April 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

JAMES SNOW PARKWAY UNDERPASS ASSESSMENT, HIGHWAY 401 WIDENING FROM TRAFALGAR ROAD TO REGIONAL ROAD 25
HALTON REGION
W.O. 07-20024

Submitted to:
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REPORT

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JAMES SNOW PARKWAY UNDERPASS ASSESSMENT
HIGHWAY 401 WIDENING, TRAFALGAR ROAD TO REGIONAL ROAD 25
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future widening of Highway 401 from Regional Road 25 to Trafalgar Road (approximately 9 km) in the Regional Municipality of Halton, Ontario.

This report addresses the assessment of the James Snow Parkway underpass structure. The terms of reference and scope of work for the preliminary foundation engineering services for this structure site are outlined in MTO's amended Terms of Reference dated September 15, 2011, and in Golder's scope change letter dated October 06, 2011.

A previous foundation investigation for the existing underpass structure was carried out by MTO in 1979, and the factual data from this previous investigation report (referenced below) was used in the preparation of this preliminary report.

- Ministry of Transportation, Ontario, "Foundation Investigation Report for James Snow Parkway Underpass", W.P. 27-78-02, Site 10-312, Hwy. 401, District 6, Toronto, GEOCRES No. 30M12-138, dated May 25, 1979.

2.0 SITE DESCRIPTION

The existing Highway 401/James Snow Parkway Interchange is a Parclo A-4 interchange located approximately 1.4 km west of Highway 401 at 5th Line; and 1.2 km east of Highway 401 at Steeles Avenue in the Regional Municipality of Halton, Ontario. At this site Highway 401 is a six-lane divided freeway and James Snow Parkway crosses Highway 401 via a two-span underpass.

A hydro corridor is oriented generally parallel to and about 125 m east of James Snow Parkway. The Milton Crossroads shopping centre is located at the northwest quadrant of this interchange and a residential subdivision exists in the southwest quadrant of the interchange. The topography across the site is flat with light vegetation consisting of grass and small shrubs.

3.0 INVESTIGATION PROCEDURES

MTO undertook the field work for the James Snow Parkway Underpass on April 17 and April 18, 1979. Three boreholes were drilled and sampled to depths ranging from 9.6 m to 15.7 m and their locations are shown on the Borehole Locations and Soil Strata Drawing (Drawing No. 277802-A) in Appendix C.

The boreholes were advanced with a continuous flight auger machine equipped with hollow stem augers. Samples of the overburden soils were obtained with a 50 mm outer diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the physiographic region of Southern Ontario referred to as the Peel Plain whose topography slopes gradually and gently towards Lake Ontario. Etobicoke Creek and other rivers have cut deep valleys across the Peel Plain.

The Peel Plain is known to consist of generally clayey and silty soils that cover the central portion of the regions of York, Peel and Halton. There are exceptions to be noted in these major soil groups. Trains of sandy alluvium can be found at various places in the stream valleys. These overburden soils at this site are underlain by the Queenston Formation.

4.2 Subsurface Conditions

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings. An overall description of the stratigraphy at this site is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic section (Drawing No. 277802-A) are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface soils at this site as encountered in Boreholes 1, 2 and 3 consist of granular roadway fill and a very stiff to hard glacial till deposit consisting of a clayey silt matrix with some sand, trace of gravel with occasional weathered shale partings. A 0.6 m thick layer of granular roadway fill was encountered at ground surface in Borehole 2 only. Bedrock was not encountered in any of the borings.

4.2.1 Granular Fill

Granular roadway fill extending to a depth of 0.6 m (Elev. 206.1 m) below ground surface was encountered at this site in Borehole No. 2.

4.2.2 Clayey Silt Till

The site is underlain by a clayey silt till deposit that extends at least to borehole termination depths ranging from 9.6 m to 15.7 m below ground surface or to elevations ranging from 189.4 m to 197.1 m.

The till generally consists of a clayey silt matrix containing some sand, trace gravel and occasional weathered shale partings. In Borehole 1 a 0.8 m thick silty sand layer and a 0.4 m thick layer of weathered shale partings were encountered. Cobbles and boulders zones ranging in thickness from 0.4 m to 0.8 m were encountered in Boreholes 2 and 3.

Six samples of the clayey silt till were subjected to grain size distribution tests and the grain size distribution envelope is illustrated in Figure 2 contained in Appendix B. The grain size distribution envelope is summarized below; till soils can also be expected to contain random cobble and boulder inclusions.

Soil Constituent	Per Cent By Mass
Gravel	6% – 22%
Sand	19% – 24%
Silt	47% – 56%
Clay-size	10% – 18%

Atterberg Limits tests were carried out on nine samples of the clayey silt till and the results shown on the plasticity chart (Figure 1, Appendix B) indicate a generally low plasticity (CL-ML to CL) clayey silt matrix. The results from these tests are summarized below.

Liquid Limit:	16% – 29 %
Plastic Limit:	13% – 19 %
Plasticity Index:	3% – 10 %
Natural Moisture Content:	9% – 18 %

The Standard Penetration Test ‘N” values measured in the clayey silt till range from 21 blows to 121 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

4.3 Groundwater Conditions

During the boring operations of April 1979, unstabilized water levels of 6.1 m (Elevation 198.8 m) and 6.4 m (Elevation 198.7 m) were recorded in Boreholes 1 and 3 respectively. Perched water is also expected to occur in the more permeable silty sand and cobble and boulder zones.

5.0 CLOSURE

The factual data provided in this Preliminary Foundation Investigation Report was extracted from the following report:

- "Foundation Investigation Report for James Snow Parkway Underpass, W.P. 27-78-02, Site No. 10-312, Highway 401, District 6, Toronto", GEOCRES No. 30M12-138, prepared by Ministry of Transportation, Ontario, dated May 25, 1979.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
JAMES SNOW PARKWAY UNDERPASS ASSESSMENT
HIGHWAY 401 WIDENING, TRAFALGAR ROAD TO REGIONAL ROAD 25
HALTON REGION
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This Preliminary Foundation Design Report presents a comparison of various retaining structure options to assess the feasibility and constructability of cutting into the abutment foreslopes at the James Snow Parkway underpass, to fit a widened Highway 401 cross-section without compromising the integrity of the existing "perched" abutment footings. The discussion and recommendations presented in this report are based on the existing factual data obtained from a previous investigation at this site (MTO GEOCRES No. 30M12-138). These geotechnical recommendations are for planning and preliminary design purposes only as part of the assessment of the feasibility and constructability of potential retaining structures. Further investigations and geotechnical and structural analyses will be required during detail design.

The James Snow Parkway underpass is approximately 76.5 m long and 32 m wide. The bridge spans are approximately 38 m between the abutments and centre pier and the finished James Snow Parkway grades are about Elevation 214.1 m and Elevation 214.3 m at the north and south abutments respectively.

The structural design drawings for the existing James Snow Parkway underpass, which are contained in Appendix D for reference, provide the following foundation data:

- The north and south abutments are supported on spread footings approximately 32.5 m long and 4.1 m wide that are "perched" within the James Snow Parkway approach embankments above the Highway 401 grade. The underside elevations of the north and south abutment footings are Elevation 207.9 m and Elevation 208.5 m respectively, relative to the Highway 401 grade which varies from about Elevation 206 m on the north side of the westbound lanes, to about Elevation 207 m on the south side of the eastbound lanes.
- Both abutment footings were constructed on a Granular "A" pad that is estimated from the design drawings to be approximately 3.0 m thick.
- The abutment foreslopes are sloped at approximately 2 horizontal to 1 vertical (2H:1V).
- Four spread footings each measuring approximately 6.5 m x 6.5 m support the centre pier.

Based on the planning and preliminary design study completed to date for the widening of Highway 401 from Regional Road 25 to Trafalgar Road in the Regional Municipality of Halton, it is understood that the future widening at this site will consist of two additional lanes in both the eastbound and westbound directions on Highway 401. In order to fit the widened highway platform through the existing James Snow Parkway underpass structure, it would be necessary to cut into the existing abutment foreslopes and construct permanent retaining structures. It is understood that depending on the approach geometry for the speed change lanes, the proposed retaining structures could be located at distances ranging from approximately 0.5 m to 5.0 m in front of the front edge of the abutment footing. The vertical wall height would vary from about 4 m to 1.5 m or shallower, depending on the proximity of the proposed retaining structures to the front edge of the existing abutment footings.

6.2 Design and Constructability Considerations

The basic approach to designing excavation support and retaining structures includes:

- Designing for the minimum requirements to satisfy load carrying capacity (lateral earth pressures, hydrostatic pressures and imposed foundation loads) and global system stability; and
- Designing to control deformations.

For this site, the selection of appropriate, practical and feasible temporary and permanent retaining systems in front of the “perched” abutment footings at the James Snow Parkway underpass will depend on a number of considerations, key among them the following:

- The selected retaining structure type(s) must be constructable within the limited available headroom beneath the underside of the existing bridge deck. Based on the existing Highway 401 grade and the underside elevation of the bridge deck, it is estimated that there is a maximum of approximately 6 m of headroom in front of the north abutment, and about 4.7 m of headroom in front of the south abutment, although this available height is reduced further by the presence of the existing abutment foreslope. It is noted that it may be possible to excavate narrow slots (oriented perpendicular to the face of the abutment wall) within the forward slope to reduce the impacts of broader excavation on the existing footings while providing more equipment headroom for construction of certain types of walls.
- Because the abutments are “perched” on spread footings that are founded approximately 1.5 m to 1.9 m above the Highway 401 grade, it will be critical to limit and control wall deflection if the retaining structure is close to the abutment footings, to minimize lateral and vertical deformation of the ground behind the retaining structure that could result in movement of the bridge footings and its superstructure. This aspect must also consider both ground and groundwater control during construction of the wall itself.
- The cost relative to the risk of wall movement, ground deformation and resulting impacts on the existing abutment footings and superstructure.

To aid in the assessment of the constructability and feasibility of various retaining structure options, Golder met on site with a foundation contractor (Deep Foundations Inc.) to review the site constraints and discuss the constructability aspects of various retaining structure options, and also contacted Anchor Shoring Inc. regarding the dimensions and capabilities of low headroom equipment.

6.2.1 Preliminary Assessment of “Zone of Influence” of Existing Abutment Footings

For preliminary assessment of the existing abutment footings which are perched on a compacted Granular A pad in the approach embankments, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 850 kPa, and the geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be taken as 300 kPa. For preliminary assessment of resistance to lateral loads (sliding resistance), the coefficient of friction, tan ϕ', between the cast-in-place concrete footing and the compacted Granular A pad may be taken as 0.70.

The location of the retaining structure relative to the existing abutment footings is a prime consideration in the selection of the temporary and permanent retaining structures. As noted in Section 6.1, the existing underside of the north and south abutment footings is located at approximately Elevation 207.9 m and 208.5 m, respectively, which is approximately 1.9 m and 1.5 m above the proposed final highway grade in front of the retaining walls (at approximately Elevation 206 m and 207 m, respectively). It is anticipated that the excavation for the highway widening will extend approximately 1.5 m lower than this to allow for construction of the widened pavement structure (i.e., to approximately Elevation 204.5 m to 205.5 m on the north and south sides of Highway 401, respectively).

For the purposes of the preliminary assessment conducted as part of this assignment, the zone of influence of the existing abutment footing loading has been considered based on 1H:2V slopes extending outward and downward from the front edge of the footings (based on the increase in pressure with depth due to footing loads). In addition, given the potential for ground loss in the cohesionless embankment fill material (as well as potentially within cohesionless layers or zones within the glacial till deposit), and the dependence of wall performance on the quality of the construction techniques, an additional 0.3 m to 0.5 m wide “buffer zone” has been considered to protect the zone of influence of the footing.

Based on these considerations, it is conservatively estimated that the boundary of the “zone of influence” of the existing abutment footings could be taken to extend to a horizontal distance of approximately 2 m in front of the front edge of the abutment footings.

If any portion of the retaining structure is located within a distance of less than 2 m from the front edge of the abutment footing, it will be within the zone of influence or could potentially impact the zone of influence of the abutment footing loads. The lateral loads on such walls will be relatively high, and there will be a greater risk of impacting the existing footing and superstructure. These walls must be sufficiently rigid to resist the lateral loads while maintaining wall and ground deformations and footing displacements to acceptable limits.

For distances equal to or greater than 2 m from the front edge of the abutment footing, walls will be located outside of the zone of influence of the footing loads. The lateral loads on these walls will be relatively low, and there will be a relatively low risk of wall or ground deformations impacting the existing footing and superstructure. In this case, a more flexible temporary protection system or permanent retaining structure could be used.

6.2.2 Preliminary Assessment of Tolerable Deformations

For the purpose of this preliminary assessment, it is estimated that the deformation of the existing perched abutment footings resulting from the construction of the retaining structure and removal of a portion of the abutment foreslopes should be limited to approximately 5 mm. This estimation assumes that differential settlement has occurred between the abutments, and between the abutments and centre pier, following the original construction. Further assessment of the tolerable magnitude of deformation should be addressed at the detail design stage.

Construction of excavations supported by vertical in situ walls can induce localized displacements of the adjacent ground (Peck 1969, Clough and O’Rourke 1990, Goldberg et al. 1976, Boone and Westland 2006). The magnitude and pattern of such displacements varies and depends on factors such as the following:

- type and structural stiffness of the wall system installed;
- depth of cut;

- ground conditions (strength and deformation properties);
- type, number, and spacing of horizontal support (tie-backs or struts);
- degree of pre-stressing of the horizontal supports;
- depth of penetration of the wall below the base of the excavation;
- whether or not the horizontal supports are removed during construction; and
- construction workmanship.

A number of measures are available to limit the displacements of retaining structures, including pre-stressing of horizontal restraints, stiffening of the vertical wall systems and extending the depth of wall penetration.

Wall displacements and resulting ground deformations, and their impact on the existing abutment footings, should be evaluated in greater detail during the detail design. The preliminary comments presented in this report are intended to facilitate refinement of conceptual alternatives and should be updated as additional project and subsurface information are developed.

If a temporary protection system is required at this site and is located within the zone of influence or could potentially impact the zone of influence of the existing abutment footing loading, it is recommended that the temporary system be designed to meet Performance Level 1A as set out in OPSS 539. This level specifies a maximum angular distortion of 1:1000 and a maximum horizontal displacement of 5 mm. Similar tolerances are recommended for permanent retaining structures located within the zone of influence of the existing abutment footings. Where temporary protection systems or permanent retaining structures are located outside the zone of influence, less stringent deformation criteria will be acceptable.

6.3 Overview of Retaining System Options

Earth retaining systems can be grouped into two categories based on the means by which they are constructed:

- **In situ walls** are constructed by building a wall face in the ground either before the ground is excavated or while the ground is excavated to create the grade difference. In situ walls generally do not require excavation behind the wall face, which is a primary advantage. Such systems do not incorporate free-draining granular backfill behind the wall facing and other measures must be taken to resist or control groundwater and frost pressures.
- **Gravity walls** are constructed “bottom up” from the base of an excavation, and then backfilled. Gravity wall systems are generally constructed such that the weight of the wall and entrained earth resists the lateral loads and consequent overturning forces from the earth pressures and imposed loads behind the wall. Gravity wall systems are generally back-filled with free-draining granular soils to control water and frost pressures.

Depending on the type of wall constructed, the combination of the wall embedment below the excavation bottom, the base friction, and/or the wall’s structural capacity and rigidity may be sufficient to resist the horizontal earth load and imposed loads. If this is not feasible, horizontal displacement of the wall may be restrained by including steel rods or wires (strands) drilled into and anchored in the ground behind the wall (tie-backs, ground anchors, or soil nails).

The following sections discuss the temporary and/or permanent retaining structures that may be feasible and practical for the James Snow Parkway underpass site, as follows:

- Secant pile (caisson) walls;
- Soldier pile and lagging walls;
- Soil nail walls;
- Use of micro-piles to underpin the existing abutment footings, with a temporary and/or permanent retaining wall then constructed at close proximity in front of the footings;
- Use of micro-piles or mini-piles installed in front of the existing abutment footings, likely with horizontal restraint in the form of tie-backs, and with a permanent concrete facing installed on the front of the arrangement; and
- Conventional concrete retaining walls (both cast-in-place and pre-cast).

The feasibility of each system has been assessed on a conceptual/preliminary level based on technical and constructability considerations such as compatibility with ground conditions as understood based on available information; installation and workspace requirements; and economic factors.

The advantages, disadvantages, risks and approximate costs for these types of retaining systems are summarized and compared in Table 1 following the text of this report, and are also discussed in Sections 6.4 to 6.11 of this report. Identification of preferred options from a foundations perspective is provided in Section 6.13, following the sections that discuss the various retaining structure types.

6.4 Secant Pile (Caisson) Walls

A secant pile wall (or “caisson wall”) is constructed by drilling holes between 0.9 m and 1.2 m in diameter to the full depth of the wall, inserting steel reinforcement in the form of steel beams or reinforcing bars, and filling the holes with concrete. The secant pile wall is formed by having each pile overlap the adjacent pile. A permanent secant pile wall often has a permanent cast-in-place or precast concrete facing attached to the front surface to fill any gaps between piles and provide a smooth or architecturally appropriate surface finish. A secant wall can be designed as a cantilever wall (up to a site-specific limiting height), with permanent tie-backs. In some cases, where tie-backs or bracing are not feasible, piles as large as 2 m in diameter can be constructed to allow higher unbraced/unrestrained cantilever walls. Permanent soldier pile and lagging walls must also include provisions for frost protection (as discussed in Section 6.10) and control of any groundwater seepage.

The main advantages of a secant pile wall are increased wall stiffness compared to the more flexible sheet pile, soldier pile and lagging or soil nail wall systems, control of ground and groundwater by pile interlock, and the ability to be used in difficult ground containing cobbles or boulders. The main drawbacks are that vertical tolerances may be hard to achieve for deep piles (on the order of 30 m deep), they are relatively expensive to construct and waterproofing may be difficult to achieve at the joints. For this site, where the existing abutment footings are perched on a compacted Granular A pad, there is also the potential for ravelling of the Granular A when exposed during construction, which would necessitate the use of temporary liners.

Construction equipment for installing a secant pile wall can generally operate within a working area of about 7 m to 10 m width with the wall at nearly any position within that window. Equipment for constructing a secant pile wall generally consists of a mobile drill rig (which may be based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In conventional construction, the height of caisson rigs can vary significantly; however, based on discussions with foundation contractors in the Toronto area, it is understood that smaller equipment is available that can work within heights of approximately 3.5 m to 5.5 m. To fit within the headroom available at the James Snow Parkway underpass structure, it may be necessary to excavate narrow slots (perpendicular to the abutment wall) within the abutment foreslope to provide sufficient vertical clearance even for low headroom equipment, while minimizing the impacts of these excavations on the existing footings. The use of smaller equipment may necessitate the construction of smaller diameter piles.

For the conceptual/preliminary assessment of retaining structure performance at the James Snow Parkway underpass site, it is anticipated that secant pile (caisson) walls will undergo a maximum horizontal and vertical displacement of about 0.1 per cent of the total excavation depth. For maximum wall heights of approximately 1.5 m to 4 m, this translates to less than about 5 mm of deformation at this site, provided that appropriate construction procedures and workmanship are adopted.

Preliminary geotechnical recommendations are provided in Appendix E (Section E.1) to support the structural assessment that may be required as part of the preliminary design for this type of retaining structure, if adopted.

6.5 Soldier-Pile and Lagging Walls

Soldier pile and lagging systems are commonly used for earth retention and can be constructed in a variety of ground conditions. Given the low headroom at this structure site, the wall can be installed by boring a series of 0.5 m to 1.0 m diameter holes, spaced 2 m to 3 m apart, into which H-piles (soldier piles) are installed; the annular space is then filled with a relatively low strength sand-cement concrete mix. It may be necessary to splice sections of H-pile together given the low headroom. Also, because the existing abutment footings are perched on a compacted Granular A pad at this site, temporary liners would be required during construction of the soldier pile holes to minimize the risk of ravelling and ground loss from the Granular A.

After installation of the soldier piles the excavation proceeds and lagging (timber boards, concrete or steel sheeting) is inserted behind the front flanges or placed against the piles and attached to the front flange using fasteners. The lagging is often installed in lifts of 1 m to 1.5 m, depending on the ground conditions. For permanent installations, pre-cast concrete lagging is normally used but the alignment must be closely controlled during installation of the soldier piles to ensure a proper fit. Permanent soldier pile and lagging walls must also include provisions for frost protection (as discussed in Section 6.10) and control of any groundwater seepage.

To resist lateral forces and to control lateral wall movement, soldier pile and lagging walls typically require horizontal restraints. For this site, where the face of the excavation would need to be free from obstructions, tie-backs would likely be required depending on the final wall height. The use of tie-backs is contingent upon the absence of underground utilities and the presence of suitable soils or rock in which to install anchors in the tieback area. The requirement to obtain subsurface easements is not considered to be an issue as tiebacks should remain within the right-of-way of the MTO highway and/or the local road. Excavations must be monitored for subsidence and lateral movement particularly when structures are nearby.

Construction equipment for installing a soldier-pile and lagging wall can generally operate within a working area of about 7 m to 10 m width with the wall at nearly any position within that window. Equipment for construction of soldier pile walls generally consists of mobile drill rigs (which may be based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. As for soldier pile wall construction, the height of conventional rigs can vary significantly; however, based on discussions with foundation contractors in the Toronto area, it is understood that smaller, “low headroom” equipment is available that can work within heights of approximately 3.5 m to 5.5 m. To fit within the headroom available at the James Snow Parkway underpass structure, it may be necessary to excavate narrow slots (perpendicular to the abutment wall) within the abutment foreslope to provide sufficient vertical clearance even for low headroom equipment, while minimizing the impacts of these excavations on the existing footings.

For the conceptual/preliminary assessment of retaining structure performance at the James Snow Parkway underpass site, it is anticipated that soldier pile and lagging walls will undergo a maximum horizontal and vertical displacement of about 0.2 per cent of the total excavation depth. For maximum wall heights of approximately 1.5 m to 4 m, this translates to up to about 8 mm of deformation at this site. This estimated displacement assumes high quality workmanship; however, it is noted that ground loss can be more common with this type of retaining system, especially during lift excavation and installation of lagging elements, and there is some risk of potential for higher displacements in the event of ground loss or otherwise poor workmanship.

Preliminary geotechnical recommendations are provided in Appendix E (Section E.2) to support the structural assessment that may be required as part of the preliminary design for this type of retaining structure.

6.6 Soil Nail Walls

Temporary and permanent retaining walls can be constructed using the soil nailing technique whereby the ground is supported by inserting and grouting reinforcing steel rods (nails) into the ground on a regularly spaced grid and then covering the excavation face with steel mesh and shotcrete structurally connected to the nails. The permanent facing can then be constructed of successive layers of shotcrete, precast panels, or a cast-in-place concrete face. The solid steel reinforcing bars are the main component of the system and tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation activities. Where more stringent deformation control is required, post-tensioning of the soil nails can be undertaken, although this measure would increase the project cost.

Soil nail walls are constructed from within the area to be excavated and require between about 6 m and 10 m of working space in front of the wall for equipment. The working space required will depend on the equipment chosen, staging, and routes required for earth moving equipment. In general, the equipment is portable and can operate in limited headroom environments as the nail installation is horizontal to sub-horizontal. It requires relatively little space and generates less noise and requires less manpower than other methods. The design of a soil nail wall can be readily adapted to fit curved forms.

The length of the soil nails is usually 0.6 to 1 times the height of the wall and less than what is used in conventional tie-back or soil anchor construction. Soil nailing is most economical in ground that can stand unsupported for at least one day on a vertical or steep slope cut 1 m to 1.8 m high and in which drill holes can remain open for at least several hours. This method of construction is best suited for use in deposits of dense granular and stiff low plasticity clayey soils.

Where the excavated face cannot stand unsupported for the required period of time (which could be the case at the James Snow Parkway underpass site, where Granular A fill is expected be encountered), the forward slope may be employed to stabilize the unsupported face section. In this case the soil nails would be installed and grouted first through the forward slope along its entire length, then the forward slope would be excavated and shotcrete applied along the entire excavation level. This technique would impact on construction costs and project schedule.

During construction and after its completion, a soil nail wall and the soil behind it tend to deform outwards. Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements of the wall at the facing are generally small, and are on the same order of magnitude as the horizontal movements at the top of the wall. Empirical data show that for soil nail walls with a typical Nail Length/Wall Height ratio between 0.7 and 1.0, negligible surcharge loading, and a typical global factor of safety value of 1.5, the maximum long-term horizontal and vertical wall displacements at the top of the wall, δ_h and , δ_v can be estimated as follows:

δ_h = $(\delta_h/H)_i \times H$

where $(\delta_h/H)_i$ = a ratio dependant on the soil conditions (assume 0.002 for Granular A material)

H = wall height

For the conceptual/preliminary assessment of the performance of retaining structures up to approximately 1.5 m to 4 m in height at this site, it is anticipated that soil nail walls will undergo maximum displacements of up to about 8 mm (similar to soldier pile and lagging walls). In addition, post-construction monitoring of soil nail wall displacements indicates that movements tend to continue after wall construction, sometimes for periods of up to six months, depending on the ground type. Typically, the post-construction deformation is about 15 per cent of the deformations observed soon after construction.

Preliminary geotechnical recommendations are provided in Appendix E (Section E.3) to support the structural assessment of soil nails that may be required at this preliminary design stage.

6.7 Micropiles to Underpin Existing Abutment Footings

A micropile is a small-diameter (typically less than 300 mm), drilled and grouted pile that is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads and they can be installed through, and bonded within, existing structures, providing direct connection with competent underlying strata.

Micropiles are an ideal choice for underpinning existing foundations as well as for earth retention. These piles can provide the high individual capacities typically required by structural support applications in transportation projects. Micropile structural capacities rely on high capacity steel elements to resist most or all of the applied load.

Special drilling and grouting methods are used in micropile installations to achieve high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used.

Micropile installation methods cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in restricted-access environments (including low headroom environments such as is the case at the James Snow Parkway underpass site) and in all soil types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to underpin existing structures.

The drilling and grouting equipment used for a micropile installation is relatively small and can be mobilized in restrictive areas that would prohibit the entry of conventional equipment. Micropiles can be installed within a few millimetres of existing walls or foundations, provided that there is space above for the drill-head and safe work zone or the piles are battered to provide this space. Also, drilling and grouting procedures associated with micropile installations do not cause damage to adjacent existing structures or affect adjacent ground conditions when proper drilling and grouting procedures are utilized.

6.8 Micropiles Used in Front of Existing Abutment Footings

Alternatively, consideration could be given to the use of micropiles installed vertically in front of the edge of the existing abutment footings, rather than drilling micropiles through the footings; a conceptual plan and section for this arrangement is shown (in comparison to a soldier pile and lagging wall type) on Figure 1. The discussion presented in Section 6.7 above would apply to this retaining wall concept. However, in this application, rather than using the micropile elements to extend the underpass structure load below the base of the retaining wall, the micropiles would serve as the retaining wall itself.

As for the option presented in Section 6.7, micropiles are advantageous in that they can be constructed in low headroom, narrow-access conditions. They can be installed quite close to the front edge of the existing footings, because the risk of ground loss during micropile installation is relatively low (particularly when compared to larger diameter caissons for secant pile wall or soldier pile installation); in particular, the use of flush-joint casing for the installation of the micropiles will control the potential for loss of ground in the Granular A fill, and will provide structure support to the Granular A fill prior to excavation. This option does not require alteration or undermining of the existing abutment footings, and displacements should be able to be adequately controlled.

For conceptual design, it is anticipated that the micropiles would consist of flush-joint, approximately 150 mm diameter, heavy-wall casing, drilled into place using a limited access drill rig at centre-to-centre spacings of approximately 2 to 2.5 diameters; hollow stem flight augers should not be permitted for the micropile installation to minimize the risk of ground loss. The micropiles or mini-piles will have to penetrate into the clayey silt till deposit sufficiently to develop passive resistance, with the depth to be determined during detail design analyses.

It is anticipated that the micropiles would require horizontal restraint to limit lateral movement. This could be provided by tie-backs fully grouted through the Granular A and into the clayey silt till deposit, with the length to be determined during detail design. (Further discussion is provided in Section 6.9 on horizontal restraints, and preliminary geotechnical recommendations have been provided in Appendix E (Section E.3) for use in structural assessment that may be required as part of the preliminary design for horizontal restraints.) The tie-backs should consist of hollow-core bars, grouted as they are drilled, and pre-tensioned prior to excavation below the tie-back level. Hollow stem flight augers should not be permitted for the tie-back installation to minimize the risk of ground loss.

Concrete facing panels could be attached to the front face of the micropiles to complete the permanent structure, or a permanent wall could be cast-in-place in front of the micropiles.

6.9 Horizontal Restraint for In Situ Walls – Tie-Backs/Ground Anchors

Tie-backs, also called ground anchors, are constructed by drilling horizontal or sub-horizontal holes into the ground behind the wall as the excavation proceeds downward. After the hole is drilled, steel rods or high-strength steel strands are inserted into the hole and an “anchor zone” is then created by filling the annular space around the steel rods or strands with cement grout. Often, the cement grout is injected under pressure. The anchor zone is typically located beyond the “active” earth zone behind the wall (the mass of earth that deforms and places load on the wall). After the grout is cured, the anchor is pre-stressed to its design load, structurally connected to the wall and the remaining annular space between the anchor zone and the wall face, called the “free” length,” is backfilled.

Tie-backs offer an unrestricted excavation once they are in place but permanent tie-backs can limit future subsurface use since the integrity of the tied-back walls depends on the ground around the tie-backs remaining undisturbed. For planning purposes, it may be assumed that the anchors may extend back from the face of the wall in distance equal to twice the excavation depth.

The horizontal and vertical spacing of the tie-backs will largely depend on the stiffness of the vertical wall elements, the loads that are distributed to the tie-backs, the capacity of the ground in which they are anchored to resist the load, tolerable displacements of the ground and facilities around the excavation, and the cost for installing the tie-backs. Typically, the spacing of tie-backs (both vertically and horizontally) is limited to about 5 m. Larger spans can be achieved but the required bending moment capacity of the vertical wall elements must be substantially greater than typical excavation support installations. It may also be necessary to install walers (long structural sections that support the wall horizontally) between supports. Walers can consist of steel sections or, in the case of permanent installations, cast-in-place concrete.

For the James Snow Parkway underpass site, permanent tie-back installations can be made in the native, very stiff to hard clayey silt till. The presence of the existing abutment footing will have to be considered relative to the location of the proposed retaining structure to ensure that tie-backs can be installed to their required length (which would require that they extend below the existing footing). Depending on the angle at which tie-backs are installed, the vertical component of the tie-back load can be significant and the design of earth retaining systems must take this vertical load into account. Vertical wall members must be capable of supporting the vertical load component while maintaining vertical settlement within tolerable limits. Excessive vertical wall movement can cause loss of tension in the tie-backs and poor performance of the entire excavation support system.

Preliminary geotechnical recommendations have been provided in Appendix E (Section E.3) for use in structural assessment that may be required as part of the preliminary design for horizontal restraints.

If required, improved anchor bond strengths can be achieved within the embankment fill and the clayey silt till by post-grouting. Apart from increasing the apparent bond stress, the technique also has the advantage of allowing individual anchors to be re-grouted and improved, if proof-testing shows a particular anchor to be deficient. Because the anchor bond zones are expected to be formed within the native clayey silt till deposit, and based on the anticipated geometry, it is expected that secondary pressure grouting will not have a negative impact (i.e., heave) on the existing abutment footings.

6.10 Frost Protection and Drainage Requirements for In Situ Walls

In situ walls will be subjected to freezing ambient temperatures at the wall face during winter. The walls will also be in direct contact with the ground behind the wall. It is anticipated that the wall materials will serve as a thermal conductor and unless insulation is provided at the wall face, the freezing temperatures may cause ice lenses and frost pressures behind the wall. The design and construction of such walls will require that consideration be given to providing the face of the wall with insulation and a protective wall facing. This is consistent with other projects in Ontario where permanent in situ walls have been fitted with an insulation layer to prevent such pressures.

For in situ wall systems, drainage can be achieved using prefabricated drainage elements placed behind the permanent facing. Where precast concrete panels are used, the space between the temporary wall face and the permanent facing may be backfilled with a drainage element consisting of an approved granular drainage layer or pre-fabricated drainage material. Water intercepted by the drainage layer will flow downward to the base of the wall where it can be removed by a subdrain arrangement, or conveyed through the permanent facing in a weep hole per OPSD 3190.100 *(Retaining and Abutment Wall Drain).*

6.11 Conventional Concrete Retaining Walls

Conventional cast-in-place concrete walls are highly adaptable, and well-established in their design and construction methods; however, their cost can exceed the cost of other wall options for similar project conditions, especially if the walls are relatively high and/or if a temporary protection system is required for construction of the permanent wall. A number of pre-cast versions of conventional cantilever or “counterfort” retaining walls are also available. In general, the walls are constructed at concrete pre-casting plants to standard panel dimensions. Once at a construction site, the pre-cast panels are then attached to a cast-in-place concrete footing with similar dimensions as for cast-in-place concrete walls. These walls offer the advantages of construction speed and potentially reduced costs as formwork is largely eliminated.

In general, concrete retaining walls can be of a cantilever or gravity design, which support the retained earth through the following mechanisms:

- overturning moment is resisted by the counteracting direction of the soil weight on the footing;
- sliding of the wall is resisted by friction along the wall base and any soil in front of the wall; and

- the integrity of the wall is maintained by the structural capacity of the wall face and the footing connection.

Preliminary geotechnical recommendations for the design of concrete retaining walls are provided in Appendix E (Section E.4). Select, free-draining granular fill meeting the specifications of OPSS 1010 Granular A or Granular B Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the concrete retaining wall.

Conventional concrete wall systems can be constructed within a temporary excavation support system or an open-cut if space permits. At the James Snow Parkway site, it is expected that a temporary protection system would be required behind the permanent wall to facilitate construction of this wall type, unless the ramp geometry can be adjusted so that the permanent wall is located sufficiently far away from the abutment footings to permit a short, temporary open-cut for a toe wall for example.

6.12 Global Stability of Abutment Footing-Retaining Structure System

The global stability of retaining structures in front of the abutment foundations at this site will be dependent on the type of wall, its geometry and location relative to the abutment footing, and the characteristics of the fill and native soils. For preliminary assessment, a minimum target factor of safety of 1.5 has been adopted.

Preliminary global stability analyses were carried out using the commercially available slope stability program Slide 6.0 (developed by Rocscience Inc.), employing the Morgenstern-Price and Spencer methods for stability analysis, for proposed wall arrangements based on the following considerations and assumptions:

- The existing abutment footings are constructed on a 3 m thick compacted Granular A pad with a trapezoidal cross-section consisting of a 2H:1V forward slope, a 1H:1V back slope and a width of 9.5 m at the footing level.
- The Granular A fill behind the existing abutment footings conforms to the geometry depicted in OPSD 3101.150 *(Abutment Walls – Backfill – Minimum Granular Requirements).* The fill material outside of this zone has been assumed to consist of local earth fill.
- The applied footing stress is estimated to be approximately 310 kN/m^2, based on input from URS.
- In situ walls (secant pile walls, micropiles) are located within a distance of less than approximately 2 m from the front edge of the abutment footing and more flexible retaining structures or conventional concrete retaining walls are located a distance of more than 2 m from the front edge of the abutment footing.
- The soil parameters used in the analysis for long-term global stability are as follows:
    - Granular A: $\phi' = 35$ degrees, $c' = 0$ kPa and $\gamma' = 22.8$ kN/m^3
    - Local earth fill: $\phi' = 28$ degrees, $c' = 0$ kPa and $\gamma' = 19.0$ kN/m^3
    - Clayey silt till: $\phi' = 35$ degrees, $c' = 5$ kPa and $\gamma' = 21.0$ kN/m^3

The preliminary global stability analyses indicate that the target factor of safety of 1.5 can be achieved for an in situ wall provided that its minimum embedment depth (measured from below the current highway grade) is approximately 3.0 m. The preliminary global stability analyses also indicate that a target factor of safety of 1.5 can be achieved for conventional concrete retaining walls located a distance of more than 2 m from the front edge of the abutment footing, although this factor of safety is sensitive to the geometry and further assessment will be required during detail design.

6.13 Recommended Alternative from a Foundations Perspective

As discussed in Section 6.2, the location of the wall alignment relative to the existing abutment footings has a significant impact on the selection of permanent and/or temporary retaining structures. The existing underside of the north and south abutment footings is located at approximately Elevation 207.9 m and 208.5 m, respectively, which is approximately 1.9 m and 1.5 m above the proposed final highway grade in front of the retaining walls (at approximately Elevation 206 m and 207 m, respectively).

For the purposes of this preliminary assessment, the zone of influence of the existing abutment footing loading has been considered based on 1H:2V slopes extending outward and downward from the front edge of the footings (based on the increase in pressure with depth due to footing loads). In addition, given the potential for ground loss in the cohesionless embankment fill material (as well as potentially within cohesionless layers or zones within the glacial till deposit), and the dependence of wall performance on the quality of the construction techniques, an additional 0.3 m to 0.5 m wide “buffer zone” has been considered to protect the zone of influence of the footing.

Based on this, it is conservatively estimated that the boundary of the “zone of influence” of the existing abutment footings extends to a horizontal distance of approximately 2 m in front of the abutment footing.

If any portion of the retaining structure is located within a distance of less than 2 m from the front edge of the abutment footing, it will be within the zone of influence or could potentially impact the zone of influence of the abutment footing loads. The lateral loads on such walls will be relatively high, and there will be a greater risk of impacting the existing footing and superstructure. These walls must be sufficiently rigid to resist the lateral loads while maintaining wall and ground deformations and footing displacements to acceptable limits.

For distances equal to or greater than 2 m from the front edge of the abutment footing, walls will be located outside of the zone of influence of the footing loads. The lateral loads on these walls will be relatively low, and there will be a relatively low risk of wall or ground deformations impacting the existing footing and superstructure. In this case, a more flexible temporary protection system or permanent retaining structure could be used.

The advantages, disadvantages, risks and approximate costs for the various types of retaining systems discussed in this report are summarized and compared in Table 1 following the text of this report. Based on these considerations, the preferred retaining structure options from a foundations perspective are summarized in the following table:

Preferred Option	Location of Retaining Structure Relative to Front Edge of Existing Abutment Footing	
	< 2m From Front Edge of Footing	> 2m From Front Edge of Footing
1	Micropiles or mini-piles installed in front of abutment footings, with horizontal restraint provided by tie-backs, and a permanent facing installed in front of the micro-piles	Flexible, in situ (“top-down” construction) retaining system such as a soil nail wall (easier to construct in limited headroom) or soldier pile and lagging wall, with permanent facing attached to the front of the wall

Preferred Option	Location of Retaining Structure Relative to Front Edge of Existing Abutment Footing	
	< 2m From Front Edge of Footing	**> 2m From Front Edge of Footing**
2	Underpinning of existing abutment footings with micropiles to extend the structure load below the highway grade, followed by construction of a more flexible, in situ retaining system such as a soil nail wall (easier to construct in limited headroom) or soldier pile and lagging wall	Use of a temporary protection system as above, with construction of a conventional concrete wall in front for the permanent retaining structure – unless the conventional concrete wall can be located sufficiently far from the abutment face that only temporary steepening of the abutment foreslope is required for an “open-cut” excavation

6.14 Recommended Additional Studies for Detail Design

It is recommended that the following issues be considered during the future detailed design studies:

- Carry out detailed field investigations at the structure sites to confirm the thickness and founding elevation of the existing footings, the thickness and properties of the compacted granular pad below the perched footings, the composition and properties of the fill material comprising the abutment foreslopes, and the properties of the clayey silt till behind the abutments within potential anchor bond zones.
- Confirm the groundwater level(s), perched or otherwise, at the site.
- Confirm and further refine the preliminary geotechnical recommendations as set out in Appendix E of this report.
- Complete more rigorous assessments of wall and ground deformation, suitable for detail design of the retaining structures at this site.
- Complete global stability assessment based on actual wall configuration.
- Develop an instrumentation and deformation monitoring program to measure horizontal and vertical displacements during construction. This instrumentation should consist of settlement points and tilt beam sensors mounted at predetermined locations on the abutment walls and/or footings. Appropriate review and alert levels should be established during detail design, based on further structural and geotechnical/foundations assessments.

7.0 CLOSURE

This Preliminary Foundation Design/Assessment Report has been prepared based on existing subsurface information to assist the project team with evaluating conceptual retaining wall alternatives. The preliminary recommendations provided herein should be reviewed and revised as necessary as and if further information is developed with respect to design concepts, more detailed retaining wall locations, and subsurface information.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index **Relative Density**	**N** **Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

	c_u, s_u	
	kPa	**psf**
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Percent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
μ	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_{(w)}$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1 $\tau = c' + \sigma' \tan\phi'$

2 shear strength = (compressive strength)/2

TABLE 1
COMPARISON OF RETAINING STRUCTURE ALTERNATIVES

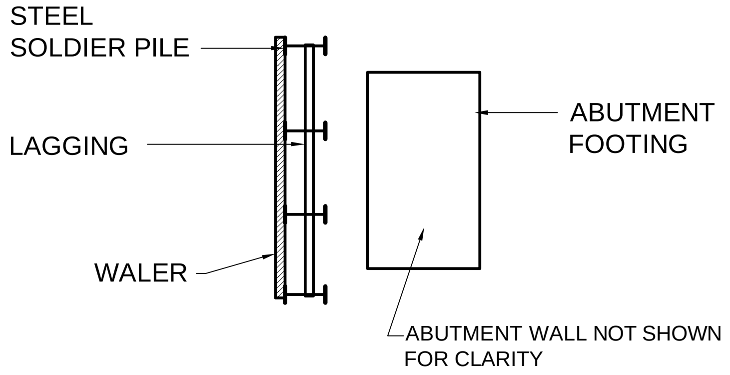
Secant Pile (Caisson) Wall	Soldier Pile and Lagging Wall	Soil Nail Wall	Micropiles to Underpin Existing Footings or in Front of Existing Footings	Conventional Concrete Retaining Wall
Advantages: i. Uses top-down construction ii. Can withstand relatively large horizontal wall loads compared to soldier pile and lagging walls, with less deformation behind the wall due to the greater stiffness of the retaining system (estimated at approximately 0.1% of wall height, or less than about 5 mm for this site). iii. Excellent performance behavior demonstrated in similar applications in which retaining structures were required very close to existing foundations (for buildings, etc.). iv. Can be installed relatively close to the existing abutment footing v. Can serve for both temporary and permanent retaining structure solution. ***Disadvantages:*** i. Wall construction is controlled by the concrete curing time and the construction window will be longer compared to other options. ii. Requires specialized equipment to work in the limited headroom. iii. Construction effort will increase significantly if groundwater is encountered in the excavations. A liner will be required, its installation will be difficult and maintaining a sufficiently dry excavation to permit high quality construction would be challenging. iv. Relatively expensive to construct, especially given the low headroom at this site.	***Advantages:*** i. Uses top-down construction ii. Can withstand relatively large horizontal wall pressures without requiring a significant increase in wall section. iii. Construction procedures are well understood by local contractors. iv. Proven reliable performance. ***Disadvantages:*** i. Not appropriate for applications where very strict deformation control is required for structures located within the zone of influence behind the wall; this type of wall system is likely too flexible (deformations estimated to be approximately 0.2% of wall height, or up to about 8 mm) to use within the zone of influence of existing footing loading as it could have a greater risk of impact on the existing footings and superstructure. ii. Anchored tiebacks in permanent walls require permanent underground easements, although this is not anticipated to be an issue at the James Snow Parkway site where the tiebacks would likely be within the MTO and/or local road right-of-way. iii. Performance can be very dependent on construction technique; requires careful fitting of lagging elements and backfilling voids behind the lagging to minimize soil displacement and soil creep. iv. Generally suitable as a temporary system to facilitate construction of permanent retaining structure, although it is also possible to adopt this as a permanent structure (with concrete panels instead of timber lagging) in appropriate locations. v. Requires specialized equipment to work in the limited headroom.	***Advantages:*** i. Uses top-down construction, and there is no need to embed any structural element below the bottom of the excavation. ii. Installation is relatively rapid and uses typically less construction materials than soldier pile and lagging walls. iii. Easy adjustments of nail inclination and location can be made when obstructions are encountered. iv. Overhead construction requirements are smaller than those for other wall types. v. Although this is a relatively more flexible retaining system (see “Disadvantages” below), it may be appropriate in conjunction with underpinning of the existing footings if in close proximity to the existing footings, or on its own if the wall is located sufficiently far from the existing footings. vi. Although more commonly used as a temporary protection system, could be used as a permanent retaining solution if existing foundations are underpinned. In this case, more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used. vii. Shotcrete facing is typically less costly than the structural facing required for other wall systems. ***Disadvantages:*** i. May not be appropriate for applications where very strict deformation control is required for structures located within the zone of influence behind the wall; estimated deformations of up to approximately 8 mm for this site. ii. Horizontal position of ground anchors is more difficult to modify making adjustments in the field costly. iii. Permanent soil nail walls require permanent underground easements although this is not anticipated to be an issue at the James Snow Parkway site where the tiebacks would likely be within the MTO and/or local road right-of-way. iv. Construction of the wall requires specialized and experienced contractors. v. Less experience and information on soil nail wall performance locally.	***Advantages:*** i. Uses top-down construction. ii. Ability to be installed from very restricted access locations including low headroom. iii. Minimum ground and/or structural disturbance when using flush-joint, heavy-wall casing. In particular for this site, controls potential for ground loss/ravelling through Granular A and offers structural support to Granular A prior to excavation. iv. Excellent performance behaviour demonstrated in underpinning existing structure foundations or in using micropiles in front of existing foundations. v. In the case of underpinning, very low potential for movement of existing underpass foundations because the footing loads would be transferred below the zone of influence of the proposed excavation and wall construction. vi. Load carrying capacity of the micropile can be easily confirmed by load testing. vii. Maximum width of highway widening can be achieved as temporary and/or permanent retaining structures can be constructed in very close proximity to the underpinned footings with negligible risk of ground deformation. ***Disadvantages:*** i. Requires specialized and experienced contractors, although such contractors are present in the greater Toronto area. ii. In the case of micropiles installed in front of the existing footings, permanent tie-backs would be required, along with a permanent underground easement. iii. Relatively expensive compared to other options.	***Advantages:*** i. Relatively easy to construct and does not require specialized construction skills. ii. Excellent demonstrable performance behaviour. iii. Panels can be pre-cast which reduces the level of construction effort, increases the speed of construction and reduces formwork costs. ***Disadvantages:*** i. Impractical to install this wall relatively close to the abutment footings without disturbing the surrounding soil and compromising the structural integrity of the bridge. Would require a temporary protection system to minimize ground deformations prior to constructing this type of wall, unless the concrete retaining wall is located sufficiently far from the front edge of the existing abutment footing so that only temporary steepening of the abutment foreslope is required (i.e., “open-cut” excavation). ii. Maximum highway widening width cannot be achieved with this option since the wall must be installed outside of the zone of influence of the abutment footings, and/or in conjunction with a temporary protection system such as a soldier pile and lagging wall. iii. Cost increases significantly as wall height increases.

Secant Pile (Caisson) Wall	Soldier Pile and Lagging Wall	Soil Nail Wall	Micropiles to Underpin Existing Footings or in Front of Existing Footings	Conventional Concrete Retaining Wall
Risks/Consequences i. There is some potential for perched water in the granular pad below the existing abutment footings. Whether this fill is "dry" or saturated, caisson holes through the Granular A pad will not stand unsupported (although the risk of ground loss is higher where perched groundwater is present), with moderate to high risk for loss of ground that could impact the existing footing if within the zone of influence of the footing loading. ii. A temporary liner is recommended to minimize the risk of loss of ground in the Granular A pad. However, it may be difficult to use a temporary liner due to the lower torque associated with low headroom equipment. If a temporary liner can be employed in a low headroom environment, this option is recommended for use along a wall alignment located where the deformations must be restricted to less than 0.1% of the wall height/excavation depth; for the proposed abutment foreslope cut geometry at this site, this would be less than approximately 2 m away from the front edge of the abutment footing.	***Risks/Consequences*** i. Risk of unacceptable wall deformation with associated ground settlement and footing settlement behind the wall if this type of wall is constructed relatively close to the abutment footing. This is attributable partly to the more flexible nature of this system, and also to the greater potential for loss of ground during installation of lagging between the soldier piles (when the excavated face must temporarily stand vertically). ii. As for a secant pile wall, a temporary liner is recommended to minimize the risk of loss of ground in the Granular A pad. However, it may be difficult to use a temporary liner to advance the caisson holes due to the lower torque associated with low headroom equipment. iii. If a temporary liner can be employed in a low headroom environment for temporary support of the soldier pile holes, this option is recommended for use along a wall alignment located where wall deformations of approximately 0.2% of the wall height/ excavation depth can be tolerated; for the proposed abutment foreslope cut geometry at this site, this would be equal to or more than approximately 2 m away from the front edge of the abutment footing.	***Risks/Consequences*** i. Risk of unacceptable wall deformation and consequent abutment footing settlement if the wall is constructed relatively close to the abutment footing. ii. Recommended for use along a wall alignment located where wall deformations of approximately 0.2% (or greater) of the wall height/excavation depth can be tolerated; for the proposed abutment foreslope cut geometry at this site, this would be equal to or more than approximately 2 m away from the front edge of the abutment footing.	***Risks/Consequences*** i. Very low risk of failure. ii. Recommended for use along a wall alignment where the deformations must be restricted to less than approximately 0.1% of the wall height/excavation depth; for the proposed abutment foreslope cut geometry at this site, this would be less than approximately 2 m away from the front edge of the abutment footing.	***Risks/Consequences*** i. Very low risk of failure provided the wall is located outside the zone of influence of the footing loads. ii. Recommended for use along a wall alignment where wall deformations of approximately 0.2% (or greater) of the wall height/excavation depth can be tolerated (to account for use of temporary protection system); for the abutment foreslope cut geometry at this site, this would be recommended for use along a wall alignment located equal to or more than 2 m away from the edge of the abutment footing.
Approximate Cost: $1,800/m^2	***Approximate Cost: $1,500/m^2***	***Approximate Cost: $800/m^2***	***Approximate Cost: $1,200/m^2***	***Approximate Cost: $700/m^2 plus cost of temporary protection system***

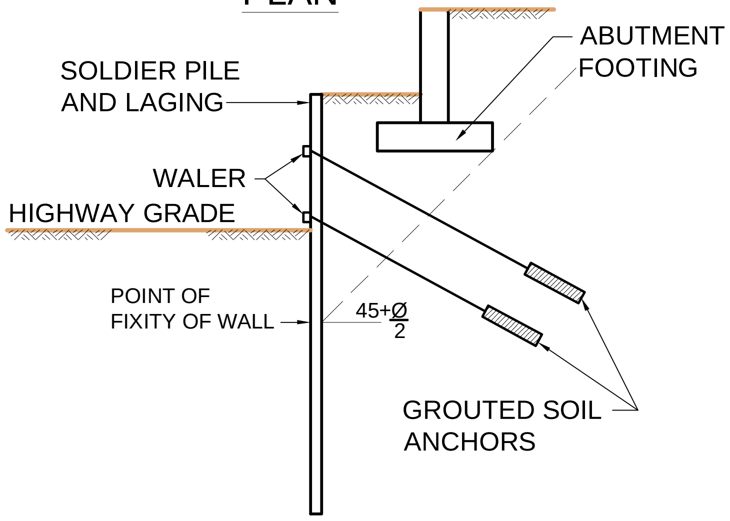
NOTES: Costs are preliminary estimates and are intended to provide a comparison between alternatives. Actual construction costs may vary.
Table to be read in conjunction with accompanying report.

PLOT DATE: January 23, 2012
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-FA- James Snow Parkway\09-1111-6036FA005.dwg

SOLDIER PILE AND LAGGING ARRANGEMENT

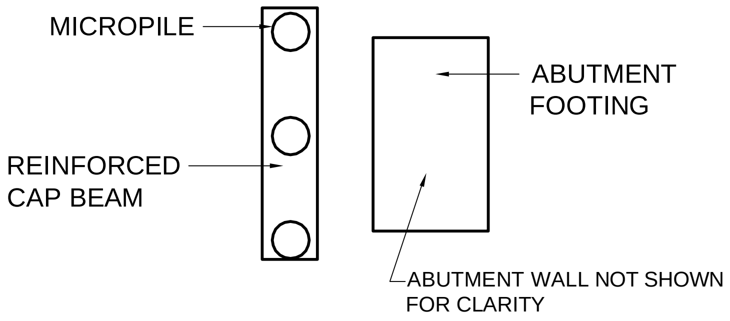


PLAN

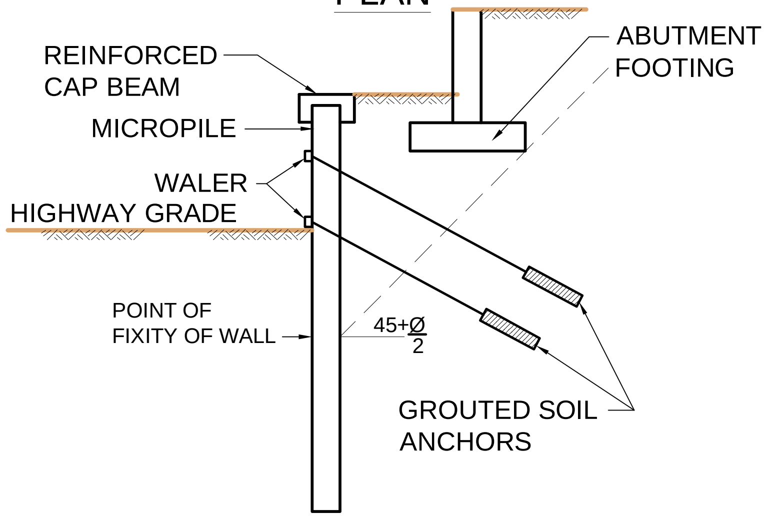


SECTION

VERTICAL MICROPILE WALL ARRANGEMENT



PLAN



SECTION

Golder Associates
Mississauga, Ontario, Canada

SCALE N.T.S.
DATE Jan. 23, 2012
DESIGN
CAD JFC
CHECK RAA
REVIEW LCC

FILE No. 09-1111-6036FA005.dwg
PROJECT No. 09-1111-6036
REV. A

TITLE

TYPICAL WALL ARRANGEMENTS PLANS AND SECTIONS

HIGHWAY 401/JAMES SNOW PARKWAY UNDERPASS

FIGURE 1

APPENDIX A

Borehole Records
GEOCRES No. 30M12-138

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

OFFICE REPORT ON SOIL EXPLORATION

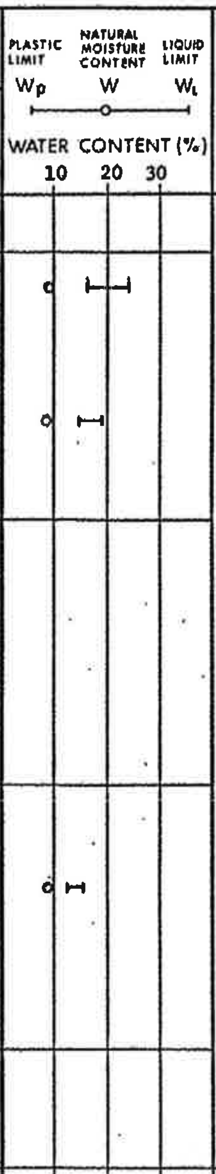
RECORD OF BOREHOLE No 1

W P 27-78-02
DIST 6 HWY 401
DATUM Geodetic
LOCATION Coords. N 15 821 095; E 903 855
BOREHOLE TYPE Continuous Flight Auger Hollow Stem Augers
DATE April 17, 1979
ORIGINATED BY TK
COMPILED BY T.K.
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.9 / 672.1	Ground Surface						
0.0 / 0.0	Weathered Firm					670	
	Clayey Silt Some Sand Trace Gravel (Glacial Till)	1	SS	42			22 19 49 10
		2	SS	50			
	Hard	3	SS	56			20 20 47 13
	Red	4	SS	32		660	
		5	SS	29			
		6	SS	31			
	0.8m Silty Sand				▼		
		7	SS	53		650	
		8	SS	41			
	0.4m Weathered Shale Partings	9	SS	121		640	
193.8 / 635.6		10	SS	60			
11.1 / 36.5	End of Borehole Note: Water Table Encountered During Boring Operations						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

UNIT WEIGHT γ



Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION

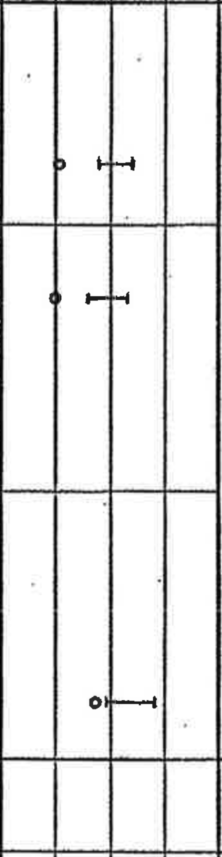
RECORD OF BOREHOLE No 2

W P 27-78-02
LOCATION Coords. N 15 821 117; Z 903 726
ORIGINATED BY TK
DIST 6 HWY 401
BOREHOLE TYPE Continuous Flight Auger Hollow Stem Augers
COMPILED BY TK
DATUM Geodetic
DATE April 18, 1979
CHECKED BY M.J.

OFFICE REPORT ON SOIL EXPLORATION

	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES						
206.7 0.0	678.2	Ground Surface									
	0.0	Granular Roadway Fill									
206.1 0.6	2.0	Clayey Silt Some Sand Trace Gravel (Glacial Till) Very Stiff to Hard Red									
			3	SS	34						11 22 51 16
							670				
		0.4m Occasional Cobbles	4	SS	36						
		Sand Seams	5	SS	46						11 22 49 18
			6	SS	36						
			7	SS	26						
							660				
			8	SS	35						
		Clayey Silt Layer 0.4m	9	SS	21						
							650				
197.1 9.6	646.7		10	SS	45						
	31.5	End of Borehole									

2



Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

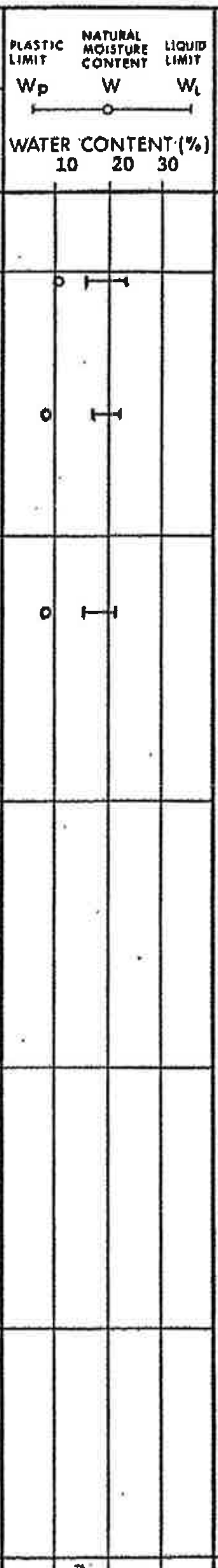
RECORD OF BOREHOLE No 3

W P 27-78-02 LOCATION Coords. N 15 821 218; E 903 655 ORIGINATED BY TK
DIST 6 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.K.
DATUM Geodetic DATE April 18, 1979 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
205.1 0.0 672.9	Ground Surface									
0.0	Weathered Firm									
	Clayey Silt Some Sand Trace of Gravel (Glacial Till) Hard Red	1	SS	41		670	6	23	56	15
		2	SS	51						
		3	SS	44			10	24	51	15
		4	SS	29						
						660				
		5	SS	38						
		6	SS	36						
	0.8m Cobbles and Boulders	7	SS	75/2"	▼					
						650				
		8	SS	55						
		9	SS	60						
						640				
		10	SS	59						
		11	SS	76						
						630				
189.4 621.4		12	SS	90						
15.7 51.5	End of Borehole Note: Water Table Encountered During Boring Operations									

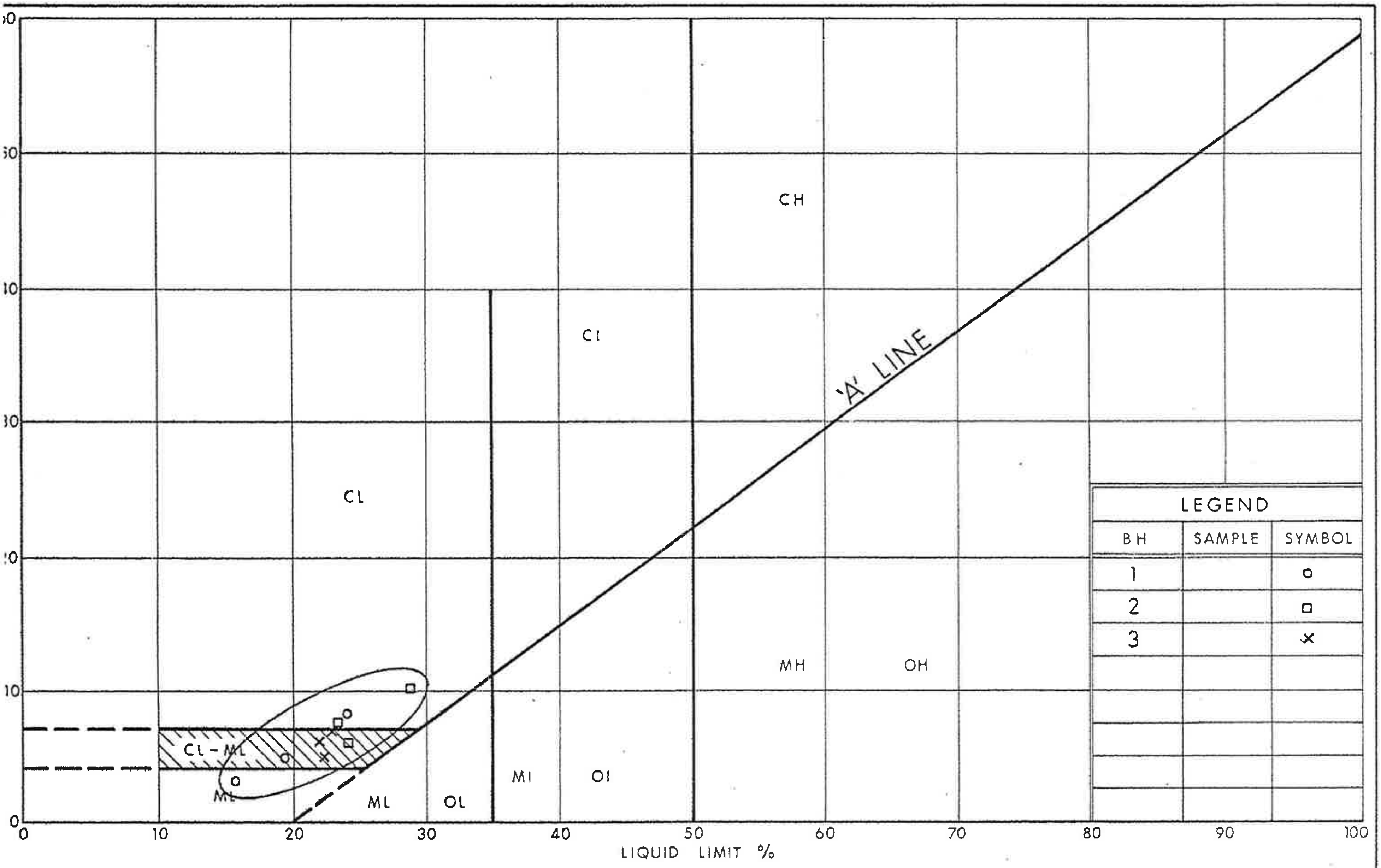
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE



UNIT WEIGHT γ

APPENDIX B

Laboratory Test Results
GEOCRES No. 30M12-138



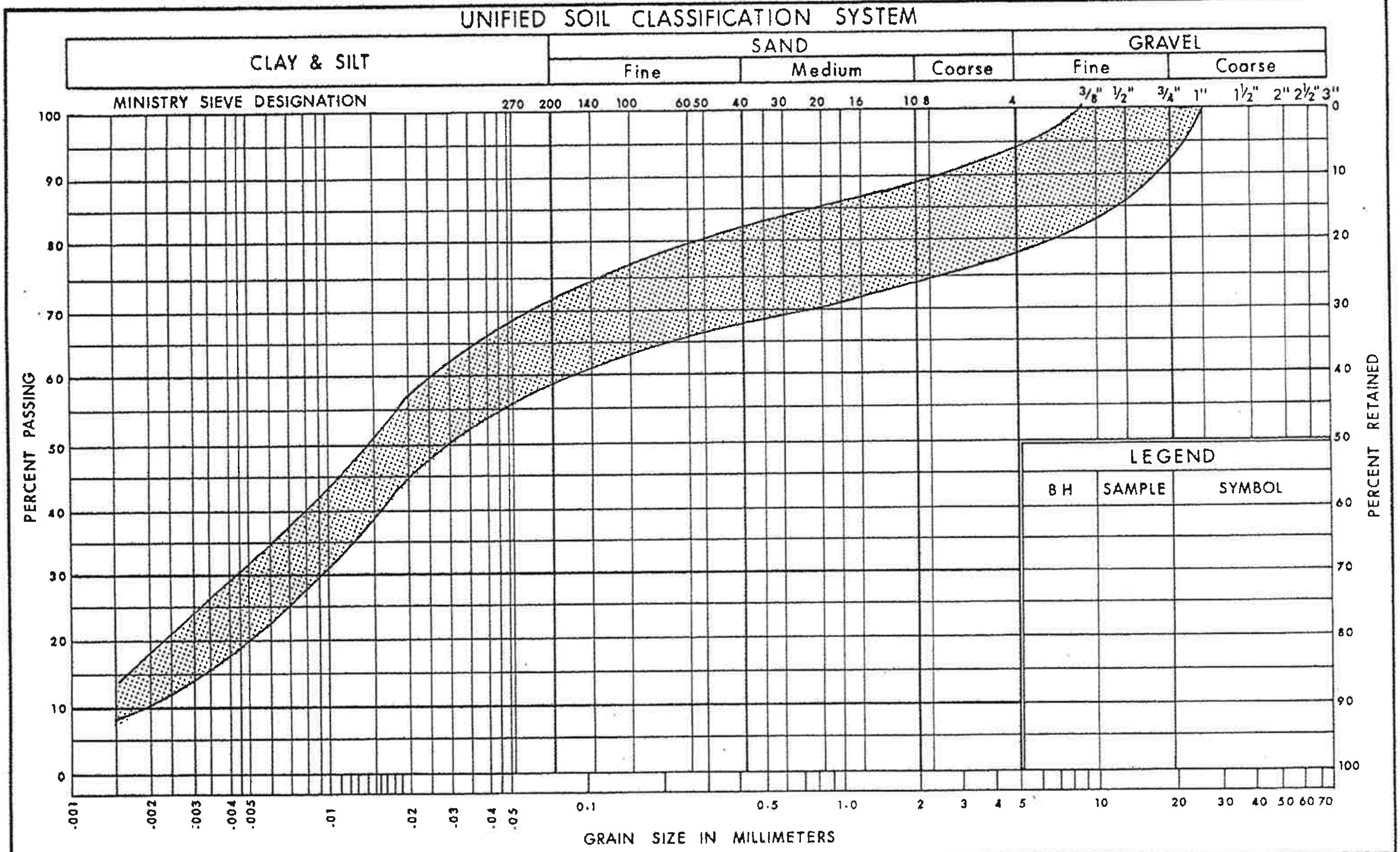
Ministry of
Transportation and
Communications
ntario
GINEERING SERVICES BRANCH

PLASTICITY CHART
GLACIAL TILL
CLAYEY SILT SOME SAND TRACE OF GRAVEL

FIG No 1

W P 27-78-02

Oct 75, FF-S-22



Ministry of
Transportation and
Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
CLAYEY SILT SOME SAND TRACE OF GRAVEL

FIG No 2

W P 27-78-02

APPENDIX C

Borehole Locations and Soil Strata
GEOCRES No. 30M12-138 – Drawing No. 277802-A

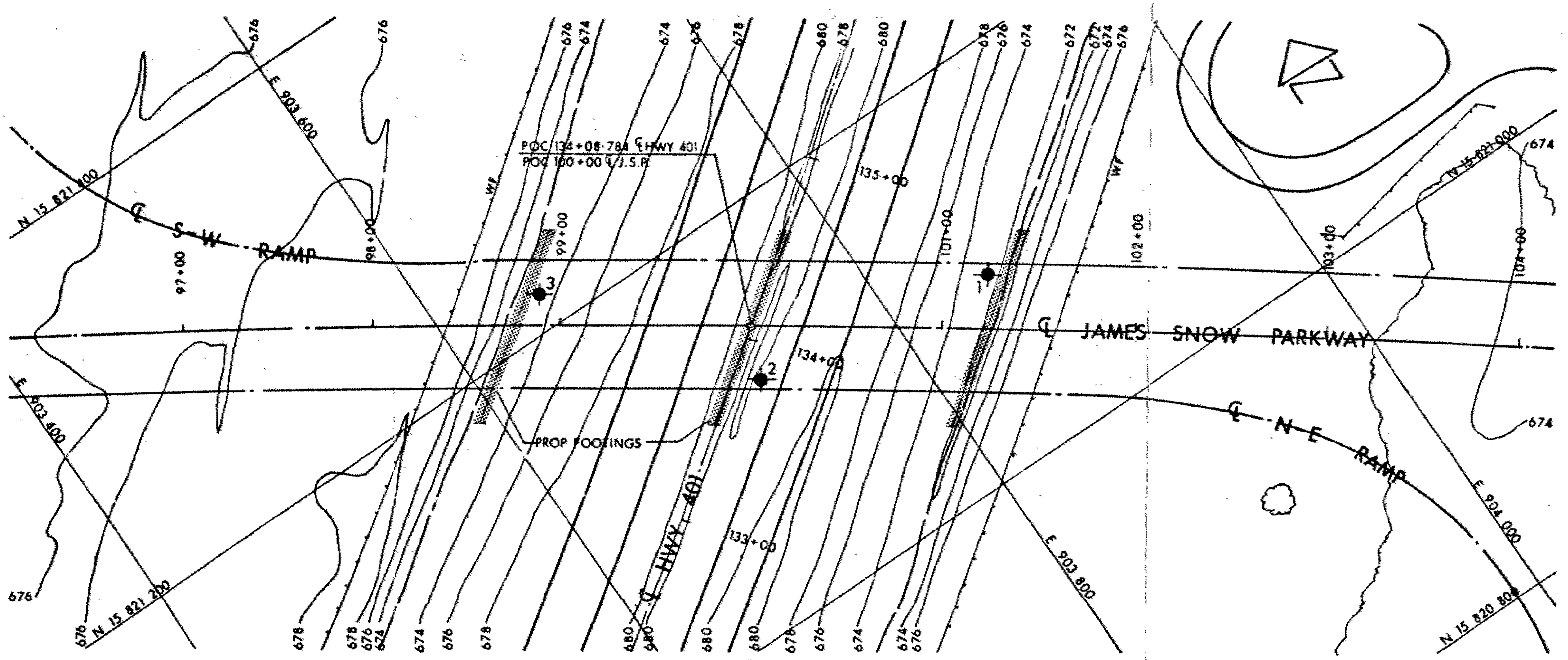
MINISTRY OF TRANSPORTATION, ONTARIO

CONT No
WP No 27-78-02

JAMES SNOW PARKWAY UNDERPASS

BORE HOLE LOCATIONS & SOIL STRATA

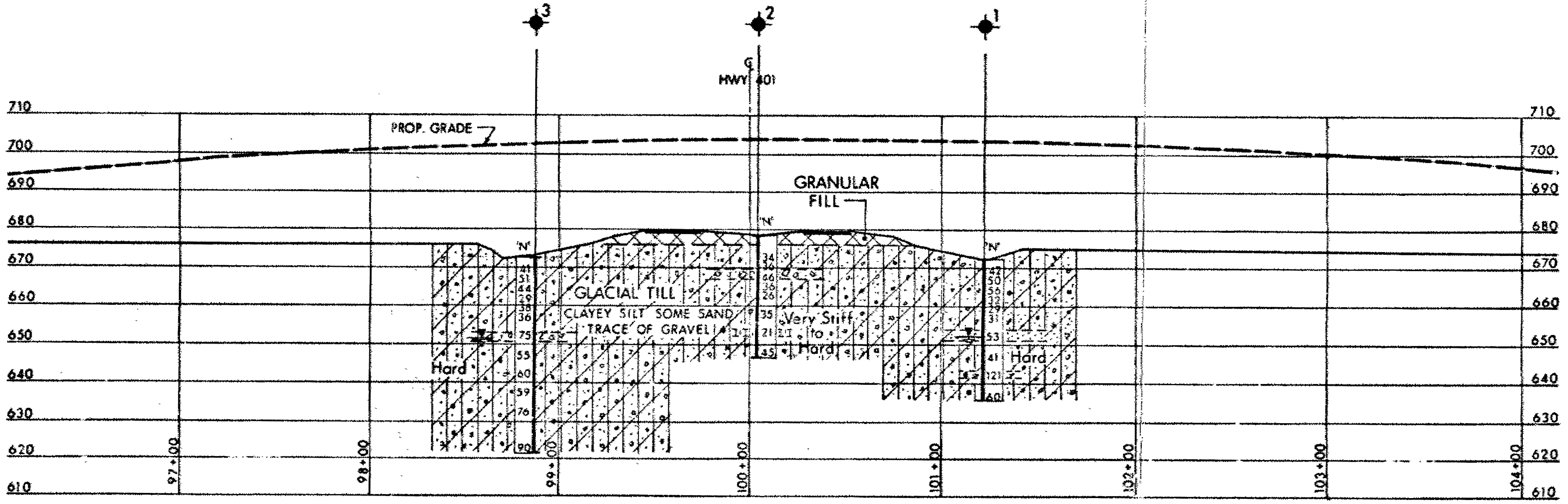
SHEET



PLAN

SCALE

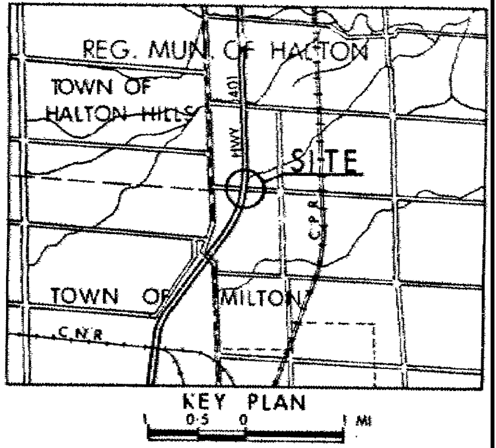
40 20 0 40 FT.



℄ PROFILE J.S.P.

SCALE

VERT. 40 20 0 40 FT.
HOR. 20 10 0 20



KEY PLAN

1 0.5 0 1 MI

LEGEND

- ● Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◈ Bore Hole & Cone
- 'N' Blows/ft (Std Pen Test 350 ft lbs energy)
- CONE Blows/ft (60° Cone, 350 ft lbs energy)
- ↓ WL at time of investigation APR 1979
  NO WL Established BH No 2

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	672·1	15 821 095	903 855
2	678·2	15 821 117	903 726
3	672·9	15 821 218	903 655

-NOTE-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS		
DATE	BY	DESCRIPTION

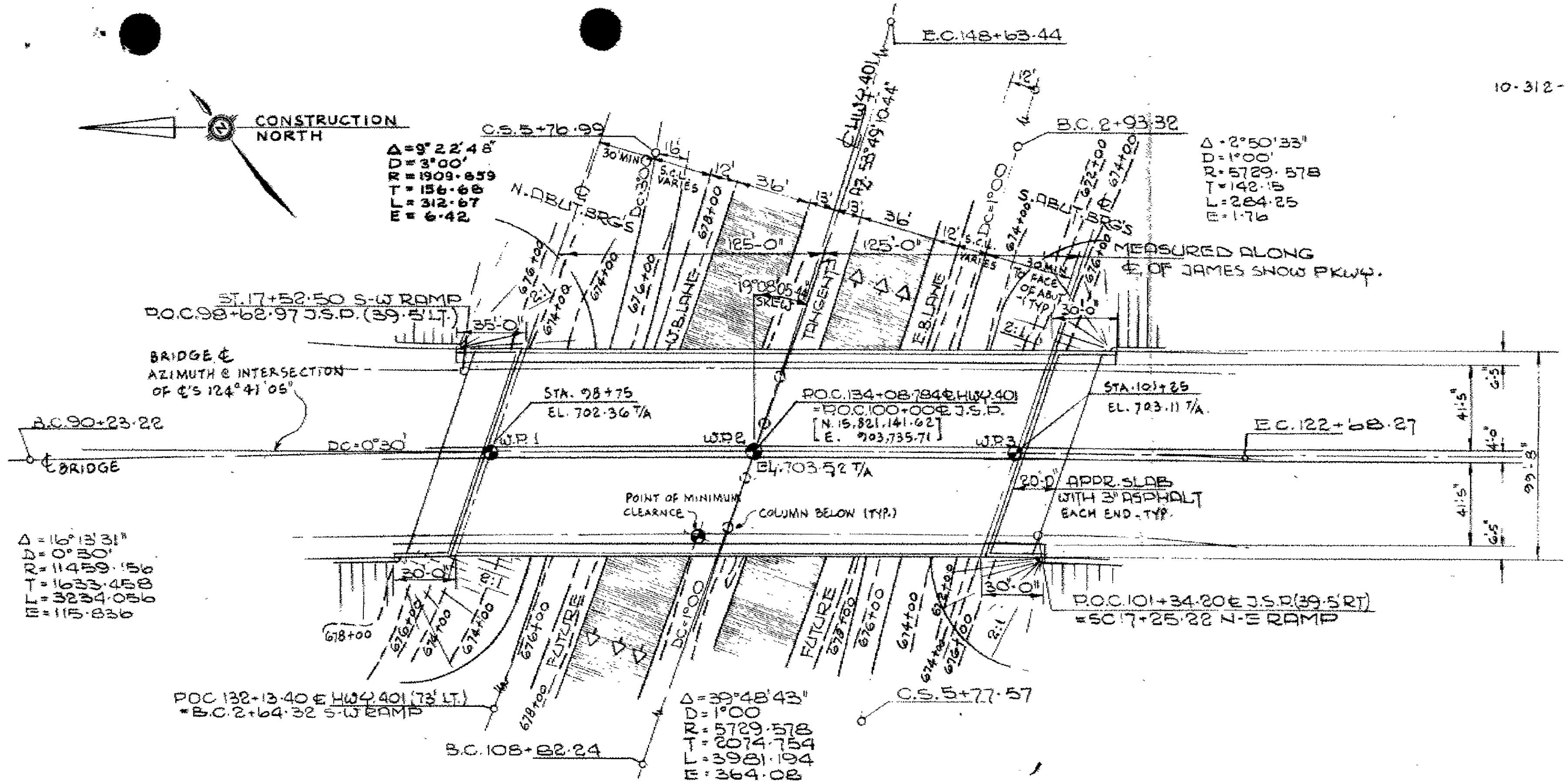
HWY No JAMES SNOW PARKWAY | DIST 6
SUBM'D T K | CHECKED | DATE 79 05 15 | SITE 10-312
DRAWN OL J | CHECKED | DWG 277802-A

PLOT DATE: January 12, 2012
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-FA- James Snow Parkway\09-1111-6036FA001.dwg

APPENDIX D

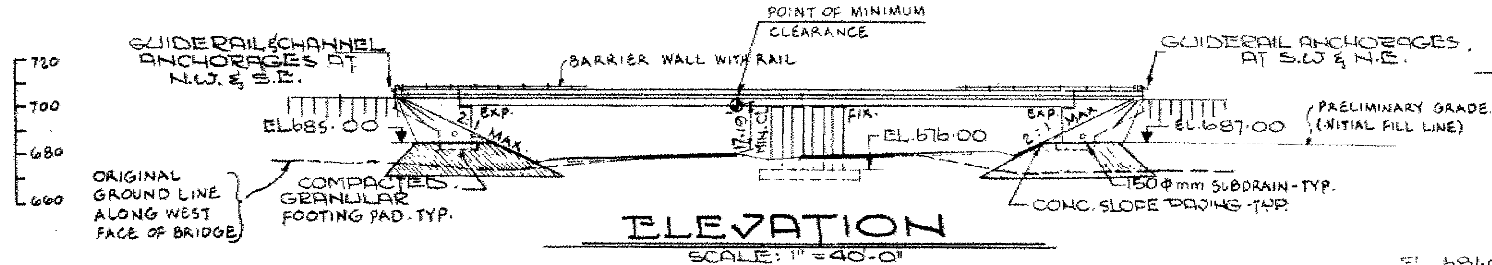
Structural Drawings
W.P. No. 27-78-02 – James Snow Parkway Bridge

MINISTRY OF TRANSPORTATION, ONTARIO

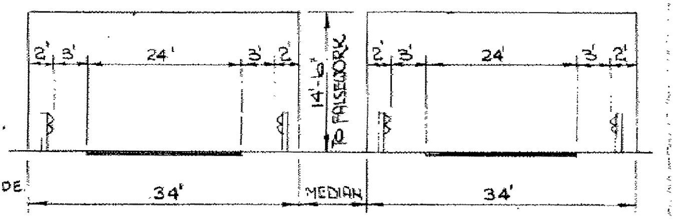


PLAN
SCALE: 1" = 40'-0"

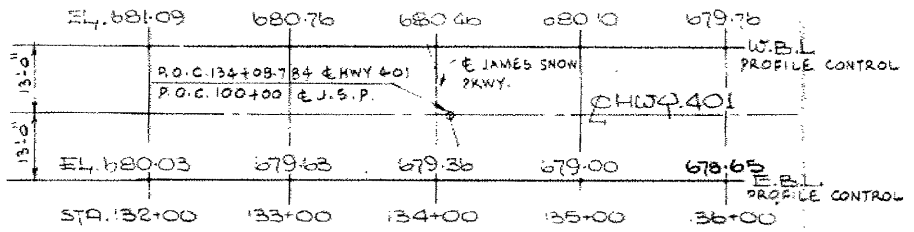
NOTES: T/A DENOTES TOP OF ASPHALT.
W.P. DENOTES WORKING POINT.
FOR ROADWAY PROTECTION ALONG PIER, SEE DETAIL ON DRAWING 7.



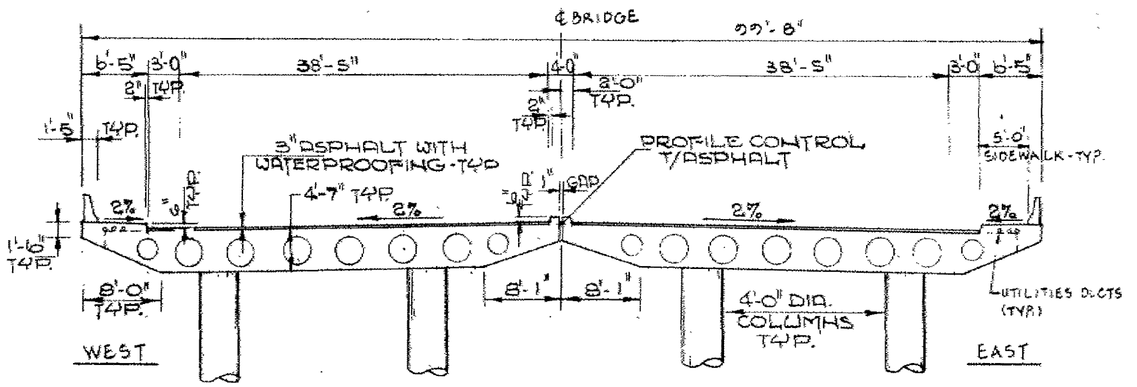
ELEVATION
SCALE: 1" = 40'-0"



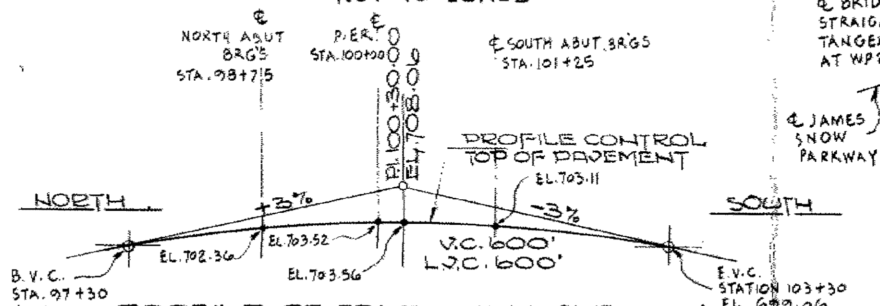
FALSEWORK CLEARANCES
4 LANE FACILITY (2 LANE IN EACH DIRECTION)
NOTE: ALL HORIZONTAL DIM. ARE MIN. DIMENSIONS REQUIRED.



HWY. 401 PAVEMENT ELEVATIONS (EXISTING)
(AT PROFILE CONTROL, 13' EITHER SIDE OF ℄ 401)
NOT TO SCALE



TYP. DECK CROSS-SECTION
SCALE: 1" = 10'-0"



PROFILE OF JAMES SNOW PKWY
NOT TO SCALE

LIST OF DRAWINGS :

- 10-312-1. GENERAL LAYOUT
- 2. BOREHOLE LOCATIONS & SOIL STRATA
- 3. FOUNDATION LAYOUT
- 4. NORTH ABUTMENT LAYOUT
- 5. SOUTH ABUTMENT LAYOUT
- 6. WINGWALL DETAILS
- 7. PIER AND BEARING DETAILS
- 8. DECK SCREED ELEVATIONS
- 9. DECK REINFORCEMENT DETAILS
- 10. LONGITUDINAL CABLES LAYOUT
- 11. TRANSVERSE CABLES LAYOUT.
- 12. PRESTRESSING AND REINFORCEMENT DETAILS I
- 13. PRESTRESSING AND REINFORCEMENT DETAILS II
- 14. BARRIER WALL WITH SIDEWALK
- 15. RAILING FOR BARRIER WALL
- 16. 20 FT. APPROACH SLAB
- 17. DETAILS OF CONCRETE SLOPE PAVING.
- 18. STANDARD DETAILS I
- 19. STANDARD DETAILS II
- 20. STANDARD DETAILS III
- 21. STANDARD DETAILS IV
- 22. BRIDGE DATE & SITE NUMBER DATA
- 23. AS CONSTRUCTED ELEVATION & DIMENSION

A. TORK

79-307
A. T.H. CHAN

CONT No
WP No 27-78-02

JAMES SNOW PARKWAY BRIDGE
AT HIGHWAY 401
GENERAL LAYOUT.

SHEET

Morrison, Hershfield,
Burgess & Huggins, Limited
Consulting Engineers

MHBH

GENERAL NOTES

CLASS OF CONCRETE

DECK & PIER COLUMNS ———— 35 MPa
BARRIER WALLS & ABUTMENTS —— 30 MPa
REMAINDER ———— 20 MPa

REINFORCING STEEL

ALL REINFORCING STEEL SHALL BE GRADE 400.

CLEAR COVER ON REINFORCING STEEL

FOOTINGS ———— 3"
PIER, ABUTMENTS & WINGWALLS ———— 2"
DECK TOP ———— 2"
DECK BOTTOM (TO SUPPORT BARS) ———— 1½"
APPROACH SLABS & SIDEWALKS ———— 2"
AND/OR AS NOTED ON DRAWINGS

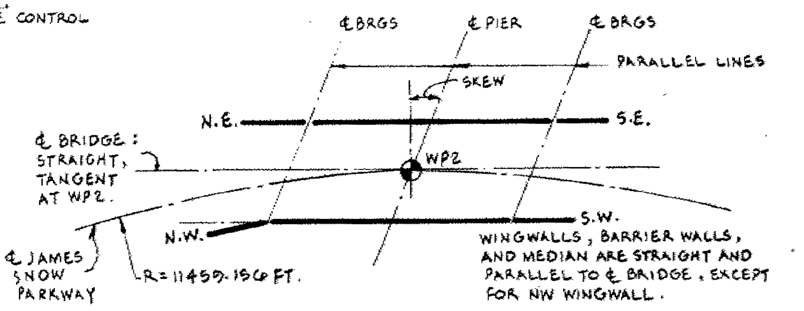
CONSTRUCTION NOTES

- THE CONTRACTOR SHALL FINISH THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8"
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND GROUTED.
- TO ACHIEVE THE MINIMUM CLEAR COVER OF 2" SPECIFIED, THE TOP LAYER OF DECK STEEL SHALL BE PLACED PRIOR TO CONCRETING WITH A CLEAR COVER OF 2½" ± ½" TOLERANCE.
- REINFORCING BARS WITH THE DESIGNATION "C" AT THE END OF BAR MARKS SHALL BE COATED BARS.

CONCRETE QUANTITIES :

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE LUMP SUM TENDER ITEMS.

- CONCRETE IN PIERS, ABUTMENTS & WINGWALLS ———— 493 C.Y
- PRESTRESSED CONCRETE BRIDGE DECK ———— 2923 C.Y
- CONCRETE IN BARRIER WALLS ———— 47 C.Y
- CONCRETE IN APPROACH SLABS ———— 120 C.Y
- CONCRETE IN SLOPE PAVINGS ———— 97 C.Y



SCHEMATIC DIAGRAM OF BRIDGE LAYOUT
N.T.S.

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN A.C.	CHECK A.T.	LOADING HS20-44	DATE 14.3.80
DRAWING [illegible]	CHECK A.C.	SITE No 10-312	DWG 1

PLOT DATE: January 12, 2012
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-FA- James Snow Parkway\09-1111-6036FA002.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

DIST. No 4
CONT No
WP No 27-78-02

JAMES SNOW PARKWAY BRIDGE
AT HIGHWAY 401

FOUNDATION LAYOUT

SHEET

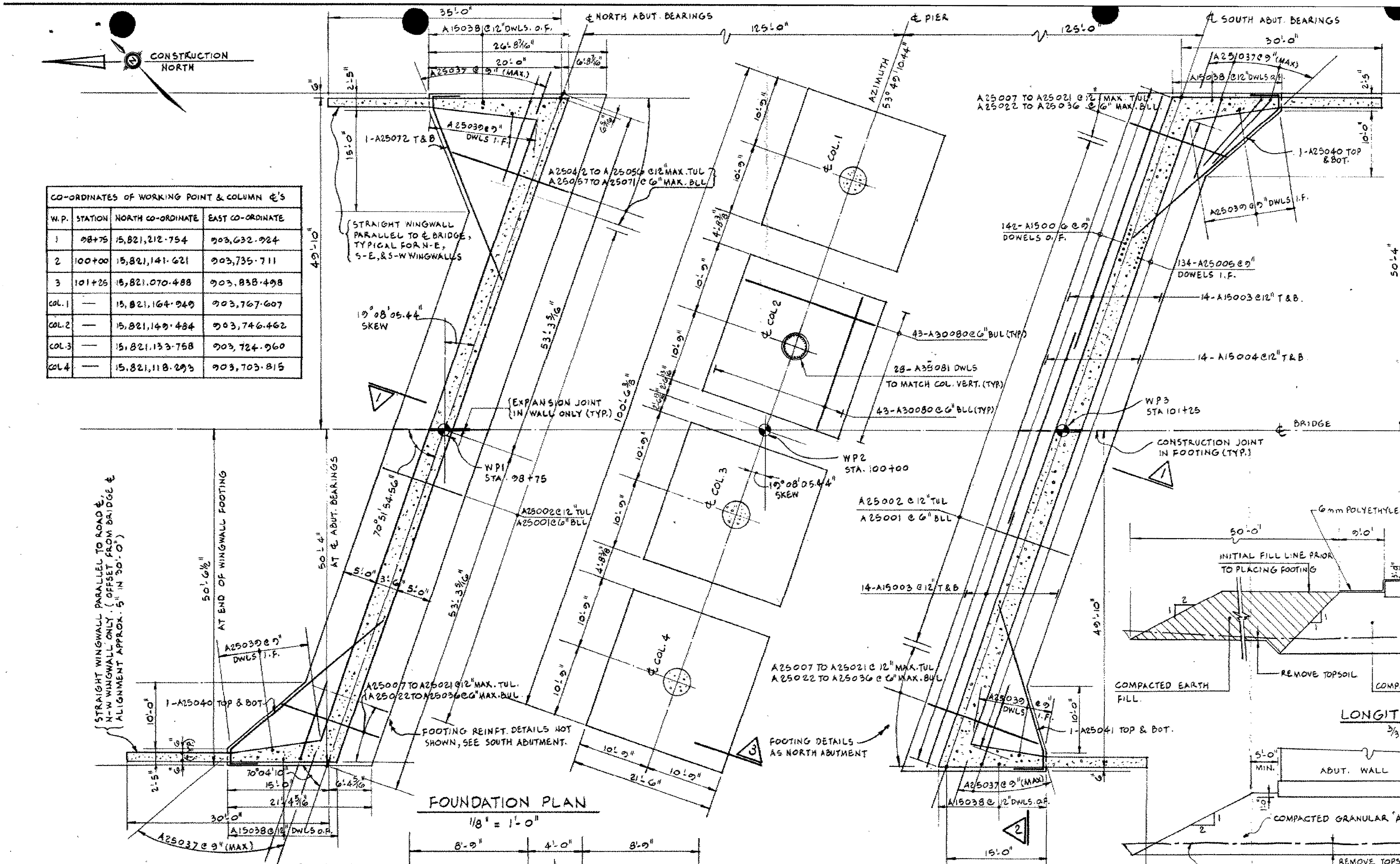
Morrison, Hershfield,
Burgess & Huggins, Limited
Consulting Engineers

MHBH

A. TORK

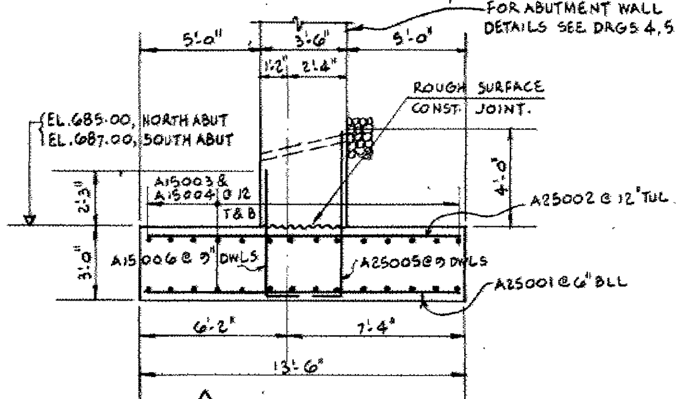
79-307
A. M. CHAN

CO-ORDINATES OF WORKING POINT & COLUMN ℄'S			
W.P.	STATION	NORTH CO-ORDINATE	EAST CO-ORDINATE
1	98+75	15,821,212·754	903,632·924
2	100+00	15,821,141·621	903,735·711
3	101+25	15,821,070·488	903,838·498
COL. 1	—	15,821,164·949	903,767·607
COL. 2	—	15,821,149·484	903,746·462
COL. 3	—	15,821,133·758	903,724·960
COL 4	—	15,821,118·293	903,703·815

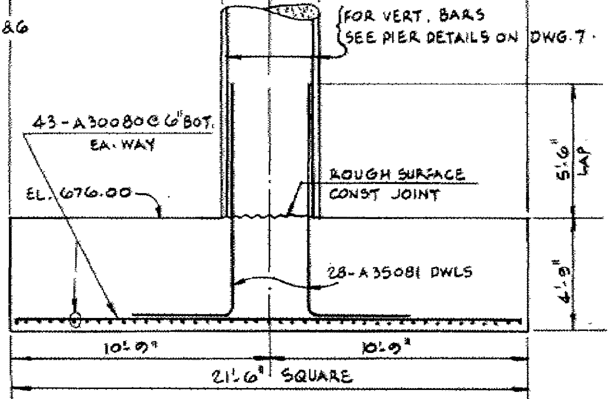


FOUNDATION PLAN

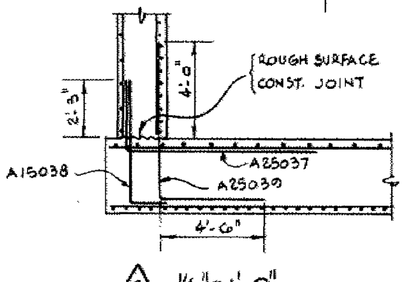
1/8" = 1'-0"



1 1/4" = 1'-0"



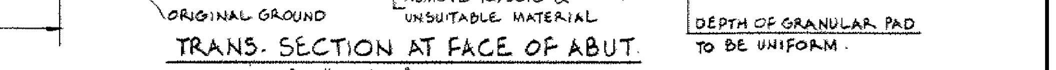
3 1/4" = 1'-0" (TYPICAL FOR ALL COLUMN FOOTINGS)

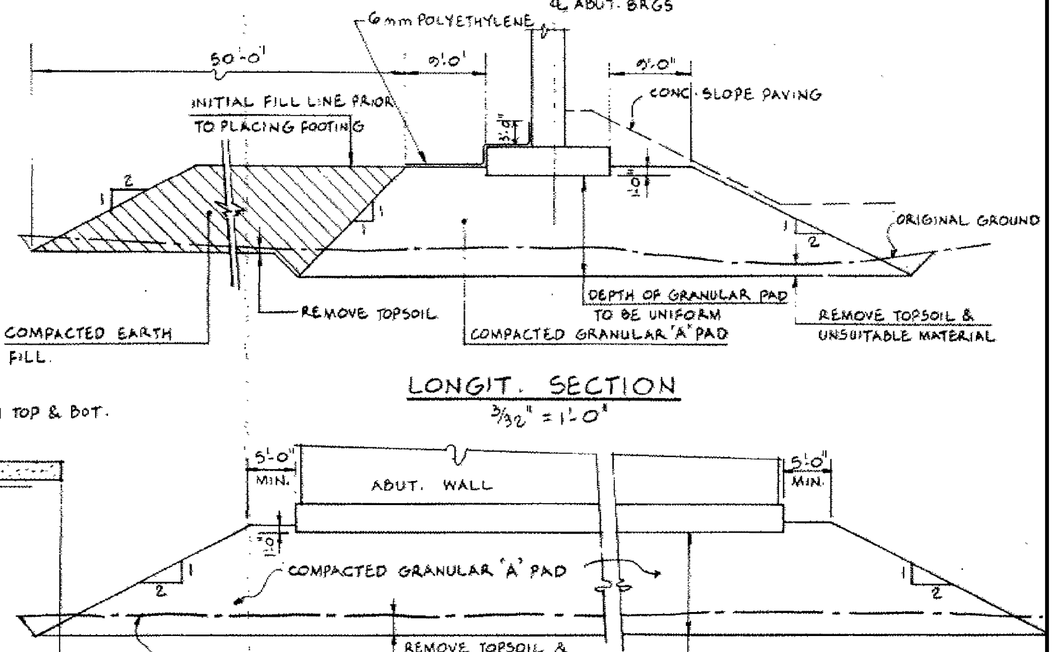


2 1/4" = 1'-0"

NOTES :

- I.F. DENOTES INSIDE FACE.
- O.F. DENOTES OUTSIDE FACE.
- BOTTOM AND SIDES OF FOOTINGS TO BE CAST AGAINST UNDISTURBED SOIL.
- FOR ABUTMENT AND WINGWALL DETAILS, SEE DRAWINGS 4, 5 AND 6
- FOR PIER DETAILS, SEE DRAWING 7
- TUL DENOTES TOP UPPER LAYER
- TLL DENOTES TOP LOWER LAYER
- BUL DENOTES BOTTOM UPPER LAYER
- BLL DENOTES BOTTOM LOWER LAYER
- UNLESS OTHERWISE NOTED, MINIMUM LAP LENGTHS FOR REINF. BARS :
  15 M = 2'-3"
  20 M = 2'-9"
  25 M = 4'-0"





LONGIT. SECTION
3/32" = 1'-0"

TRANS. SECTION AT FACE OF ABUT.
3/32" = 1'-0"

DETAILS OF GRANULAR 'A' PAD

CONSTRUCTION SEQUENCE FOR ABUTMENTS

1. STRIP TOPSOIL, EXCAVATE, AND CONSTRUCT GRANULAR 'A' PAD.
2. CONSTRUCT EARTH FILL TO INITIAL FILL LINE SHOWN.
3. RE-EXCAVATE AND PLACE ABUTMENT FOOTING.
4. COMPLETE ABUTMENT WALL AND WINGWALLS.
5. COMPLETE ABUTMENT BACKFILL AND EARTH FILL TO REQ'D GRADE.

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN A.C	CHECK A.T	LOADING HS20-44	DATE 14.3.80
DRAWING P.Mo	CHECK A.C.	SITE No 10-312	DWG 3

PLOT DATE: January 12, 2012
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-FA- James Snow Parkway\09-1111-6036FA003.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

DIST. No 4
CONT No
WP No 27-78-02

JAMES SNOW PARKWAY BRIDGE
AT HIGHWAY 401.
NORTH ABUTMENT LAYOUT

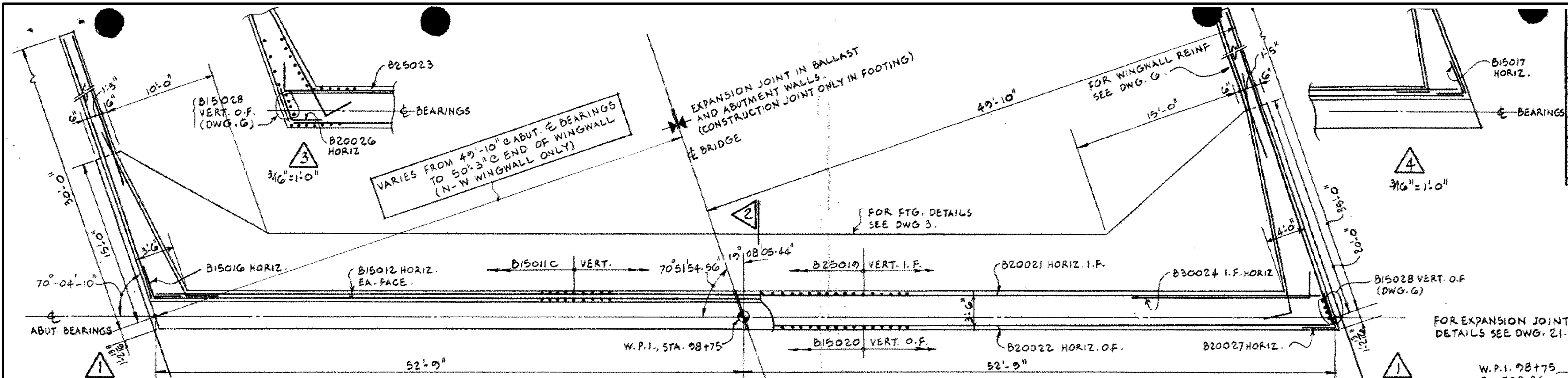
SHEET

Morrison, Hershfield, Burgess & Huggins, Limited
Consulting Engineers

MHBH

A. TORK

79-30?
A. CHAN

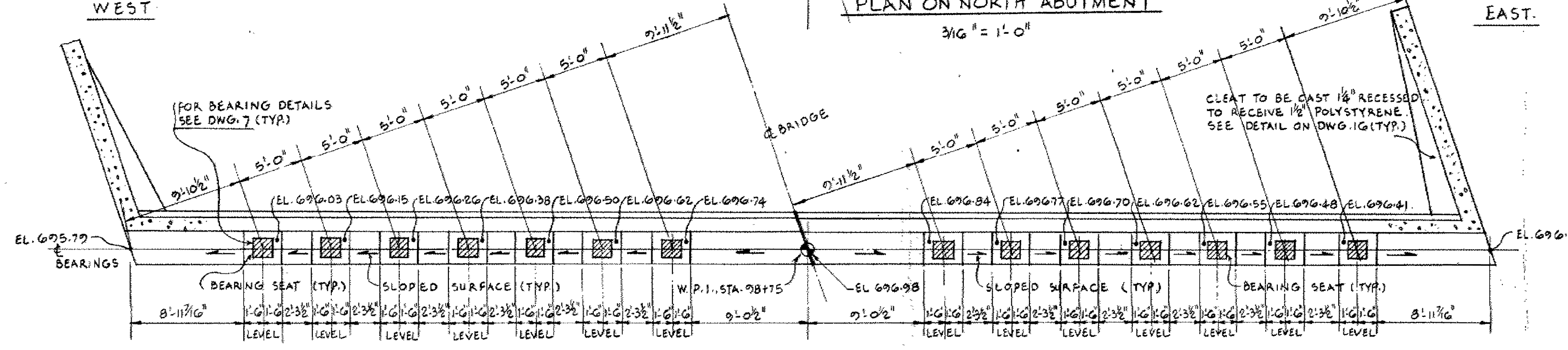


WEST

PLAN ON NORTH ABUTMENT

3/16" = 1'-0"

EAST.

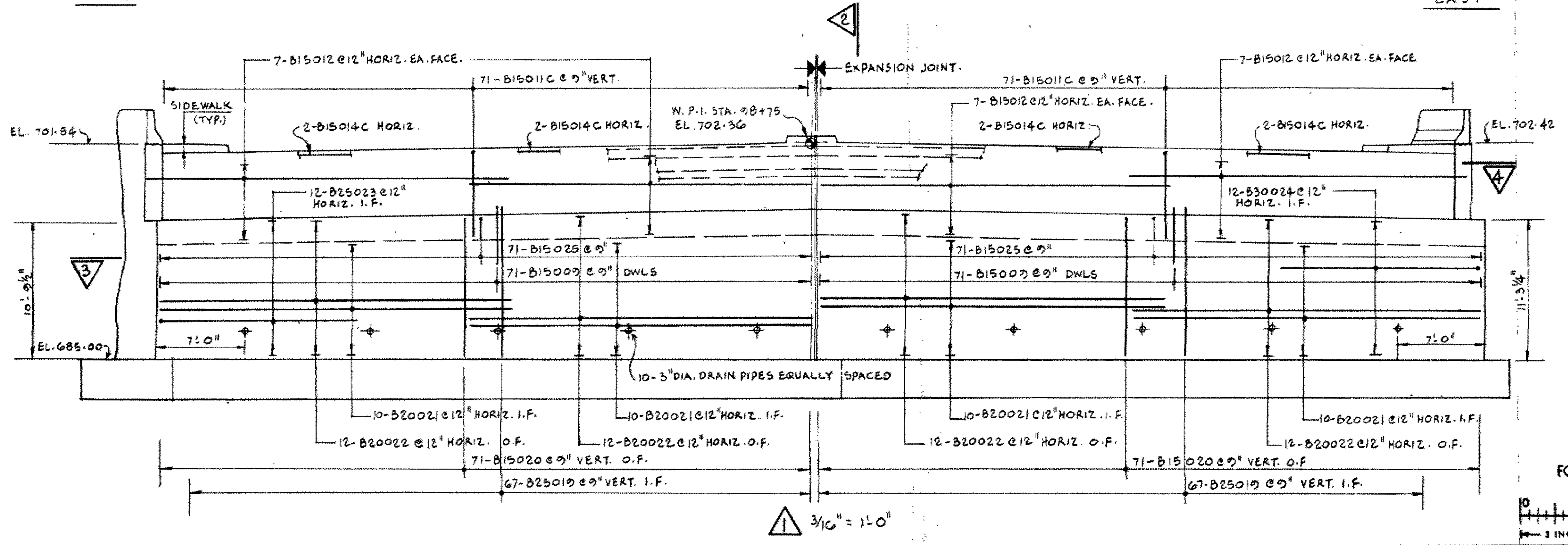


PLAN ON BEARING SEATS

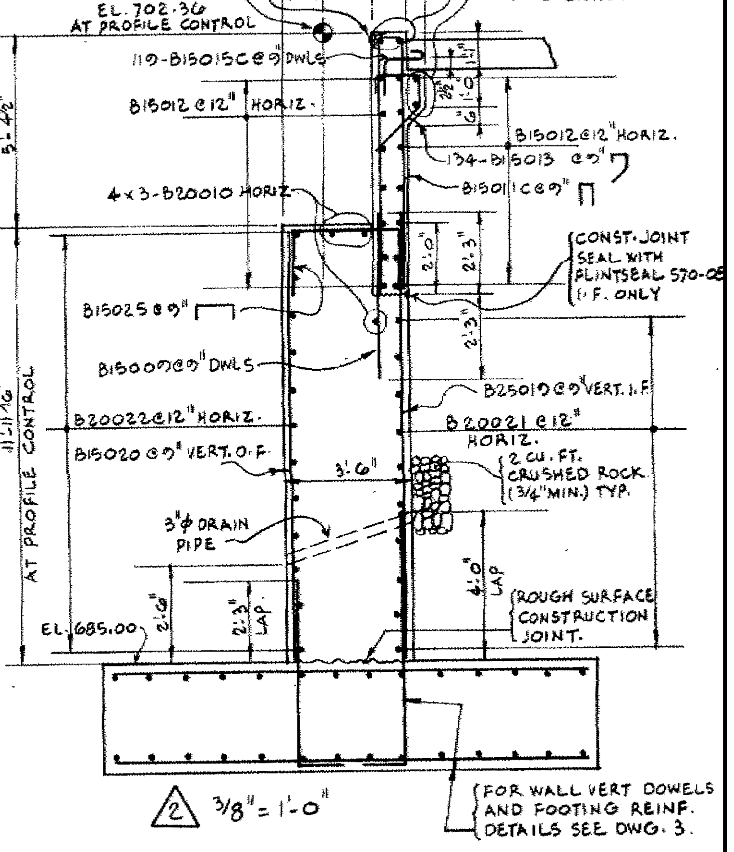
3/16" = 1'-0"

WEST

EAST



1 3/16" = 1'-0"



2 3/8" = 1'-0"

NOTES:

- E.F. DENOTES EACH FACE.
- I.F. DENOTES INSIDE FACE.
- O.F. DENOTES OUTSIDE FACE.
- CONSTRUCTION JOINT WITH 1" VEE GROOVE CAULKED WITH FLINTSEAL 570-05 ALONG INSIDE FACE (SEE DWG. 19)
- UNLESS OTHERWISE NOTED, MINIMUM LAP LENGTHS FOR REINF. BARS:
  15 M = 2'-3"
  20 M = 2'-9"
  25 M = 4'-0"
- FOR LAYOUT OF WINGWALLS, SEE DWGS. 3 & 6

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

REVISIONS					
DATE	BY		DESCRIPTION		
DESIGN A.C.	CHECK A.T.		LOADING HS 20-44	DATE 14.3.80	
DRAWING P.Mc	CHECK A.C.		SITE No 10-312	DWG 4	

PLOT DATE: January 12, 2012
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-FA- James Snow Parkway\09-1111-6036FA004.dwg

APPENDIX E

Design Recommendations for Structural Assessment in Preliminary Design of Retaining Structures

E. GEOTECHNICAL RECOMMENDATIONS FOR STRUCTURAL ASSESSMENT

This appendix provides geotechnical recommendations to support structural assessment as required during the preliminary design of the retaining structures at this site.

E.1 Secant Pile Wall – Design Considerations

For a preliminary secant wall design, the lateral resistance of the caissons may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $n_h \,.\, z / D$ [cohesionless soils] (kN/m^3)
k_s = 67 S_u/D [cohesive soils] (kN/m^3)
p_{ult} = 3 . γ . z . K_p [cohesionless soils] (kPa)
p_{ult} = 9 S_u [cohesive soils] (kPa)

where z = depth of embedment of caisson (m)
D = caisson diameter (m)
S_u = undrained shear strength (kPa) as provided in the table below
n_h = coefficient of horizontal subgrade reaction (kN/m^3)
γ = unit weight (kN/m^3) as provided in the table below
K_p = passive earth pressure coefficient

The above equations and recommended parameters may be used to analyze the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the caisson. For preliminary design purposes, a lateral load of 300 kN (ULS) can be used for caissons located less than 2 m away from the edge of the abutment footing. The caissons should be designed to act as rigid elements (i.e., a length-to-diameter ratio (L/d) of not greater than 10).

The spring constant, K, for analysis may be obtained by the expression, K = k_s x L x D (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the caisson diameter (m) and L is the length (m) of the caisson segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}$ x L x D.

The following table provides the recommended geotechnical parameters for use in the design approach outlined above.

Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation*8	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)*
North Abutment BH No. 3	210.0 – 204.8 204.8 – 189.4	Granular “A” Clayey Silt Till	22.8 21	35 0	– 200	6600 –
South Abutment BH No. 1	210.0 – 204.8 204.8 – 193.8	Granular “A” Clayey Silt Till	22.8 21	35 0	– 200	6600 –

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992
** Granular “A” elevations are estimates based on James Snow Parkway Bridge Drawings, W.P. No. 27-78-02.

E.2 Soldier Pile and Lagging Wall – Design Considerations

The shape of the soil pressure distribution diagram behind a shoring system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The shoring system can be restrained, fixed or flexible. The sequence of work may also alter the shape of the pressure diagram during the various construction phases.

Earth pressure computations must also take into account the groundwater level. Above the groundwater level, earth pressure is computed using the bulk unit weight of the retained soil. Below the groundwater level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained. Earth pressures acting on the retaining structure should be computed in accordance with Clause 6.9 of the CHBDC 2006. The appropriate pressures can be computed from the expression:

$$P_h = K[\gamma (h - h_w) + (\gamma' h_w) + q] + h_w \gamma_w$$

where P_h = horizontal pressure on the wall (kPa)
K = earth pressure coefficient
γ = bulk unit weight of retained soil
γ' = submerged unit weight of soil ($\gamma - \gamma_w$)
γ_w = unit weight of water (9.81 kN/m^3)
h = depth below surface (m)
h_w = depth below the ground water level (m)
q = value of any surcharge (kPa)

If the retaining structure is anchored at more than one level, then it is recommended that the design of the system be undertaken based on an earth pressure distribution consisting of a uniformly distributed pressure defined by the expression:

$$P = 0.65\,K[\gamma\,h + q] + h_w\gamma_w$$

where K = earth pressure coefficient

γ = bulk unit weight of retained soil

γ_w = unit weight of water (9.81 kN/m^3)

h = depth below surface (m)

h_w = depth below the ground water level (m)

q = value of any surcharge (kPa)

The design should be based on the at-rest earth pressure coefficient (K_o). For “kick out” design, the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p). The lateral earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC, 2006.

For the toe design of the soldier pile in the cohesive clayey silt till, the ultimate horizontal resistance can be estimated as $4c_u$, where c_u is the undrained shear strength of the clayey silt till. An undrained shear strength of 250 kPa can be used for preliminary design assessment.

The recommended unfactored values of the parameters for use in the design of structures subject to unbalanced earth pressures are given in the following table.

Earth Pressure Coefficients

Soil	ϕ (deg)	γ (kN/m^3)	K_a	K_o	K_p
Granular “A” Fill	35	22.8	0.27	0.43	3.70
Clayey Silt Till	30	21	0.33	0.50	3.00

E.3 Tie-Backs, Ground Anchors and Soil Nails – Design Considerations

For the purposes of the preliminary design assessment, the capacities of the anchors may be calculated using the method provided in the Canadian Foundation Engineering Manual (CFEM, 2006) as follows:

For cohesive deposits:

$$P_{ar} = \Phi\, s_u\, A_s\, L_s\, \alpha_c$$

where P_{ar} = pull-out resistance of the grouted ground anchor (kN)

Φ = resistance factor = 0.6 (ULS)

s_u = average undrained shear strength of the soil (kPa)

A_s = effective unit surface area of the anchor bond zone (i.e., circumference) (m^2/m)

L_s = effective length of the anchor bond zone (limited to approximately 8 m)

α_c =adhesion (reduction) factor related to the undrained shear strength (assume 0.4)

Based on the available borehole information for the James Snow Parkway underpass site, an undrained shear strength of 250 kPa can be used for preliminary design estimates of the bond strength of grouted soil anchors installed in the clayey silt till deposit.

For the Granular "A" pad:

$$P_{ar} = \Phi\ \sigma'_z\ A_s\ L_s\ \alpha_g$$

where P_{ar} = pull-out resistance of the grouted ground anchor (kN)

Φ = resistance factor = 0.6 (ULS)

σ'_z = effective vertical stress at the midpoint of the load carrying length (kPa)

A_s = effective unit surface area of the anchor bond zone (i.e., circumference) (m_2/m)

L_s = effective length of the anchor bond zone (limited to 8 m)

α_g = adhesion/bonding factor (assumed 2.0 for design)

Anchors may be sized based on the following factored soil-to-grout bond stresses at ULS:

- Granular "A" fill: $\Phi\ \sigma'_z\ \alpha_g = 135$ kPa
- Clayey silt till: $\Phi\ s_u\ \alpha_c = 60$ kPa

Improved ground anchor capacity within both the Granular "A" fill and the clayey silt till can be achieved by post-grouting. Apart from increasing the apparent bond stress, the technique also has the advantage of allowing individual anchors to be re-grouted and improved, if proof testing shows a particular anchor to be deficient.

E.4 Conventional Concrete Retaining Wall – Design Considerations

Strip footings placed on the properly prepared, hard clayey silt, at or below a depth of 1.2 m (for frost protection purposes), should be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa, and a geotechnical resistance at Serviceability Limit States (SLS) of 400 kPa. This SLS value is based on a maximum footing width of 2.5 m. These preliminary geotechnical resistance values will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation and the proposed retaining structure geometry.

Resistance to lateral forces/sliding resistance between the concrete footing and the subgrade soils should be evaluated in accordance with the CHBDC 2006. The following ultimate coefficient of friction values are recommended between concrete and the subgrade soils, which are expected to consist of either compacted granular fill or native clayey silt till.

- Compacted Granular A – ultimate coefficient of friction of 0.7; and
- Clayey silt till – ultimate coefficient of friction of 0.58.

Earth pressures are generally calculated using the following expression:

$P_h = K(\gamma h + q)$

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient

γ = unit weight of retained soil

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC 2006 and according to Clause 6.9.3 of the CHBDC 2006, a compaction surcharge should also be added. Earth pressure coefficients for retaining structures are dependent on the material used as backfill and typical values are provided in the following table for preliminary design purposes.

Earth Pressure Coefficients

Wall Condition	**Earth Pressure Coefficient (K)**			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.30	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-

The earth pressure coefficients in the table above are “ultimate” values that require certain structural movements for the respective conditions to be mobilized. The values to use in the preliminary design can be estimated from Figure C6.16 in the Commentary to the CHBDC, 2006.

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