DRAFT

INTERIM REPORT ON

GEOTECHNICAL CONSIDERATIONS FOR BELOW-GRADE APPROACH ROADWAYS, CUT AND COVER, AND TUNNEL OPTIONS DETROIT RIVER INTERNATIONAL CROSSING

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September 2006 04-1111-060 (5100)

EXECUTIVE SUMMARY

This report provides a overview of various retaining wall systems, grade cuts and tunnel options being considered for the extension of Highway 401 from its existing terminus at Highway 3 northwest to Huron Church Road, along Huron Church Road to the intersection with E.C. Row Expressway. It is understood that between Huron Church Road and Ojibway Parkway that the Highway 401 corridor is proposed to be constructed at or above existing ground elevations and parallel to the E.C. Row Expressway. The extension of Highway 401 is an integral part of the Detroit River International Crossing (DRIC) project.

The available subsurface information and evaluations completed to date as part of this study suggest that construction of open-cut (depressed roadway) sections may be made to assist in separating traffic grades with permanent side slopes of approximately 2.5:1 (horizontal:vertical) or with permanent retaining structures (using a variety of systems) provided that the cuts are no deeper than about 10 to 14 m at Highway 401 and Highway 3 decreasing to about 5 to 7 m at Huron Church Road and E.C. Row Expressway and westward.

Cut and cover tunnels should be feasible for the entire length of the approaches, however, base stability conditions may require either temporary ground improvement measures or other temporary wall and base stability enhancements during construction of excavations deeper than about 10 m from about the Highway 3 and Todd Lane near the intersection of Huron Church Road and E.C. Row Expressway. From Huron Church Road proceeding westward along E.C. Row Expressway, the depths of cut requiring additional stabilization measures decrease such that near the intersection of Ojibway Parkway and E.C. Row Expressway, the maximum depth of cut is limited to about 6 m. The potential cut depths near the Grand Marais and Cahill Drains will require further evaluation and it is anticipated that additional stability enhancement measures may be required in these areas.

Notwithstanding any traffic safety, capacity, or accessibility issues, which are not part of this report, construction of bored tunnels is considered impractical because of the potential difficulties associated with approach cut construction, the limited thickness of overburden that results in low cover over the tunnel, and the potential for unacceptable settlements created at the surface from bored tunnelling.

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1.0 INTRODUCTION

This report presents geotechnical evaluations and recommendations related to the highway access route portion of the Area of Continued Analysis (ACA) associated with the Detroit River International Crossing (DRIC) between Windsor, Ontario, and Detroit, Michigan. This work was undertaken at the request of URS Corporation as part of an on-going study for a joint partnership between the Ministry of Transportation Ontario, Transport Canada, the Michigan Department of Transportation (MDOT), and the US Federal Highway Administration (FHWA). The existing site conditions and scope of the overall project are described in an earlier report completed by Golder Associates Ltd. in July, 2005, titled "Interim Foundations & Geotechnical Engineering Report, Detroit River International Crossing, Windsor, Ontario, W.O. 04-33-002 (Geocres No. 40J6-14)". This report is provided to describe further evaluations conducted specifically for the ACA selected based on the outcome of the July 2005 report and other studies undertaken by URS. Reference should be made to the previous report for discussion of other aspects of the DRIC project.

2.0 PROJECT DESCRIPTION

It is understood that several conceptual designs are under consideration, typically involving six-lane urban freeway sections. The concepts revolve around three main methods of freeway construction through the corridor which are at-grade, depressed roadway and below-grade tunnel constructed as either a cut and cover structure or a bored tunnel with cut and cover approaches. The various options for each method of constructing the urban freeway sections are summarized as follows:

- At-grade with retaining walls, noise berms/walls, and service roads;
- Depressed roadway in an open excavation with sloped sides or retaining walls and service roads;
- Depressed roadway within cut and cover (or top-down) tunnel section with service roads and parking lanes above the tunnel roof; and
- Depressed roadway within bored or mined tunnel with service roads above the tunnel and depressed roadways and cut and cover structures leading to and from the bored tunnel.

If the approach roadways are constructed as cut and cover structures, it is anticipated that the structure will be approximately 45 to 50 m wide. These structures may be built in two halves to permit continued vehicle traffic along the routes during construction. If the six lane roadway is to be constructed within bored tunnels, three bored tunnels would each be about 15 m diameter and carry two lanes of traffic with the tunnels separated by at least 7 to 8 m (outside to outside of tunnel lining).

3.0 SITE DESCRIPTION AND GEOLOGY

The site is located in Windsor, Ontario (See Figure 1). The corridor is approximately 9 km in length and it passes through several urban residential and commercial areas. Highway 401 may be extended from its current terminus at Highway 3 (Talbot Road East) northwest along Highway 3 to Huron Church Road, along Huron Church road to the intersection with EC Row Expressway, and then adjacent to the E.C. Row Expressway to its intersection with Ojibway Parkway.

Existing subsurface data was used to complete the study described in this report. Information was gathered from Ministry of Transportation Ontario files (through the GEOCRES system), Golder Associates project files, the Ontario Ministry of Natural Resources (MNR), Ontario Ministry of the Environment (MOE), State of Michigan Geographic Data Library, published papers and books, and cooperative sharing of information with NTH Consultants (geotechnical consultants to MDOT for the DRIC partnership). Data resources and references are listed at the conclusion of this report. Figure 1 illustrates the locations where subsurface data related to soil conditions through the full overburden profile were available along the corridor.

The topography along the proposed corridor is flat to gently undulating with the ground surface sloping downward to the northwest from about elevation 187 metres near Highway 401 to elevation 180 metres at E.C. Row Expressway and Ojibway Parkway. The corridor is crossed by three municipal drains, the most significant of which is the Grand Marais (Turkey Creek) Drain. Two smaller drains, the Lennon Drain and the Cahill Drain cross the alignment just south of the intersection of Huron Church Road and Highway 3, west of St. Clair College. Figure 1 illustrates the approximate ground surface, soil stratigraphy, available deep borehole data, and the anticipated bedrock surface along this ACA alignment.

The site lies in a physiographic region known as the Essex Clay Plains which drains northwards into Lake St. Clair. According to Ontario Department of Mines and Northern Affairs Preliminary Map P.749, the overburden soils along the corridor consist of clayey silt till to the east and lacustrine sand overlying silty clay till to the west. Limestone, dolostone or shale bedrock of the Hamilton Group or Detroit River Group is expected to lie beneath approximately 25 to 35 m of overburden. Available information indicates that the bedrock surface in the study area is generally found at elevations ranging between about El. 148 m and El. 155 m, although there are local variations in the bedrock surface elevation. Along the majority of the proposed approach corridor, the bedrock is anticipated to be limestone of the Dundee Formation (Detroit River Group).

Information from boreholes advanced for previous investigations in the area indicate that the stratigraphy along the proposed alignment consists of surficial fill, sands or organic material overlying extensive deposits of relatively soft to firm silty clay or clayey silt. These soils are likely tills of glaciolacustrine origin (deposited from the base of a glacier into water). Layers of sand or sandy silt 2 to 3 m thick were encountered at some boreholes between the silty clay and the underlying bedrock.

Between the existing terminus of Highway 401 and E.C. Row Expressway, the soils are predominantly cohesive in nature and consist of a stiff upper "crust" which extends to depths of 4 to 6 metres below the ground surface. Above Elevation 173 to 180 m, the soils can be described as very stiff to hard in some areas but are predominantly very stiff based on Standard Penetration Test “N values” that average about 20 blows per 0.3 metres of penetration. The average elevation of the base of the crust is approximately 176 metres, or about 5 metres below the ground surface. Below the "crust", blow counts range between 4 and 25 per 0.3 metres but are generally less than 9 blows per 0.3 metres. The silty clay is firm to stiff based on the results of unconfined compression and in situ vane shear strength testing in which measured shear strengths ranging between 35 to 90 kPa are reported.

West of the intersection of E.C. Row Expressway and Huron Church Road the soil conditions become progressively softer. Localized zones of very soft silty clay are also known to exist in areas west of Huron Church Road. Beneath surficial layers of sand, fill and topsoil lie layers of soft to stiff, but generally firm, silty clay or, less frequently, clayey silt. Shear strengths in these soils range between 25 and 75 kPa based on the results of unconfined triaxial compression and in situ vane shear strength testing. The sensitivity of the soils vary between 1.5 and 4.0. As with the soils to the east of this intersection, a generally stiff, desiccated "crust" is present to a depth of 5 to 6 metres below the ground surface or between elevations 172 to 170 metres. Standard Penetration Test blow counts (SPT “N” values) in the "crust" are between 5 and 13 blows per 0.3 metres. Below the "crust", SPT “N” values range from the weight of the sampling hammer to 11 blows per 0.3 metres with an average of 5 blows per 0.3 metres.

Groundwater was encountered at depths of 0.6 to 6.6 m depending on the local ground surface elevations. The estimated static groundwater surface elevations range from about 175 to 180 m. In some areas, particularly near the Detroit River, where ground surface elevations are relatively low, groundwater pressures within the bedrock may be equivalent to a water elevation of about 179 m, or on the order of 2 m to 3 m above the ground surface elevation (artesian) near the river shoreline.

4.0 EARTH RETAINING SYSTEMS TECHNOLOGIES

Earth retaining systems will be required for at-grade sections that incorporate noise berms integral with retaining walls, or for depressed roadway sections constructed in cuts and for the approaches to any tunnel section where cut slopes can not be utilized. Selection of the appropriate temporary and permanent retaining systems for the corridor depends on the cost, final surface finish requirements, available subsurface easements, available surface easements for open excavation, lateral earth pressures, groundwater control requirements, and roadway clearance for horizontal braces between the walls.

Earth retaining systems typically can be grouped into two categories based on the means by which they are constructed. Gravity walls are constructed “bottom up” from the base of a cut and then backfilled. 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. Retaining systems can also be categorized considering two main components: the structural wall face; and the lateral restraint system which resists the horizontal earth pressures. The type of structural wall face can be most readily categorized by the construction technique used to form the wall. Generally walls are either constructed prior to excavation (in situ walls) or in an open excavation. Those constructed in an open excavation can be built either after the full depth has been established or from the top down as excavation proceeds. Depending on the type of wall constructed, the combination of the embedment of the wall below the excavation bottom and the structural capacity of the wall may be sufficient to resist the horizontal earth load (cantilever walls). If construction of a cantilever wall is not feasible, horizontal displacement of the excavation sidewalls is commonly restrained by inclusion of internal braces placed between two opposing wall faces; steel rods or wires (strands) drilled into and anchored in the ground behind the wall (tie-backs, ground anchors, or soil nails); or the base friction arising from the weight of either the wall itself or of backfill placed on top of an integrally connected footing.

The following discussion identifies various permanent retaining systems for each option under the two main construction methods. The feasibility of each system has been evaluated on a conceptual level based on technical considerations such as compatibility with ground conditions as understood based on available information, installation and workspace requirements; economic factors such as installation costs and minimization of traffic disruption. This information may be used by the DRIC project team to assist in analysis or refinement of alternatives.

Depressed roadway sections can be built either in sloped road cuts or in cuts where the sidewalls are permanently supported. Cut and cover tunnel sections can be constructed either in a top down or bottom up fashion. In top down construction, a permanent excavation sidewall is constructed followed by a permanent deck that can be established as a finished permanent roadway prior to undertaking further below-ground work. Excavation to form the tunnel and base then proceeds beneath the deck without further traffic disruptions. In bottom up construction, construction of the permanent tunnel walls is followed by excavation to the required depth, building the tunnel base, sidewalls, and finally constructing the deck/roof section and re-establishing the surface roadway.

The following table summarizes the types of retaining systems which were considered for each method of construction:

OPTION	PERMANENT RETAINING SYSTEMS	GENERAL WALL TYPE
At-grade Noise Berm Retention	Cast-in-Place Reinforced Concrete Wall	Gravity
	Pre-Cast Cantilever or Counterfort Wall	Gravity
	Crib and Bin Walls	Gravity
	Mechanically Stabilized Earth (MSE) Wall	Gravity
	Soldier Piles and Lagging Wall (with permanent facing)	In Situ
Depressed Roadway: Open Cut	Cast-in-Place Reinforced Concrete Wall	Gravity
	Pre-Cast Cantilever or Counterfort Wall	Gravity
	Crib and Bin Wall	Gravity
	MSE Wall	Gravity
	Soil Nail Wall	In Situ
	Soldier Pile and Lagging Wall	In Situ
	Secant or Tangent Pile (Caisson) Wall	In Situ
	Driven Sheet Pile Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Soil-Cement or Deep Soil Mix (DSM) Wall	In Situ
Depressed Roadway: Covered Cut, Top down construction	Secant or Tangent Pile (Caisson) Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Soldier Pile and Lagging Wall	In Situ
Depressed Roadway: Covered Cut, Bottom up construction	Soldier Pile and Lagging Wall	In Situ
	Soil Nail Wall	In Situ
	Driven Sheet Pile Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Secant or Tangent Pile (Caisson) Wall	In Situ
	DSM Wall	In Situ

4.1 Gravity Wall Systems

Gravity wall systems are generally constructed in such a manner that the weight of the wall and entrained earth resists the lateral loads and consequent overturning forces from the ground behind the wall. The weight of the wall structure can be provided by:

- heavy stone masonry (little used for modern walls);
- structural concrete;
- soil resting on structural concrete members of the wall (cast-in-place concrete cantilever walls); and
- soil integrally mated with a reinforced wall facing material (mechanically stabilized earth).

These mechanisms may be used separately or combined in a variety of forms. Some of the general wall types are described in greater detail below. Gravity wall systems are generally backfilled with free-draining granular soils so as to control water and frost pressures. It should be noted, however, that obtaining granular backfill in the Windsor area is generally more costly than in other regions of Ontario.

4.1.1 Cast-in-Place Concrete Walls

Until the advent of pre-cast concrete wall systems, cast-in-place concrete walls were often the most common retaining wall constructed. In the simplest form, a large mass of formed concrete can be cast-in-place with the resistance to the lateral loads of the retained earth resisted simply by the dead-weight of the concrete, the friction at the wall base, and the resistance offered by the soil at the wall toe. However, large walls of this type are relatively uncommon in modern construction due to the disproportionate cost of the concrete in comparison to other retaining wall materials. Cast-in-place concrete cantilever walls are constructed for many projects for a number of reasons:

- once the structure is in place, the backfill behind the wall can sometimes be excavated without destabilizing the wall;
- design and construction methods are well established; and
- their shape can be made to fit complex grading and site conditions.

In general, concrete cantilever walls are constructed in the shape of an inverted ‘T’ or in the shape of an ‘L’, where soil is placed on top of the horizontal wall “footing”. Typically, cast-in-place concrete walls achieve their support of 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 structural itself is maintained by the structural capacity of the wall face and the footing connection.

Conventional concrete wall systems can be constructed within a temporary excavation support system or an open cut if space permits. For wall heights in excess of 6 to 8 metres, structural support can be achieved by internal or external counterforts (buttresses) as well as structural connections with the base slab or footing. Walls of this type generally require either an open or shored excavation with a base width approximately equal to about one third to one half of the final wall face height. Though conventional cast-in-place concrete walls are highly adaptable and common in their design and construction methods, their cost can exceed the cost of other available walls for similar project conditions, especially if shoring is required to support the existing earth.

4.1.2 Mechanically Stabilized Earth (MSE) Walls

The earth behind a wall can be stabilized and included in the mechanism for resisting the lateral loads of the native ground. Typically, mechanical stabilization of wall backfill is achieved by:

- placing and compacting a layer of earth backfill (typically 0.3 to 0.6 metres thick);
- laying steel straps, steel wire grids, or plastic grids (polypropylene, polyester, polyethylene) on the surface of the backfill layer as reinforcing elements;
- attaching a structural face to the reinforcing elements – typically the face consists of interlocking concrete blocks or panels;
- placing and compacting additional backfill on top of the reinforcing elements; and
- repeating the above sequence until a structural face is provided to the required height, with the mass of earth stabilized with internal "reinforcement" behind the face.

These walls achieve their stability by virtue of the friction and interlocking of the reinforcing strips or grids and the backfill. For the wall face to fail the connection to the strips/grid must break or the strips/grid must pull out of the backfill. Many systems are available for constructing such mechanically stabilized wall systems. These systems are often patented with respect to the method of earth reinforcement, attachment of the reinforcement to the structural face, and the structural face finish and interlocking mechanisms. The global stability of the overall reinforced mass is governed by its mass and geometry in the same manner as cast-in-place gravity walls.

Some "walls" can be constructed using the principles described above but instead utilize a geotextile fabric alone or in combination with a grid to replace the concrete facing. In such walls, the fabric and grid are wrapped over the front edge of each successive layer of backfill, producing a face that is constructed primarily of fabric. Where necessary, such walls can be sprayed with concrete or seeded for landscaping if the wall/stabilized earth face is sloped.

To construct a mechanically stabilized earth system, it is necessary to have an open excavation or to construct the wall as a "fill" wall, whereby the retaining wall and backfill are placed above existing grades. Many highway ramps and grade separations are constructed using such wall systems.

Mechanically stabilized earth walls offer the advantage that they are relatively inexpensive and rapid to construct and, depending on the wall facing units, can be more tolerant of differential settlements than cast-in-place concrete walls. In some cases, however, MTO has precluded the use of some types of mechanically stabilized earth walls in areas in which the underlying soil consists of soft silty clay and where measurable differential settlements are anticipated. This restriction on the use of MSE walls arises from a concern that differential settlements could cause breakage at the corners of facing blocks that would ultimately cause premature deterioration of the wall.

Mechanically stabilized earth walls typically require an open excavation that includes a level area from which to build the wall from that is approximately equal to ¾ of the wall face height back from the wall face. Creating such an open area requires extensive earthwork where such walls are used in support of earth "cuts." In general, for support of cuts, other wall systems are often more economical. "Walls" constructed of wrapped geosynthetic products are generally not suitable where aesthetics are important elements of the project and, depending on their design and construction, can be less durable than other feasible earth support systems. Since mechanically stabilized earth systems rely on both the backfill soil and reinforcement elements for support, excavation into the reinforced zone behind the wall must be restricted, providing a constraint on future infrastructure construction behind such walls.

4.1.3 "Crib" and "Bin" Walls

Crib walls derive their generic name by virtue of their construction method. Some crib-wall systems are composed of interlocking pre-cast concrete "stretchers" (similar in size and shape to railroad ties). The stretchers are used to build an interlocking "crib" in which earth backfill is placed. "Bin" walls are generally constructed of relatively thin-walled pre-cast concrete blocks, open at their top and bottom. As the "bins" are placed, their interior is filled with either compacted earth or clean crushed stone. The bins and cribs are either constructed from the bottom of an open excavation or from the ground surface for support of fills. A flat area approximately equal to about one half of the final wall height is generally required for construction of bin and crib walls. With these wall types a self-supporting structure is created.

Bin and crib walls offer the advantage that the area required at the base of an open excavation is less than that for mechanically stabilized systems, they are relatively rapid to construct, and can be reasonably tolerant to settlement or deformation, dependent upon the details of their construction. Bin and crib walls, however, are generally more expensive than mechanically stabilized walls.

4.1.4 Pre-Cast Cantilever or Counterfort Walls

A number of pre-cast versions of conventional cantilever or "counterfort" retaining walls are 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 that formwork is largely eliminated. However, these walls can be more costly than other wall systems that might be suitable for similar project conditions.

4.2 In Situ Wall Systems

In general, in situ walls includes a broad range of retaining systems characterized by constructing the face of the wall in-place, as opposed to creating a sloped excavation and building the wall from the bottom to the top. The primary advantage of in situ walls is that they generally do not require excavation behind the wall face. Lateral support is provided by either anchoring back beyond the retained ground during mass excavation in front of the wall, or by providing bracing from within the excavation. Such systems do not incorporate free draining granular backfill behind the wall facing and thus other measures must be taken to resist or control groundwater and frost pressures.

4.2.1 Soil-Nail Wall

Temporary and permanent retaining walls can be constructed using the soil nailing technique whereby the ground is supported by inserting reinforcing steel rods (“nails”) into the ground on a regularly spaced vertical grid, covering the excavation face with steel mesh 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. In essence, soil nailing creates the “reinforcing” of the mechanically stabilized earth systems without excavating the native ground behind the wall.

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 tie-back or conventional soil anchor construction. The design of a soil nail wall can be readily adapted to fit curved or shaped topographic forms. The equipment is generally portable, requires relatively little space and generates less noise and requires less manpower than other methods.

Soil nailing is most economical in ground that can stand unsupported for at least one day on a vertical or steep slope cut 1 to 1.8 metres 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. Also, groundwater must be well controlled such that seepage does not lead to excavation face instability during the initial construction.

Soil nail walls are constructed from within the area to be excavated and require between about 6 and 10 m of working space in front of the wall for equipment. The space required for working will depend on the equipment chosen, staging, and routes required for earth moving equipment.

4.2.2 Driven Sheet-Piles

For open cut excavations in loose or soft soils, excavation support and the permanent retaining structure can sometimes be provided by driven sheeting. Such walls can be designed as cantilever walls or with "dead-men" anchors (depending on loading and easements), permanent tie-backs, or internal bracing for lateral support. The choice of cantilever or permanent horizontal restraint will depend on the height, structural details of the wall, and space restrictions. Furthermore, such walls are more commonly utilized where surface appearance is of little importance such as for shipping dock bulkheads or along freight railway corridors. These systems are often considered flexible and permanent facings, if used, are generally designed to be relatively independent of the more flexible steel sheeting.

Driven sheet piles are readily available and effective for soft ground conditions which will be encountered in the project area. In addition to conventional sheet pile sections, some driven steel walls may consist of interlocking pipe and sheet piles. Such wall systems can typically provide greater bending stiffness than conventional sheeting alone. This method is not suitable for soils that contain substantial obstructions such as boulders, or that are very dense. Installation requires use of sheet pile impact hammers or other vibratory drivers.

Construction equipment for installing a driven sheet pile wall can generally operate within a window of about 7 to 10 m width with the wall at nearly any position within that window. Equipment for installation generally consists of mobile cranes suitable for lifting both the steel sheets and operating the vibratory hammer, compressors, and other equipment for delivery of sheets. In some cases, sheeting can be installed abutting property limits or other features that are vibration tolerant.

4.2.3 Secant or Tangent Pile (Caisson) Wall

Secant or tangent pile walls are constructed by drilling holes between 0.9 and 1.2 metres 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. Tangent pile walls are constructed by having the drilled holes immediately adjacent to one another and secant pile walls are formed by having each pile overlap the adjacent pile. Secant pile walls are preferable where groundwater or soft/loose soils must be controlled. Such walls can be constructed as either temporary or permanent walls. Permanent secant or tangent pile walls often have a permanent cast-in-place or precast concrete facing to fill any gaps between piles and provide a smooth or architecturally appropriate surface finish. These walls can be designed as cantilever walls (up to a site specific limiting height), with permanent tie-backs, or with internal bracing for lateral support. In some cases, where tie-backs or bracing are not feasible, piles as large as 2 metres in diameter can be constructed to allow high cantilever walls.

The main advantages of secant or tangent pile walls are increased construction alignment flexibility, increased wall stiffness compared to sheet piles, control of groundwater by pile interlock, and the ability to be used in difficult ground with 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), increased costs compared to sheet pile walls and that waterproofing may be difficult to achieve at joints. There is also the possibility of ground loss and seepage through any gaps which may be present and which can require remedial work during and after construction.

Construction equipment for installing a secant pile wall can generally operate within a window of about 7 to 10 m width with the wall at nearly any position within that window. Equipment for construction of secant or tangent pile walls generally consists of mobile drill rigs (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

4.2.4 Soil-Cement Mix Wall

Soil-cement mix, or deep soil mix (DSM) walls, can be used alone or in conjunction with traditional techniques. In order to reduce steel requirements for temporary shoring, DSM walls can be constructed as part of a soldier pile and tie-backs system. DSM systems can also provide increased stability in deep cuts in ground prone to deep-seated failures.

In general, soil-cement mix walls are constructed by using drilling equipment to produce a hole filled with soil cuttings, cement grout is then injected into the loosened ground, and the grout and soil are then mixed with the drilling equipment to produce a column of soil-cement slurry. The drilling, injection, and mixing can be accomplished using a variety of equipment configurations from single-flight augers modified with injection points and mixing blades, to overlapping continuous-flight augers. The type of equipment chosen for a particular project typically depends on cost and availability of proprietary systems developed and patented by various contractors. The columns of soil-cement mix walls can be reinforced by inserting steel H or W sections into the soil-cement column. As with the secant and tangent pile walls, a final facing is generally required for architectural purposes, drainage, and frost protection and, with soil-cement walls, for surface durability as well.

Soil-cement mix walls have particular application for soft soils as the procedure modifies the ground properties so that they are similar to a soft rock or low-strength concrete. This method is not suitable for soils containing more than 10 per cent peat and mixing of soft clay soils must be carefully controlled to avoid significant pockets of untreated soils.

Construction equipment for installing soil mix walls can generally operate within a window of about 7 to 10 m width with the wall at nearly any position within that window. Equipment for construction of these walls generally consists of mobile drill rigs (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

4.2.5 Soldier-Pile and Lagging

Soldier pile and lagging systems are commonly used and can be constructed in a variety of ground conditions. Soldier pile and lagging systems can be used in place of sheet piling where the soil is bouldery or quite dense. To avoid the noise and vibration usually associated with sheet pile installation, the piles are typically installed in pre-drilled holes. The wall is installed by boring a series of 0.5 to 1.0 metre diameter holes, spaced 2 to 3 metres apart, into which H piles (soldier beams) are installed and the annular space is filled with a relatively low strength sand-cement concrete mix. As the excavation proceeds, 50 to 100 millimetre thick boards are inserted behind the front flanges or placed against the piles and clipped to the front flange using fasteners. Concrete lagging, shotcrete or steel sheeting can be used in place of wood. For permanent installations pre-cast concrete lagging may also be used provided that alignment is closely controlled during installation of the piles in pre-drilled holes. Permanent soldier pile and lagging walls must also include provisions for frost protection and control of any groundwater seepage. The lagging is often installed in lifts of 1 to 1.5 metres, depending on the ground conditions.

Soldier pile and lagging can be installed at relatively low cost and the installation method can be adapted to poor ground conditions. However, horizontal restraints in the form or wales and struts, rakers or tiebacks are required. Tiebacks are the best choice for minimizing obstructions in the excavation. However, the use of tiebacks for deep excavations is contingent upon obtaining subsurface easements, the presence of underground utilities and suitable soils or rock in which to install anchors. Excavations will have to be carefully monitored for subsidence and lateral movement particularly when structures are nearby. Since ground loss is more common with this system than sheet piles and some other systems, construction of soldier pile and lagging retaining systems must be carefully controlled, especially in built-up areas.

Construction equipment for installing a soldier-pile and lagging wall can generally operate within a window of about 7 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 (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

4.2.6 Cast-in-Place Concrete Diaphragm Wall

Commonly called “slurry walls”, cast-in-place concrete diaphragm walls are constructed by excavating a deep, narrow trench, filling the trench with a viscous slurry (of clay and water or polymers and water) to keep the trench from collapsing, placing reinforcing steel within the trench, and then placing the final concrete from the bottom up, displacing the slurry. Typical trench widths are in the order of 0.6 to 1 metre. These walls offer the advantage that they can serve as both temporary excavation support and the permanent wall depending on the design details. Concrete diaphragm walls that are constructed as the permanent structural wall often are provided with a cast-in-place or precast facing to improve the architectural finish and address frost protection or and/or drainage issues. Such walls can be designed as cantilever walls, with permanent tie-backs, or with internal bracing for lateral support. The choice of cantilever or permanent horizontal restraint will depend on the height, structural details of the wall, and space restrictions. If necessary, diaphragm walls can be constructed in “T” shaped sections to permit high cantilever walls without additional internal bracing or permanent tie-backs; however, construction of this type is relatively rare within North America. In addition, if carried to a suitable bearing layer or if the wall is of sufficient penetration depth within a relatively competent ground layer, concrete diaphragm walls can serve as foundation or vertical load bearing elements for overlying or attached.

Slurry walls are suitable for construction of walls in caving and cohesive soils. They may be necessary for locations where sheetpiling or soldier piles and lagging are not applicable or where greater control over ground deformations or groundwater infiltration is required. However, slurry walls can be approximately twice as expensive as these systems. Much of the high cost is attributable to requirements for specialized equipment and more stringent field control.

Construction equipment for installing a concrete diaphragm wall can generally operate within a window of about 10 m width with the wall at nearly any position within that window. Equipment for construction of these walls generally consists of mobile cranes suitable for lifting the trench excavating equipment (often a clam-shell bucket), steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

4.3 Horizontal Restraint Systems

4.3.1 Internal Struts/Braces

For temporary excavation support, the walls of the excavation can be propped using steel beams or pipe sections placed between the walls as the excavation proceeds. These struts are often removed during construction of the permanent structure if the structure and backfill over the structure will be sufficiently strong to resist the permanent earth loads. Some permanently retained walls for grade separation projects have used permanent internal struts placed near the top of the trench. Permanent struts can be constructed of steel, cast-in-place concrete, or pre-cast concrete depending on the required structural dimensions, design-life performance goals, cost, and construction considerations. Struts present a disadvantage during construction since they obstruct the working space within the excavation. Permanent struts also obstruct the space within the excavated area and are subject to weathering and thermal stresses and require long-term maintenance. However, struts offer the advantage that once in place, the excavation and all the wall systems are contained within the limits of the support walls.

In general, strut spans are limited to about 20 m (when using pipe struts) unless vertical support is provided to inhibit bending or buckling due to the combination of axial and self-weight loads. Larger spans are possible, but installation of supporting piles and multiple strut-to-pile connections can contribute to the complexity of the supports, congestion of the working space, and displacements of the wall and surrounding ground. The horizontal and vertical spacing of the struts will largely depend on the stiffness of the vertical wall elements, the loads that are distributed to the struts, and tolerable displacements of the ground and facilities around the excavation. Typically, the spacing of struts (both vertically and horizontally) is limited to about 5 m, though larger spans can be achieved. In some cases, vertical spans between struts on the order of 8 to 10 m can be achieved, though the required bending moment capacity of the vertical wall elements must be substantially greater than typical excavation support installations. It may be also necessary to install wales – long structural sections that support the wall horizontally between supports. Wales can consist of steel sections or, in the case of permanent installations, cast-in-place concrete.

4.3.2 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. Once a hole is drilled, steel rods or high-strength steel strands are inserted into the hole. 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). Once the grout is cured, the anchor is prestressed to its design load and structurally connected to the wall. After stressing, the remaining annular space between the anchor zone and the wall face, called the “free” length,” is backfilled. Tie-backs offer an unrestricted excavation or permanent underground space once they are in place. Tie-backs, however, typically cost more than internal bracing for long and narrow temporary excavations. In addition, subsurface easements are typically required from neighbouring properties if the tie-backs extend beyond existing rights-of-way or property boundaries. Permanent tie-backs can limit future subsurface uses for neighbouring areas 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 and 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, though larger spans can be achieved. In some cases, vertical spans between tie-backs on the order of 5 to 8 m can be achieved, though 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 wales – long structural sections that support the wall horizontally between supports. Wales can consist of steel sections or, in the case of permanent installations, cast-in-place concrete.

For this project, the use of tie-backs may largely be limited to temporary installations in the upper firm to stiff clay crust. The deeper soft silty clay is unlikely to be capable of providing adequate resistance for anchoring tie-backs. Should excavations penetrate relatively deep into the silty clay, it may be necessary to extend any ground anchors to bedrock to provide adequate resistance capacity. 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 settlement of the wall can lead to loss of tension in the tie-backs and poor performance of the entire excavation support system and it may be necessary to extend the vertical wall elements to bedrock.

4.4 Earth and Groundwater Pressure for Preliminary Design of Walls

Earth pressures for the design of gravity walls will likely be governed by the composition of the wall backfill materials. In general, it is recommended that all gravity walls be backfilled with granular soils such that design earth pressures are of typical magnitudes, where the active earth pressure coefficient may range between about 0.25 and 0.30, and groundwater can be drained from the backfill. Compaction pressures, however, may dominate design conditions near the top of any gravity or cantilever walls. Earth pressures for the design of in situ walls will be governed by the existing soil and groundwater conditions, and the relatively low strength of the in situ silty clay soils, where typical active earth pressure coefficients may range between about 0.33 and 0.40. The relatively high groundwater levels will also affect design of in situ walls since the native soils will not drain sufficiently to avoid groundwater pressures on the back of the wall.

4.5 Frost Protection

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 because all native soils behind the walls are considered to be frost susceptible. It should be noted that there are a number of documented cases of in situ wall distress due to pressures induced by frozen ground (e.g., Broms and Stille 1976, Eigenbroad and Burak 1992). 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 grade separation projects using permanent in situ walls (concrete diaphragm and drilled pile walls) in Ontario that have been fitted with an insulation layer to prevent such pressures.

5.0 APPLICABILITY OF RETAINING WALLS TO DRIC PROJECT

For the purposes of evaluating feasibility, it has been assumed that the average depth of excavation required along the approximately 9 km long corridor is in the range of 8 to 12 metres for a cut and cover tunnel. Deeper cuts may be required to cross under the significant municipal drains such as the Grand Marais (Turkey Creek) Drain. Construction of the retaining wall systems for a bored or mined tunnel are addressed in a subsequent section of this report.

For the purpose of a preliminary comparison of retaining systems, it has also been assumed that generally the temporary cut slopes will be stable at slopes of between 1 and 2 horizontal to 1 vertical. For planning purposes, it may be assumed that temporary cut slopes in the area near the terminus of Highway 401 could be about 1:1 (horizontal:vertical), whereas slopes closer to 2:1 may be necessary near the intersection of E.C. Row Expressway. Temporary construction slopes between these locations may be interpolated between these values based on the distance between the end locations. Permanent slopes will need to be flatter as discussed in a subsequent section of this report. As noted previously, the selection of a suitable retaining system is based on several factors including cost of installation, compatibility with soil conditions, workspace requirements (surface or subsurface easements) and limitation of movements.

5.1 Factors of Safety for Excavation Stability

Three principal conditions affect the stability of deep excavations in silty clay and clayey silt soils and these are:

1. the strength of the soil relative to the depth of the cut, sometimes called “global stability,” or stability against “base heave” (e.g. Peck 1969, NAVFACS 1986, Clough and O’Rourke 1990, CFEM 2006);
2. upward seepage of groundwater carrying fine granular soils leading to loss of ground, or “piping stability” (e.g. NAVFACS 1986, CFEM 2006); and
3. upward groundwater pressures on cohesive soil layers sufficient to overcome the weight and strength of the overlying soil and uplift the bottom of the excavation (e.g. Milligan and Lo 1970, CFEM 2006, Shirlaw 2006).

The factor of safety for excavation global stability is generally defined by the depth of the excavation and the undrained shear strength of the soil near the base of the excavation. The excavation plan dimensions (length and width) can also influence the stability factor of safety, however, for the purposes of this feasibility assessment, the excavations have been considered to beat least twice as long as they are wide. Figure 2 illustrates the base stability factor for global stability for several selected locations along the proposed approach route and excavation depths. In general, for temporary excavations, it is recommended that the factor of safety for global stability (sometimes called base stability) be at least 1.25. For permanent cuts, it is recommended that this factor of safety, based on undrained shear strength considerations be about 2, so as to limit the potential for undesirable long-term creep displacements.

Table 5.1

Maximum Temporary Excavation Depths Considering Global Excavation Stability

Location	Approximate Depth of Excavation, Factor of Safety for "Global Stability" ≅ 1.25 to 1.5
Highway 401 & Highway 3	15 to 16 m
Highway 3 & Cousineau Road	12 to 13 m
Huron Church Road & Highway 3	10 to 12 m
Huron Church Road & E.C. Row Expressway	10 to 12 m
E.C. Row Expressway & Ojibway Parkway	6 to 7 m

Stabilizing excavations that extend below these depths may be accomplished using construction techniques such as:

- extending the penetration of retaining system walls to well below the base of the excavation;
- installation of below-grade struts in slurry-filled trenches;
- construction of relatively thick concrete base slabs under slurry or water; or
- improving the ground at the excavation base using techniques such as jet grouting or deep soil mixing.

It should be noted that the factors of safety and resulting limitations to the depths of excavation calculated as part of this phase of work are based on the data available at the time this report was prepared. In general, the testing consisted of a limited number of unconfined compression tests on recovered thin-wall tube samples or field tests with rather crude hand-held devices for projects completed more than 30 years ago. The limiting depths provided in Table 5.1 above are sensitive to soil strength values. Future investigation must be undertaken to obtain information necessary to confirm these calculations as described in Section 10 of this report.

Since it is anticipated that the ground conditions will primarily consist of silty clay from near the ground surface down to near the bedrock surface, base stability of the excavation may not be influenced by piping failure mechanisms, except should uplift failure occur and free flow of groundwater into the excavation subsequently occur. It is considered that the groundwater pressures within the bedrock and granular soils that immediately overly the bedrock may be consistent with a hydrostatic pressure equivalent to a groundwater level of about 3 m below the ground surface. In this case, the maximum depth of excavation that can be achieved will be governed by the depth of soil remaining between the bottom of the excavation and the top of the granular soils or bedrock. The maximum depth of excavation to maintain a factor of safety against uplift failure of about 1.2 is shown in the table below for different locations.

Table 5.2

Maximum Excavation Depths Considering Base Uplift Stability

Location	Approximate Depth of Excavation to Maintain Factor of Safety Against Uplift $\cong$ 1.2
Highway 401 & Highway 3	17 m
Highway 3 & Cousineau Road	15 m
Huron Church Road & Highway 3	13 m
Huron Church Road & E.C. Row Expressway	13 m
E.C. Row Expressway & Ojibway Parkway	11 m

Excavation below the depths listed above will require that either groundwater pressures be lowered temporarily (e.g. Conlon et al. 1971), or that alternative construction techniques be employed in order to build a base slab that can resist the upward hydraulic pressures. In addition, it is anticipated that the permanent structures will require a structural base slab and that this slab will have to be “tied-down” to resist hydrostatic uplift pressures. Resistance to uplift pressures can be accomplished using such measures as:

- installation of tie-down anchors into competent soil or bedrock; or
- installation of piles to resist uplift loads.

In some cases, uplift pressures can be reduced by the use of permanent pressure relief wells. Temporary and/or permanent groundwater lowering (pressure relief) may not be practicable for this project because of the relatively large groundwater flows that might be required for dewatering, the presence of hydrogen sulphide gas within the groundwater, the effects of groundwater lowering on the soft compressible soils, and the potential inability to effectively cut-off groundwater flows (see Section 8 of this report). Since temporary lowering of groundwater pressures is likely not feasible, it would be necessary to construct much of the excavation base slab under slurry or water, where the slab is to be below the depths indicated above. In this case, it will also be necessary to anchor the base slab prior to draining of the excavation. Tension anchors installed for this purpose and under the anticipated site conditions would be required to be drilled into the bedrock and could consist of either conventional strand or tendon anchors, or the use of small diameter piles drilled into the bedrock. Drilling of the anchors within a flooded excavation and methods to secure the anchors to the base slab are critical technical challenges that would affect the cost and schedule of this alternative. Further, the subaqueous construction methods described above are relatively uncommon in North America and may be impractical for this project, depending on the depth of cut and the extent to which such measures may be necessary. Alternatively, it may be feasible to use jet grouting to form temporary base slabs prior to soil excavation, depending on the excavation dimensions and depths and the pressures to be resisted (e.g. Shirlaw 2006). Extensive use of jet grouting to form base slabs may also become impractical depending on the length of cut and cover sections that might require such ground improvement. The need for stability improvement measures will be sensitive to the local ground strength dimensions of the excavation and, in particular, to the depth of the excavation and relative groundwater level.

5.2 Gravity Walls

Gravity walls for support of roadway cuts are most economical for shallow excavations or for wall heights up to 6 metres. Gravity walls for roadway cuts generally require a working space behind the face of the wall in the range of 2 to 3 times the wall height to account for the base width and back slope of the cut. Use of MSE, cast-in-place or pre-cast concrete cantilever walls supported on shallow foundations located in the very stiff to hard clayey till crust is technically feasible for wall heights as illustrated in the table below.

Table 5.3

Maximum Gravity Wall Heights for Walls Supported on Shallow Foundations

Location	Limiting Wall Heights
Highway 401 & Highway 3	7 to 8 m
Highway 3 & Cousineau Road	7 to 8 m
Huron Church Road & Highway 3	7 to 8 m
Huron Church Road & E.C. Row Expressway	5 to 7 m
E.C. Row Expressway & Ojibway Parkway	4 to 6 m

Although the undrained shear strength of the silty clay soils indicates that a suitable factor of safety may be achieved for deeper cuts, the feasibility of using gravity walls without resort to pile foundation support will be sensitive to local soil strength, bearing pressure, and settlement or long-term creep displacement considerations.

For depressed roadway sections, where retaining walls will be built against cut slopes, cast-in-place concrete cantilever walls and MSE walls are the most economical types of retaining systems with crib and bin walls being the next most expensive. For cantilever or counterfort walls, pre-cast sections can be used to speed up construction at increased costs.

Ground conditions are such that deep foundations may be required for gravity walls constructed for grade cuts greater than the values provided in Table 5.1 above due to the low available bearing pressures and the need to limit settlements.

As the depth of excavation extends beyond these depths and into the softer cohesive deposits, flatter temporary side slopes or use of temporary shoring such as soldier piles and lagging, soil nail walls, or sheet piling may be required in order to maintain the stability of the excavation sidewalls and to restrict movement of the surrounding soils. Deformations are expected to become significant for cuts deeper than the values listed in Table 5.1. Displacement of the surrounding ground must be examined in detail since maintaining displacements of adjacent buildings or utilities within acceptable limits may require underpinning or alternate excavation support systems (also see Section 5.4 of this report). Based on the available information, groundwater encountered in the shallow surficial granular deposits should be able to be controlled using sump pits and pumps, though groundwater control will need to be examined in more detail near the existing drains and watercourses. The need for temporary shoring and deep foundations increases construction complexity and costs for gravity walls constructed in excavations deeper than those provided in Table 5.3. For these reasons, in situ retaining systems are preferred for construction of depressed roadway sections in cuts of greater depth.

Where the wall is to be built up from the existing grade and is to be used to retain noise berm fill or embankments, the wall heights may be limited to about 5 to 10 m in the eastern end of the area of continued analysis (i.e. near the existing terminus of Highway 401). The height of such walls may be limited to between 4 and 5 m where soils are of lower strength, particularly along the E.C. Row Expressway area between Huron Church Road and Ojibway Parkway. In this area, the embankments for the E.C. Row Expressway overpasses exhibit settlements that may not be tolerable for some types of gravity walls. Use of such walls for retaining embankments and fills will depend on further analysis considering overall stability, total settlement, and differential settlement.

5.3 In Situ Walls

An advantage of in situ walls is that the road section can be constructed using a vertical cut, resulting in a reduction of the required working space compared to gravity walls built against sloped excavation walls.

Temporary or permanent soil nail walls may be feasible for construction of depressed roadways in open excavations or covered cuts, depending on the local soil strength and depth of cut. Soil nail walls are best used in stiff to hard cohesive deposits. Due to the potential for basal instability caused by the softer underlying deposits, construction of permanent soil nail walls greater than about 6 metres in height is not recommended, as suitable anchoring of the nails may be more sensitive to local soil strength conditions. In general, it is recommended that permanent soil nail walls be excluded from consideration for cuts west of and north of Huron Church Line pending further investigation. A subsurface easement extending a minimum of 0.6 to 1 times the height of the wall is required for installation of the soil nails because of the anticipated soft to firm clays. Further investigation may delineate areas of stiffer clays were such walls may be feasible. Where the excavation includes granular soils near the surface, special construction provisions may be required to maintain a stable cut face during installation of the nails and facing materials. In some cases, vertical facing elements or ground reinforcement can be installed in situ in the granular soils prior to construction of the primary soil nail wall system.

Driven sheet piles should be suitable for temporary support of excavations where depressed roadway sections are to be constructed. Sheeting of the "Z" shape or interlocking pipe and sheet sections may be best for resisting large bending moments associated with deeper excavations and larger spans between supports. A subsurface easement of approximately about 1.5 to 2 times the height of the excavation will be required if dead-men or tie-backs are used. Internal bracing can be used instead of anchors if subsurface easements cannot be obtained. Driven piles are generally not suitable for construction adjacent to structures which are within one excavation depth of the work area due to vibration induced settlement and damage. In some cases, interlocking pipe and sheet piles may be used for construction of permanent retaining walls. The interlocking pipe piles are advantageous in some instances because the steel is located on the outside of the bending structure, providing relatively high bending moment capacity. Surface finish and frost protection issues must be considered for these walls to form permanent structures for this project. In addition, permanent cantilever walls should be limited in height to about 7 to 8 m, depending on displacement considerations. Where cantilever retaining structures are greater than about 6 m in height, permanent buried struts may be required to limit long-term creep displacements in areas where the factor of global stability is less than about 2. The need for permanent buried base struts should be examined once additional soil data is available.

Soldier pile and timber lagging shoring systems are commonly used in Southern Ontario for temporary support of excavations. If the soldier piles are founded on bedrock, it may be possible for this system to serve as a support of excavation wall for top-down or cut and cover tunnel construction. Soldier pile and lagging is most economical for excavations which extend below the crust to depths of 5 to 7 metres where the risk of damage to settlement prone structures or utilities is low. For deeper excavations in soft ground, concrete lagging or shotcrete can be used between the soldier piles to increase the wall stiffness. Since only the soldier piles will be embedded beneath the base of the excavation, use of soldier pile and lagging is not recommended for excavations deeper than about 8 metres because of the potential for basal instability. Soldier piles and lagging are not considered to be suitable for permanent grade separation for this project.

Secant or tangent (caisson) walls are suitable for deeper excavations and can be used in both open and covered cuts. Carefully constructed caisson walls can form an adequate barrier to inflow of groundwater, though some localised seepage through the wall should also be expected. Since they can be readily extended beneath the base of the excavation and are relatively stiff in bending, they can be used to reduce the deformations which can occur in deep excavations that penetrate softer cohesive deposits. These walls are, however, more expensive to construct than sheet pile or soldier pile and lagging walls. It is anticipated that for this project, secant pile walls may be considered for the following purposes:

- To maintain a relatively dry excavation in the areas of the Grand Marais and Lennox Drains or for roadway sections constructed parallel to and a municipal drain as needed;
- Areas very close (within one depth of excavation) to existing settlement-sensitive structures or utilities;
- To maintain stability in soft ground which can be expected adjacent to and beneath the municipal drains and towards the northwest end of the alignment;
- To support decking so that traffic can be carried above the excavation, with the decking and traffic loads carried to bedrock (if needed);
- In top-down construction to provide permanent structural walls between which a roof slab would be constructed with backfill, pavement, and traffic loads transferred to the walls and down to bedrock, and;
- To construct relatively high permanent cantilever walls that will have no horizontal supports such as rakers, tie-backs or struts.

Surface finish and frost protection issues must be considered for these walls to form permanent structures for this project. In addition, permanent cantilever walls should be limited in height to about 7 to 8 m. Where cantilever retaining structures are greater than about 6 m in height, permanent buried struts may be required to limit long-term creep displacements in areas where the factor of global stability is less than about 2. The need for permanent buried base struts should be examined once additional soil data is available.

Deep soil mix (DSM) walls can be used for support of open excavations and covered cut sections built using bottom up construction and are subject to similar global and uplift factors of safety concerns as with all walls. DSM walls may be comparatively expensive to construct and, depending on the anticipated soil conditions, internal bracing may still be required. Further, the native silty clay soils may result in difficulty with achieving the desired strength and homogeneity of the soil-cement mix. Surface finish and frost protection issues must be considered for these walls to form permanent structures for this project. Where cantilever retaining structures are greater than about 6 m in height, permanent buried struts may be required to limit long-term creep displacements in areas where the factor of global stability is less than about 2. The need for permanent buried base struts should be examined once additional soil data is available.

Slurry or diaphragm walls should be suitable for construction of support of excavation walls in both open and cut and cover excavations. If slurry walls are to be incorporated in top-down tunnel construction where the excavation support wall is to support roof loads, the structural diaphragm walls must extend to bedrock. Specialty equipment may be required for construction of diaphragm walls greater than 30 metres in depth. There must be sufficient workroom for both the equipment and storage of the both the slurry and spoil material. As with secant pile walls, slurry walls would be best used in areas of soft ground where the depth of excavation is greater than 8 metres, a continuous groundwater cut-off is required, or settlement sensitive structures are relatively close with a distance less than the excavation depth. These walls may also be used for permanent cantilever walls for support of open cuts. Surface finish and frost protection issues must be considered for these walls if they are to form permanent structures. As with secant pile walls, permanent cantilever walls should be limited in height to about 7 to 8 m unless special construction is applied in which the structural shape is modified (“T” shape) such that a buttress or counterfort is constructed integral with the wall. Where cantilever retaining structures are greater than about 6 m in height, permanent buried struts may be required to limit long-term creep displacements in areas where the factor of global stability is less than about 2. The need for permanent buried base struts should be examined once additional soil data is available.

5.4 Displacements Associated With Deep Excavations

Construction of excavations, with the sides either sloped or supported by vertical retaining structures, will cause displacement of the ground to differing degrees. When sloped excavations are made as either permanent cut slopes or for construction of backfilled gravity walls, induced displacements will generally be minimal in magnitude and limited to affecting the ground within a distance back from the top of the cut (slope crest) equal to the depth of the slope cut, provided that the factor of safety for slope stability is satisfactory.

Construction of excavations supported by vertical in situ walls can induces greater localized displacements of the adjacent ground (e.g. 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:

- 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

For cuts typically ranging between 10 and 12 m deep, with 2 levels of strut supports below deck beams (if any), without support pre-stressing, and a depth of penetration on the order of 50% to 80% of the cut depth, preliminary calculations suggest that maximum horizontal and vertical displacements of the ground adjacent to the wall could be characterised as:

- on the order of 0.5% to 1% of the cut depth for wall systems such as soldier-piles and lagging or sheet piles; and
- on the order of 0.1% to 0.5% of the cut depth for wall systems such as contiguous drilled pile or concrete diaphragm walls.

Cuts of lesser depth may exhibit smaller proportional displacements than those suggested above since the soils closer to the ground surface are typically of greater strength than those at depth. Cuts greater than about 12 m in depth, particularly closer to the intersection of Huron Church Road and E.C. Row Expressway may experience greater proportional displacements than those suggested above.

A number of measures are available to limit the displacements of such retaining structures including pre-stressing of horizontal supports, stiffening of the vertical wall systems, extending the depth of wall penetration, providing buried struts prior to excavation, or improving the ground at the base of the cut using a variety of grouting or soil mixing techniques (e.g. Shirlaw 2006). Displacements and their effects on nearby facilities should be evaluated in greater detail during further stages of analysis and design (e.g. Boone et al 1998, Boone 2001, ITIG 2006). The preliminary evaluation above is intended to facilitate refinement of conceptual alternatives and should be updated as additional project and subsurface information is developed.

5.5 Other Geotechnical Issues for Depressed Roadways

Construction of depressed roadways, whether in a cut supported by retaining structures that remains open, or within a cut and cover tunnel structure, will require the excavation and removal of relatively large volumes of soil. A soil management plan should be developed to address disposal locations, testing, and transportation routes for excess soils. If the depressed roadway option is considered further through design, consideration will also need to be given to appropriate field instrumentation and monitoring during construction to assure that the effects of such work on nearby facilities is maintained within achievable and specified limits.

6.0 CUT SLOPES

The stability of cut slopes is primarily dependent upon the local soil type, shear strength, static groundwater level, cut depth, slope angle and the length of time the excavation remains open. A review of slopes cut into similar soils in Welland and Sarnia, Ontario, and Port Huron, Michigan, suggest that excavations with depths of between 15 and 18 m with side slopes of between 1.5:1 (horizontal:vertical) to 2.5 to 1 have failed repeatedly (e.g. Conlon et al. 1971, Lo 1971, Dittrich et al. 1997). Final cut slopes in Welland for the canal underpass tunnel cuts, where the depth of cut were on the order of 24 m, required permanent groundwater lowering and side slopes ranging between about 3:1 near the ends of the approach roadways where the cuts were the most shallow, to about 8:1 at the deepest parts of the cut. Stable slopes were achieved in Sarnia with overall slopes of about 3.5:1, though these included 3:1 slopes of limited height with intermediate benches. It is further understood that in Detroit, where the soils may be of somewhat greater strength, cut slopes along the highways are initially cut at 2:1 but continued maintenance is required and some flattening of slopes or buttressing of the slope toes has occurred such that finished surface slopes closer to 2.5:1 are achieved.

Based on the available subsurface data it is considered that temporary slopes in the very stiff to hard cohesive soils should remain stable during short-term construction at a maximum slope of 1 horizontal to 1 vertical to a maximum depth of about 5 to 7 m. Temporary cut slopes in firm to stiff cohesive soils are expected to be stable at a maximum inclination of 1.5 horizontal to 1 vertical. In both these soil types, it is anticipated that shallow cuts on the order of 1.5 m or less (e.g. for soil nail walls) should maintain a vertical or near vertical face for a sufficient length of time (but less than a shift) to permit soil nail wall construction. Excavations in very soft to soft cohesive deposits and very loose to loose granular soils may be stable in the short term at maximum slopes of 2 horizontal to 1 vertical or flatter, provided that groundwater in granular soils is adequately controlled. It is anticipated that shoring will be required for excavations greater than the depths identified in Table 5.3 since the previous boreholes indicate that the shear strength of the cohesive deposits generally decrease with depth. It is generally recommended that, for preliminary planning purposes, temporary slopes no steeper than 1:1, 1.5:1, and 2:1 (horizontal:vertical) be planned for the east, central, and west sections of the project, respectively.

A preliminary slope stability analysis was conducted to ascertain the long-term stability of cut slopes along the corridor. Assuming average cut slope heights equal to or less than those listed in Table 5.4, below, a bulk unit weight of 19 to 20 kN/m3, and an effective angle of internal friction of 30^{o}, the results of the analysis indicates that permanent cut slopes should be stable at an inclination of 2.5 horizontal to 1 vertical. Maintaining an adequate long-term factor of safety of about 1.3 or greater, however, is predicated on adequate slope drainage where the phreatic water surface level is maintained at least 1 m below the ground surface at all locations. Achieving this condition may require that the slope includes a flat “bench” at the approximate mid-height of the slope with a subsurface drain placed along the up-hill edge of the bench. These recommended values for cut side slopes also consider that the area being cut undergoes no significant slope displacement during construction of the cut nor has experienced slope instability in the past.

Table 5.4

Maximum Heights for Permanent Cut Slopes

Location	Limiting Cut Slope Heights
Highway 401 & Highway 3	7 to 8 m
Highway 3 & Cousineau Road	7 to 8 m
Huron Church Road & Highway 3	7 to 8 m
Huron Church Road & E.C. Row Expressway	5 to 7 m
E.C. Row Expressway & Ojibway Parkway	4 to 6 m

7.0 SOFT GROUND TUNNELLING

7.1 Overview of Tunnelling Conditions

Tunnels have been mined or bored in the soil deposits of the Windsor area. In the upper portions of the extensive silty clay deposit, relatively small diameter tunnels, on the order of 2 to 4 m in diameter, have been mined using open-face shields. Within the deeper and softer parts of the silty clay, when tunnelling using open-face shields, clay squeezed into the face on a number of projects. For this reason, modern tunnels at depth or of large diameter within soft to firm silty clay are most often constructed using closed-face tunnel boring machines. Older tunnels utilized compressed air chambers at the face to assist in maintaining face stability and groundwater control. Tunnels within the bedrock have also encountered difficulties due to groundwater and hydrogen sulphide gas conditions. Some of the tunnels constructed near the Detroit River, or through similar ground between Sarnia (Ontario) and Port Huron (Michigan), are listed below as a historical perspective of tunnelling in the area and the issues that such tunnelling encountered.

- A soft ground tunnel was to be constructed for a railroad crossing in the late 1800s to connect the downtown areas of Detroit and Windsor. After a number of attempts, the tunnel effort was abandoned due to ground water inflows and hydrogen sulphide gas. It is understood that two workers were killed by exposure to toxic hydrogen sulphide gas.

- In 1890, a rail tunnel was constructed between Sarnia, Ontario, and Port Huron, Michigan. Initial construction attempts involved the installation of a lower drainage tunnel. This tunnel was abandoned during construction due to flowing sand and water within the face. Soft clay squeezing into another 2 m diameter test tunnel (at about 27 m below ground surface) was so fast that the test tunnel was abandoned. Methane gas, squeezing clay, and groundwater control problems also resulted in abandonment of a similar test tunnel on the Canadian side of the works. Subsequent work on large diameter shafts for another attempt at a full size tunnel was abandoned due to squeezing clay between 20 m and 30 m depths. The use of shafts was abandoned as a tunnel construction technique and the approaches were moved further from the shores. The approaches to the tunnel were constructed in open-cut to depths of between 10 m and 15 m. During construction the open-cut side slopes failed on two occasions. The tunnel was finally constructed through the soft clay using an open face shield and compressed air for face support. Gilbert (1991) and Busbridge et al. (1993) provide additional details on this case history.

- In 1910, the Detroit River Rail Tunnel was completed beneath the Detroit River. The river crossing was completed as an immersed tube tunnel (where the bottom of the river was dredged to allow burial of the tunnel sections). The approaches to the tunnel were constructed through soft ground using an open face tunnel shield and compressed air for face support with the shallower sections constructed using cut and cover methods. It is understood that the top of this tunnel is about 1 to 2 m below the bottom of the river bed.

- Between about 1928 and 1930, the Detroit River Car Tunnel was constructed similarly to the Detroit River Rail Tunnel, using immersed tube, shield and compressed air, and cut and cover methods. In Windsor, the tunnel started at about Park Street between Ouellette Avenue and Goyeau Street and is located on a northern alignment to and beneath the Detroit River. From the tunnel portal on south side of Park Street, to north of University Avenue, a distance of about 190 metres, the tunnel was constructed using open cut techniques to depths of about 7 to 14 m below the ground surface. This section was located generally within the right-of-way of Freedom Way and is referred to as the approach tunnel. North of the approach, the tunnel was advanced using conventional shield tunnelling techniques using compressed air to support the tunnel face. The shield driven tunnel portion was about 380 m long, and extended beneath the Detroit River where it connected to the sunken tube portion of the structure. The shield driven portion of the tunnel is about 9.8 m in diameter, and the top of the tunnel is located at depth of about 14 m below the ground surface at Riverside Drive. On the Riverfront lands adjacent to the Detroit River the top of the tunnel is located about 10 m below the ground surface. The immersed tube section of the tunnel is covered by about 2 to 3 m of backfill up to the river bed elevations.

- The Belle Isle River Intake Tunnel connects the northern end of Belle Isle with the Michigan mainland for drinking water supplies. This tunnel was constructed in the 1930s within the bedrock. Hydrogen sulphide gas, liquid petroleum, and artesian water inflows all complicated and slowed construction of this tunnel.

- The Detroit River Outfall Tunnel No. 1 (DRO-1) was constructed in 1936 from near Jefferson Avenue and the Rouge River through soft clay within an open shield using compressed air at a depth of about 20 m.

- The Southwest Intake Tunnel (Land Section) was constructed in the 1950s from near the intersection of Goddard and interstate highway I-75 through soft silty clay with sand layers with an open shield about 12 m below the ground surface. It is understood that two workers were also killed during construction of this project by exposure to toxic hydrogen sulphide gas.

These older tunnel case histories provide useful information related to soil strength and subsurface conditions. However, modern tunnelling techniques may permit more rapid tunnel construction in conditions that would have not been possible to tunnel through using the older technologies. The most relevant tunnel constructed in conditions similar to those in Windsor using modern tunnelling technologies is the St. Clair River Tunnel constructed for CN Rail between Sarnia, Ontario, and Port Huron, United States of America (Golder Associates Ltd. 1992, Charalambu et al. 1993, Kramer et al. 1993, Harrison et al. 1994, Kramer et al, 1994).

The St. Clair River rail tunnel, constructed in the early 1990s, was built using an approximately 9.5 m diameter earth-pressure-balance (EPB) tunnel boring machine (TBM). At the time, this was the largest soft ground TBM used in North America. At the lowest point of the crossing, the tunnel passes within 5 m of the river bed and about 1 to 2 m of the bedrock surface. Apart from where the tunnels passed beneath the portal cut slopes, the top of the tunnel was approximately 10 to 16 m below the ground surface. The approaches to this tunnel were also constructed in open cut and concrete diaphragm walls with underground struts were used to provide additional stability to the base of the excavation and side slopes.

Because of anticipated ground surface settlements arising from the tunnelling, a number of settlement protection measures were implemented during construction. An extensive compensation grouting program was undertaken to maintain settlements to tolerable levels where the tunnel passed approximately 10 m beneath the foundation level of the Imperial Oil research building. This program included several 4.5 m diameter shafts on either side of the building, installation of an array of grout pipes beneath the building, and extensive monitoring throughout grouting and tunnel construction. In addition to the research building, facilities considered to be at risk from tunnelling included: oil refinery pipe bridges, oil storage tanks, other buried services (sewers, water lines, electrical ducts, abandoned product lines), electrical substation and pump house, marine dock structures, warehouse building and, hard landscape features. A number of settlement protection works were carried out from the surface to limit damage to these facilities including: removal of the facility (or portions thereof), temporary diversion with closure of the main facility and monitoring, abandonment, installation of secondary support systems, and close monitoring with contingency plans to manage unanticipated events.

During construction a shaft was also built to access the TBM for unexpected repairs. Squeezing clay near the bases of the drilled shafts, used for support of the walls, and near the base of the excavation caused considerable difficulty during shaft construction. After TBM repair, the tunnel was constructed successfully.

Settlements during tunnelling generally ranged between about 10 mm and a maximum of about 425 mm, though one area near the TBM repar shaft settled over 1 m due to the combined effects of shaft construction and tunnelling performance. Average settlements were on the order of about 70 mm, within the anticipated 40 to 90 mm range that was assessed prior to construction based on the ground conditions, expected tunnel performance, and tunnel size and depth. Heave of the ground surface of up to 30 mm was also experienced in some areas.

Other smaller diameter tunnels constructed in the area within the last 10 to 15 years include:

- In 1995 the Second Street Sewer was built to cross below the E.C. Row Expressway. This approximately 1.3 metre diameter sewer tunnel was constructed using pipe jacking techniques with an open faced shield at a depth of about 11 metres through mainly stiff to very stiff silty clay till materials. Difficulties were encountered during the work caused by squeezing of the soils into the open face.

- The Detroit River Outfall Tunnel No. 2 (DRO-2), near the location of DRO-1, was started in 2000 using an open rock tunnel boring machine at a depth of about 85 m below the ground surface but abandoned due to difficulties with grouting, water inflows, and hydrogen sulphide gas.

- The approximately 2.1 metre diameter McDougall Avenue Storm Sewer Tunnel was constructed about 11 metres below existing grade and from the Detroit River to Tuscarora Street in the City of Windsor, Ontario. The tunnel was constructed in 1997 using a TBM through the firm to very stiff grey silty clay till.

- The Prince Road Storm Sewer, measuring about 1.8 to 2.3 metre diameter, was constructed in phases between 1980 and 1998 at depths of between 6 and 9 metres below the ground surface. The tunnel was constructed in the soft to firm grey silty clay using pipe jacking techniques and compressed air was applied in some of the deeper sections to control clay squeeze.

The health and safety measures required to allow tunnelling under compressed air conditions make this methodology inefficient as compared to tunnelling using modern closed-face tunnel boring machines. Thus, it is anticipated that any machine tunnelling that would be done for the project through the soil deposits would be constructed with a closed face tunnel boring machine (TBM). Either slurry or earth pressure balance TBMs capable of applying a controlled pressure to the tunnel face could be used for construction. Such machines would be required to resist the squeezing pressures of the soft silty clay, the high groundwater pressures, and the presence of hydrogen sulphide gas. Soft-ground tunnel boring machines are generally limited in size to about 15 m in diameter or less, though TBMs of 10 to 15 m diameter are relatively unusual, with the largest soft-ground tunnel boring machine ever constructed being about 15.4 m in diameter. Without significant ground improvement, through replacement techniques such as jet grouting, or pre-support methods, such as jacked pipe or small diameter tunnel arches and inverts, mining of tunnel headings without a TBM is not considered possible given the anticipated ground conditions and the required tunnel size.

Bored tunnelling of “mixed faces” of soil and rock are problematic. Tunnelling through such conditions may be more suitable where the soft ground is relatively stable and an open face can be maintained to allow use of different excavation methods appropriate to the face conditions. Mixed face conditions are particularly problematic for tunnel boring machines where loose granular soils (either dry or saturated) or soft cohesive soils overlie harder ground. Where such conditions have been encountered on other projects, the granular soils are often improved by grouting/replacement techniques or by using specially modified earth pressure balance or slurry tunnelling machines. Where such conditions are not adequately addressed or unexpectedly encountered, such mixed-face tunnelling has been known to cause significant losses of ground and settlement, development of sinkholes at the ground surface, significant alignment difficulties, and obstruction of tunnelling (e.g. Shirlaw and Boone 2005). Where such conditions are known to exist, efforts to control ground losses and settlement can be made by improving the granular soils by grounting/replacement techniques or by using specially modified earth pressure balance or slurry pressure balance tunnelling machines; however, these techniques are not always successful. Therefore, for this project, it is considered that tunnelling using a tunnel boring machine should avoid any mixed faces of soil and rock.

7.2 Tunnel Depth and Stability

Available subsurface information indicates that bedrock elevations along the corridor between Highway 401 and Ojibway Parkway along Huron Church and Highway 3 and the E.C. Row Expressway range between about Elevation 148 m to 155 m, resulting in overburden thicknesses in the range of 30 to 35 m, though the thickness of overburden and depth to bedrock decrease from E.C. Row Expressway toward the Detroit River. Since tunnelling through a mixed face of soil and rock is problematic as discussed above it would be inadvisable to consider a tunnel with an invert elevation lower than about el. 153 m to 160 m such that there would be about 5 m between the planned tunnel invert and the top of rock. This prudent buffer distance is recommended so as to minimize the potential for encountering localized high points in bedrock and the potential for encountering larger boulders that typically have a higher frequency of occurrence near the soil/bedrock interface. Assuming a maximum tunnel diameter of 15 m, this limitation to the tunnel depth results in a maximum ground surface to crown of tunnel distance (cover) of about 10 m to 15 m, or cover-to-diameter ratios of between 0.6 to 1.0. The extent of the buffer distance might be reduced pending the results of further investigations to better define the elevations and variation of the bedrock surface.

The face stability of tunnels is dependent on the relative strength of the ground, the depth of the tunnel, and any internal pressure applied to the tunnel face. The most common approach to assessing the stability of tunnel faces in cohesive soils (silt and clay) uses the stability number, N, defined as:

$$N=(\gamma H-\sigma_t)/S_u$$

where

γ = total unit weight,

H = depth to tunnel axis

σ_t = pressure at tunnel face

S_u = undrained shear strength

Larger values of stability number indicate a less stable tunnel face (i.e. the ratio of forces tending to destabilize the face as compared to the capability of the soil strength to resist these forces is greater). Mair (1992) showed based on centrifugal modelling and case history analysis that a critical stability number, Nc, threshold is reached at which collapse is imminent and that this value depends on the thickness of soil overlying the tunnel (cover), C, and the tunnel diameter, D (i.e. the "cover to diameter" ratio, C/D). For low C/D values, of about 0.2, the Nc value is approximately 2.5, and as C/D values increase to about 1.5, the Nc value is about 6.5. As N approaches Nc the potential for and magnitude of settlements generally increase. If calculated N values are greater than Nc values, internal pressures must be increased to avoid instability.

In the case of a bored tunnel of about 15 m diameter with an axis depth ranging between about 17 and 22 m below the ground surface, calculated stability numbers range between about 3.5 and 8 using typical soil strength values from the available data, with values greater than 15 in some localized areas. For C/D values ranging between 0.6 and 1.0, Nc ranges between about 3.7 and 5.3 indicating that stability of the tunnel faces will be dependent on and sensitive to pressures applied to the tunnel face during construction. Calculated stability numbers for the historical tunnelling cases in the Windsor area (in which no face pressure was applied) support the general conclusions regarding the potential for squeezing (displacement) of ground into the tunnel face.

7.3 Preliminary Assessment of Surface Effects of Tunnelling

In general, where tunnels may be mined beneath existing surface features that are sensitive to ground displacement, it is best to maintain a distance of at least twice the tunnel diameter between the crown of the tunnel and the underside of any overlying facility or feature. Detailed evaluations of potential ground displacements will, however, be required for any such undertaking. For the purposes of this evaluation, however, a preliminary assessment of potential surface displacements given different tunnelling conditions was completed and is presented below.

Settlement and horizontal deformations at the ground surface and at various subsurface levels were estimated using the combined methods of Lee et al. (1992), Mair (1992), and Loganathan and Poulos (1998). The volume of soil that intrudes into the tunnel face, U3D, as a result of unbalanced pressures at the face was estimated by

$$U_{3D} \quad = \quad k\Omega RP_o/(2E_s)$$

where

$k = 1$ (for soft to firm clay)

Ω = dimensionless displacement factor depending on the stability of the tunnel face

R = tunnel radius

$P_o = K_o\sigma'_v + u - \sigma_t$

E_s = secant elastic modulus (Young's modulus)

σ_t = pressure at tunnel face

u = porewater pressure;

σ'_v = vertical effective stress; and

K_o = coefficient of horizontal earth pressure at rest (in situ state).

Deformations into the tunnel face increase beyond a characteristic stability number until a critical stability number at collapse is reached, N_c, and this depends on the depth of cover to tunnel diameter ratio C/D (Mair 1992). The undrained shear strength of the silty clay soils was assumed to be about 50 kPa, though values ranging between 25 and 90 kPa have been reported within a range of depths consistent with the potential springline elevation of a bored tunnel for this project. The relationship between Ω and N is a relatively constant value of about 1.12 when $N<2.5$ and for $N>2.5$, Ω increases exponentially; however, determination of Ω generally requires the use of charts derived from 3-D numerical modelling results (Lee et al. 1992). To implement the above relationships in the estimates of ground displacement it was considered that N_c represented an asymptote for Ω versus N and Ω could be determined using the following equation:

$$\Omega = 1.12+e^{\{N - [2+5\ln(C/D+1)]/2\}}$$

Control of face pressure during machine tunnelling depends on ground conditions, machine design, advance rate, rate of soil extraction from the front TBM chamber, slurry/muck viscosity and density, and operator experience. Based on experience and published information, it was considered that σ_t could typically vary between 50% to 120% of the planned pressure and that a consistent applied pressure equal to 80% of the total overburden stress, σ_v, (or about 110% of the in situ horizontal earth and water pressures) should represent a practical estimate of routinely achievable construction control for preliminary evaluation purposes. Such variability in EPB pressure control has been noted on projects worldwide including China, Italy, Singapore, Spain, Toronto, the United Kingdom and other locations (e.g. Chang et al. 2000, Lee et al. 2000, Gaj et al. 2003, Shirlaw et al. 2002 and 2003, Minguez et al. 2005, Shirlaw and Boone 2005, Borghi and Mair 2006, Minguez 2006). To examine the effect of variable face pressure on surface settlement, face pressure was varied between $\sigma_t = 50\% \sigma_v$ and $\sigma_t = 120\% \sigma_v$, where $\sigma_t = 50\% \sigma_v$ would represent unacceptable quality control, but possible performance in limited instances.

Closure of the gap between the cut diameter and the lining diameter, G_p, will be governed by the grouting around the lining. To consider the combined effects of variation in workmanship and ground conditions may have on grout placement, a "grouting efficiency", *Eff*, was defined as follows:

$$Eff = \frac{\text{Volume of Grout Injected}}{\text{Theoretical Gap Volume}}$$

Based on experience and published reports, it was considered that *Eff* = 80% would represent consistently achievable construction control in the soft soil conditions. Variability in grouting (both pressure and volume) is a key factor in development of surface settlement (e.g. Lee et al. 2000, Bakker et al. 2000, Minguez et al. 2006). To examine the effect of *Eff* on surface settlement, *Eff* was varied between 50% and 100%, where *Eff* = 50% would represent unacceptable quality control, but possible poor performance in limited instances.

The degree of over-excavation due to machine alignment difficulties that will be experienced is difficult to assess prior to construction; however, some allowance for such over-excavation must be made to provide a realistic settlement estimate. An equivalent radial over-excavation gap, ω, due to TBM alignment was using the approach of Lee et al. (1992) and Rowe and Lee (1992).

The sum $U_{3D}+G_p+\omega$ is equal to the "gap parameter", *g*, that represents an equivalent radial deformation including all of the above effects. The total "volume loss" at tunnel level can then be readily calculated based on *g*. It was assumed that the volume of ground "lost" at the tunnel level would be fully reflected at the ground surface and that the surface effects of three tunnels could be represented by superposition without consideration of dilation because of the soft soils and limited depth of cover. Potential additional compression of the pillar of soil between the tunnels or additional settlement resulting from disturbance of the ground by the preceding tunnel was not considered, though this is known to result in additional settlement in soft ground (e.g. Cording and Hansmire 1975, Addenbrook and Potts 2001, Cooper et al. 2002, Hansmire 2002, Chapman et al. 2006). Assessment of this effect will depend on refinement of potential tunnel geometry and further investigations and testing.

Parametric evaluations of face pressure and grouting efficiency were carried out on a single tunnel with a 15 m nominal diameter tunnel with a cover to diameter ratio of about 0.8, reflecting what might be typical conditions for the Area of Continued Analysis. The results of these evaluations are illustrated in Figure 3. This figure illustrates that the short-term ground surface settlements:

- could be less than 25 mm under ideal conditions and ideal workmanship;
- are sensitive to the control of grouting and face pressures;

- may typically be in the range of 100 mm to 200 mm for a single tunnel (or a surface settlement trough unit volume about 1.5% to 3% of the face unit volume); and
- the settlement trough of a single tunnel with the invert about 26 m below the surface could be on the order of 80 m wide.

The above range of settlement estimated for a single 15 m diameter tunnel is consistent with the St. Clair River tunnel experience in which a large diameter earth pressure balance machine was advanced through similar ground. In that case, average settlements for the 9 m diameter single tunnel were typically between 30 and 75 mm, excluding events with excessive settlements or heave, as described previously. When this average value is scaled up to represent a 15 m diameter tunnel, settlements ranging between about 80 and 195 mm would be expected.

Figure 4 illustrates an example estimated settlement trough for one tunnel bored with an EPB TBM in the Windsor soils under reasonable operating conditions and workmanship. Also shown in Figure 4 are superposed settlement troughs for two and three tunnels. The settlement troughs illustrated in Figure 4 are based on some simplifying assumptions and it has been shown in a number of case histories and analytical studies that the cumulative settlement trough can be greater than the superposition of individual settlement troughs, particularly when the tunnels are separated by distances of less than one tunnel diameter in soft soils. For the combined three-tunnel approach, short-term surface settlements may be on the order of about 100 to 200 mm.

These preliminary estimated short-term settlement ranges are typical of “volume loss” (or surface settlement trough unit volume) performance of tunnels in soft soil conditions similar to those that may be anticipated in the project vicinity. For all the conditions evaluated, the surface settlement trough unit volumes (for single tunnels) ranged between 1% and a maximum of 9% of the face volume, with typical values of about 2% to 4%. These typical values of 2% to 4% of the tunnel face unit volume are consistent with many case histories of single tunnels constructed in soft clay (e.g. Peck 1969, Lee et al. 1992, Mair 1992, Kramer et al. 1994, Shirlaw et al. 2002 and 2003, Shirlaw et al. 2006). It should be noted, however, that long-term surface settlements may increase due to consolidation of soft soils long after construction (Shirlaw et al. 1994). Excess face pressures or excess pressures during grouting around the lining may induce excess pore water pressures in soft soils so that as these pressures dissipate, the ground subsequently consolidates and subsides. These factors have not been considered in the preliminary analysis of surface settlements but should be considered during any further and more detailed analysis of a bored tunnel option.

The preliminary estimates provided above are for a tunnel constructed under typical conditions based on the information available and assuming a reasonable degree of quality in workmanship. If face pressures are not maintained within a relatively narrow operating range, however, surface settlements could be significantly greater than estimated above. In addition, it has also been shown that there remains some risk of unanticipated events (encounters with large boulders, bedrock intruding into the tunnel horizon, machine malfunctions, grout problems, sporadic loss of face pressure, etc.) leading to potentially larger surface settlements in localized areas and it is recommended that additional analysis and risk management activities be undertaken should the bored tunnel option be considered further (e.g. Shirlaw et al. 2003 and 2006).

For typical operating conditions and reasonably good workmanship, the estimates above suggest that surface centre-line settlements could be in the range of 100 mm to 200 mm. It is further anticipated that these settlements would be intolerable for many surface structures such as buildings, roadway structures, and critical utilities. Further evaluation of a bored tunnel option should include an assessment of the potential damage to surface facilities (e.g. Boone et al. 1998, Boone 2001, Harris and Franzius 2006, Boone 2007). The nature and degree of potential damage that such settlements may induce, the probability that such events might occur, and the potential mitigation measures will need to be examined in greater detail should a bored tunnelling option be further considered.

7.4 Tunnel Launch/Recovery Portals and Roadway Approaches

For preliminary assessment of a bored tunnelling alternative, the alignment should be planned to maintain a depth of cover of at least 0.5 times the tunnel diameter, or about 8 m. It should be noted, however, that the potential for relatively large settlements and uncontrolled losses of ground at the start of tunnelling and along the initial sections of the tunnel drive are greater than may be acceptable (as discussed above), particularly if areas of thinner cover are planned. In the case of maintaining a minimum C/D ratio of 0.5, the invert of the tunnel would likely be about 17 to 23 m below the ground surface. In all cases along the alignment this depth of excavation results in inadequate factors of safety for base uplift and “global” stability (as described previously) without adopting additional stabilization measures.

It is anticipated that the maximum slope permissible for the approach roadways would be between about 3% and 5%. In this case, the minimum length of approach may be on the order of 400 to over 750 m to reach the minimum invert depth for tunnelling and, due to the size and distance required between the tunnels, may be on the order of 60 m to 80 m wide. Considering that at depths of between 11 m and 17 m there will be an inadequate factor of safety against uplift and that the required approach cuts would exceed these limits by more than 6 m, mitigating measures for anchoring the base slab (either temporary or permanent) would be necessary along about 100 to 200 m at the eastern end of the project (near Highway 401) and 200 to 400 m at the western end of the project (near E.C. Row Expressway). The depth of the excavations and the need to provide horizontal support to the excavation walls may necessitate staged construction of the approaches for each tunnel bore or use of unusually long tie-back anchors to bedrock. The relatively large vertical span necessary to permit launching and construction of a 15 m diameter TBM must also be considered during conceptual and later design of tunnel approach structures.

As discussed in Section 8, the use of temporary deep dewatering of the bedrock or overlying granular soils or cut-off of groundwater pressures may not be practicable because of hydrogen sulphide gas, settlement induced by groundwater lowering and the large volumes of groundwater that may have to be extracted. Thus, it may be necessary to construct the base slabs and uplift-mitigation measures (anchors, piles, etc.) under slurry or water. Construction of such tunnel approaches in soft ground and under slurry or water is sometimes used in such locations as the Netherlands. A number of examples of such construction are discussed in van Beek et al. (2003). In such cases, friction piles driven deep into the soils below the base of the excavation are used to assist in resisting uplift. In other areas such as Singapore, extensive jet-grouting is used to form base slabs around anchor piles prior to excavation (Shirlaw 2006). For the approach structures necessary for a bored tunnel as described above, it would likely be necessary to anchor the base of the excavation and roadway structures to the bedrock, either temporarily or permanently, using rock anchors or drilled piles. The subaqueous construction methods described above are relatively uncommon in North America and may be impractical for this project, particularly given the required size of the tunnel launch and recovery areas and approach roadway sections.

7.5 Other Issues Related to Bored Tunnels

For earth pressure balance machine tunnelling, it is becoming more common for soil conditioning agents to be added to the spoil within the machine to assist in controlling machine wear and tunnelling face pressures. Such additives, however, may affect the suitability of the spoils for other uses or may complicate disposal options depending on the chemical composition of the conditioning agents. Issues associated with management and disposal of excavated materials should be examined if the bored tunnel is pursued further with respect to spoil volumes, consistency, disposal options, and environmental regulations.

At the time this report was prepared, the bored tunnel concepts suggested that the tunnels would be separated by distance approximately equal to half the tunnel diameter. In some cases, depending on the soil strength, tunnelling operation, and tunnel lining design, the separation distance may need to be increased to avoid inducing unacceptable unbalanced stresses on adjacent tunnel linings (e.g., the stresses on the first tunnel while the second and third tunnels are constructed). Structural design of the tunnel lining, while not the subject of this study, will also need to consider the potential effects of unbalanced strength, loading, and deformation that can develop during construction of any asymmetrical openings within the circular tunnel for ancilliary works such as emergency exits or cross-passages as such stresses will be influenced by both the soil strength and the degree to which the soil is disturbed during tunnel boring.

If the depressed roadway option is considered further through design, consideration will also need to be given to appropriate field instrumentation and monitoring during construction to assure that the effects of such work on nearby facilities is measured and maintained within achievable and specified limits.

8.0 INTERACTION BETWEEN BELOW-GRADE CONSTRUCTION AND GROUNDWATER CONDITIONS

As noted in Section 5, groundwater conditions inducing uplift pressures from the bedrock or overlying granular soil aquifer will significantly affect the feasibility of constructing deep excavations unless other excavation stability enhancement measures are implemented. Such stability enhancements commonly include dewatering or depressurization of groundwater levels. Significant temporary dewatering and permanent depressurization of groundwater levels was undertaken in similar soil conditions in Welland during construction, and later operation, of the Townline Road and Main Street tunnels beneath the Welland Canal (Farvolden and Nunan 1970, Frind 1970, Olpinski 1970, Golder Associates project files). Dewatering the bedrock aquifer in Welland for these tunnel projects resulted in extracted water volumes on the order of 1,500 to more than 6,000 litres per minute with a zone of influence (where the groundwater levels were drawn down) of many kilometers surrounding each pumping site. The long-term depressurization of the groundwater has created consolidation within the overlying silty clay, though because the area was primarily rural at the time, few surface facilities were affected.

Based on existing information, groundwater in the Windsor area contains hydrogen sulphide and managing this dissolved gas, should dewatering be undertaken, will be critical to the safe completion of the project. There is also the potential for groundwater flows through fractures within the upper horizons of the bedrock to be significant and/or prohibitive for temporary construction dewatering of relatively large areas. In addition, depressurization or dewatering of granular soils near the bedrock interface or bedrock will induce measurable consolidation settlements within the overlying silty clay soils. Depending on the local strength and compressibility of these soils, such settlements may cause damage to structures over a wide area because the zone of influence of such depressurization could be extensive unless measures such as grout curtains or re-injection systems are implemented. Detailed investigations, testing, and analyses will be required to adequately assess the feasibility of dewatering or depressurization of the groundwater levels within the bedrock or the granular soils separating the bedrock from the overlying silty clay deposits.

Creating permanent open depressed roadways within the native clays using slopes or supported with retaining walls that do not cut off groundwater pressure gradients from the adjacent higher grades, will result in a permanent lowering of the groundwater level within the clay soils. Based on the limited available information, and for preliminary planning purposes, it is anticipated that the zone of influence of such near-surface groundwater lowering within the silty clay should be assumed to be a distance equal to about 5 to 10 times the depth of cut. Such groundwater lowering will induce settlement within the silty clay subsoils within this zone. It is anticipated that if low permeability in situ walls (e.g. contiguous caisson walls or concrete diaphragm walls) are used for excavation support or for permanent below grade structures, that the influence of the excavation on near-surface groundwater would be minimal. Further refinement of this zone of influence and the magnitude of potential settlement requires additional site specific investigation and analyses.

9.0 FURTHER INVESTIGATION

The evaluations described in this report are based on a review of the documented subsurface conditions gathered for other projects and highway access route concepts developed and provided by URS. These evaluations have been provided to allow a general assessment of the different retaining wall and tunnel construction systems which can be used along the proposed Highway 401 extension. It is essential that detailed geotechnical and hydrogeological investigations are conducted prior to further design and consideration of these alternative construction options because the strength of the soils, the groundwater levels, and the hydraulic conductivity of the soils and bedrock will have significant effects on the conceptual design and final design alternatives and costs. It is recommended that for further examination of these concepts, the following subsurface investigations be completed:

- Four boreholes should be completed at selected locations along the route at likely critical structure or route locations. These boreholes would be drilled to bedrock and then cored into bedrock. Field (in situ) testing should be completed in each of the four boreholes, consisting of field vane shear testing (VST) and piezocone penetration testing (CPTu). This will permit correlation of in situ tests to the results of geotechnical laboratory testing that should be carried out on "undisturbed" and "disturbed" samples recovered from the boreholes.

- Groundwater observation wells and stand-pipe piezometers should be installed in the boreholes to allow measurement of groundwater pressures and preliminary assessment of permeability of the rock and water-bearing soil deposits.

- Laboratory testing should be conducted on the soil samples recovered from the boreholes. Soil strength should be determined using consolidated, undrained triaxial compression tests with pore water pressure measurements, and one dimensional consolidation tests to assist in defining the compression characteristics of the silty clay soils. Conventional index property tests (water content, Atterberg Limits determinations) should also be carried out on selected samples.

- Additional CPTu tests should be completed along the alignment at approximately ½ km intervals to assess compressibility and strength of soils between the boreholes (critical for evaluating embankment and depressed roadway issues). Each of these tests would be conducted from the ground surface to refusal.

The level of investigation effort described above, while essential for continuing further evaluation of project alternatives, will not be sufficient for final design. Preparation of investigation programs suitable for design of the selected alternative should be developed once future concepts are further refined. At the time this report was finalized, investigations meeting the above recommendations were underway.

10.0 SUMMARY & CONCLUSIONS

The available subsurface information and preliminary evaluations completed as part of this study suggest that:

- construction of open-cut (depressed roadway) sections may be made to assist in separating traffic grades with permanent side slopes of approximately 2.5:1 (horizontal:vertical) or with permanent retaining structures (using a variety of systems) provided that the cut depths are limited to values consistent with the transition in ground strength and groundwater conditions from east to west along the corridor;

- cut and cover tunnels should be feasible for the entire length of the approaches, however, base stability conditions may require either temporary ground improvement measures or other temporary wall and base stability enhancements during construction of excavations particularly in the areas in which the tunnel passes beneath the Grand Marais and Cahill Drains and areas closer to E.C. Row Expressway and westward; and

- construction of multiple bored tunnels on the order of 15 m diameter may imply that preservation of existing structures above the alignment could be achieved; however, the limited thickness of soil above the tunnels and the realistic limits on tunnel construction quality control are likely to result in unacceptable ground surface settlements that could preclude preservation of existing structures, utilities, and roadways. This, and the anticipated difficulties with approach cut construction increase the risks and cost of the bored tunnel alternative for this project.

The feasible cut depths for excavations are particularly sensitive to local soil strength values and, along much of the alignment, the potential cut depths are near the threshold of permissible values; therefore, further investigation and testing will be necessary for evaluating appropriate excavation support systems and any stability enhancement measures.

Each of the below-grade construction alternatives discussed in this preliminary evaluation induce differing degrees of risk to adjacent facilities and overlying roadways; some of these risks may be acceptable while others may not. The options also carry differing degrees of risks to the project design and the construction cost and schedule. The tolerance of the adjacent and overlying facilities to displacements should be addressed in progressively greater detail as the project evolves, taking into account changes in conceptual and, at later stages, design geometry in combination with future subsurface investigation results. The owner’s tolerance of varying levels of technical, cost, and schedule risks, and the risks to third parties should also be examined in detail as the project develops and appropriate risk analysis, mitigation, and management strategies developed appropriate to the stage of design or construction (e.g. ASCE 1997, Westland et al. 1998, Eskessen et al. 2004, Boone 2007, ITIG 2006). Based on the information available to date and our experience with underground construction on similar large infrastructure projects, it is considered that, from a geotechnical perspective, the aforementioned risks are greater for bored tunnel alternatives on this project.

This report has been prepared to assist the DRIC project team with evaluating conceptual alternatives associated with the highway access route to the Detroit River crossing. As this work was prepared to assist with conceptual alternatives and is based on available data, the recommendations provided within this report should be reviewed and revised as necessary as further information is developed with respect to design concepts, more detailed structure locations, and subsurface information.

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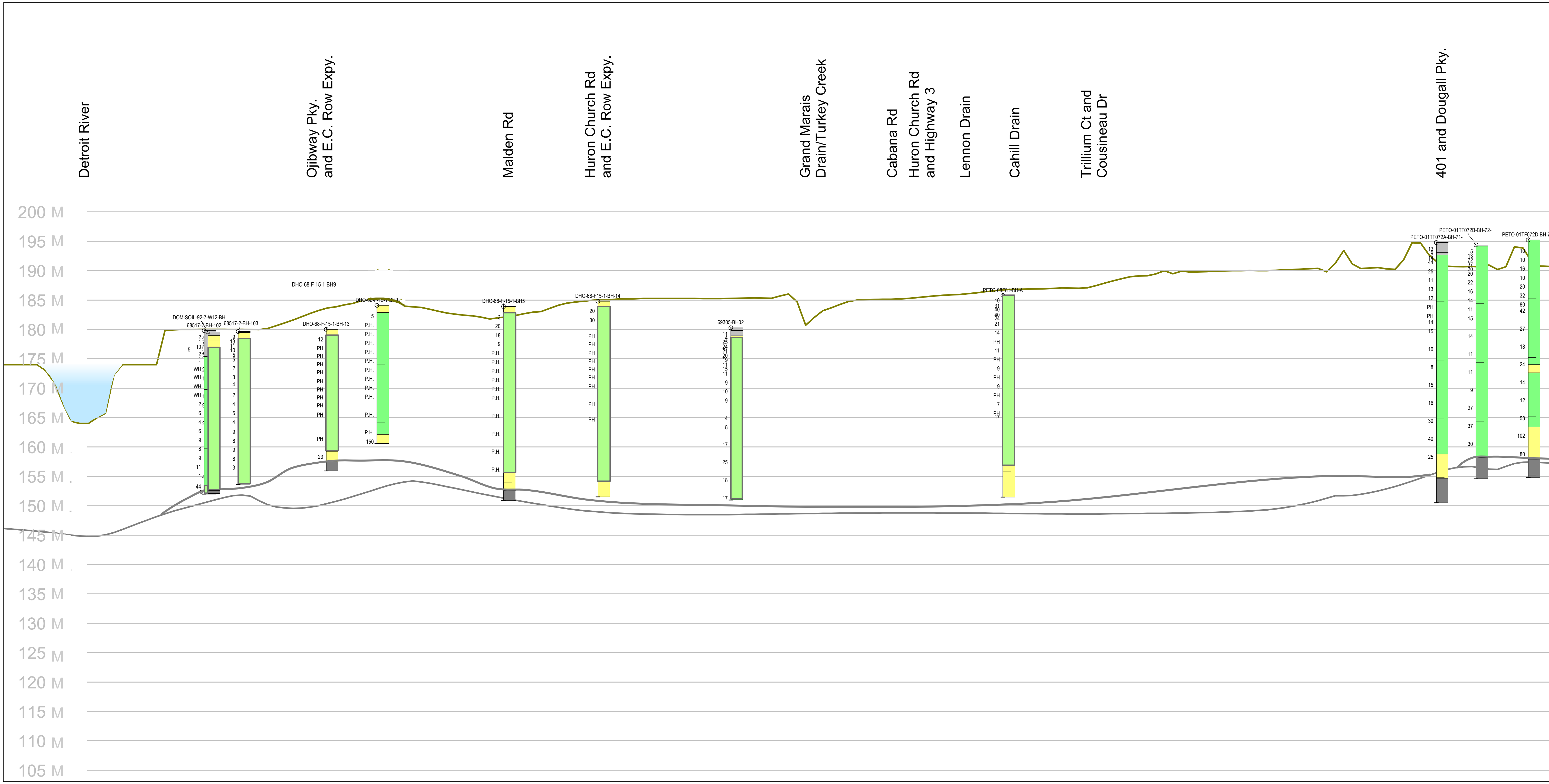
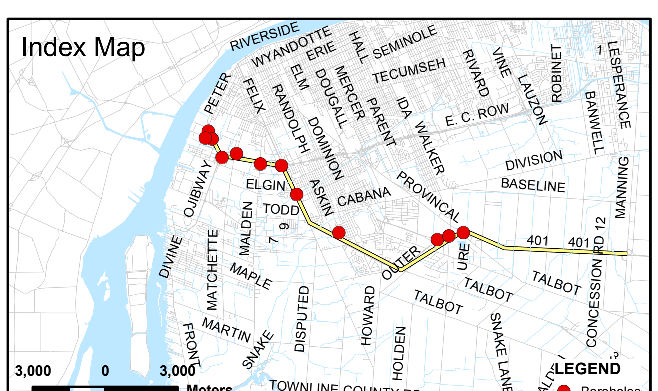
SUMMARY OF GRADE SEPARATION SUPPORT OPTIONS

Grade Separation System	Depth of Cut Limitation (m)[1]					Approximate Construction Area Behind Wall or Slope Toe (m)[2]	Relative Construction Cost[3]	Environmental Considerations[4]	Notes
	Hwys 401/3	Hwy 3 & Cousineau	H.C.Rd, & Hwy 3	H.C.Rd. & E.C.R. Expwy.	E.C.R. Expwy. & Ojibway Pkwy.				
Permanent Slopes	7 – 8 (G)	7 - 8 (G)	7 - 8 (G)	5 – 7 (G)	4 - 6 (G)	2H	Low	Groundwater control on slopes required	
Gravity Wall Systems									
Cast-in-Place Concrete Walls						(0.3H to 0.5H) + *f*H	High	Disposal of excess cut	Granular backfill required
Mechanically Stabilized Earth (MSE) Walls	7 – 8 (G)	7 - 8 (G)	7 - 8 (G)	5 - 7 (G)	4 - 6 (G)	0.75H + *f*H	Medium	Disposal of excess cut	Granular backfill required
Crib and Bin Walls						(0.3H to 0.5H) + *f*H	Medium	Disposal of excess cut	Granular backfill required
Pre-Cast Cantilever or Counterfort Walls						(0.3H to 0.5H) + *f*H	High	Disposal of excess cut	Granular backfill required
In Situ Wall Systems									
Soil-Nail Wall	7 – 8 (G)	7 - 8 (G)	7 - 8 (G)	5 - 7 (G)	4 - 6 (G)	< 1 - 2 m (I)	Low		Low industry capacity in ON/MI. Most suited to temporary construction. Can not extend below (G) or (U) limits. Work must be performed from within the excavated area.
Driven Sheet-Piles						1 – 2 m (I) 7 – 10 m (O)	Low	Vibrations during driving. Below (U) limit will require significant groundwater pressure control or resistance	Significant industry capacity in ON/MI. Pipe-sheet pile wall may form permanent wall. Depth may be extended below limitations shown with special provisions.
Secant or Tangent Pile (Caisson) Wall						1 – 2 m (I) 7 – 10 m (O)	Medium to High	Slurry management for deep holes. Below (U) limit will require significant groundwater pressure control or resistance	Significant industry capacity in ON/MI. May form permanent tunnel or retaining walls. May be suitable for top-down construction. Depth may be extended below limitations shown with special provisions
Soil-Cement Mix Wall	15 - 16 (G) 17 (U)	12 - 13 (G) 15 (U)	10 – 12 (G) 13 (U)	10 – 12 (G) 13 (U)	16 - 17 (G) 11 (U)	1 – 2 m (I) 7 – 10 m (O)	Medium	Disposal of excess mix. Below (U) limit will require significant groundwater pressure control or resistance	Low industry capacity in ON/MI. May form permanent retaining walls with precast panels. Depth may be extended below limitations shown with special provisions
Soldier-Pile and Lagging						1 – 2 m (I) 7 – 10 m (O)	Low	Vibrations during driving, slurry management for pre-drilled deep holes.	Significant industry capacity in ON/MI. May form permanent retaining walls with precast panels.
Cast-in-Place Concrete Diaphragm Wall						1 – 2 m (I) 7 – 10 m (O)	High	Slurry management. Below (U) limit will require significant groundwater pressure control or resistance	Low industry capacity in ON/MI. May form permanent tunnel walls. Depth may be extended below limitations shown with special provisions. May be suitable for top-down construction.

Notes:

1. Depth limitations shown (G) for global stability (deep-seated stability of ground mass) and (U) for uplift related to groundwater pressures in bedrock or near-bedrock granular soils.
2. H = exposed height of the wall, *f* = factor for temporary cut slope being 1 near Highway 401 and Highway 3, transitioning to 1.5 near Highway 3 and Huron Church Road, to 2 near Huron Church Road and E.C. Row Expressway and westward, (I) indicates space required behind the wall if the equipment is operating from Inside the excavation area, (O) indicates space behind the wall if the equipment is operating from Outside the excavation area;
3. Relative construction costs are relative to each of the two main categories (i.e. High cost of Cast-in-Place Concrete Wall relative to Low cost of MSE wall). Cost for in situ permanent walls are typically 1.5 to 2 times as costly as gravity wall systems unless deep foundations are required. Relative costs shown in table above do NOT include cost of permanent wall systems for in situ walls and cost comparison only addresses relative costs if in situ walls are considered temporary excavation support structures.



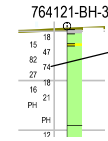


LEGEND

— Ground Surface
— Bedrock Surface

Hole Stratigraphy

- Bedrock
- Fill &/or Organic
- Massive to well laminated clayey silt to silty clay diamict and Glaciolacustrine-derived silty to clayey till
- Sand & Gravel
- Sand & Silt



764121-BH-3 - Borehole Identification
Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

Note:
1. This simplified stratigraphy is based on a limited number of boreholes projected onto the profile line. Specific ground conditions between boreholes and along profile line will be different than as shown. Refer to report text for discussion of the limitations of this report.
2. Index map illustrates plan location of boreholes shown on profile.

REFERENCE

Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

750 375 0 750 1,500
Meters

PROJECT
ROUTE PLANNING
DETROIT INTERNATIONAL CROSSING
CANADIAN SIDE

TITLE
SITE LOCATION AND SIMPLIFIED STRATIGRAPHIC PROFILE

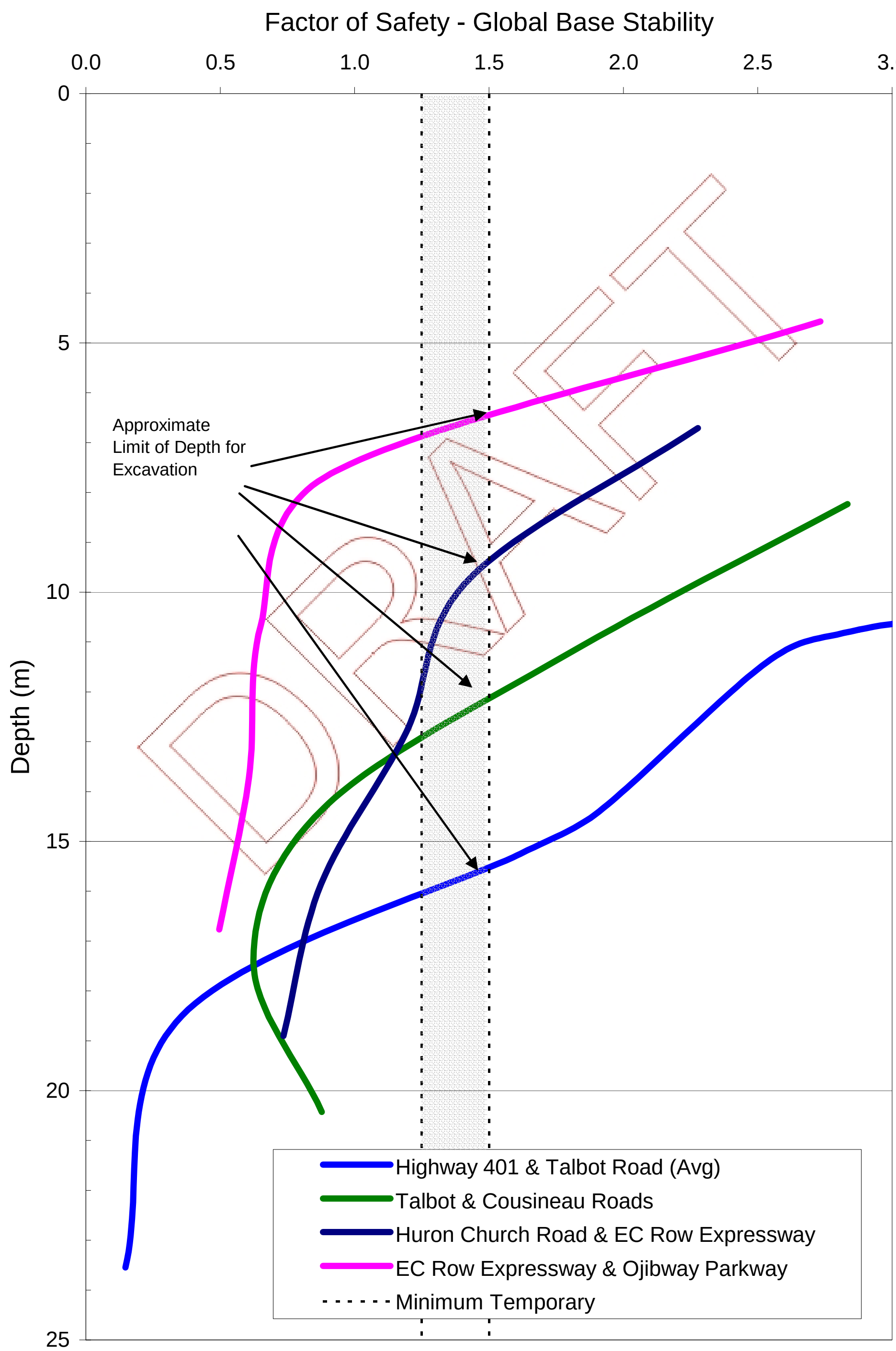
Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 20,000	REV. 0
DESIGN	AW	15 Mar. 2005	FIGURE: 1	
GIS	CC	29 May 2006		
CHECK				
REVIEW				

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\Feb_06_ProfileF\FigureX_ProfileF Rough_v2.mxd

DETROIT RIVER INTERNATIONAL CROSSING
Area of Continued Analysis - Feasibility Study
Factors Influencing Surface Settlement – Bored Tunnels

FIGURE 2



Notes:

1. Factor of safety estimates based on limited unconfined compression test data and assumed excavation geometry
2. See text for additional description and limitations of analyses in report prepared by Golder Associates Ltd., June, 2006, titled “Below-grade Approach Roadways, Cut And Cover And Tunnel Options, Detroit River International Crossing”..

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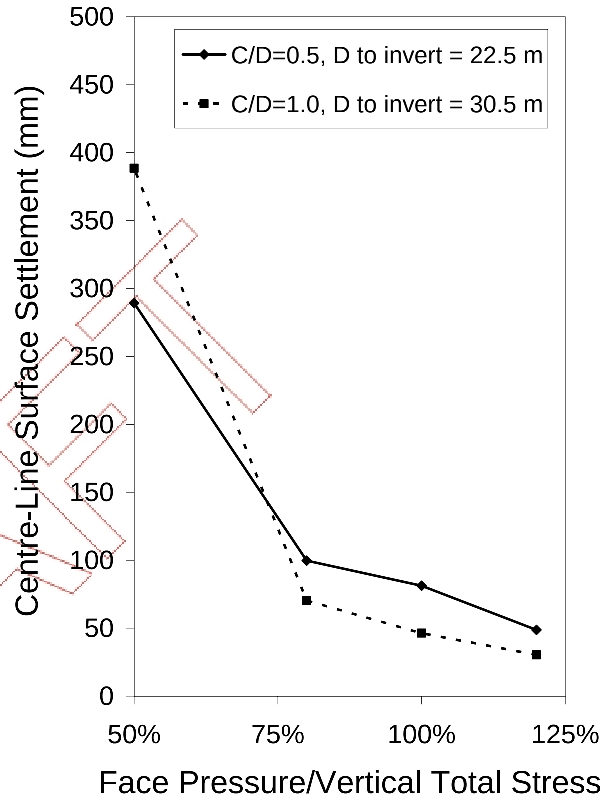
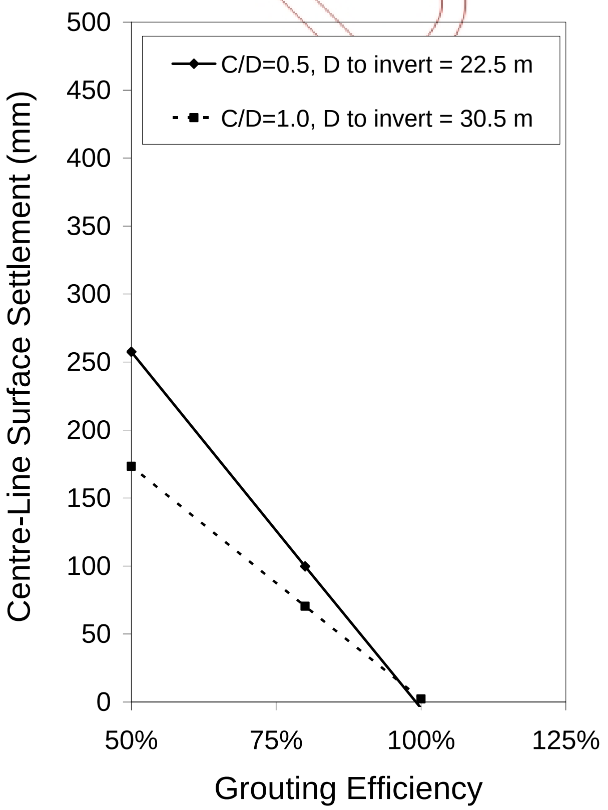
Project: 04-1111-060 Drawn: SJB Reviewed: FH/MD Rev.: June 2006

DETROIT RIVER INTERNATIONAL CROSSING
Area of Continued Analysis - Feasibility Study
Factors Influencing Surface Settlement – Bored Tunnels

FIGURE 3





Notes:

1. Example centre-line surface settlement values shown for single tunnel with diameter (D) = 15 m, undrained shear strength = 50 kPa
2. Example settlement values are provided for illustration purposes only
3. See text for additional description and limitations of analyses in report prepared by Golder Associates Ltd. titled “Interim Report on Geotechnical Considerations for Below-grade Approach Roadways, Cut And Cover And Tunnel Options, Detroit River International Crossing”.

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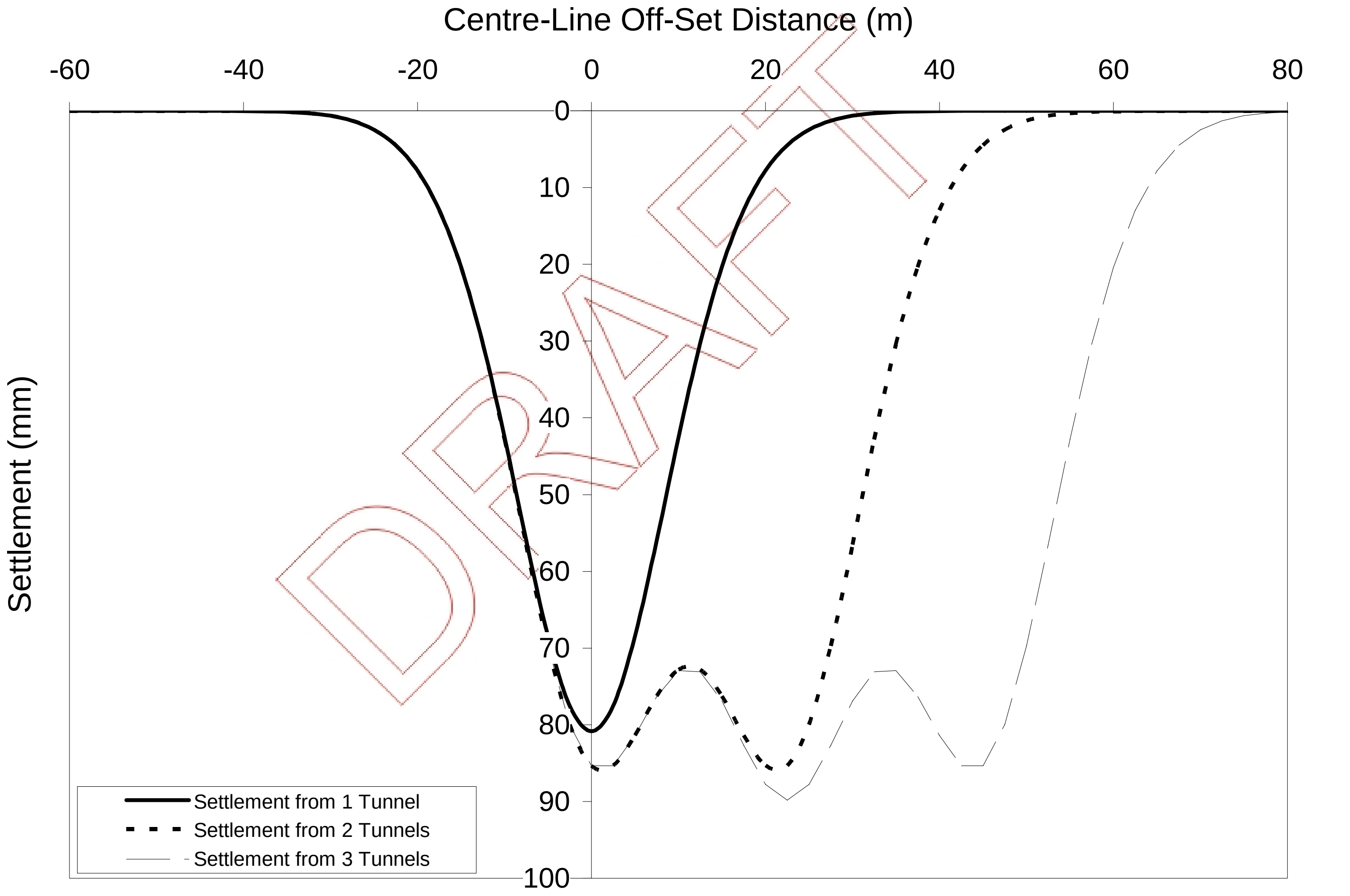
DETROIT RIVER INTERNATIONAL CROSSING
Geotechnical Considerations, Below-Grade Approach Roadways
Surface Settlement Trough – Bored Tunnels

FIGURE 4



Notes:

1. Example surface settlement trough shown for three tunnels with tunnel diameter (D) = 15 m, C/D = 0.8, face pressure = 80% total vertical stress, undrained shear strength = 50 kPa, centre-centre tunnel spacing = 1.5D, 80% grouting of annular space
2. Example settlement trough is provided for illustration purposes only
3. See text for additional description and limitations of analyses in report prepared by Golder Associates Ltd., June, 2006, titled "Interim Report on Geotechnical Considerations for Below-grade Approach Roadways, Cut And Cover And Tunnel Options, Detroit River International Crossing".

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