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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Oshawa Creek Bridge Structure Site No. 22-175 Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

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REPORT



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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
OSHAWA CREEK BRIDGE
STRUCTURE SITE NO. 22 - 175
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario.

This report addresses the proposed widening of the existing Oshawa Bridge Creek. This report was developed with information from a geotechnical/foundation investigation completed in 1973 at the Oshawa Creek Bridge site, reported as follows:

- **MTO GEOCREs No. 30M15-033:** *Foundation Investigation Report for Widening of Oshawa Creek Bridge, Highway 401, Oshawa, Site 22-175. W.P. 44-71-08 – W.O. 73-11022*, by MTO Foundations Section, dated June 21, 1973.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2010-E-0062, dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of the *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

2.0 SITE DESCRIPTION

The existing Oshawa Creek bridge is located approximately 1.5 km east of the Stevenson Road interchange, between Cubert Street and Simcoe Street in the City of Oshawa, in the Regional Municipality of Durham. The existing bridge is an approximately 56.4 m long, three-span arch structure. The original structure and a subsequent widening in 1977 are supported on spread footings founded on very dense glacial till.

The natural ground surface in the vicinity of this site is at about Elevation 94 m and 92 m west and east of the creek, respectively. The base of the creek channel is at approximately Elevation 89 m within the centre span of the arch structure. The channel base consists of a 150 mm thick reinforced concrete slab that is founded on approximately 150 mm of Granular A padding at an average elevation of 88.6 m.

Highway 401 has been constructed on embankments that are about 5 m to 6 m high relative to the natural ground surface, with the pavement grade at about Elevation 99.4 m at the west abutment, and about Elevation 98.0 m at the east abutment.

3.0 INVESTIGATION PROCEDURES

Twelve boreholes were advanced at this site as part of the previous investigation by the Foundations Section of the Ministry of Transportation and Communications, Ontario, in December 1972 and May 1973. For the purpose of this Preliminary Foundation Investigation Report, the boreholes have been re-numbered such that the 30M15-series GEOCREs number precedes the original borehole number. For example, Borehole 12 from GEOCREs Report No. 30M15-033 is referred to throughout this report and on the drawings as Borehole 033-12. The approximate locations of the boreholes are shown on Drawing 1; these borehole locations have been interpreted based on scaling measurements from the plan shown in the 1973 GEOCREs report.



Boreholes 033-9, 033-10, 033-12, 033-14, 033-15 and 033-16 were advanced on the north side of Highway 401, and Boreholes 033-1, 033-3, 033-4, 033-5, 033-6 and 033-7 were advanced on the south side of Highway 401, within the footprint of the widening that was constructed in 1977. The boreholes were advanced through the overburden using both wash-boring and augering techniques, with equipment supplied and operated by Canadian Longyear Limited of Rexdale, Ontario. Soil samples were obtained at regular intervals using split-spoon samplers driven by a manual hammer, and the resulting Standard Penetration Test (SPT) 'N' values were recorded, as shown on the appended Records of Borehole. Dynamic cone penetration tests were completed adjacent to Boreholes 033-1, 033-6, 033-9, 033-14 and 033-15, and these results are shown on the applicable borehole record.

The groundwater conditions in the open boreholes were observed during and immediately after the 1972/1973 drilling operations, and these measurements are noted on the borehole records contained in Appendix A.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984) and *Urban Geology of Canadian Cities* (Brennand, 1998). The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the 1972/1973 investigation and the results of in situ and laboratory testing are presented on the borehole records and laboratory test figures contained in Appendix A. Interpreted stratigraphic profiles along the north and south sides of the bridge are shown on Drawing 2.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles on Drawing 2 are inferred from observations of drilling progress and non-continuous sampling. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profiles shown on Drawing 2 represent simplifications of the subsurface conditions at the site. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of surficial layers of topsoil and fill associated with the existing Highway 401 embankments, underlain by a non-cohesive deposit of interlayered sand, silt, and gravel. This interlayered deposit is in turn underlain by a silty sand to sandy silt till deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.



4.2.1 Topsoil

Approximately 180 mm of topsoil was encountered in Boreholes 033-4 and 033-6, immediately below the ground surface at the time of the 1972/1973 investigation. This material may have been removed, or largely removed, in conjunction with the 1977 widening works undertaken in this area.

4.2.2 Fill

Fill material was encountered immediately below the then-existing ground surface (or a thin topsoil layer) in Boreholes 033-4, 033-7, 033-9, 033-10, 033-14 and 033-16, during the 1972/1973 investigation. The thickness of the fill ranged between approximately 0.6 m and 7.3 m, with the base of the fill encountered between Elevations 89.5 m and 93.5 m.

The fill material encountered in the boreholes was generally non-cohesive, ranging in composition from gravelly sand, to sand and gravel, to sand, to silt and sand. The results of grain size distribution tests on selected samples of the fill are annotated on the borehole records in Appendix A, and included within the envelopes of grain size distribution shown on Figures 1 to 3 in Appendix A. The natural water content measured on selected samples of the fill varied between approximately 5 and 32 per cent.

The Standard Penetration Test (SPT) 'N' values measured within the fill material range from 4 blows to 77 blows per 0.3 m penetration, indicating a loose to very dense (but typically compact to dense) relative density.

4.2.3 Interlayered Silt to Sand to Sandy Gravel/Gravelly Sand Deposit

A non-cohesive deposit with variable proportions of silt, sand and gravel was encountered immediately below the ground surface or underlying the topsoil or fill layers, in eight of the twelve boreholes advanced during the 1972/1973 investigation. The base of this deposit was encountered between approximately Elevation 87.7 m and 89.1 m in the boreholes, and the thickness ranges from about 2.1 m to 5.2 m.

The deposit consists of interlayers of non-cohesive soil that vary in composition from sandy gravel or gravelly sand, to sand, to silty sand, to silt and sand, to sandy silt, containing trace gravel. In general, the coarser (sandy gravel to gravelly sand) layers were encountered near the eastern portion of the site, in Boreholes 033-1, 033-4, 033-15 and 033-16, as well as in Borehole 033-6 near the west pier. Trace quantities of organic matter were observed in samples recovered within this deposit, as noted on the borehole records. The results of grain size distribution tests completed on samples of this deposit are annotated on the borehole records, and envelopes of grain size distributions are shown on Figures 1 to 3 in Appendix A. The measured water contents of samples of this deposit range from about 8 to 39 per cent.

The SPT 'N' values measured within this deposit range from about 7 blows to greater than 100 blows per 0.3 m of penetration, indicating a loose to very dense relative density. However, generally the SPT 'N' values are below 30 blows per 0.3 m of penetration indicating on average a compact relative density.

4.2.4 Silty Sand to Sandy Silt Till

A deposit of silty sand to sandy silt till was encountered underlying the interlayered non-cohesive deposit, with all boreholes terminating within this deposit with the exception of Boreholes 033-16, where the till deposit was not encountered within the investigated depth. The thickness of the till deposit varied from approximately 1.2 m to 6.0 m.



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The till deposit varies in composition from silty sand, to silt and sand, to sandy silt, and typically contains trace gravel. The results of grain size distribution tests completed on selected samples of the till deposit are annotated on the borehole records, and an envelope of the grain size distributions is shown on Figure 4 in Appendix A. Natural water content measurements on selected samples of this deposit range from about 8 to 25 per cent.

The SPT 'N'-values measure within the silty sand to sandy silt till are generally greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

4.3 Groundwater Conditions

During the 1972/1973 investigation, water levels were observed in the open boreholes at the completion of drilling and are summarized as follows:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level below 1973 Ground Surface (m)	Approximate Groundwater Elevation (m)	Date
033-1	90.2	Above ground surface	Artesian, encountered at 83.5	Dec 5, 1972
033-3	92.9	3.2	89.7	Dec 8, 1972
033-4	97.7	8.3	89.4	May 3, 1973
033-5	91.2	Above ground surface	Artesian, encountered at 83.2	May 10, 1973
033-6	91.1	1.4	89.7	May 10, 1973
033-7	99.0	5.2	93.8	May 4, 1973
033-9	94.8	5.2	89.6	May 9, 1973
033-10	99.0	8.0	91.0	May 3, 1973
033-12	89.8	0.0	89.8	May 10, 1973
033-14	90.1	0.3	89.8	May 10, 1973
033-15	93.1	3.1	90.0	May 8, 1973
033-16	97.8	6.1	91.6	May 7, 1973

Water level was found at or immediately below the then-existing ground surface in Boreholes 033-12 and 033-14, which are located immediately adjacent to the creek channel. On May 8, 1973, the water level in the creek was at approximately Elevation 89.8 m at the north end of the Bridge and Elevation 89.3 m at the south end (i.e., at or near the ground surface in Boreholes 033-12 and 033-14).

Artesian water pressures were encountered in Boreholes 033-1 and 033-5 (also located immediately adjacent to the creek channel) within the till deposit at about Elevation 83.2 m to 83.5 m, and the water level was noted to rise above the then-existing ground surface to about Elevation 91.4 m.

It should be noted that the water levels observed in the open boreholes on completion of drilling may not represent long-term stabilized groundwater levels, nor the current groundwater regime at the site. The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.



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5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., EIT, and was reviewed by Ms. Nikol Kochmanova, P.Eng. Ms. Lisa Coyne, P.Eng., Designated MTO Foundations Contact and a Principal of Golder, conducted an independent technical and quality review of this report.

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**PRELIMINARY FOUNDATION REPORT
OSHAWA CREEK BRIDGE, W.O. 10-20011**

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
OSHAWA CREEK BRIDGE
STRUCTURE SITE NO. 22-175
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
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6.0 DISCUSSION AND PRELIMINARY GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed widening of the existing Highway 401-Oshawa Creek bridge (MTO Structure Site 22-175) and associated wingwalls. These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1972/1973 subsurface investigation at this site. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents in the future. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect the equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road, the existing Oshawa Creek bridge will be widened. Based on the preliminary General Arrangement drawing provided by AECOM, the widening of the existing bridge structure varies from about 10.65 m to 11.98 m on the south side, and from about 1.575 m to 3.575 m on the north side.

The following design drawings are available for the existing Oshawa Creek bridge:

- Contract No. 77-133, WP No. 44-71-08, Sheet 1: "Widening of Existing Oshawa Creek and Highway 401 Structure, General Plan", prepared by the Ministry of Transportation and Communications - Ontario, dated November 1974.
- Contract No. 77-133, WP No. 44-71-08, Sheet 2: "Oshawa Creek Borehole Location and Soil Strata", prepared by the Ministry of Transportation and Communications - Ontario, dated June 1973.
- Contract No. 77-133, WP No. 44-71-08, Sheet 3: "Widening of Existing Oshawa Creek and Highway 401 Structure, Footings – Dimensions and Reinforcement", prepared by the Ministry of Transportation and Communications - Ontario, dated November 1974.

Based on these drawings, the original structure is supported on spread footings founded in the very dense glacial till, at approximately Elevation 90.5 m on the west abutment, Elevation 88.3 m at the piers and Elevation 90.3 m at the east abutment; at the east abutment, the footing has been placed on approximately 0.7 m of tremie concrete, which extends to about Elevation 89.6 m. The bridge was widened by about 9 m on the north and south sides in 1977 under Contract 77-133; this widening included the construction of new spread footings founded at the same elevation as the original structure.



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Both shallow and deep foundation options have been considered for support of the required abutment and pier widening for the Oshawa Creek bridge. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded within the compact to very dense silt to sand deposit:** Shallow footings are feasible at this site due to the relatively shallow depth to suitable founding soils. Additionally, this foundation type will match the existing foundations which have proven to perform satisfactorily on this site. The abutment footings should be founded at similar levels to the existing, to avoid undermining or loading the existing footings. This will require excavation to a depth of up to approximately 3 m at the west abutment and 2.5 m at the east abutment, relative to the existing ground elevation within the creek valley; deeper excavation will be required through the existing Highway 401 embankment to reach these footing founding levels, in conjunction with temporary protection systems. At the piers, excavation will be required to a depth of approximately 3 m, extending to slightly below the base of the adjacent creek channel. Groundwater control/cofferdams will be required as the proposed founding levels are below the groundwater level at the site. There is some potential for differential settlement between the existing and new foundations, on the order of 10 mm or less for the anticipated structure loading, and this could be addressed by using construction joints.
- **Driven steel H-piles or pipe piles founded within the very dense till:** Driven steel H-piles or pipe piles are feasible for support of the abutments, and would permit design of conventional, semi-integral (pipe piles) or integral abutments (H-piles). Although this option would allow the abutment pile caps to be founded higher than spread footings, minimizing excavation, dewatering and protection system requirements, this foundation option is not compatible with the existing structure. In addition, due to the relatively shallow depth to very dense, “100-blow” soils, pre-augering through the very dense till deposits would likely be required to achieve minimum penetration depth for the required pile length and capacity, and to ensure that the piles remain aligned and are not damaged. Driven pile foundations are not considered feasible at the pier locations, where “100-blow” soil is present immediately below the anticipated footing/pile cap level. Based on these considerations, driven pile foundations are not presented further in this report.
- **Caissons founded within the very dense till:** Caissons are considered feasible for the support of the abutment and pier widening, if this foundation type can be made structurally compatible with the existing bridge. This option may have significant advantages if it is possible to raise the pile cap (at the abutment locations) or eliminate the pile cap (at the pier locations, assuming that each caisson carries a single pier column extending to the underside of the bridge deck), to minimize excavation depths, dewatering and protection system requirements relative to a spread footing option. However, artesian groundwater pressures were encountered within the till deposit at depth. If caissons are adopted, these will extend through water-bearing non-cohesive deposits, and temporary liners would be required during caisson construction to balance the artesian pressures, and control potential ground losses and disturbance at the caisson base.

Based on the above considerations, shallow foundations are preferred from a geotechnical/foundations perspective, because these will match the existing structure foundations and will be founded on or near the “100-blow” till deposit. However, the use of caisson foundations offers some advantages with respect to excavation depth, dewatering and protection systems if this option can be made structurally compatible with the existing



bridge; the risks associated with water-bearing soils and artesian groundwater pressures must be considered for this option.

6.3 Assessment of Existing Foundations

The existing bridge is supported on 2.8 m to 3 m wide spread footings founded on the compact to very dense silt to sand deposit at the abutments, and the very dense glacial till at the pier locations, at approximately Elevation 90.5 m (west abutment), 88.3 m (piers) and 90.3 m (east abutment). The preliminary geotechnical assessment of the existing structures may be based on a factored geotechnical resistance at Ultimate Limit State (ULS) of 650 kPa. The geotechnical resistance at Serviceability Limit State (SLS, for 25 mm of settlement) may be taken as 500 kPa at the abutments for preliminary design.

6.4 Shallow Foundations

6.4.1 Founding Elevation

For support of the widened abutments and piers, spread footings should be founded on the compact to very dense silt to sand deposit (at the abutments) or very dense till deposit (at the piers). Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

At this preliminary stage, it is recommended that the abutment footings for the widening be founded to match the existing structure, to avoid transmitting load onto or undermining the existing footings. The table below summarizes these founding levels:

Foundation Element	Founding Elevation (m)	Founding Soil
West Abutment	90.5	Compact to dense silt to sand, underlain by “100-blow” till
West and East Piers	88.3	Very dense (“100-blow”) silty sand to sandy silt till
East Abutment	89.6 ¹	Tremie concrete slab on compact to very dense sand and gravel

¹An approximately 0.7 m thick tremie concrete slab was placed underneath the footing at the east abutment. Actual footing elevation is at 90.3 m.

The groundwater level at the site was between approximately Elevation 89.4 m and 91.6 m during the 1972/1973 investigation, although this may not represent the stabilized or current groundwater level at the site. The water level in the creek was measured at Elevation 89.8 m. The maximum (high) water level at the creek is understood to be at approximately Elevation 94.2 m, based on the General Arrangement drawing provided by AECOM. Groundwater control/cofferdams will be required during construction to maintain stable subgrade conditions for foundation construction.

The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately, a 100 mm thick concrete working slab (of 20 MPa compressive strength concrete) should be placed on the prepared subgrade within four hours of its inspection and approval.



6.4.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) may be used for preliminary design of spread footings founded on the properly prepared subgrade at the design elevations given in the preceding section:

Founding Element	Factored Geotechnical Resistance (ULS) kPa	Geotechnical Reaction at SLS (for 25 mm of settlement) kPa
Abutments	650	500
Piers	650	500

NOTE: The geotechnical resistance/reaction values given above are estimated for 2.8 m to 3 m wide spread/strip footings.

These geotechnical resistances are given for loads that will be applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with the Table 10.2 in *CFEM* (2006).

These preliminary geotechnical resistances should be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation, and the final footing width and founding elevation.

6.5 Caisson Foundations

6.5.1 Founding Elevation

Caissons founded within the very dense silty sand to sandy silt till may be considered for support of the abutment and pier widening. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming approximately 5 m of penetration into “100-blow” soil. However, it is noted that these elevations extend below the base of some of the boreholes at the site and should be taken as preliminary; further investigation will be required at the abutments during detailed design:

Foundation Element	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)
West Abutment	89	84
Piers	88	83
East Abutment	88	83

If caisson foundations are adopted, the presence of water-bearing non-cohesive soils and artesian groundwater pressures must be taken into account. A temporary liner and drilling slurry would be required to support the overburden soils during construction and to balance the artesian groundwater pressures to minimize disturbance to the base. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

If caissons are adopted, based on the preliminary founding elevations given above, the factored geotechnical resistance at ULS may be taken as 2,400 kN for a 1.2 m diameter caisson. The axial geotechnical reaction at SLS (for 25 mm of settlement) will be greater than the factored axial resistance at ULS and as such, the SLS condition does not apply.



If this foundation option is adopted for the widening of the abutments and/or piers, the preliminary geotechnical resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during detailed design in consideration of any additional subsurface investigation at the foundation elements.

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

Highway 401 has been constructed on approximately 5 m to 6 m high embankments, with its grade at about Elevation 99.4 m to 98 m, declining toward the east. For widening of these embankments it is recommended that any topsoil within the footprint of the embankment widening be stripped.

Benching the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (*Benching of Earth Slopes*). The widened Highway 401 embankment side slopes should be formed at a maximum gradient of 2H:1V. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be completed as soon as practicable after widening of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (*Seed and Cover*).

6.6.2 Global Slope Stability

Preliminary static slope stability analyses have been completed for the widened highway approach embankments, using the commercially available program *Slide 6.0* from RocScience, to check that the target minimum factor of safety is achieved. A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions (CHBDC 2006). This minimum factor of safety is considered appropriate for the proposed slope widening on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses of an overall 5.5 m high abutment in long-term drained (effective stress) conditions, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m³)	Effective Friction Angle (°)
Fill	21	32
Compact silt to sand	21	32
dense to very dense sand and gravel	21	34
Very dense glacial till	21	34

The results of the static global stability analysis indicate that a minimum factor of safety of 1.3 is achieved for an approximately 5.5 m high embankment, oriented no steeper than 2H:1V, as shown on Figure 1. This preliminary assessment of the slope stability of the embankments should be reviewed and confirmed during detailed design, based on additional subsurface information as may be available at that time.

6.6.3 Settlement Under Widened Embankment Loading

Preliminary settlement assessment has been completed for the embankment widening using the commercially available computer program *Settle-3D 2.0* from Rocscience, using the estimated elastic deformation moduli given



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in the table below, based on correlation with the SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984 and Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Fill	21	-
Compact silt to sand	21	50
Dense to very dense sand and gravel	21	100
Very dense glacial till	21	150

Based on this preliminary assessment, the settlement of the foundation soils under the embankment widening is estimated to be approximately 10 mm to 15 mm. This settlement will occur immediately during construction.

The above preliminary estimate does not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for the embankment construction, settlement of the fill itself is expected to occur essentially during the embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement overtime.

6.7 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design, and for which provision may be required in the contract documents produced as part of detail design.

6.7.1 Open-Cut Excavation and Temporary Protection Systems

If adopted at the abutments, the construction of the new strip footings for the widening of the existing structure will require excavations up to about 3.4 m below the ground surface in front of the abutments, and deeper through the existing Highway 401 embankment fill. The existing fill and upper loose to compact silt to sand are classified as Type 3 soils, while the native compact to very dense soils are classified as Type 2 soils (provided these soils are dewatered), according to the Occupational Health and Safety Act (OHSA). Temporary excavations should be made with side slopes no steeper than 1H:1V.

Temporary protection systems will be required along the existing Highway 401 lanes to facilitate excavation through the existing highway embankment for widening of the abutments. Protection systems/cofferdams will also be required parallel to the foundation elements/creek. These temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the protection systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor. However, for preliminary planning purposes, it is noted that the very dense (“100-blow”) till soils that are present at or immediately below the existing footing founding levels will likely pose some challenges for driven steel sheetpiles.



6.7.2 Groundwater Control

Assuming that shallow foundations are adopted for support of the bridge widening, excavation for their construction will extend to approximately Elevation 89.6 m to 88.3 m. The groundwater level was generally measured to be between Elevation 89.6 m and 91.6 m during the 1972/1973 investigation, although this may not represent the stabilized or current groundwater level at the site. It is anticipated that the footing founding elevation will extend below the groundwater level at the site.

At this preliminary stage, it is recommended that dewatering be assumed to be required, potentially in conjunction with cofferdams to create a groundwater cut-off through the more permeable upper deposits. An active dewatering system (such as wellpoints or eductors, given the fine-grained nature the silt and till deposits) will be needed to lower the groundwater level to approximately 1 m below the proposed founding elevation and ensure a stable base. Further investigation and monitoring of the groundwater level will be required during detailed design to confirm the current groundwater levels and determine the extent of dewatering that would be required for construction.

The water bearing soils at the proposed founding elevations are generally fine to medium grained in nature (silty to sandy) and therefore will have a lower to moderate permeability. At this preliminary stage, an accurate prediction of the groundwater pumping rates cannot be made, but it is considered possible for pumping volumes to exceed 50 m³/day during initial drawdown stages and/or if multiple excavation areas are being dewatered at one time.

It is anticipated that the zone of influence for the dewatering operation would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the compact to very dense silt to sand deposits at the site. However, the potential for settlement impacts on the existing and/or new foundation structures and any adjacent utilities should be re-assessed once the groundwater levels are confirmed.

6.7.3 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in any future investigation at this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction.

6.7.4 Vibration Monitoring During Pile or Caisson Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition; lower thresholds are applicable for nearby residential and commercial facilities (between 25 mm/s and 50 mm/s). Vibration monitoring is recommended if caissons are adopted and for temporary protection system installation, to demonstrate/confirm that vibration levels do not exceed the threshold levels.

6.8 Recommendations for Further Work in Detail Design

During the detailed design phase, additional borehole investigation work is recommended to confirm the stabilized groundwater level at the site, to further assess groundwater control and protection system requirements. It is recommended that additional investigation be completed in the vicinity of foundation elements for the larger southward widening of the eastbound lanes, to confirm the soil conditions, founding elevations (or subexcavation/tremie concrete requirements) and geotechnical resistances in this area. If caisson foundations



PRELIMINARY FOUNDATION REPORT OSHAWA CREEK BRIDGE, W.O. 10-20011

are adopted, deeper boreholes will be required at the applicable foundation elements to investigate to a sufficient depth below the caisson base elevation to confirm artesian groundwater pressures, the risk of basal instability, ground and groundwater control requirements, and geotechnical resistance.

As noted throughout this report, the preliminary assessment of founding elevation, geotechnical resistances, and global stability analysis should be revisited based on the additional borehole information, geometry and other requirements at the detailed design stage.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., EIT, and was reviewed by Ms. Nikol Kochmanova, P.Eng. Ms. Lisa Coyne, P.Eng., Designated MTO Foundations Contact and a Principal of Golder, conducted an independent technical and quality review of this report.

GOLDER ASSOCIATES LTD.

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Geotechnical Group



Nikol Kochmanova, P.Eng.
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Lisa Coyne, P.Eng.
Designated MTO Foundations Contact, Principal

MGP/NK/LCC/sm

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PRELIMINARY FOUNDATION REPORT OSHAWA CREEK BRIDGE, W.O. 10-20011

REFERENCES

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Ontario Provincial Standard Specifications (OPSS)

- | | |
|---------------|---|
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV 804 | Construction Specification for Seed and Cover |

Ontario Provincial Standard Drawings (OPSD)

- | | |
|---------------|--|
| OPSD 208.010 | Benching of Earth Slopes |
| OPSD 3090.101 | Foundation Frost Depths for Southern Ontario |

Other:

- | | |
|------------------------|------------------------------------|
| Ontario Regulation 213 | Construction Projects (as amended) |
|------------------------|------------------------------------|



PRELIMINARY FOUNDATION REPORT OSHAWA CREEK BRIDGE, W.O. 10-20011

TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footing	<ul style="list-style-type: none"> Recommended for support of widened abutments and piers Compatible with existing structure 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Dense to very dense soils (with SPT “N” values greater than 30 blows per 0.3 m of penetration) present at a reasonable depth below existing ground surface, with good geotechnical resistance and settlement performance Existing abutments supported on shallow foundations, and have performed well 	<ul style="list-style-type: none"> Significant excavations below Highway 401 grade and original ground surface Excavations will extend below the groundwater level at the site, and groundwater control will be required Temporary protection systems required along Highway 401, and parallel to foundation elements/creek Potential for differential settlement between existing structure and widened abutments, although this is estimated to be less than 10 mm 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, plus the cost of protection systems and dewatering 	<ul style="list-style-type: none"> Minor to moderate risks (but conventional construction practices) associated with deep excavations, protection systems and dewatering for this site
Steel H-piles or pipe piles founded in very dense glacial till	<ul style="list-style-type: none"> Feasible for support of widened abutments but not compatible with existing foundation structure Not recommended for support of widened piers, as “100-blow” soil is present immediately below pile cap level 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system and groundwater control requirements 	<ul style="list-style-type: none"> Not compatible with existing structure Pre-augering would be required due to very dense soils present at relatively shallow depths, particularly on the west abutment Relatively short piles would have relatively lower geotechnical resistances in soil; 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction, plus the cost of protection systems and dewatering 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving



PRELIMINARY FOUNDATION REPORT OSHAWA CREEK BRIDGE, W.O. 10-20011

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
			<p>higher capacities might be achieved, but additional investigation will be required.</p> <ul style="list-style-type: none"> • Potential for damage during driving through very dense soils • Temporary excavation support still required • Design geotechnical resistance may not be achieved if piles refuse on cobble and boulder layers 		
Caissons	<ul style="list-style-type: none"> • Feasible for support of widened abutments • Feasible for support of widened piers, and may be advantageous if pile cap can be eliminated 	<ul style="list-style-type: none"> • Conventional construction methods • Abutment pile caps could be maintained higher than spread footings potentially reducing depth of excavation, protection system and groundwater control requirements • May allow pile cap to be eliminated at pier locations, minimize excavation, protection system/cofferdam and dewatering requirements 	<ul style="list-style-type: none"> • Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance; must address artesian groundwater pressures • Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base 	<ul style="list-style-type: none"> • Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary liners, plus the cost of protection systems and dewatering for pile caps 	<ul style="list-style-type: none"> • Risk of loss of ground during construction of caissons through water bearing layers and at the base, although the risk can be mitigated with the use of temporary liners, drilling slurry and the placement of concrete by tremie methods

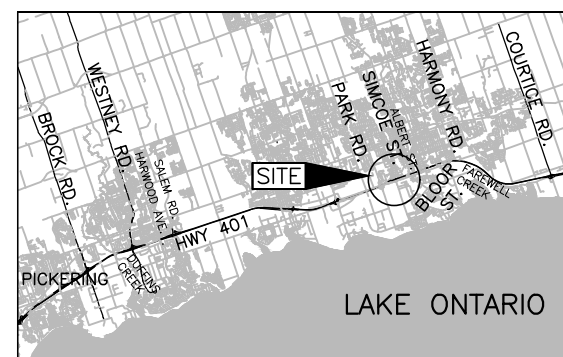
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 10-20011



OSHAWA CREEK BRIDGE
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS

SHEET



KEY PLAN

SCALE

4 0 4 8 km

LEGEND

Borehole - 1972 to 1973 Investigation (Geocres No. 30M15-033)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
033-1	90.2	4860300.6	356380.7
033-3	92.9	4860293.8	356395.2
033-4	97.7	4860306.2	356393.8
033-5	91.2	4860294.4	356361.4
033-6	91.1	4860285.5	356360.9
033-7	99.0	4860289.7	356345.7
033-9	94.8	4860330.7	356334.3
033-10	98.9	4860320.9	356336.8
033-12	89.8	4860327.2	356350.5
033-14	90.1	4860333.3	356367.1
033-15	93.1	4860345.9	356378.3
033-16	97.8	4860336.8	356382.6

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by AECOM, drawing file no. Oshawa Creek Bridge GA.dgn.



NO.	DATE	BY	REVISION
Geocres No. 30M15-300			
HWY. 401		PROJECT NO. 11-1184-0143	
SUBM'D. MGP		CHKD. MGP	DATE: 4/5/2017
DRAWN: DD		CHKD. NK	APPD. LCC
			DWG. 1

PLAN

SCALE

5 0 5 10 m



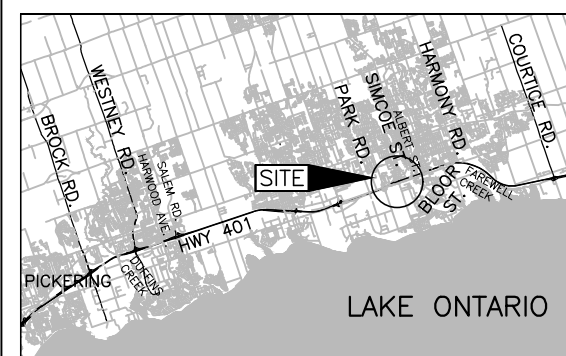
CONT No.
WO No. 10-20011

OSHAWA CREEK BRIDGE

HIGHWAY 401 IMPROVEMENTS

SOIL STRATA

SHEET



KEY PLAN

SCALE

4 0 4 8 km

LEGEND

- Borehole — 1972 to 1973 Investigation (Geocres No. 30M15-033)
 N Standard Penetration Test Value
 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 ▼ WL upon completion of drilling, 1973
 * Artesian water encountered at the elevation shown

BOREHOLE CO--ORDINATES			
No.	ELEVATION	NORTHING	EASTING
033-1	90.2	4860300.6	356380.7
033-3	92.9	4860293.8	356395.2
033-4	97.7	4860306.2	356393.8
033-5	91.2	4860294.4	356361.4
033-6	91.1	4860285.5	356360.9
033-7	99.0	4860289.7	356345.7
033-9	94.8	4860330.7	356334.3
033-10	98.9	4860320.9	356336.8
033-12	89.8	4860327.2	356350.5
033-14	90.1	4860333.3	356367.1
033-15	93.1	4860345.9	356378.3
033-16	97.8	4860336.8	356382.6

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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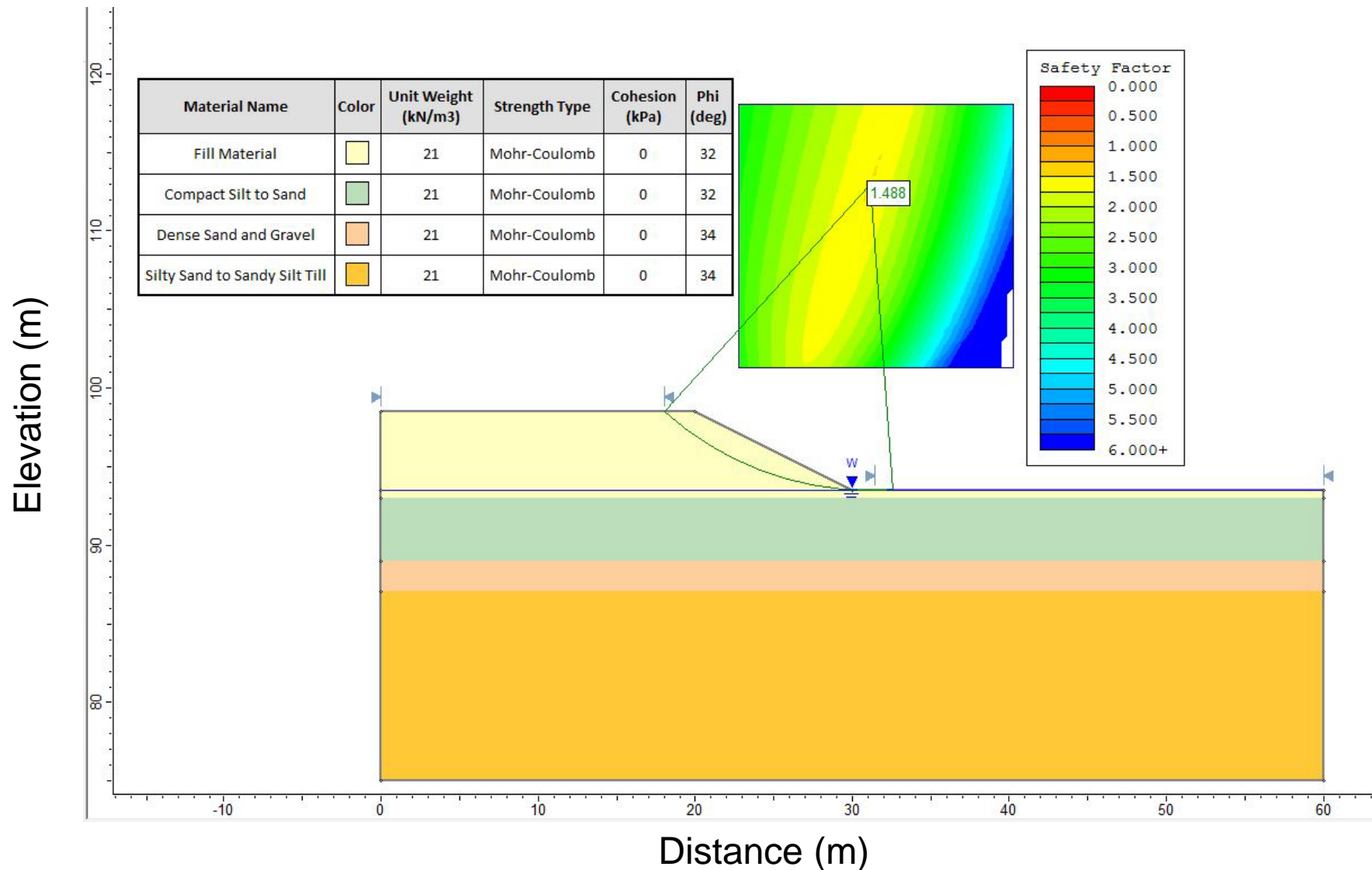


NO.	DATE	BY	REVISION
Geocres No. 30M15-300			
HWY. 401		PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. MGP	CHKD. MGP	DATE: 4/5/2017	SITE: 22-175
DRAWN: DD	CHKD. NK	APPD. LCC	DWG. 2



STATIC GLOBAL STABILITY OSHAWA BRIDGE CREEK – APPROACH EMBANKMENTS

Figure 1





APPENDIX A

Borehole Records and Laboratory Test Results GEOCRES No. 30M15-033 – 1972 to 1973 Investigation

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-1

JOB 73-11022

LOCATION Co-ords. 15,945,138 N; 1,169,182 E.

ORIGINATED BY EAW

W.P. 44-71-08

BORING DATE Dec. 5, 1972

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 25 50 75 100 125	LIQUID LIMIT —WL PLASTIC LIMIT —WP WATER CONTENT —W Wp — W — WL WATER CONTENT %	BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
90.2	Ground Level									
0.0	Sandy gravel, some silt.		1	SS	28					47 45 (8)
87.9	Brown Compact to Dense		2	SS	41	290				44 30 (26)
2.3	Silt with sand, traces of gravel.		3	SS	100	6"				
			4	SS	100	2"				
	Very Dense		5	SS	100	4"	280			3 37 (60)
	Grey		6	SS	100	2"				
82.7	(Till)		7	SS	100	6"				49 (44)
			8	SS	100	6"				Artesian
7.5	End of Borehole					270				water encountered

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-3

JOB 73-11022 LOCATION Co-ords. 15,945,116 N; 1,169,228 E.

W.P. 44-71-08 BORING DATE December 8, 1972

DATUM Geodetic BOREHOLE TYPE Washboring

ORIGINATED BY EAW

COMPILED BY FK

CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT _____		LIQUID LIMIT _____w _L PLASTIC LIMIT _____w _p WATER CONTENT _____w		BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT % w _p w w _L			
92.9	Ground Level										P.C.F.	GR.SA.SI.CL.
0.0	Brown Sand with silt, traces of organics. Loose to Very Dense	1	SS	10	300						WL Estimated 89.7 m 0 71 (29) 2 50 (48)
			2	SS	11							
			3	SS	7							
			4	SS	33							
			5	SS	100/6"		290					
87.7			6	SS	100/6"							
5.2 86.5	(Till) Silty sand, trace of gravel. Grey. V. Dense											
6.4	End of Borehole Water level not established					280						

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-4

JOB 73-11022 LOCATION Co-ords. 15,945,156 N; 1,169,224 E. ORIGINATED BY PK
W.P. 44-71-08 BORING DATE May 2 to 3, 1973 COMPILED BY PK
DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY PK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.			WATER CONTENT %				
							W_p	W	W_L	10	20	30		
97.7	Ground Level					320								GR.SA.SI.CL
0.2	Topsoil		1	SS	75									24 49 (27)
	Brown (Fill)		2	SS	77	310								
	Sand, some gravel and silt.		3	SS	53									29 53 (18)
92.2	Very dense		4	SS	16									5 52 (13)
5.5	Silty sand, traces of gravel & organics. Compact.		5	SS	11	300								
89.6	Grey to Dark Grey		6	SS	31									38 52 (10)
8.1	Sand with gravel, some silt. Grey		7	SS	61	290								50 44 (6)
87.9	Dense to Very Dense (Till)		8	SS	100.4"									19 56 (25)
9.8	Silt with sand, traces of gravel. Grey		9	SS	100.6"									
85.3	Very Dense		10	SS	100.4"									4 33 (63)
12.4	End of Borehole													

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-5

JOB 73-11022 LOCATION Co-ords. 15,945,118 N; 1,169,118 E.
 W.P. 44-71-08 BORING DATE May 4 and May 10, 1973
 DATUM Geodetic BOREHOLE TYPE Auger and Washboring

ORIGINATED BY PK
 COMPILED BY PK
 CHECKED BY JK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w w_p — w — w_L WATER CONTENT % 10 20 30			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
91.2	Ground Level														
0.0	Brown Sand, some silt, gravel, traces of organics. Compact		1	SS	22	290								8 71 (21)	
89.1			2	SS	13										
2.1	Silt with sand, traces of gravel. Compact to Very Dense Grey (Till)		3	SS	140										8 35 (57)
			4	SS	105.9"										
			5	SS	100.9"										
			6	SS	151	280									
83.1		7	SS	155									4 37 (59)		
8.1	End of Borehole					270									Artesian water encountered

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-6

JOB 73-11022

LOCATION Co-ords. 15,945,090 N; 1,169,118 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 9 and 10, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT			BULK DENSITY	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT					WATER CONTENT %						
						25	50	75	100	125	SHEAR STRENGTH P.S.F.			WATER CONTENT %				
											○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			W _L ——— PLASTIC LIMIT ——— W _P W _L ——— W ——— W _P				
91.1	Ground Level																	
0.2	Sand with gravel, some silt.																	
88.1	Brown Compact		1	SS	26													
3.0	Sandy silt, traces of gravel.		2	SS	21													
	Very Dense		3	SS	100/10"													
	Grey		4	SS	100/10"													
	(Till)		5	SS	100/6"													
84.0			6	SS	100/10"													
7.1	End of Borehole																	

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-7

JOB 73-11022 LOCATION Co-ords. 15,945,102 N; 1,169,066 E.
 W.P. 44-71-08 BORING DATE May 3 and 4, 1973
 DATUM Geodetic BOREHOLE TYPE Washboring

ORIGINATED BY PK
 COMPILED BY PK
 CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			WATER CONTENT % 10 20 30				
99.0	Ground Level													GR. SA. SI. CL.
0.0	Gravelly sand. Compact Fill	⊗				320								
27.4 1.6	Silt with sand, traces of gravel (fill) Brown	⊗	1	SS	18									
		⊗	2	SS	35									
		⊗	3	SS	7	310								3 26 (71)
93.5	Loose to Dense.	⊗												
5.5	Sand with silt, traces of gravel & organics.	⊙	4	SS	7									
	Loose to Dense	⊙	5	SS	14	300								3 62 (35)
		⊙	6	SS	31									
89.9	Dark Grey	⊙	7	SS	12									
9.1	Silt with sand, traces of gravel. Grey (Till)	⊙	8	SS	100/20"	290								2 39 (59)
87.8	Very Dense	⊙	9	SS	14 17/5"									
11.3	End of Borehole													
						280								

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-9

JOB 73-11022

LOCATION Co-ords. 15,945,232 N; 1,169,030 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 8 and 9, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY


SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT ——— w_L			BULK DENSITY	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT			PLASTIC LIMIT ——— w_p						
							25	50	75	100	125	WATER CONTENT — w				
						SHEAR STRENGTH P.S.F.			w_p ——— w ——— w_L			WATER CONTENT %				
						○ UNCONFINED + FIELD VANE						10 20 30				
						● QUICK TRIAXIAL × LAB VANE										
94.8	Ground Level													P.C.F.	GR. SA. SI. CL.	
0.0	Sand, some silt and gravel.	X X X				310										
	Loose Brown		1	SS	4											
91.8	Fill		2	SS	10											
3.0	Sand with silt, traces of organics.	• • •	3	SS	10	300									0 61 (39)	
	Dark Brown		4	SS	13											
89.6	Compact		5	SS	17											1.4 organic content
5.2	Silt with sand, traces of gravel.		6	SS	100	290									3 37 (60)	
	Very Dense		7	SS	157											
	Grey		8	SS	100											
84.7	(Till)					280									1 40 (59)	
10.1	End of Borehole					270										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE N^o 033-10

JOB 73-11022 LOCATION Co-ords. 15,945,202 N; 1,169,038 E. ORIGINATED BY PK
 W.P. 44-71-08 BORING DATE May 3, 1973 COMPILED BY PK
 DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY PK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			WATER CONTENT % w_p — w — w_L						
99.0	Ground Level										10	20	30	P.C.F.	GR. SA. SI. CL.	
0.0	Silt with sand, traces of gravel. Loose to Dense Brown (Fill)		1	SS	6	320						○				8 36 (56)
			2	SS	11											
			3	SS	12	310						○				5 38 (57)
			4	SS	32											
91.6			5	SS	11	300						○				
7.4	Sandy silt, traces of gravel & organics. Compact Grey		6	SS	18											 5 43 (52) organic content
89.5			7	SS	9										8.4	
9.5	Silt with sand, traces of gravel. Grey Very Dense (Till)		8	SS	100/2"	290						○				1 35 (64)
			9	SS	100/6"											
86.4			10	SS	100/9"											
12.6	End of Borehole					280										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-12

JOB 73-11022 LOCATION Co-ords. 15,945,220 N; 1,169,082 E.
 W.P. 44-71-08 BORING DATE May 10, 1973
 DATUM Geodetic BOREHOLE TYPE Auger

ORIGINATED BY PK
 COMPILED BY PK
 CHECKED BY PK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	W _p	W _L	W _p		
89.8	Ground Level												
0.0	Silty sand, traces of gravel. Grey		1	SS	100	290							
87.6	Very Dense (Till)		2	SS	100	290							1 54 (45)
2.2	End of Borehole												
						280							

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-14

JOB 73-11022

LOCATION Co-ords. 15,945,240 N; 1,169,136 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 10, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
90.1	Ground Level															
89.5	Gravelly sand. Grey Fill	X														
0.6	Silt with sand, traces of gravel. Grey		1	SS	100	290										4 33 (63)
			2	SS	100/26"											
			3	SS	100/27"											
	Very Dense		4	SS	100/28"											4 33 (63)
84.6	(Till)		5	SS	100/30"	280										
5.5	End of Borehole					270										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-15

JOB 73-11022

LOCATION Co-ords. 15,945,280 N; 1,169,175 E.

ORIGINATED BY PK

W.P. 14-71-08

BORING DATE May 7 and 8, 1973

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 25 50 75 100 125	LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W W _P — W — W _L	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE					
93.1	Ground Level								
0.0	Sandy silt.								
	Loose		1	SS	8				0 47 (53)
89.4	Brown		2	SS	9				
3.7	Sand with gravel, traces		3	SS	17				32 55 (13)
88.5	of silt. Compact		4	SS	72				12 51 (37)
4.6	Sand with silt, some gravel. (Till)		5	SS	100	5"			
	Grey								
85.4	Very Dense		6	SS	100	6"			9 90 (1)
7.7	Sand, traces of gravel		7	SS	100	8"			
84.4	& silt. Very Dense								
8.7	End of Borehole								

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 033-16

JOB 73-11022

LOCATION Co-ords. 15,945,252 N; 1,169,188 E.

ORIGINATED BY PK

W.P. 44-71-08

BORING DATE May 4 to May 7, 1973

COMPILED BY PK

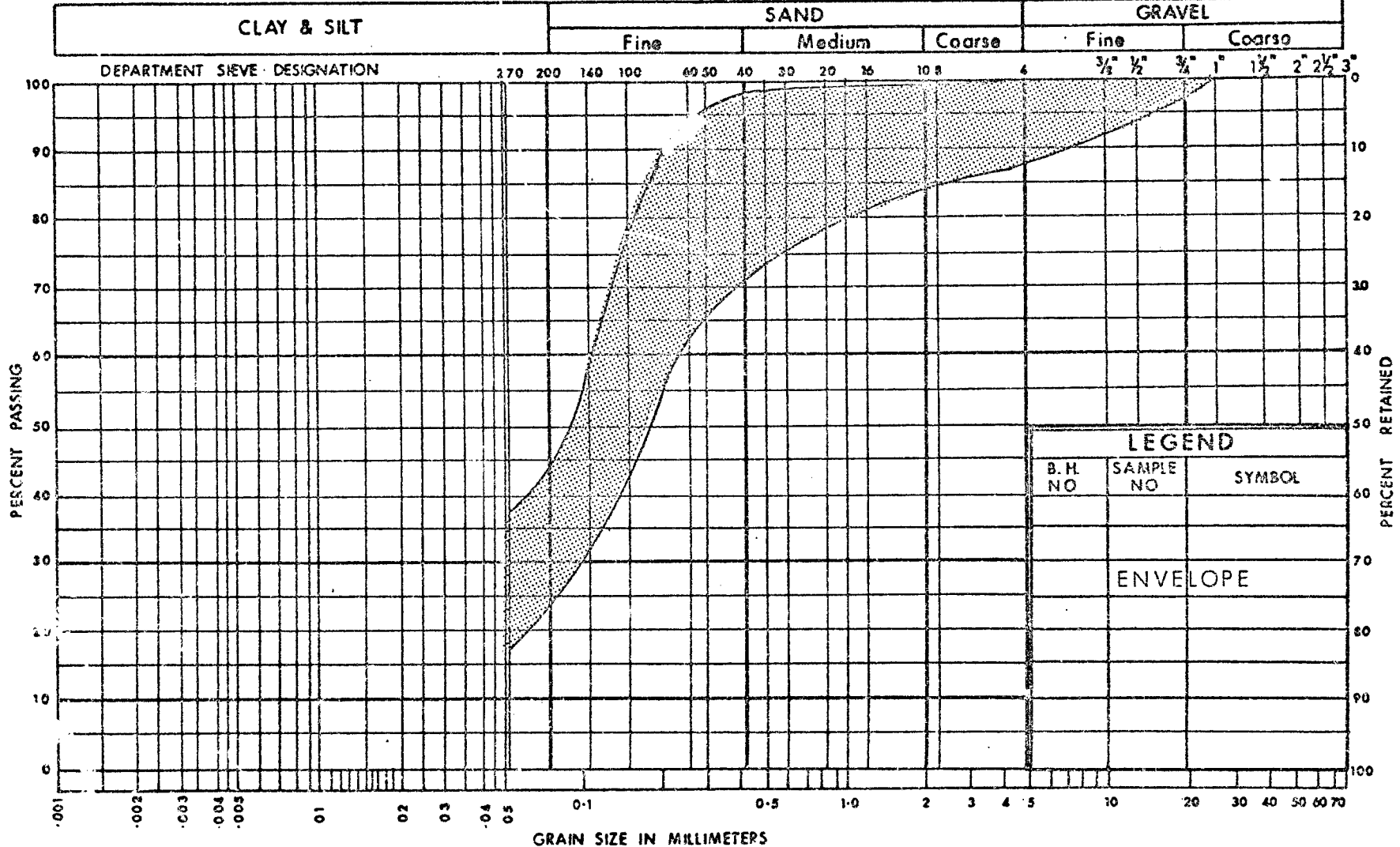
DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.				WATER CONTENT %				
							\circ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				w_p w w_L 10 20 30				
97.8	Ground Level														
0.0	Sand with gravel, some silt. (Fill)					320									
			1	SS	28										
			2	SS	10										34 48 (18)
92.6	Brown Compact		3	SS	9										
5.2	Sandy silt, traces of gravel.		4	SS	17	300									
90.1			Compact	5	SS	14									 3 43 (54)
7.7	Sand with gravel, traces of silt. Grey		6	SS	57										
87.7			Very Dense	7	SS	100 2"	290								36 54 (10)
			8	SS	100 2"										
10.1	End of Borehole					280									

UNIFIED SOIL CLASSIFICATION SYSTEM



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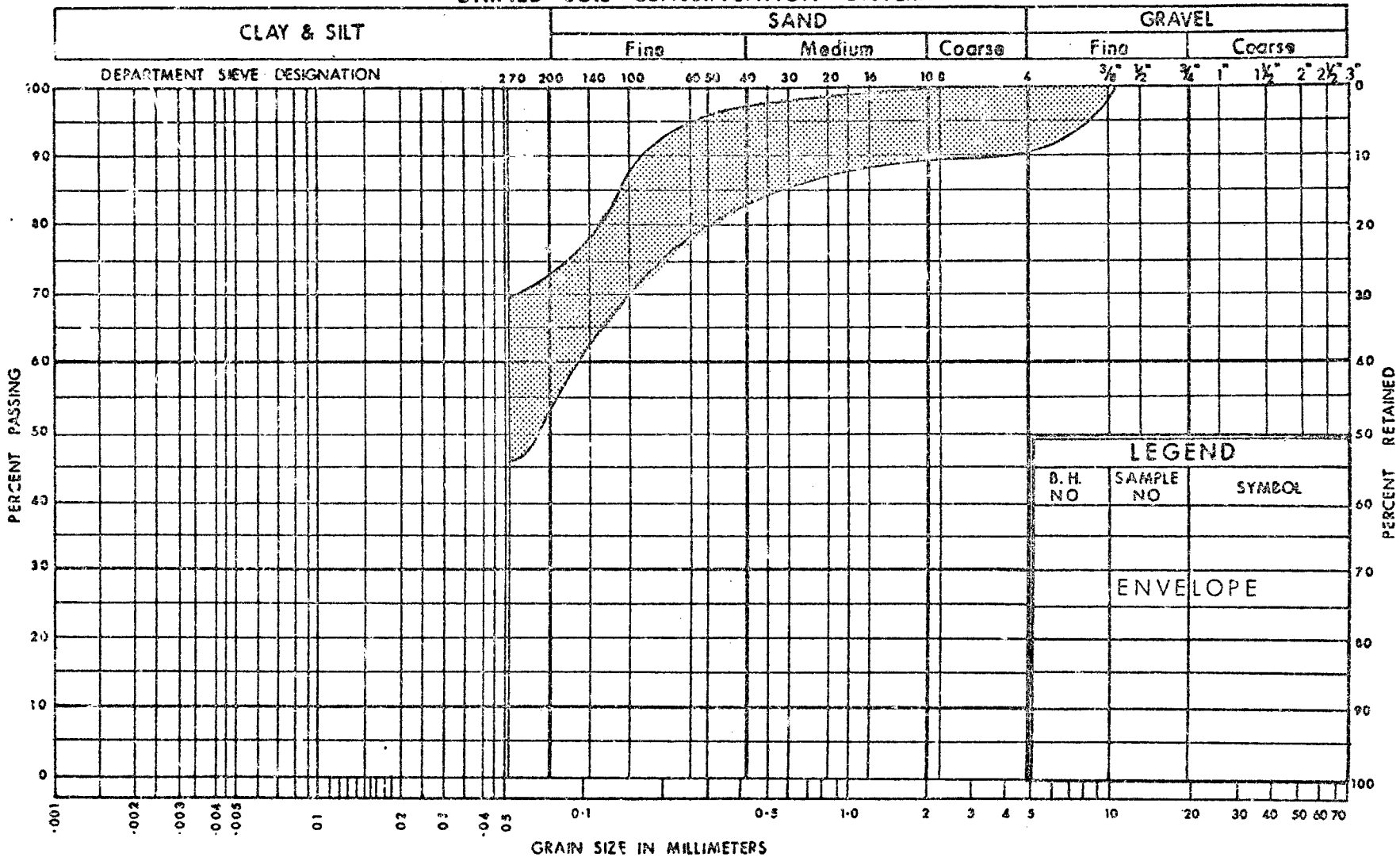
GRAIN SIZE DISTRIBUTION
SAND
WITH SILT, TRACES OF GRAVEL

W.P. No. 44-71-08

JOB No. 73-11022

FIG.2

UNIFIED SOIL CLASSIFICATION SYSTEM

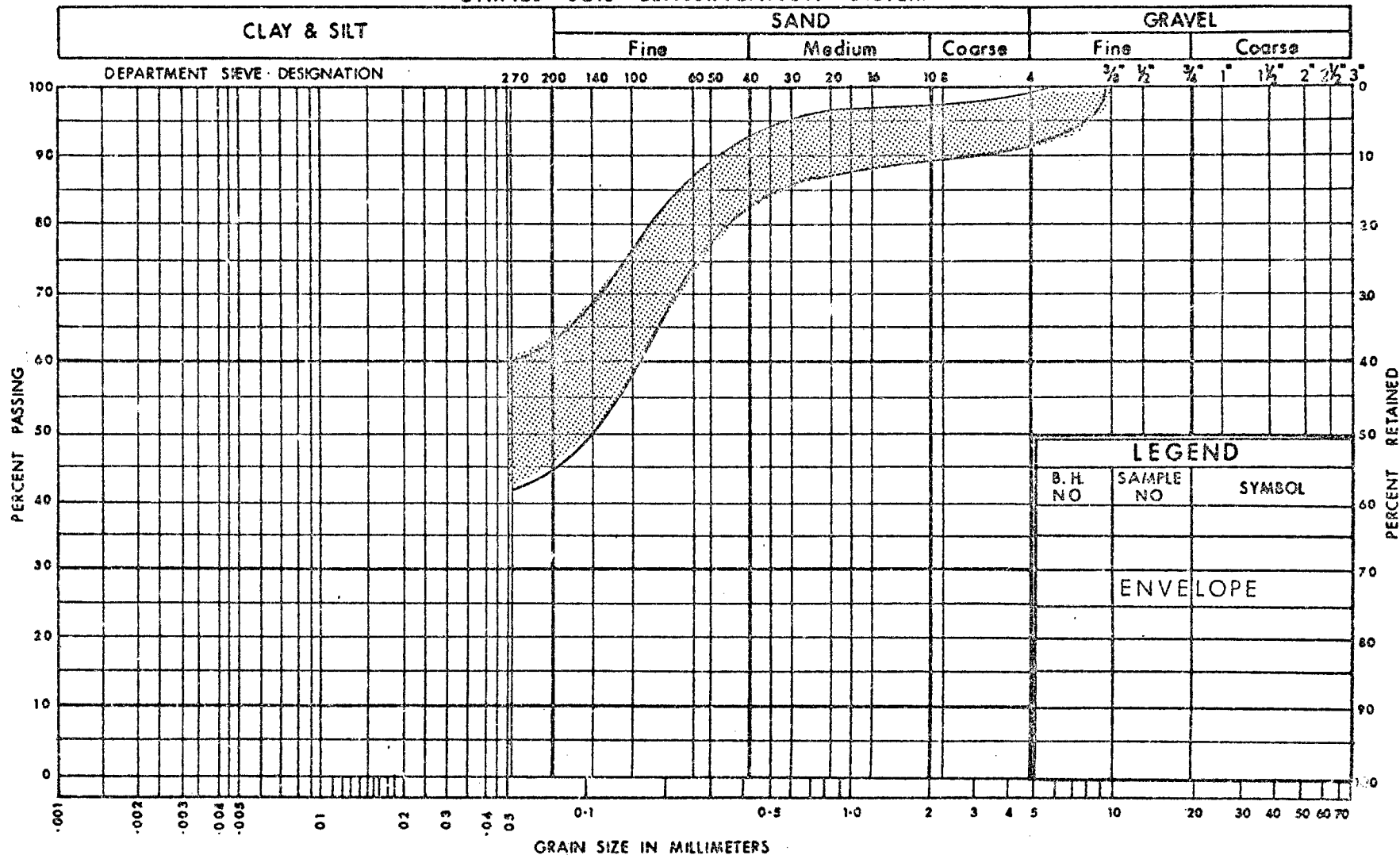


DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
SILT
WITH SAND, TRACES OF GRAVEL

W.D. No. 44-71-08
JOB No. 73-11022
FIG.3

UNIFIED SOIL CLASSIFICATION SYSTEM



DESIGN SERVICES
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GRAIN SIZE DISTRIBUTION
SILT (GLACIAL TILL)
WITH SAND, TRACES OF GRAVEL

W.P. No. 44-71-08

JOB No. 73-11022

FIG. 4

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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