so founded, may be designed for a safe bearing pressure of 2.5 t.s.f. Settlement of the foundation subsoil at and below the footing level will occur due to the induced footing pressure and to a small extent, by the adjacent embankment loading; the magnitude of these settlements is discussed in Sub-section 6.3.

No major dewatering problems are anticipated during construction of the footings, in view of the relatively impermeable nature of the subsoil. Care should be taken to prevent softening of the subsoil at the footing levels due to minor groundwater seepage or surface run-off. In this regard it is recommended that the foundation base be protected by pouring a mat of lean concrete as soon as subgrade level is reached.

6.2.2) Abutment Foundations:

The abutments of the proposed structure could be constructed, within the approach fills, on spread footings. The fill material, below the spread footings, should consist of well compacted G.B.C. 'A' material and should extend for a horizontal distance of at least 10 feet from the footing edges in the plane of the footing tops. This portion of the fill should be constructed with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for at least a distance of 50 feet behind the abutments before re-excavating for the abutment footings. A design load of 2.0 t.s.f. may be used for footing design.

cont'd. /10 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Foundations: (cont'd.) ...

6.2.2) Abutment Foundations: (cont'd.) ...

As an alternative, the proposed abutments may be constructed within the approach fills and supported on 12-3/4 inch O.D. x 1/4 inch wall thickness closed-end steel tube piles driven about 10 feet into the upper very stiff to hard portion of the cohesive deposit; in no case should the final pile tip elevation be lower than 565. The piles can be designed for a safe capacity of 20 tons/pile.

Care should be taken to ensure that rock or bouldery fill is not placed within the areas in which piles have to be driven.

The differential settlement between the various structure elements is the governing factor in determining the type of structure to be employed at this location. The subsection to follow will discuss this aspect in detail.

6.3) Settlement Considerations:

Computations, based on Schmertmann's* method, have been carried out to determine the consolidation settlement of the foundation subsoil due to the pier footing and the embankment loading.

Results of the analysis are summarized in tabular form below:

| | |
|--|----------------|
| 1) Ultimate settlements at the pier locations - | |
| Induced by the footing pressure of 2.5 t.s.f. \approx 2-1/2" | |
| (footing size 50' x 8') | |
| Induced by embankment loading | \approx 1/2" |
| (30-ft. height) | |
| Total -- | \approx 3" |

*Schmertmann, J. E. -

"The Undisturbed Consolidation Behaviour of Clay" -
American Society of Civil Engineers" - 1955.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.3) Settlement Considerations: (cont'd.) ...

- 2) Ultimate settlement at the abutment locations $\approx 10"$
(Induced by embankment)

These values represent the total consolidation settlements, the majority of which should occur within 1 to 1-1/2 years with about 50% occurring within 4 to 6 months.

Because of the predicted relatively rapid rate of settlement, it would be extremely advantageous to construct the approach fills prior to construction of the structure footings, in order to reduce the differential settlements between the various elements. In this regard it is recommended that, if scheduling allows, the fills be constructed and remain in place for a period of between 4 and 6 months. For example, it is estimated that if a staging period of 6 months is allowed, the differential settlement between the abutments and piers would be of the order of 2 to 2-1/2 inches.

6.4) Approach Embankments:

The approach embankments will have a maximum height of about 30 feet. For fills of this magnitude, no stability problems are anticipated, provided standard 2:1 slopes are adopted. The maximum computed consolidation settlement of the embankment will be of the order of 10 inches. The time rate of settlement will be similar to that discussed previously.

7. SUMMARY:

A foundation investigation for the proposed overhead structure at the crossing of the C.N.R. tracks and East Side Highway is reported.

Subsoil at the site consists of a deposit of hard to stiff silty clay to clay some 70 to 82 ft. thick, followed by a

7. SUMMARY: (cont'd.) ...

thin, competent glacial till deposit. The glacial till is, in turn, underlain by dolomite bedrock, the surface of which is some 79 to 84 ft. below ground surface.

Pier foundations for the structure should be supported on spread footings located at least 4 ft. below the ground surface; a safe bearing pressure of 2.5 t.s.f. can be applied.

The abutments can be founded within the approach fill either: 1) within a zone composed of properly compacted granular fill using an allowable bearing pressure of 2.0 t.s.f., or 11) on 12-3/4" O.D. closed-end pipe piles driven about 10 feet into the hard to very stiff silty clay; the allowable load per pile will be about 20 tons.

The anticipated settlement of the structure foundations and approach fills are discussed in the Section, "Discussion and Recommendations". In order to reduce the magnitude of the differential settlement between the abutments and piers, it would be advantageous to construct the approach fills prior to construction of the structure foundations, as discussed in the report.

No major dewatering problems are anticipated for the pier footing excavations.

No stability problems are anticipated for the approach fills, provided 2:1 slopes are employed.

8. MISCELLANEOUS:

The field work, performed during October 17 to November 1, 1968, was supervised by Mr. W. Hutton, Project Foundation Engineer, who also prepared this report.

The investigation was carried out under the supervision of Mr. M. Devata, Supervising Foundation Engineer, who reviewed the report.

Equipment used was owned and operated by F. E. Johnston Drilling Co. Ltd.

December 1968.

APPENDIX I.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

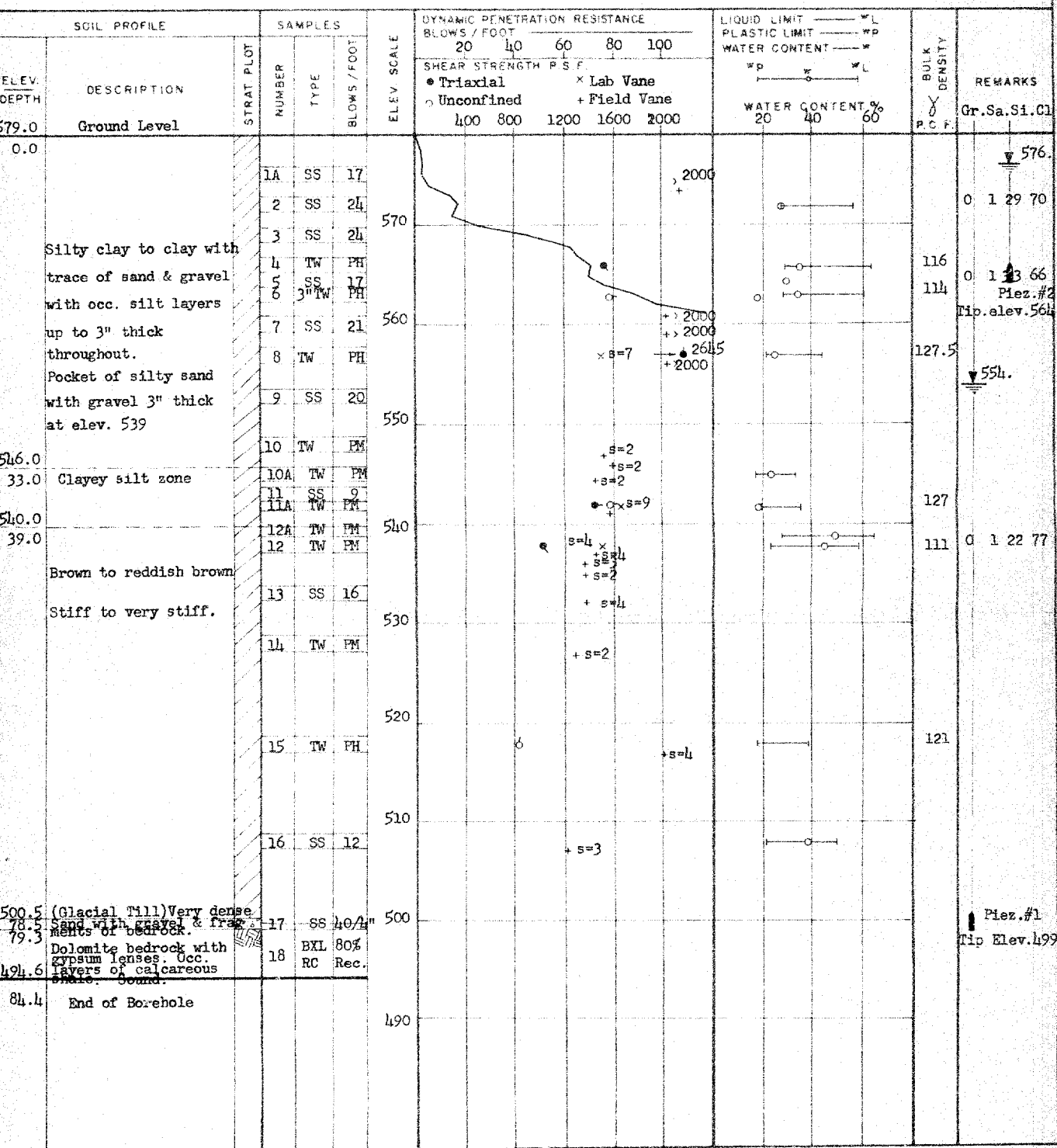
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

| | | | | | |
|-------|----------|---------------|--|---------------|----|
| JOB | 68-F-73 | LOCATION | Sta. 216+50 1/2 East Side Hwy. o/s 25' Rt. | ORIGINATED BY | WH |
| W.P. | 60-68-03 | BORING DATE | Oct. 17 - Nov. 1, 1968 | COMPILED BY | WH |
| DATUM | Geodetic | BOREHOLE TYPE | Cont. Flight auger & diamond drill | CHECKED BY | |



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION
JOB 60-F-73 LOCATION Sta. 219+10 @ East Side Hwy. o/s 38th Lt. ORIGINATED BY WH
W.P. 60-68-03 BORING DATE Oct. 23-29, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Cont. flight auger & diamond drill CHECKED BY XX

| SOIL PROFILE | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT | | | | | LIQUID LIMIT ——— % PLASTIC LIMIT ——— % WATER CONTENT ——— % | | | BULK DENSITY P.C.F. | REMARKS |
|----------------|--|-------------|--------|------|--|-------------|----|----|----|--|-----|------|---------------------------|------------|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | 20 | 40 | 60 | 80 | 100 | W.P. | W.L. | |
| 582.0 | Ground Level | | | | | | | | | | | | | |
| 0.0 | | | | | | | | | | | | | | |
| | Clayey silt to clay with trace of sand & gravel | | 1 | SS | 26 | 580 | | | | | | | | |
| | | | 2 | SS | 21 | | | | | | | | | 0 1 (99) |
| | | | 3 | SS | 21 | | | | | | | | | |
| | Occ. very thin grey silt seams containing clear gypsum crystals above elev. 559. | | 4 | TW | PH | 570 | | | | | | | | 116 |
| | | | 5 | SS | 14 | | | | | | | | | |
| | | | 6 | TW | PH | | | | | | | | | |
| | | | 7 | SS | 24 | 560 | | | | | | | | |
| | | | 8 | TW | PH | | | | | | | | | 128 |
| | | | 9A | SS | 10 | 550 | | | | | | | | |
| | | | 10 | TW | PH | | | | | | | | | 125 |
| | | | | | | 540 | | | | | | | | |
| | | | 11 | TW | PH | | | | | | | | | |
| | | | 12 | SS | 12 | 530 | | | | | | | | |
| | Brown to reddish brown | | | | | | | | | | | | | |
| | | | 13 | TW | PH | 520 | | | | | | | | 121 |
| | Firm to very stiff | | | | | | | | | | | | | |
| 510.0 | | | 14 | SS | 2 | 510 | | | | | | | | |
| 72.0 | Occ. thin sand seams up to 1/2" thick | | 15 | TW | PH | | | | | | | | | 112 108 |
| | | | | | | | | | | | | | | |
| 500.0 | | | 16 | TW | PH | 500 | | | | | | | | |
| 482.0 | Occ. thin clayey silt with sand, gravel and gypsum. Very stiff. | | 17 | SS | 29 | | | | | | | | | |
| 497.5 | | | | | | | | | | | | | | |
| 84.5 | Dolomite bedrock with a bed of gypsum 2' thick | | 18 | BXL | 76% | | | | | | | | | |
| 193.0 | Sound. Grey. | | | RC | Rec | | | | | | | | | |
| 89.0 | End of Borehole | | | | | 490 | | | | | | | | |

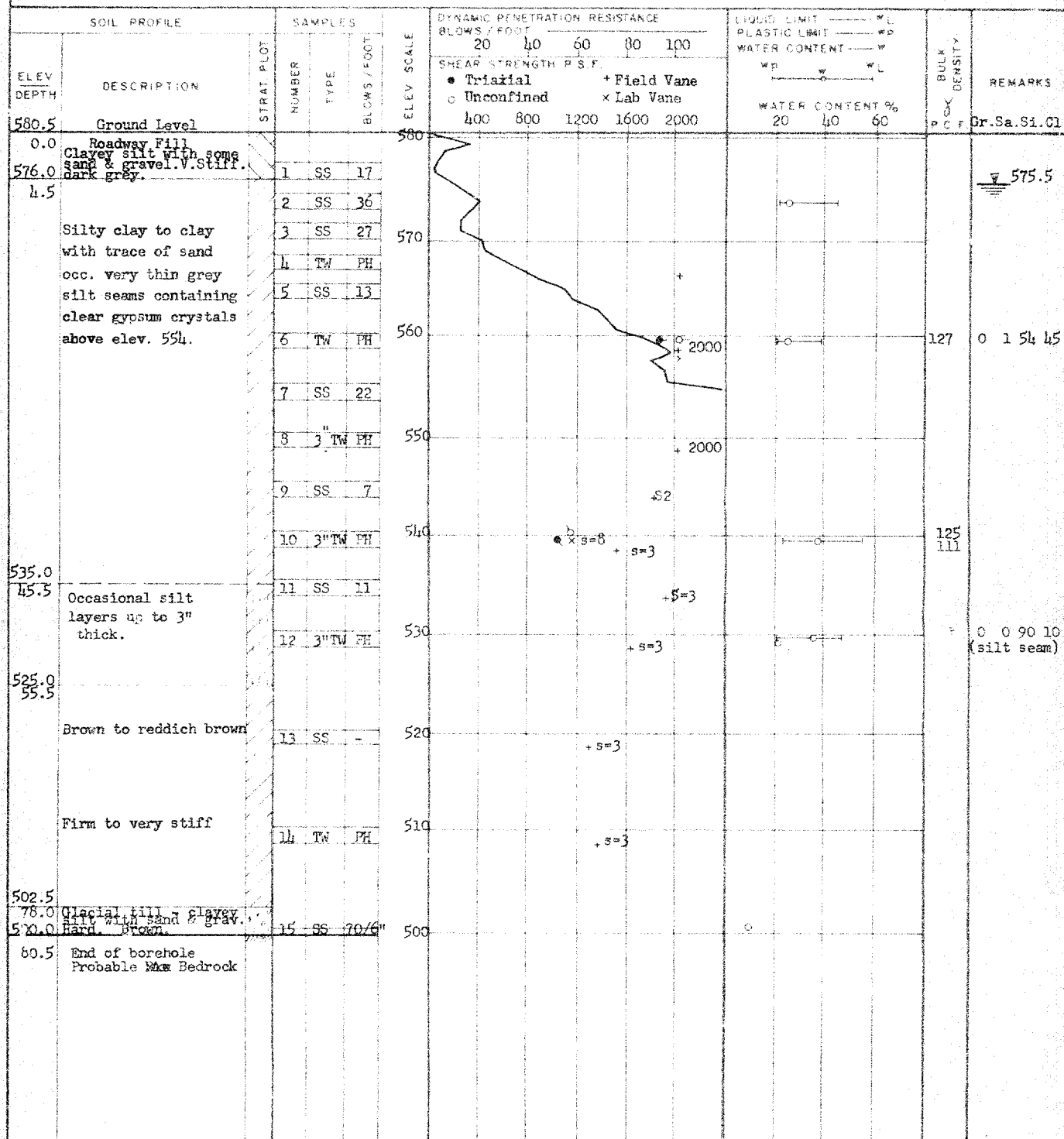
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 217+53 E East Side Hwy. o/s 73' Rt. ORIGINATED BY WH
W P 60-68-03 BORING DATE Oct. 28-29, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

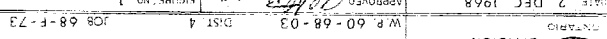
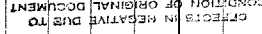
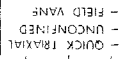
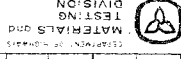
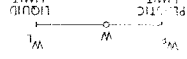
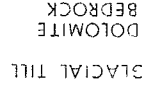
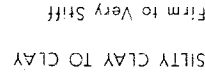
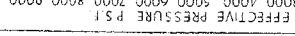
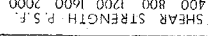
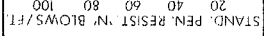
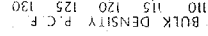
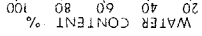
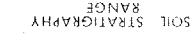
RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

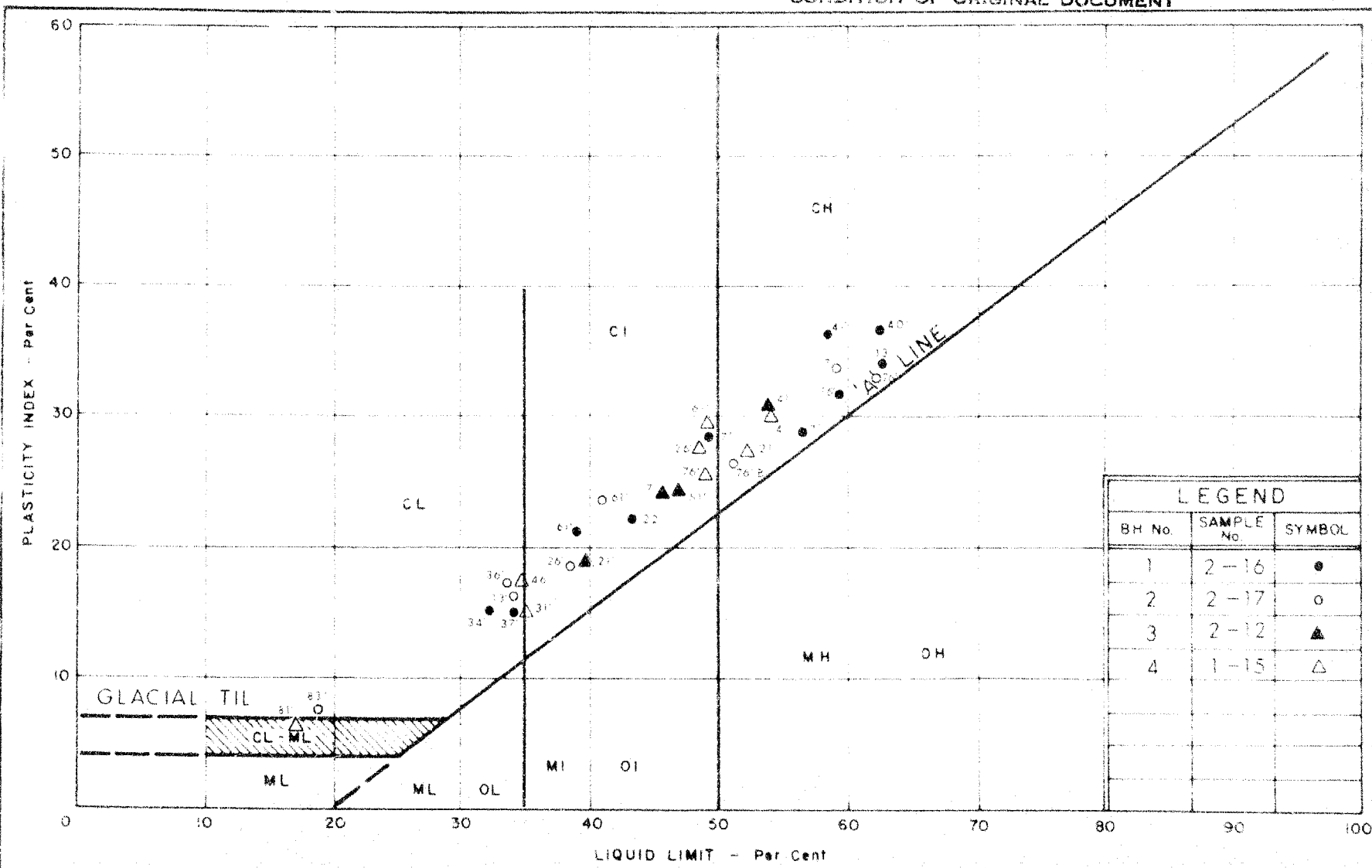
JOB 68-F-73 LOCATION Sta. 221+00 @ East Side Hwy. o/s 5¹ Lt. ORIGINATED BY WH
W.P. 60-68-03 BORING DATE Oct. 30-31, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY _____

| SOIL PROFILE | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT | | LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — % | | BULK DENSITY P.C.F. | REMARKS |
|---------------|--|-------------|--------|------|--|------------|---|-----------------|---------------------------|-------------|
| ELEV DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV SCALE | SHEAR STRENGTH P.S.F. • Triaxial + Field Vane ○ Unconfined Comp. x Lab Vane | WATER CONTENT % | | |
| 582.0 | Ground Level | | | | | | 400 800 1200 1600 2000 | 20 40 60 | | Gr.Sa.Si.Cl |
| 0.0 | | | | | | | | | | |
| | Silty clay to clay with trace of sand & gravel | | 1 | SS | 76 | 580 | | | | |
| | | | 2 | SS | 23 | | | | | |
| | | | 3 | SS | 19 | | | | | |
| | | | 4 | TW | FM | 570 | | | | |
| | occ. very thin grey silt seams containing clear gypsum crystals above elev. 560 | | 5 | SS | 17 | | + 2000 | | | |
| | | | 6 | SS | 17 | 560 | s=4 x0 | | 124 | |
| | Brown to reddish brown | | 6A | SS | 13 | | + 2000 | | | |
| | Stiff to hard | | 7 | TW | FM | 550 | s=2 | | 127 | |
| | | | 8 | SS | 14 | | + s=2 | | | |
| | | | 9 | TW | FM | 540 | s=5 + s=2 | | 123 | 0 2 61 37 |
| 534.0 | | | 10 | TW | FM | | + s=3 | | | |
| 48.0 | Occ. layers of silt up to 3" thick | | 11 | SS | 19 | 530 | + s=3 | | | |
| | | | 12 | TW | FM | 520 | s=3 + s=3 | | 117 | |
| 520.0 | | | | | | | | | | |
| 62.0 | | | | | | | | | | |
| | | | 13 | SS | 12 | 510 | s=3 + s=2 | | | |
| | | | 14 | TW | FM | | + 2000 | | | |
| 504.0 | | | | | | | | | | |
| 78.0 | Glacial till-clayey silt with sand & grav. Hard. Brown to grey. | | 15 | SS | 81 | 500 | | | | |
| 497.3 | | | 16 | SS | 647 3" | | | | | |
| 84.7 | End of Borehole Probable Bedrock | | | | | 490 | | | | |

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



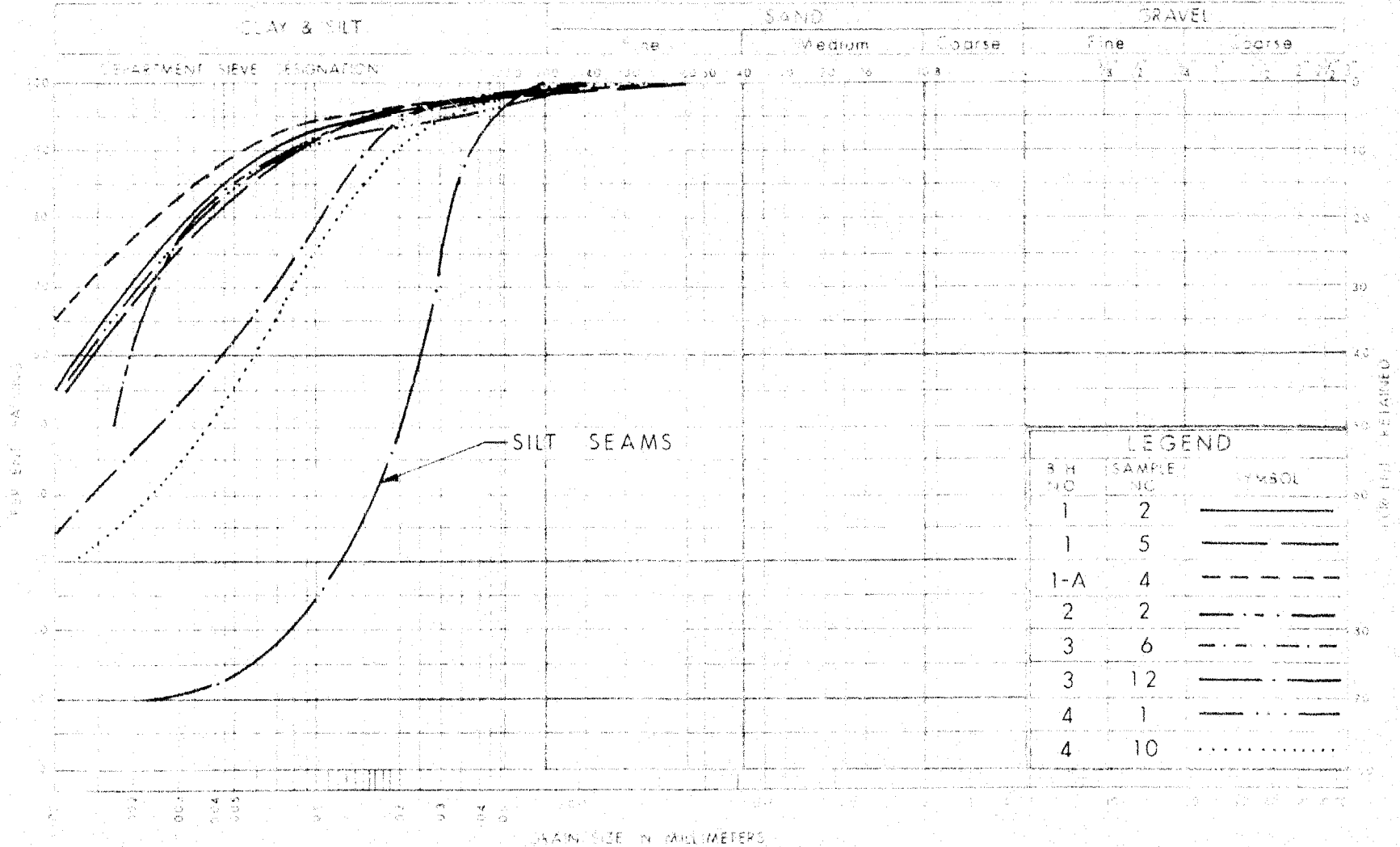
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART SILTY CLAY TO CLAY OCC ZONES OF CLAYEY SILT

WP No. 60-68-03
JOB No. 68-F-73
FIG. NO. 2

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY
WITH TRACE OF SAND AND GRAVEL

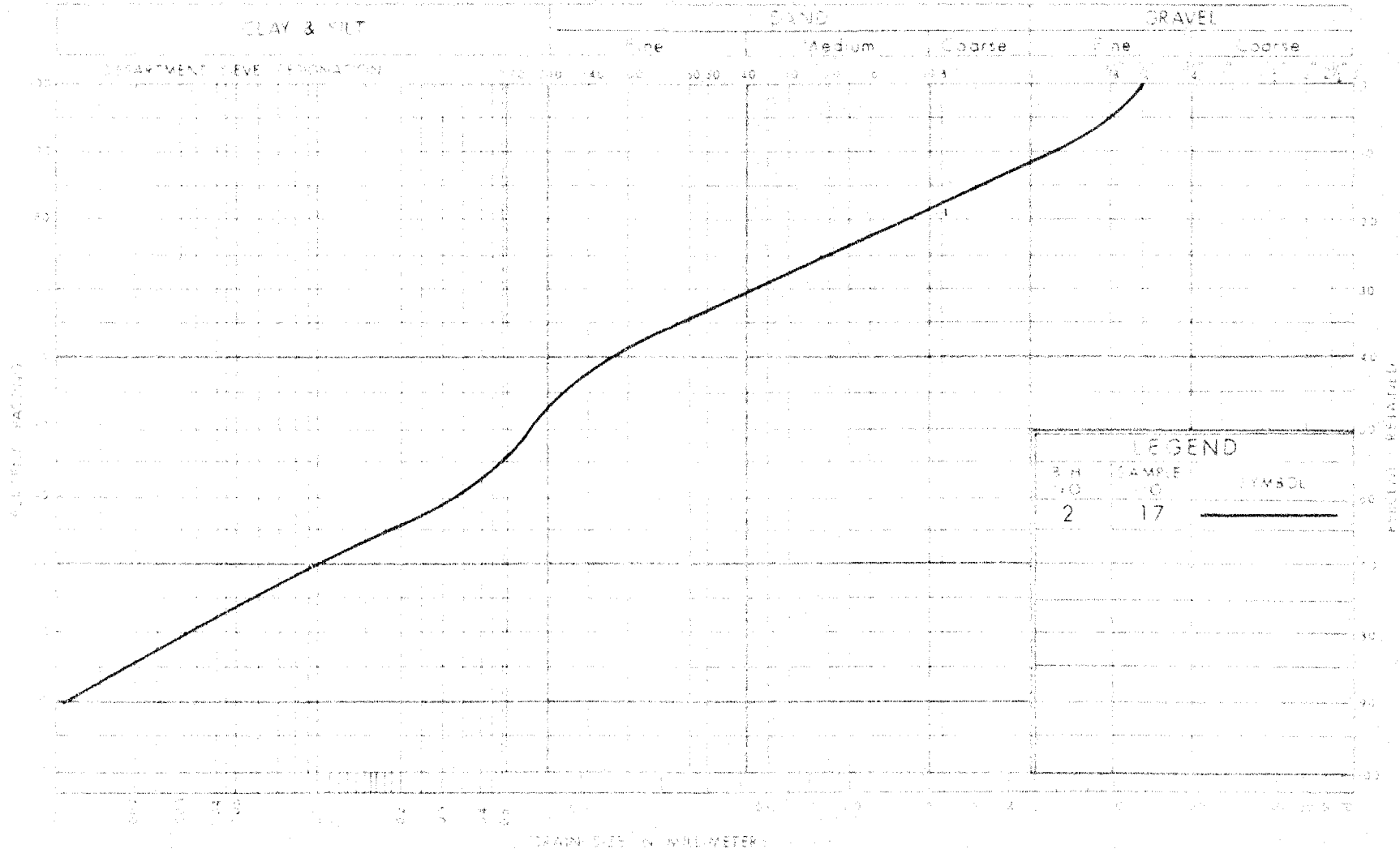
W 12-10 60-68-03

JOB No. 68-F-73

FIG NO. 3

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
CLAYEY SILT WITH SAND & GRAVEL

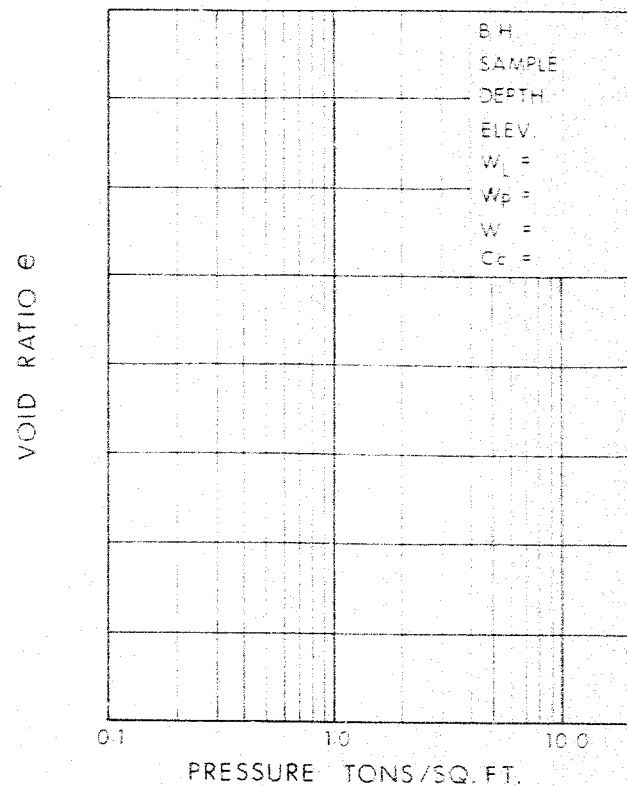
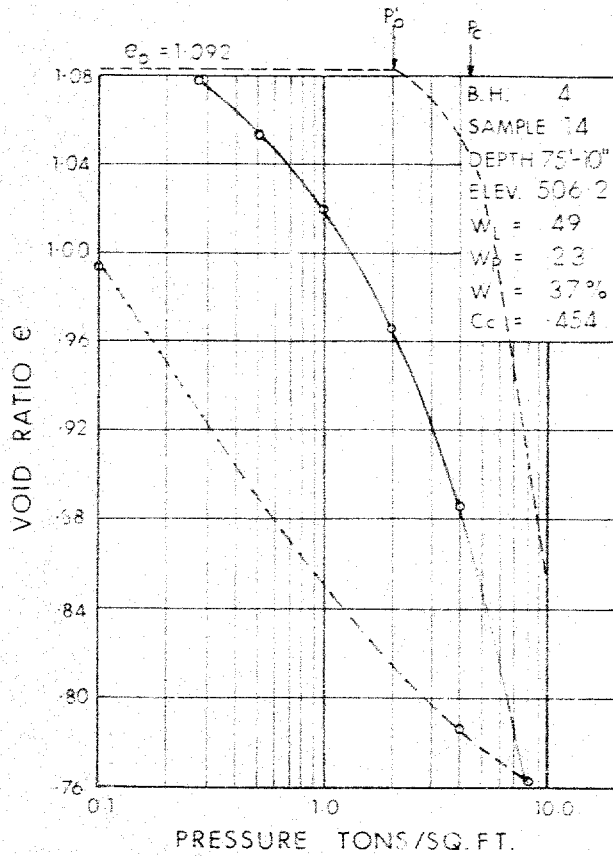
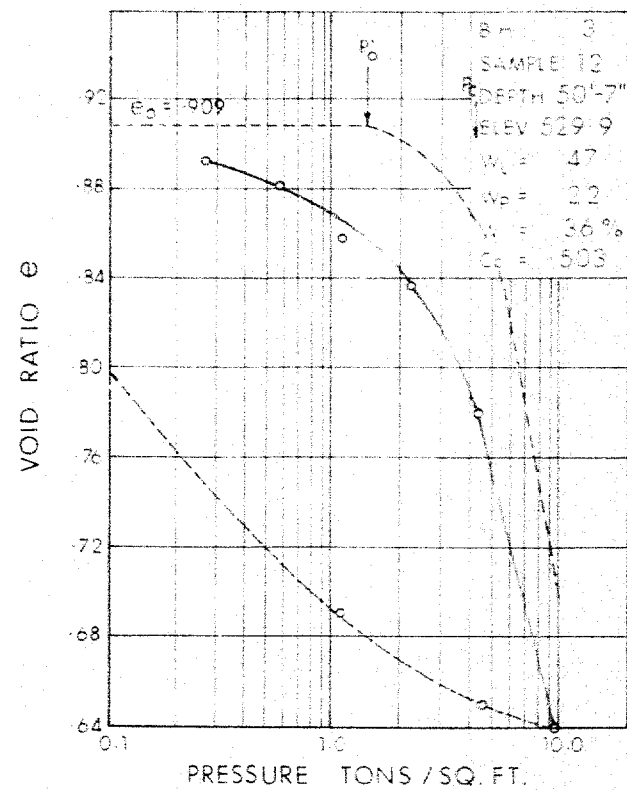
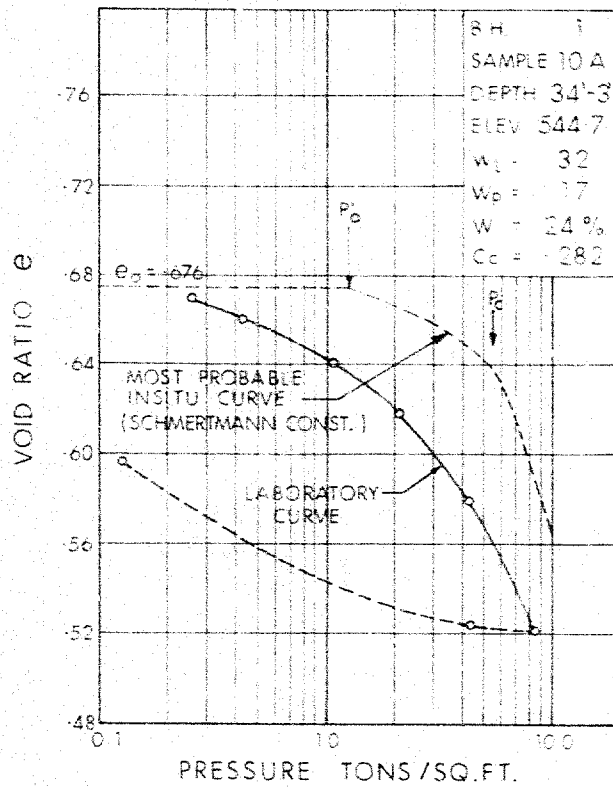
W.C. No 60-68-03

JOB No 68-F-73

FIG. NO 4

VOID RATIO - PRESSURE CURVES

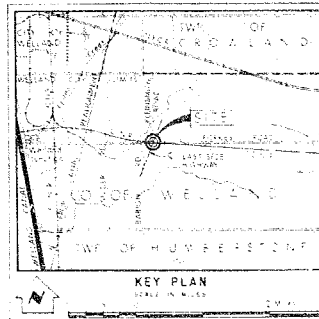
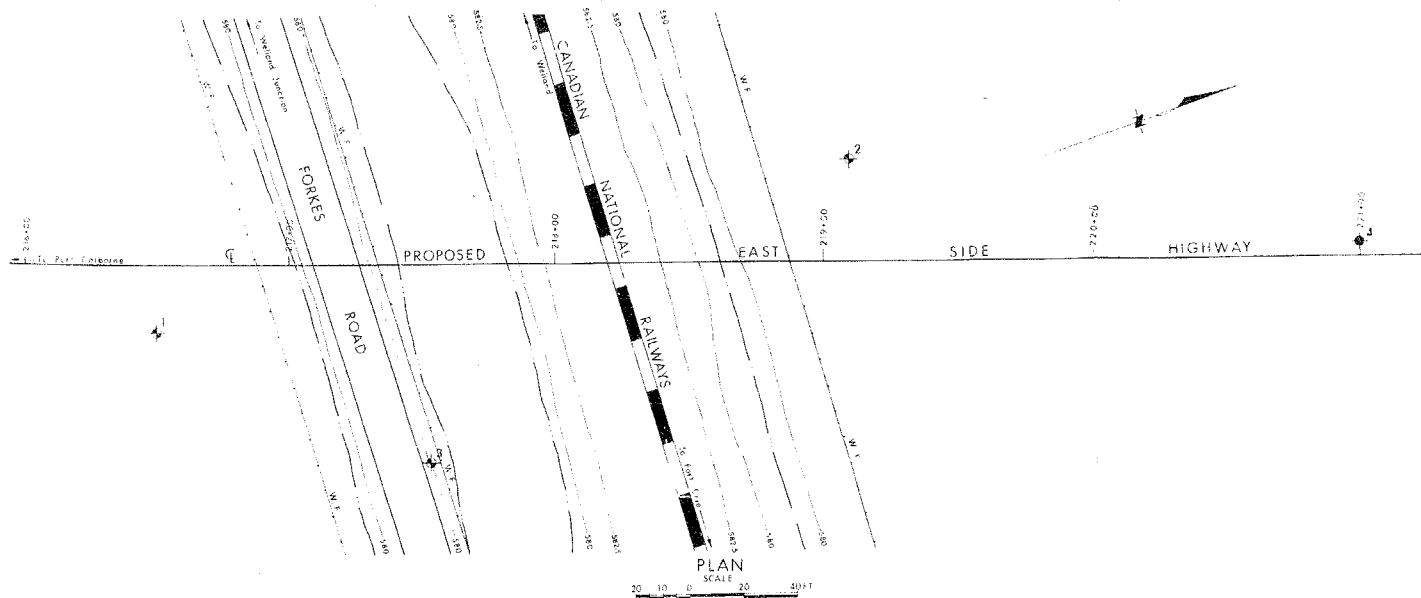
JOB NO. 68-F-73



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT.

FIG. 5

P_0 - EFFECTIVE OVERBURDEN PRESSURE
 P_c - MOST PROBABLE PRECONSOLIDATION PRESSURE

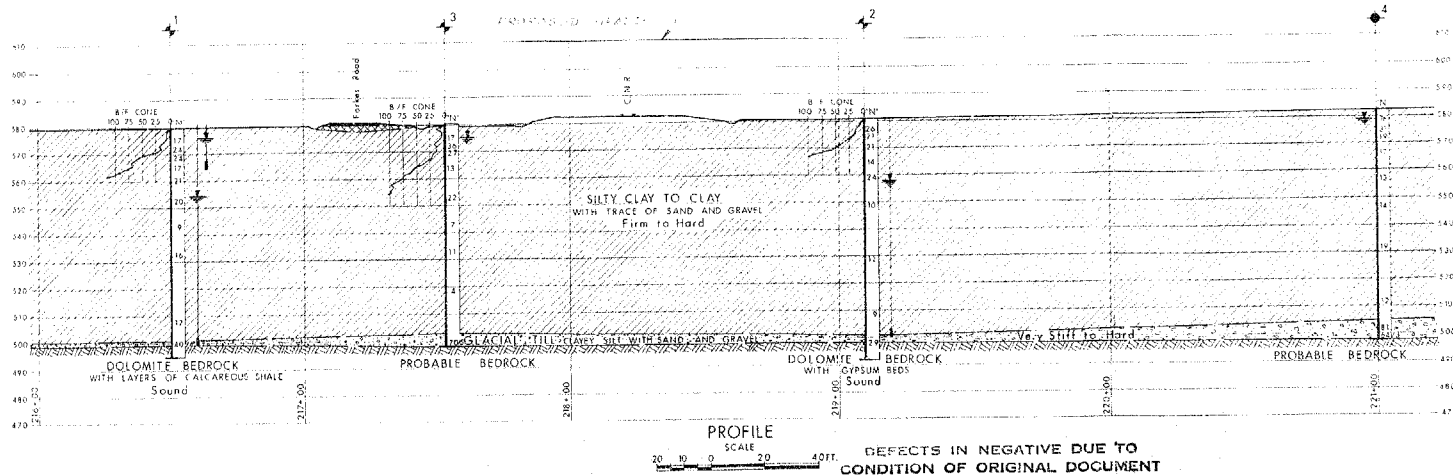


LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation
- Proposed

| NO. | ELEVATION | STATION | DEPTH |
|-----|-----------|---------|-------|
| 1 | 579.0 | 210+00 | 23.47 |
| 2 | 582.0 | 210+00 | 23.47 |
| 3 | 580.5 | 210+03 | 23.47 |
| 4 | 582.0 | 221+00 | 8.17 |

NOTE
The boundaries between soil strata have been located and only of Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to some degree of error.



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - HIGHWAY 26698

CANADIAN NATIONAL RAILWAYS OVERHEAD
(LINEAR FORKES ROAD)

KING'S HIGHWAY NO. EAST SIDE HIGHWAY DIST. NO. 4
CO. WELLAND LOT 18 & 22 TWP. HUMBURSTONE

BORE HOLE LOCATIONS & SOIL STRATA

SUBNO. IN H. CHECKED BY: J.P. NO. 60-08-03 V.S. DRAWING NO. 68-F-73A
DRAWN BY: P. CHECKED BY: J.P. NO. 68-F-73
DATE: Nov. 27, 1968 SITE NO. 68-F-73A
APPROVED BY: J.P. NO. 68-F-73A

Department of Highways Ontario

Copy for the information of

28-110-73

Mr. M. Devata,

Foundation Section

C. R. Robertson, P. Eng.,
District Engineer,
HAMILTON District

Bridge Office,
Downsview, Ontario

J. Regan


July 8, 1971

W.P. 60-68-02
Site 34-234,
Cont. 70-212,
C.N.R. Overhead
(Perkes Road)
Hwy. 140, Dist. 4

This will confirm a conversation with the project supervisor of the above structure concerning the piles at the abutment. I said that providing that the fill was up to the correct elevation and properly compacted, I could see no reason why we can not let the Contractor start pile driving operations. I wanted to know when these operations would start and keep in touch with the progress during pile driving.

I also suggested that the Contractor be written a letter concerning the tilting of the piers. More fill was placed than called for on the drawings, certainly this was so on the north side. The Contractor should be asked how he is going to correct the situation after the fill slopes have been trimmed to the correct line. I have some ideas on how this should be done but first we should hear from the Contractor.

PMcW/mw
c.c. M. Devata


P. McWatt, P. Eng.,
Bridge Construction Engineer

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Building

From: J. Lin,
Bridge Office

Attention: Mr. M. Devata

Date: April 30, 1970

Our File Ref.

IN REPLY TO

Subject: C.N.R. Overhead (Forkes Road)
S.P. 60-68-02, Site 34-234
Highway 140, District No. 4

8-5-73

Please find attached prints showing the revision to the abutment footing design as a result of the meeting with you last week.

Concrete piles are now used instead of spread footing on compacted fill as previously recommended.

W. Lin

WL:rd

W. Lin,
Regional Bridge Design Engineer

Attach.

Prints should be drawn to standard

M. Lin

6th May 1970

CP-F-73

C.N.R. @ Forbes Rd.

Starna recommended (Jan 13, 70) abutment
on spread footings

Devote Referred to no abutment to be
supported on ~~floor~~ concrete piles
~~W. Lee~~ Lin letter (April 30, 70). B

The estimation drawing referred to spread
footing abutments (April 9, 1970)

MEMORANDUM

Telephone: 248-3415

To: Mr. A.G. Stermac,
Principal Foundation Engineer,
Materials & Testing,
Lab Building.

From: A.G. Kelly,
Toronto Regional Road Design.

ATTENTION:

DATE: April 10, 1970.

OUR FILE REF.

IN REPLY TO

SUBJECT: Re: Highway 140, W.P. 60-68-04, Grading, Drainage,
Granular Base and Paving, Townline Road Northerly
to East Main Street, and W.P. 60-68-02, C.N.R.
Overhead Forkes Road.


Under separate cover I am forwarding for your perusal contract documents for two locations where foundation investigations have been executed.

W.P. 60-68-04 - At Sta. 342/30 Culvert Site.

A report (W.J. 69-F-64) was issued for the multi-plate arch culvert. Your report requested a granular pad with a minimum thickness of 12 inches to be placed in conjunction with standard No. DD-808-A. We believe this should be Standard No. DD-808-B, however, both are in the contract. All of the documents for the whole contract have been forwarded.

W.P. 60-68-02 - C.N.R. Overhead.

A report (W.J. 68-F-73) was issued for this site. I have forwarded the drawing for both the highway and bridge but only the bridge D-4 and specials. If you require D-4 and specials for the highway work, I will forward same on request.


A.G. Kelly
Sr. Project Design Engineer
For:
G.K. Hunter
Regional Road Design Engineer

ACK/GE

c.c. D. Barr

MEMORANDUM

Telephone: 248-3446.

To: Mr. C.R. Robertson,
District Engineer,
District 4, Hamilton.

FROM: H.G. Kunzelmann,
Toronto Regional Road Design.

ATTENTION:

DATE: April 14, 1970.

OUR FILE REF.

IN REPLY TO

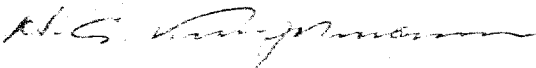
SUBJECT: RE: W.P. 157-64-01,
Q.E.W.-McLeod Road
Interchange,
District 4, Hamilton.

Enclosed please find the following contract information:

One set of drawings and breakdown sheets,
B4,
Complementary summary sheets,
Data for corrugated steel pipe,
Schedule of Materials,
List of Special Provisions,
Proposed Special Provisions,
Schedule of plans and standards.

This is for your use in preparation for the Regional Review.

The advertising date for this contract is October 21, 1970.


H.G. Kunzelmann
Asst. Projects Co-Ordinator
For:
W.C. Friedmann
Expressway Consultant Control Engineer

HCK/mj

c.c. T. Kovich
A.G. Stermac
R. Minaker
C. Fraser
D. Barr

Encl.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Office, Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. W. L. Lin

DATE: January 15, 1970

OUR FILE REF.

IN REPLY TO

SUBJECT:

Re: C.N.R. Overhead
Proposed Hwy. 140
W.P. 60-68-02
District #4 (Hamilton)

The present design for the above mentioned crossing calls for a three-span, simply-supported structure.

It is recommended that the abutments be also supported on spread footings founded on (within) the approach embankments. It is also recommended that only normal acceptable fill material be used for the ends of the approach fills where otherwise, in such instances, granular material is specified.

The District forces, which are supervising construction, should be made fully aware of the need for the specified compaction of this portion of the approach fill (under and around the abutment footing location) to be achieved under any circumstance. It is also recommended that the fill be built to its full height and the area for the abutment footings be subsequently excavated. This procedure will enable more uniform compaction of this area.

Any staging of the grading and structure contracts would be beneficial. However, this requirement is to be considered as very desirable, but not absolutely necessary.

As far as we know, this would be the first bridge of this type - i.e., the first bridge whose abutments have spread footings founded on normal, compacted earth fill. It is imperative that very close observation be maintained during and after construction. Such monitoring would provide warnings should something seem to be going wrong, and would also provide valuable information for future work. It is suggested, therefore, that the District be required to advise the Foundation Section ahead of the start of the grading of the approach embankments, so that the necessary instrumentation and control can be initiated and carried out.

AGS/MdeF

cc: Foundations Files
Gen. Files

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Materials & Testing, Lab. Bldg.
Downsview, Ontario.

FROM: Bridge Office,
Downsview, Ontario.

ATTENTION:

DATE: January 2, 1970.

OUR FILE REF.

IN REPLY TO

SUBJECT: C.M.R. Overhead (near Forkes Rd.)
Hwy. 140 - WP 60-68-02, Dist. 4

This is to confirm the results of our meeting of December 31, 1969, wherein it was decided to place the approach fills as soon as possible to reduce post construction settlements and to eliminate the piling, supporting the abutments on well compacted fill. You are to arrange to instrument these fills, and if the required stability is not realized, advise us such that we can re-introduce the pile system.



E. R. Davis,
Bridge Engineer.

BRD/vh

cc/ Messrs: C. Grebski
W. Lin
M. Stoyanoff

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Building

FROM: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: November 18, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT: C.N.R. Overhead (Forkes Road)
W.F. 60-68-02, Site No. 34-234
Highway 140, District No. 4

60-68-02

Attached herewith we are submitting the final
bridge drawings which show the foundation design for
this structure.

Kindly give us your comments at your earliest
convenience.



C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Section

Advised Mr Lin that the piles should be driven to rock
(not to refusal).

M. Levada

Nov 20th 1969



MEMORANDUM

To: Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Office,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION:

DATE: August 8, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT:

C.N.R. Overhead at the Crossing of
Proposed Hwy. 140 (Line 'A')
(Near Forkes Road) --
District No. 4 (Hamilton)
W.J. 68-F-73 -- W.P. 60-68-02

We have reviewed Bridge Drawing No. D-6698-P for the
above mentioned structure and submit the following comments:

Based on information obtained by previous studies
carried out by this Section as well as other agencies, the
extensive cohesive deposit encountered in the Welland area
has a high sulphate content. The sulphates recorded were
often in the order of 2000 p.p.m. range. Such sulphate
concentration could cause deterioration of buried steel piling
due to electro-chemical corrosion activities. In view of this,
consideration should be given to support the abutment footings
on pre-cast concrete piles driven to bedrock surface. For
example, Herkules (hexagonal) "Type 800" pile may be designed
for 100 tons per pile. The piles should be formed of sulphate
resistant cement.

MD/MdeF

cc: Messrs. S. McCombie
W. S. Melinyshyn
Foundations Files ✓
Gen. Files

M. Devata

M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Materials & Testing Office,
107 Lab. Bldg. Downsview.

FROM: Bridge Office,
Downsview, Ontario.

ATTENTION:

DATE: February 20, 1969.

OUR FILE REF.

IN REPLY TO

SUBJECT:

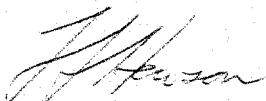
Your WJ 68-F-73
C.N.R. Overhead on Hwy. 140
(near Forkes Road)

In my request for this foundation investigation dated 23 Sept. 1968 I inadvertently quoted an incorrect W.P. number and this error, of course, now appears on your Report which has received rather wide circulation.

Please arrange to have all your records corrected from W.P. 60-68-03 to W.P. 60-68-02. Since costs on this are recoverable to some extent it will be necessary to alter your accounts also.

I am sending copies of this memo to the parties who are mentioned on your memo of December 4 to Mr. Davis.

I regret the inconvenience caused by this error.



F. I. Hewson,
Senior Bridge Liaison Engineer.

FIH/vh

cc: Messrs: B.R.Davis
H.A.Tregaskes
D.W.Farren
G.K.Hunter
H.Greenland
W.S.Melinyshyn
T.J.Kovich
B.A.Singh, O.W.R.C.

Department of Highways Ontario

Copy for the information of

file with report, please
AGB

Mr. A.G. Stermac.

Mr. D. Waller,
Construction Engineer,
District 4, Hamilton.

Bridge Office,
Downsview, Ontario.

February 1, 1971.

RE: Contract 70-212, C.W.B. Overhead,
Qty. 140, District 4, Hamilton.

W/O 100-100-73
W/O - 68-F-73

The following changes in the pre-cast concrete piles for this structure have been approved, subject to the satisfactory performance of a pile during a test driving.

- 1) The main longitudinal bars changed from #5 bars to #4 bars, with the equivalent area being provided.
- 2) Clear cover on the main steel to be 1 1/2 inches.

AKM/si

cc. Mr. G. Meyrink,
Birmingham Const. Ltd.
C.S. Grebski,
S. Cant,
A.G. Stermac.



A.E. McKim,
Bridge Control Engineer.

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Foundations Engineer, Lab. Bldg.

file 103

Bridge Office,
Downsview, Ontario,
January 26, 1971.

Birmingham Construction Limited,
Wellington Street Marine Terminal,
Hamilton 21, Ontario.

Attention: Mr. G. Meyrink,
Chief Engineer.

Re: Contract 70-212 C.N.R. Overhead
Hwy. 140, Dist. #4

*660-68-01 12
68-F-73*

Dear Sir:

In reply to your submission of January 21, the precast concrete pile and connection proposed as alternates to those details given on D6698-3, are approved subject to the following conditions:

- 1) Compression tests on cores taken from a 12" long unreinforced section of piles indicate an average concrete strength of 8000 psi at 28 days.
- 2) Satisfactory driving of the piles is accomplished on site.
- 3) The splices perform satisfactorily during actual driving at the site. If there is any indication that the splices are not adequate modifications will have to be made.

We understand you plan on driving a test section, and would like to observe this if possible. A phone call to Mr. Waller in the District Office a day or so prior to the driving would be appreciated. We would also appreciate receiving details of jobs on which these piles have been driven satisfactorily.

Yours very truly,


A. E. McKim,
Bridge Control Engineer.

AEMCK/vh
cc/ Mr. D. Waller
Mr. A. Stermac
Mr. S. C. ant

BERMINGHAM

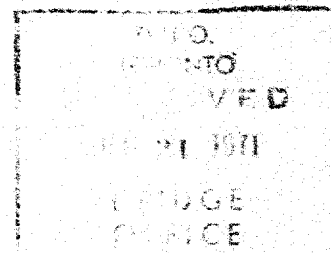
CONSTRUCTION LIMITED

WELLINGTON STREET MARINE TERMINAL

HAMILTON 21, ONTARIO

HAMILTON
JAN 20 8-1924
TORONTO
EM 6 6779

January 21, 1971



Attention: Mr. Alan L. McKim

Dear Sir:

Re: DHO 70-212 Fort Colborne

Further to our meeting at your office on January 20, 1971, I enclose two copies of our drawing showing the proposed 13 inch diameter spun concrete pile and our K-joint connector.

Also enclosed, please find a copy of Les Réseaux Centrifuges Itée, letter of January 13, 1971. The above company manufactures the spun 13 inch pile for a piling contractor in Montreal, and our proposed pile will be identical except for the connector.

As mentioned yesterday, it required an axial pull of 40 tons to separate the male and female components of the K-joint connector. At this stage we do not think that bending moment the K-joint will be able to resist. We feel that it will be more than adequate for our purpose. Once we have some actual values, we have to establish the maximum bending moment that the joint will resist.



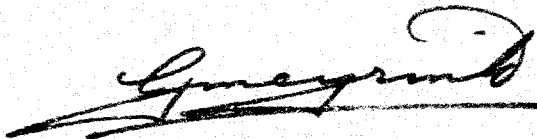
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CONDITION OF ORIGINAL DOCUMENT

Established 1897

In the meantime, we would appreciate it very much if you would check the designs of our proposed pile, and advise whether we may go ahead and use them at the Port Colborne project.

Yours very truly,

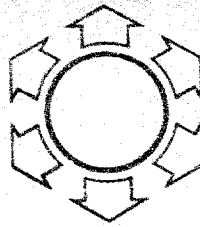
BIRMINGHAM CONSTRUCTION LIMITED

A handwritten signature in dark ink, appearing to read 'G. Meyrink', with a large, stylized flourish at the end.

G. Meyrink

Chief Engineer

CM/hb



BUREAU: 695-3195
USINE: ~~XXXXXX~~
~~XXXXXX~~
Coteau du Lac, Que.

LES BETONS CENTRIFUGES LTEE

3125 MONTÉE ST-CHARLES — VILLE DE KIRKLAND, QUE.

January 13, 1971

Birmingham Construction
Wellington Street
Marine Terminal
Hamilton, Ont.

Attention Mr. George Heyrick

Dear Sirs,

Further to our telephone conversation, please find enclosed a copy of our drawing SU 1 showing a general arrangement of a 13 inch spun concrete pile, and also illustrating the S-M connector on which St-Aubin & Mariet Enr. owns a patent covering Canada and United States.

With the use of National Building Code of Canada 1970, our design capacity on the piles is as follows:

- 1) Article 4.2.2.4.(f) - Concrete cover of 1 inch
- 2) Article 4.2.5.8 - Piles as columns
- 3) Article 4.2.6.15 - Stresses in pile materials
- 4) Although article 4.2.5.8 refers to section 4.5, we have been informed that NBC supplement N° 4 is not yet issued and could be published this coming spring. This last supplement N° 4 is entitled "Code for the Design of Plain and Reinforced Concrete Structures".
- 5) NBC supplement N° 4 being not yet available, we discussed with a local piling contractor using precast concrete piles, and he came to the conclusion to use the following as the basic formula to determine the design capacity of our pile:

$$P = 0.2 A_c f'_c + 0.3 A_s f_{yp}$$

P is the design capacity of the pile
A_c is the gross concrete area (126 sq. inch)
f'_c is the concrete strength (8000 psi)
A_s is the reinforced steel area (1.86 sq. inch)
f_{yp} is the yield stress (75000 psi)

The design capacity is then 244 kips.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Page 2

Birmingham Construction

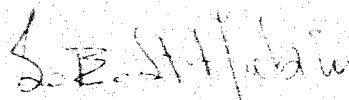
January 13, 1971

We hope the previous information will meet your requirements, and if we can be of any assistance to you, do not hesitate to get in touch with us.

To meet the undersigned curiosity, we would like to have some details about the connector you propose to use.

Yours very truly,

LES BETONS CENTRIFUGES LTEE



Bernard E. St-Aubin, Eng.
General Manager

BESA/nl

Encl.

c.c. Mr. William C. Schwenger
Ontario Stress-Crete Ltd
Burlington, Ont.