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WINDSOR-ESSEX PARKWAY

Subsurface Conditions Baseline Report

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REPORT

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1.0 INTRODUCTION

This is the Subsurface Conditions Baseline Report (SCBR) for the Windsor-Essex Parkway from North Talbot Road to Ojibway Parkway in Windsor, Ontario. The area of the site is illustrated on Figure 1.1. This report consolidates and summarizes the results of geotechnical explorations and testing carried out on behalf of the Ministry of Transportation, Ontario. This report is to be read together with the Contract Drawings and Specifications prepared by the project designers (URS Canada Inc.) and the Ministry of Transportation, Ontario.

The purpose of this report is to describe and summarize the subsurface conditions anticipated along the project alignment and to establish the baseline geotechnical conditions for the Contract. Methods of testing and interpretation of the test results are described as background to the presentation of baseline geotechnical parameters and as a means for comparing the results from different test methods to the baseline parameters.

This Subsurface Conditions Baseline Report has been prepared for a Design-Build construction contract for the Windsor-Essex Parkway. Therefore, the content of this report departs from typical practice (ASCE 2007) for design-bid-build (DBB) projects and does not provide discussions on the anticipated ground behaviour in relation to construction since the final design and construction means and methods were not identified or characterized at the time this report was prepared. Consistent with ASCE (2007) guidelines for Design-Build projects, this report provides baseline subsurface conditions and geotechnical engineering parameters for use during the tender design period and forms the basis from which to judge whether or not the conditions actually encountered during construction are different from those detailed in this report.

This report provides a number of tables and figures that summarize data and present baseline conditions and parameters. The tables are provided within the report text and figures follow the text. All tables and figures are numbered consistent with the report sections within which they are first referenced. Where alignments and stationing are shown on the figures or referred to in the text, they are based on the preliminary information available at the time this report was prepared.

2.0 PROJECT DESCRIPTION

The Windsor-Essex Parkway project includes extending the existing Highway 401 from near its current terminus at Highway 3 (near North Talbot Road) northwest along Highway 3 to Huron Church Road, along Huron Church Road to near the intersection with E.C. Row Expressway, and then adjacent to the E.C. Row Expressway to its intersection with Ojibway Parkway, as shown on Figure 1.1. The highway is to be constructed within a cut section, declining from near North Talbot Road and continuing below existing grade elevations until rising to meet the existing ground surface near the intersection of Huron Church Road and the E.C. Row Expressway. West of the Huron Church Road and E.C. Row Expressway intersection, the new highway section will then be parallel to and incorporate portions of the E.C. Row Expressway on embankments graded to permit overpasses at Malden and Matchette Roads, Ojibway Parkway, and the Essex Terminal Railway. In addition, underpasses for ramps built on high embankments will be constructed along this section.

3.0 SOURCES OF INFORMATION

The documents listed in this section have been used in developing this Subsurface Conditions Baseline Report, but these are not to be considered part of the Subsurface Conditions Baseline Report. A number of documents listed below represent interpretive reports prepared for field and laboratory investigations and design initiated by the Ministry of Transportation Ontario or others for this and adjacent projects. In addition, a number of publications were used in the development of this report and are referenced herein for information purposes only. Although there is considerable overlap between these reports and referenced documents, this Subsurface Conditions Baseline Report represents the most recent interpretation of conditions for design and construction and serves as the only Subsurface Conditions Baseline Report for this Contract.

The Geotechnical Data Report (GDR), Geocres No. 40J6-27, referenced below, was used as the primary source of data for development of this Subsurface Conditions Baseline Report (SCBR). Where precise determination of deposit boundaries or geotechnical engineering parameters are necessary for the design, safety and stability of the works, or for other construction concerns, or in instances where specialized subsurface properties or analytical parameters are required, but are not presented in the SCBR, these boundaries and parameters should be identified and determined by supplementary investigations and testing during design and prior to construction. The subsurface materials as characterized at specific sample locations within the boreholes can be relied upon and reference should be made to the specific subsurface data available in the GDR. However, the interpretation of engineering properties and parameters for the deposits and the stratigraphy as interpreted between samples as presented in the SCBR are the baselines for this project. In the event of conflict between the GDR and the SCBR, the SCBR shall be given precedence for the purpose of tendering and evaluating claims for unforeseen ground conditions.

3.1 Subsurface Data

Subsurface data gathered from multiple sources have been used in development of this report. The principal source of data is the Geotechnical Data Report:

Golder Associates Ltd. (Golder) (2009). Geotechnical Data Report, Windsor-Essex Parkway. GEOCRES 40J6-27.

Other sources have been used to supplement this information and these sources included:

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Ministry of Transportation Ontario (1994). Foundation Investigation Report for Highway 401/Highway 3, High Mast Lighting Foundations. WP#143-91-00, GEOCRES 40J-40.

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3.2 Publications

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4.0 ADJACENT STRUCTURES AND FACILITIES

Construction of the Windsor-Essex Parkway extension to Highway 401 will extend through the Towns of LaSalle and Tecumseh, and the City of Windsor. As such, the construction will influence and will be influenced by nearby existing structures and facilities. Major structures or utilities for which subsurface data were available are discussed within this section, and, if adequate information existed, discussions are also provided relative to the geotechnical performance to date of these facilities, some of which exhibited unsatisfactory performance.

4.1 E.C. Row Expressway/Matchette Road Overpass

The existing E.C. Row Expressway/Matchette Road overpass structures were built in two phases. The westbound overpass structure was constructed in 1982 as the E.C. Row Expressway was first constructed as a two lane highway on a single embankment. The eastbound structure was built in the late 1980s when the highway was twinned. Each of these structures consisted of an approximately 38 m long three-span bridge supported by two abutments and two piers. All foundations consisted of steel piles driven to bedrock or refusal. It is understood that these piles were drilled with closed ends and filled with concrete. Pile driving records were not available at the time this report was prepared. The 1978 design drawings indicate that the top of pile cap elevations for the abutments and piers were to be at Elevations 181.9 m and 178.0 m, respectively, for the westbound structure and that 324 mm diameter driven tube (pipe) piles were to be used for foundation support. The 1988 design drawings indicate that the top of pile cap elevations for the abutments and piers were to be at Elevations 181.3 m and 178.0 m, respectively, for the eastbound structure. The 1988 design drawings available at the time this report was prepared did not indicate the dimensions of the specified steel H piles for the foundations. Approach slabs approximately 6.0 m long (parallel to the road centre-line) were used for both structures. The approach embankments were approximately 6.2 m high at this overpass and constructed with 2 horizontal to 1 vertical side slopes.

The foundations investigation completed for this site in 1968 and reported in 1978 (GEOCRES 40J6-8) included four test borings to depths ranging from 14. 6 m to 25 m, with the deepest of these including about 1.65 m of rock coring. Foundation recommendations prepared at the time indicated that long-term settlement of the approach embankments was estimated to be about 125 mm to 150 mm.

During design of the eastbound structure, construction documents from the earlier construction of the westbound lanes and structure were reviewed by Ministry of Transportation, Ontario (MTO). A letter dated April 6, 1987 (GEOCRES 40J6-8-2), noted that the Construction Report for Contract 81-05 indicated settlement of the high fills being most notable at the bridge approaches, “considerable cavitation” (or void formation) occurred beneath the approach slabs due to settlement, and that the approach slabs cracked. The referenced Construction Report was not available at the time this report was completed and the total differential settlement producing the observed damage is not known. It is understood that the damaged approach slabs were repaired at the time and the voids filled, though it is unknown whether these voids were found on the west or east side approach slabs, or on both sides. The duration of approach embankment construction or the period of elapsed time between westbound embankment and structure construction remains unknown. A subsequent design memorandum prepared by MTO and dated April 15, 1987 (GEOCRES 40J6-8-2) recommended that the eastbound embankments be constructed and permitted to settle for as long as possible prior to constructing the new eastbound bridges. No construction reports or other documents for construction of the eastbound lanes and bridge were available at the time this report was prepared to indicate construction or post-construction performance of the bridge, approach embankments, or approach slabs for the eastbound lanes. However, during bridge repair work in 2008, voids on the order of 300 mm thick were again found beneath the Matchette Road westbound bridge east side approach slabs and voids of about 20 to 30 mm thick were found beneath the west side approach slabs. Voids were also discovered beneath the concrete surface treatments of the fore-slopes beneath the bridges abutting Matchette Road. These discovered voids were subsequently repaired by filling them with grout.

4.2 E.C. Row Expressway/Malden Road Overpass

The existing E.C. Row Expressway/Malden Road overpass structures were built in two phases. The westbound overpass structure was constructed in the early 1980s as the E.C. Row Expressway was first constructed as a two lane highway. The eastbound structure was built in the late 1980s when the highway was twinned. Each of these structures consisted of an approximately 39.5 m long three-span bridge supported by two abutments and two piers. All foundations consisted of steel piles driven to bedrock or refusal. It is understood that these piles were drilled with closed ends and filled with concrete. Pile driving records were not available at the time this report was prepared. The 1978 design drawings indicate that the top of pile cap elevations for the abutments and piers were to be at Elevations 183.8 m and 179.8 m, respectively, for the westbound structure and that 324 mm diameter driven tube piles were to be used for foundation support. The 1983 design drawings indicate that the top of pile cap elevations for the abutments and piers were to be at Elevations 183.0 m and 179.8 m, respectively, for the eastbound structure. The 1983 design drawings available at the time this report was prepared did not indicate the dimensions of the specified steel H piles for the foundations. Approach slabs approximately 6.0 m long (parallel to the road centre-line) were used for both structures. The approach embankments were approximately 6.2 m high at this overpass and constructed with 2 horizontal to 1 side slopes.

The foundations investigation completed for this site in 1968 and reported in 1978 (GEOCRES 40J6-9) included four test borings to depths ranging from 12.6 m to 32.9 m, with the deepest of these including about 1.65 m of rock coring. Foundation recommendations prepared at the time indicated that long-term settlement of the approach embankment was estimated to be about 100 mm to 125 mm.

No construction reports or other documents for construction of the westbound or eastbound lanes and bridges were available at the time this report was prepared to indicate construction or post-construction performance.

4.3 E.C. Row Expressway Interchange/Huron Church Road

The E.C. Row Expressway crosses Huron Church Road on two bridge structures carrying two lanes of traffic each. These two span bridges are approximately 56.5 m long between abutments with one pier near the centre of the bridges. The approach embankments are about 6 m high above the original grades and were constructed with 2:1 (horizontal:vertical) side slopes.

The foundations investigation completed for this site in 1968 (GEOCRES 40J6-3 and 40J6-10) included four test borings to depths ranging from 16.2 m to 36.3 m, with the deepest of these including about 3.0 m of rock coring. Boreholes designated BH-14 through BH-17, originally drilled in 1968, were renumbered BH-1 to BH-4 for a subsequent report prepared in 1978. The 1978 report provides a number of foundation design recommendations for support of these structures on either driven steel H piles or on spread footings. Settlement estimates prepared at the time indicated that long-term total settlements of the approach embankments and spread footings for abutments founded in the embankments could be about 100 mm to 125 mm and about 35 mm to 50 mm for the pier foundations, should these be constructed using spread footings. Final design drawings prepared in 1983 indicate that the bridges for this interchange were supported by driven HP310x79 H piles. Top of pile cap elevations varied between about 182.3 m for the piers and 184.9 m for the abutments. Construction reports or other documents were not available at the time this report was prepared to indicate post-construction performance.

4.4 Huron Church Road Turkey Creek/Grand Marais Drain Bridge

A single span bridge currently carries Huron Church Road over Turkey Creek/Grand Marais Drain. This bridge span is about 23.8 m long with a width of 25.6 m and is constructed of precast concrete girders supported by spread footings founded at approximately Elevation 176.5 m. The watercourse at this location is within a concrete lined channel with side slopes of about 1.5 horizontal to 1 vertical and an invert elevation of about 176.2 m, or about 5.8 m below the existing road surface in the vicinity of the crossing. Additional detail regarding this creek channel slopes is provided in Section 4.5, below.

A foundations investigation was completed for the bridge structure in 1969 (Golder 1969, GEOCRES 40J6-5) and consisted of two boreholes to auger and sampler refusal depths of 33.5 m and 33.8 m. Settlement of the spread footings for the abutments was estimated to be about 25 mm for an applied bearing pressure of 90 kilopascals (kPa).

No construction reports or other documents were available at the time this report was prepared to indicate construction or post-construction performance.

4.5 Turkey Creek/Grand Marais Drain

Early records indicate that the original Turkey Creek channel was widened and channelized in 1886 and significant channel work was completed in 1958 when it was deepened and widened using side slopes of 1.5:1 (horizontal:vertical) between South Cameron Boulevard and Todd Lane. Some remedial work was carried out in 1964 and 1965 on this section but the extent and nature of this work is not known. A geotechnical investigation and slope stability analysis were carried out and reported in 1969 (Golder, 1969). This work also included a review of the slope conditions along the creek. The review concluded that the slopes exhibited poor conditions including relatively large failures, loss of native granular soils from the slope crests, and erosion problems. The slopes at the time the review was conducted varied from 1.5:1 to 2:1. To achieve adequate slope stability, it was recommended that the channel reconstruction be provided with a concrete lining as well as a 3 m deep cut-off drain system located about 4.5 m from and parallel to the slope crest. No construction reports or other documents were available at the time this report was prepared to indicate construction or post-construction performance. The watercourse at the location of Huron Church Road is presently within a concrete lined channel with side slopes of about 1.5:1 and an invert elevation of about 176.2 m, or about 5.8 m below the existing ground surface in the vicinity of the crossing. There was no evidence of significant post-remediation repairs made to this drainage channel in the immediate vicinity of Huron Church Road based on a site visit by a Golder staff member in 2008.

4.6 Highway 401 (Westbound)/Highway 3 Underpass (Site No. 6-067)

The Highway 3 bridge over Highway 401 (westbound) was constructed as a two span, cast-in-place, concrete rigid frame structure in the mid-1950s. This structure was built with spread footing foundations ranging in width from about 1.28 m to 3.93 m. The 21.5 m long footings beneath the abutments were about 1.28 m wide. The centre pier footing being 1.97 m wide. The top of footing elevations were approximately 186.5 m based on the original 1955 design drawings. It has been estimated that these abutments experience an un-factored dead load of about 4,426 kilonewtons (kN), resulting in an applied bearing pressure of about 161 kilopascals (kPa). It has been estimated that the pier foundation experiences an un-factored dead load of about 7,756 kN resulting in an applied bearing pressure of about 183 kPa. A structure settlement study, carried out in 2006 (Golder, 2006), indicated that this structure has experienced settlements ranging from about 57 mm to 107 mm at the abutments and about 43 mm to 60 mm at the pier. The approach embankments measure about 8 m high above the adjacent ground surface and were constructed with 2:1 (horizontal:vertical) side slopes There are indications that the approach embankments were typically constructed to heights of about 3 m prior to completion of the superstructure. Therefore, the magnitude of total settlement induced by the embankments preceding the superstructure construction is unknown.

Subsurface investigations carried out for the underpass structure were reported on the 1955 design drawings. These investigations consisted of six “percussion test” holes and two auger boreholes to depths of about 3 to 3.4 m with generalized soil descriptions provided on the drawings. No quantifiable data were derived from these explorations.

4.7 Highway 401/North Talbot Road Underpass (Site No. 6-068)

The North Talbot Road underpass bridge was constructed as a cast-in-place concrete rigid frame structure in the mid-1950s. This structure was built with spread footing foundations ranging in width from about 2.39 m to 3.35 m. The 15.5 m long footings beneath the abutments were about 2.64 m wide. The top of footing elevations were approximately 188.75 m based on the original 1955 design drawings. The approach embankments are approximately 8.5 m high above the Highway 401 grades. It has been estimated that these abutments experience an un-factored dead load of about 9,121 kN, resulting in an applied bearing pressure of about 222 kPa. A structure settlement study, carried out in 2006 (Golder, 2006), indicated that this structure has experienced settlements ranging from about 114 mm to 150 mm. There are indications that the approach embankments were typically constructed to heights of about 3 m prior to completion of the superstructure. Therefore, the magnitude of total settlement induced by the embankments preceding the superstructure construction is unknown.

Subsurface investigations carried out for the North Talbot Road underpass structure were reported on the 1955 design drawings. These investigations consisted of four “percussion test” holes and two auger boreholes to depths of about 3 m to 3.4 m with generalized soil descriptions provided on the drawings. No quantifiable data were derived from these explorations.

4.8 Other Local Relevant Construction Experience

4.8.1 Tunnel Crossings of E.C. Row Expressway, Matchette Road and 2nd Street

In 1996, a tunnel was constructed at Second Street, crossing beneath the E.C. Row Expressway, approximately 250 m east of Malden Road. The crossing was constructed at a depth of about 10.5 m below the original ground surface. Two boreholes were drilled in the area, north and south of the Expressway. The shaft was constructed as a vertical excavation with a pre-engineered support system lowered into the excavation. The area surrounding the excavation was unloaded by excavation to a depth of about 3 m prior to the excavation taking place. After about three lengths of 910 mm concrete pipe were jacked into place, squeezing of the ground was observed at the tunnel face. Samples of the soil against the bulkhead revealed low plasticity silty clay with a natural water content of about 30 per cent. It was concluded at the time that the squeezing problems encountered in the Second Street tunnel were likely due to the presence of an isolated softer zone within the silty clay till combined with unsupported excavation in front of the pipe shield resulting in movement, remoulding, and weakening of the soil. The remainder of the tunnel was completed using a tunnel boring machine.

4.8.2 Windsor Regional Hospital Western Campus, Long Term Care Facility

The Windsor Regional Hospital Long Term Care facility (Malden Park), located at 1453 Prince Road, near the western end of the planned highway extension, was constructed starting in the spring of 1993. During the months of May through August of 1993 a labour strike shut down construction work after excavations were made to planned subgrade levels. These excavations, including a utility tunnel and trenches as well as a large open foundation area, extended into both the crust and the underlying unweathered clayey silt and silty clay soils. Where these exposed soils were not protected from moisture loss, shrinkage cracking developed. Adjacent to unsupported vertical and sloped excavations, the tension cracks extended to depths close to the total excavation depth, such that the excavation sides became unstable and sections of the side walls or slopes caved into the excavations. In the relatively large and flat area of exposed unweathered soils, shrinkage cracks on the order of 20 mm to 100 mm wide opened and extended to depths on the order of 1 m to 1.5 m below the exposed surface. The horizontal spacing between these shrinkage cracks varied on the order of 1 m to 1.5 m, with the cracks forming irregular and interconnected polygons at the ground surface.

4.8.3 Other Cut Slopes in Windsor Region

The drain at Concession 2, located between Jefferson Boulevard and Lauzon Road, is generally between 3 m and 4.5 m deep. This drain was modified in 1967 and 1968 with side slopes of about 1.25 horizontal to1 vertical. Between one and two years later, surface sloughing of the side slopes was observed.

The Canadian Pacific Railway cut, between University Avenue and Riverside Drive, varies between about 3 m and 6 m deep with side slopes of 2 horizontal to1 vertical. This cut was constructed in the early 1900s. No evidence of rehabilitation or flattening could be observed at the time this report was prepared.

A review of slopes cut deep into soils of similar geologic origin and composition in Sarnia, Ontario, and Port Huron, Michigan, suggests that excavations with depths of between 10 m and 18 m with side slopes of between 1.5 horizontal to 1 vertical and 2.5 horizontal to 1 have failed repeatedly (Lo 1971, Dittrich et al. 1997). Stable slopes were achieved in Sarnia with overall slopes of about 3.5 horizontal to 1 vertical, though these included 3 horizontal to 1 vertical slopes of limited height with intermediate benches. The soils in Sarnia, though similar in geology, generally exhibit lower undrained shear strength (short-term) than the soils in Windsor, but exhibit similar drained (long-term) strength parameters. In Detroit, where the soils may be of somewhat greater strength than in parts of Windsor, cut slopes along the highways ranging in depth between 3 m and 7 m, have commonly been initially cut at 2 horizontal to 1 vertical but continued maintenance is required and some flattening of slopes or buttressing of the slope toes has occurred such that finished and stable cut slopes closer to 2.5 horizontal to 1 vertical are achieved.

5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geological Conditions

The project area is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plains. Within this region, Essex County and the southwestern part of Kent County are normally discussed as a subregion known as the Essex Clay Plain. The clay plain was deposited during the retreat of ice sheets (late Pleistocene Era) when a series of glacial lakes inundated the area. In general, the ice sheets deposited materials with a glacial-till-like gradation in the area of Windsor. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the materials may have been directly deposited at the contact between the ice sheet and the bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of the ice were deposited through the lake water (lacustrine depositional environment). Glacial till, in its common usage, often indicates a very dense or hard composition resulting from consolidation and densification under the weight of the ice sheet and the mineral soil particles typically have a distribution of grain sizes ranging from cobbles to clay. In many areas of Windsor and Detroit, however, the majority of the soils described as “glacial till” were deposited through water and have a soft to firm consistency below a “crust” that has since become stiff to hard through weathering and desiccation.

The major soil stratum in the study area, consisting primarily of silty clay and clayey silt, typically ranging in thickness from about 20 m to 35 m, exhibits a till-like structure exemplified by a random distribution of coarser particles within the primarily fine-grained silt and clay deposit (also called “diamict”). For the purposes of this report, these soils are not described as glacial till. In most of the eastern and northern parts of the Windsor metropolitan area below frost depth, the near-surface clayey soils are generally firm to hard and brown. Underlying this “crust”, the soil becomes grey-brown, and firm to stiff in consistency. Below the groundwater level, the soil becomes soft to firm, particularly in the western and southern areas of metropolitan Windsor. It is considered that this deposit is geologically slightly over-consolidated, having experienced no major overburden stresses in excess of existing stresses in the project area. The apparent preconsolidation in the crust identified by laboratory and field tests is considered to result from wetting and drying cycles, fluctuations in the groundwater level, and cementation from carbonates and other minerals from weathering processes.

Surficial layers or pockets of more typical layered lacustrine (lake-deposited) silty clay, silt, or sand may be encountered overlying the extensive stratum of “till-like” (in terms of gradation) silty clay. Silt and sand deposits, on the order of 2 m in thickness, can often be found near the ground surface in areas near the western side of Windsor. A relatively thin stratum, on the order of 1 m to 6 m in thickness, of very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface.

Above the oldest Precambrian bedrock, Southwestern Ontario is underlain by relatively flat-lying sedimentary bedrock of Paleozoic age. These sedimentary rock formations were formed in shallow marine environments within what is now geologically referred to as the Michigan Basin, a regional bowl-shaped depression with shallow relief centred on south-central Michigan. The Devonian Dundee Formation of the Hamilton Group of Formations, and the underlying Devonian Lucas Formation of the Detroit River Group of Formations, are the relevant bedrock strata for this project.

5.2 General Site Stratigraphy

A total of 42 boreholes drilled to the bedrock surface or cored into bedrock and 55 cone penetration tests (CPT) have been completed along the alignment for this project. An additional 152 boreholes have been completed along this alignment, penetrating to depths on the order of 1.5 m or less, for defining the near surface conditions relevant to pavement design and surface earthworks. Twenty-seven boreholes were drilled to total depths of between 5 and 9.6 m near the eastern end of the project for a proposed noise wall and 5 boreholes were drilled at various locations along the alignment to depths of between 8.08 and 8.23 m. Exploration locations are illustrated on Figures 5.1A to 5.1I. Baseline stratigraphic profiles are also provided on Figures 5.1A to 5.1I. During preparation of this report, data from previous explorations for this project and others as relevant were revised to be consistent with the classification system used for this project and shown on these profiles. These baseline stratigraphic profiles are considered applicable below a depth of 1.5 m within areas of existing pavements and those areas that will include future pavements as indicated on Figures 5.1A to 5.1I. Above the 1.5 m depth within these areas, Figures 5.2A to 5.2H present baseline stratigraphic conditions for the near surface materials. Subsurface constructed features including remnant foundations and operational or abandoned utilities are not illustrated on these Figures. Baseline criteria related to demolished facilities or structures are addressed elsewhere in the Contract Documents.

Figures 5.1A to 5.1I and 5.2A to 5.2H represent a simplification of the subsurface conditions and are presented to illustrate the anticipated distribution of major soil deposits beneath the site. The boundaries between major deposits and major intra-deposit changes in soil type are illustrated on Figures 5.1A to 5.1I and 5.2A to 5.2H and these represent baseline conditions for tendering purposes. Although interpreted strata boundaries are illustrated on the figures included in this report, it must be understood that actual contacts between deposits will typically be gradational as a result of natural geologic processes. Variations in the deposit boundaries and the boundaries of major intra-deposit zones from those illustrated must be anticipated both along and perpendicular to the profile lines. Therefore, designs and construction equipment and procedures must be selected to accommodate significant variations in the deposit boundaries. Where precise determination of deposit boundaries is necessary for the design, safety and stability of the works, or for other construction concerns, they should be verified by supplementary investigations and testing during design and prior to construction.

In summary, the stratigraphy at the site (based on the borehole data) consists of relatively thin surficial layers of topsoil and fill, overlying a thick deposit of clayey silt to silty clay. In some areas, this silty clay to clayey silt deposit is overlain by a deposit of fine sandy silt to silty sand on the order of 1 m to 3 m thick. These near-surface native sand and silt deposits have been grouped and labelled “Upper Granular Deposits” as a means of reference within this report. The clayey silt to silty clay deposit ranges in thickness between about 20 m and 35 m, based on the data reviewed for this report. A dense to very dense layer of silty sand and gravel is found in some areas beneath the silty clay to clayey silt deposit and immediately overlies bedrock. Stiff to hard cohesive deposits are also interbedded within these granular materials. These deposits located near the bedrock interface have been collectively labelled “Lower Granular Deposits” as a means of reference within this report. Bedrock of the Hamilton Group (Dundee Formation) or Detroit River Group (Lucas Formation) was encountered at depths ranging from about 22 m to 36 m below the ground surface.

5.3 Pavement, Topsoil, Fill, and Shallow Subsurface Conditions

As part of the investigations for this project, 84 boreholes and cores were drilled through the existing pavement structures within the project limits to determine the pavement components and to delineate the subgrade conditions at these locations. In addition, 74 boreholes were drilled along proposed new alignments outside of existing paved areas to assess topsoil thickness, fill thickness and quality as well as to assess the subgrade conditions for the new pavements. The boreholes for the pavement investigation were supplemented by the information from 59 of the boreholes drilled as part of the foundation investigation component of the project and these data are included in the following discussions. The report sections below and Figures 5.2A through 5.2H provide baseline characteristics for the existing pavements, fill, topsoil and shallow subsurface conditions. Baseline thicknesses of asphaltic concrete or Portland cement concrete pavements (if present), granular base (if present), granular subbase (if present) and buried topsoil layers are presented in a series of tables associated with various areas of the proposed construction. The baseline thicknesses provided in the tables below are to be considered 50th percentile values. For construction planning purposes, it is to be assumed that the 10th and 90th percentile thickness of the identified materials will vary by 20% below or above the 50th percentile value, respectively, in local areas unless specifically stated otherwise. Baseline subsurface conditions for the native soil deposits underlying these surficial materials and conditions deeper than 1.5 m are identified in Section 5.4, below, and on Figures 5.1A to 5.1I.

Topsoil was encountered in numerous boreholes and the thickness of the encountered topsoil layers is summarized in Figures 5.2A to 5.2H. Classification of this material was based solely on visual and textural evidence; testing of organic content, other constituents or nutrients, or its general suitability as a vegetal growth medium was not carried out. An opportunity exists for the successful proponent to selectively excavate and appropriately stockpile existing topsoil materials and reuse these materials for landscaping purposes provided that the appropriate analytical testing is carried out to confirm the suitability of these materials for the intended use.

Fill materials were encountered beneath the surficial topsoil and the thicknesses of the encountered fill layers are summarized in Figures 5.2A to 5.2H. At some of the cone penetration test locations, unidentified obstructions were encountered within the fill preventing pushing of the instrument. Pre-drilling was carried out at these locations. For some CPT locations, no samples were taken as the drilling was used only to disturb and break up the material above the start of the CPT. Other CPT locations were pre-drilled with sampling to a depth of about 3 m to identify the fill and native soil interface and the major constituents of the fill.

The fill materials encountered along the alignment are generally comprised of reworked native clayey silt to silty clay soils, sand and gravel to gravelly sand pavement granular materials or utility trench backfill materials. In addition, construction and demolition or other municipal debris (brick, concrete, asphaltic concrete, wood, glass, etc.) will be found within fill materials in some areas. It is to be assumed for baseline purposes that the fill was placed in an uncontrolled manner and will therefore exhibit great variation in both composition and engineering behaviour. Unless noted otherwise in the contract documents, it is to be assumed that fill near utilities will be found within a zone defined by a 1 horizontal to 1 vertical slope projected up to the ground surface from the invert elevation of the utility except for those specifically identified as being constructed using trenchless (tunnelling, directional drilling) methods. For baseline purposes, it is to be assumed that all existing fill will be unsuitable for reuse as engineered fill.

5.3.1 Existing Highway 401

Eleven boreholes were advanced through the Highway 401 main lanes and shoulders north and east of North Talbot Road. The boreholes were located to provide a cross section of the pavement structure in this area. Table 5.1, below, and Figure 5.2A identify the baseline thicknesses of materials anticipated for the identified lanes of the existing Highway 401 pavement structures. The main lane pavement structures were underlain by native silty clay to clayey silt deposits described in a subsequent section of this report. The buried topsoil in the westbound driving lane rounding was about 270 mm thick and was underlain by native silty clay to clayey silt deposits. Layers of buried topsoil were also encountered beneath the clayey fill materials in the boreholes, except in the eastbound driving lane shoulder. The topsoil layers were encountered at about 1.1 m to 1.2 m depth and were 150 mm to 300 mm thick. For baseline purposes, it is to be assumed that the 50th percentile thickness of buried topsoil is 225 mm and it will be found beneath the clayey fill materials in all shoulders at a depth of 1.1 m, except the eastbound driving lane shoulder. The baseline width of the existing filled subexcavation is 5 m.

Table 5.1: Baseline Pavement and Shallow Subsurface Conditions, Existing Highway 401 (main lanes) Station 10+900 to 11+300

	BASELINE 50th PERCENTILE THICKNESS (mm)			
	Westbound Lanes		Eastbound Lanes	
COMPONENT	Driving Lane	Passing Lane	Passing Lane	Driving Lane
Asphalt	170	170	140	170
Concrete	280	280	230	235
Granular Base	-	50	-	-
Subbase/Sand Fill	245	-	180	145
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Table 5.2: Baseline Pavement and Shallow Subsurface Conditions, Existing Highway 401 (shoulders) Station 10+900 to 11+300

	BASELINE 50th PERCENTILE THICKNESS (mm)						
	Westbound Lanes			Eastbound Lanes			
	Driving Lane		Passing Lane	Passing Lane	Driving Lane		
COMPONENT	Rounding	Shoulder	Edge of Pav't	Shoulder	Edge of Pav't	Shoulder	Rounding
Asphalt	-	140	30	-	565	145	-
Granular Base	30	100	170	300	235	155	300
Subbase/Sand Fill	-	360	150	-	-	400	400
Subgrade	Topsoil	Clayey Fill	Clayey Silt to Silty Clay Deposit	Clayey Fill	Clayey Silt to Silty Clay Deposit	Clayey Fill	Clayey Fill

5.3.2 Existing Highway 3/Talbot Road

Borehole and pavement cores were drilled to provide eight sections on Highway 3/Talbot Road between Highway 401 and Huron Church Road for a total of 44 boreholes and cores. Table 5.3 and Figures 5.2B and 5.2C provide the investigation results and the baseline thicknesses (50th percentile) of pavement materials anticipated for the identified lanes of the existing Highway 3/Talbot Road pavement structures. Buried topsoil was encountered beneath the pavement structure and/or fill at one location in the eastbound driving lane and three locations in the eastbound passing lane. The topsoil was encountered at about 0.7 m depth and was about 250 mm thick in the driving lane. In the passing lane, the topsoil was encountered at about 0.7 to 0.9 m depth and was about 200 mm to 350 mm thick. For baseline purposes, it is to be anticipated that 40 per cent of the existing Highway 3/Talbot Road pavement structures and fill will be underlain by 250 mm (50th percentile thickness) of buried topsoil.

Table 5.3: Baseline Pavement and Shallow Subsurface Conditions, Existing Highway 3/Talbot Road Station 10+000 at Highway 401 to 21+550 at Huron Church Line

	BASELINE 50th PERCENTILE THICKNESS (mm)			
	Westbound Lanes		Eastbound Lanes	
COMPONENT	Driving Lane	Passing Lane	Passing Lane	Driving Lane
Asphalt	300	375	285	265
Concrete	210	205	190	215
Subgrade	Upper Granular Deposits/Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Additional details of the 50th percentile baseline thicknesses, as defined by the indicated depths to the material boundaries, for the various materials encountered, including granular fill, other fill materials and buried topsoil, are provided on Figures 5.2B and Figure 5.2C.

For paved shoulders, the baseline asphalt thickness is to be assumed equal to 200 mm (50th percentile) in all areas except the westbound speed change lane east of Howard Avenue and, in this case, the baseline 50th percentile thickness is to be assumed equal to 530 mm. Additional baseline information is provided on Figures 5.2B and 5.2C.

5.3.3 Existing Huron Church Road

Boreholes were advanced to provide four sections through the travelled lanes on Huron Church Road for a total of 12 boreholes. Table 5.4 and Figure 5.2D provide baseline thicknesses of pavement materials anticipated for the identified lanes of the existing Huron Church Road. Layers of fill, native upper granular deposits, and native silty clay to clayey silt deposits were encountered beneath the pavement structure as identified on Figure 5.2D. A single borehole was advanced in the southbound left turn lane adjacent to the turnaround north of Cabana Road. This borehole encountered 280 mm of concrete pavement overlying about 560 mm of granular base on a silty clay to clayey silt subgrade.

Table 5.4: Baseline Pavement and Shallow Subsurface Conditions, Existing Huron Church Road

	BASELINE 50th PERCENTILE THICKNESS (mm)			
	Northbound Lanes		Southbound Lanes	
COMPONENT	Driving Lane	Passing Lane	Passing Lane	Driving Lane
Station 10+060 to 10+150 Eastbound and to Station 10+200 Westbound				
Asphalt	-	260	-	345
Concrete	265	275	300	275
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit
Station 10+150 Eastbound and Station 10+200 Westbound to Station 21+550				
Concrete	265	275	300	275
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Additional information regarding baseline depths for other materials encountered is provided in Figure 5.2D.

5.3.4 Existing Crossing Roads

Boreholes and cores were advanced on several of the existing roads crossing Highway 3/Talbot Road, Huron Church Road and the E.C. Row Expressway. Pavement and shallow subsurface conditions are provided in the tables below and these are to be considered the baseline conditions within the project limits for these crossing roads.

Table 5.5: Baseline Pavement and Shallow Subsurface Conditions, Existing Outer Drive

	BASELINE 50th PERCENTILE THICKNESS (mm)	
COMPONENT	**Southbound Lane**	**Southbound Shoulder**
Asphalt	20	20
Granular Base	380	335
Clayey Fill with organics	310	405
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Table 5.6: Baseline Pavement and Shallow Subsurface Conditions, Existing Highway 3 West of Highway 401

	BASELINE 50th PERCENTILE THICKNESS (m)			
	West of Highway 401		**East of Highway 401**	
COMPONENT	**Main Lanes**	**Shoulders**	**Main Lanes**	**Shoulders**
Asphalt	0.15	-	0.15	-
Concrete	0.2	-	0.2	-
Topsoil	-	0.1	-	0.1
Subbase/ Sand & Gravel Fill	0.2	0.1	0.2	0.1
Clayey Fill	4.3	4.3	4.2	4.2
Buried Topsoil	0.2	0.2	0.2	0.2
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Table 5.7: Baseline Pavement and Shallow Subsurface Conditions, Existing Howard Avenue

COMPONENT	BASELINE 50th PERCENTILE THICKNESS (mm)
Asphalt	245
Granular Base	195
Concrete	260
Subbase/Sand & Gravel Fill	180
Topsoil	230
Subgrade	Clayey Silt to Silty Clay Deposit

Table 5.8: Baseline Pavement and Shallow Subsurface Conditions, Existing Todd Lane and Existing Cabana Road

	BASELINE 50th PERCENTILE THICKNESS (mm)	
COMPONENT	Todd Lane	Cabana Road
Asphalt	135	100
Granular Base	525	460
Subbase/Sand & Gravel Fill	180	140
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Table 5.9: Baseline Pavement and Shallow Subsurface Conditions, Existing Pulford Street

	BASELINE 50th PERCENTILE THICKNESS (mm)	
COMPONENT	East of Huron Church Road	West of Huron Church Road
Concrete	280	230
Granular Base	630	330
Subbase/Sand Fill	460	660
Subgrade	Clayey Silt to Silty Clay Deposit	Clayey Silt to Silty Clay Deposit

Table 5.10: Baseline Pavement and Shallow Subsurface Conditions, Existing Bethlehem Avenue and Existing Labelle Street

BETHLEHEM AVENUE		LABELLE STREET	
Component	**Baseline 50th Percentile Thickness (mm)**	**Component**	**Baseline 50th Percentile Thickness (mm)**
Concrete	215	Asphalt	105
Granular Base	625	Granular Base	455
Subgrade	Clayey Silt to Silty Clay Deposit	Asphalt	170
		Granular Base	130
		Subgrade	Clayey Silt to Silty Clay Deposit

Table 5.11: Baseline Pavement and Shallow Subsurface Conditions, Existing Malden Road

	BASELINE 50th PERCENTILE THICKNESS (mm)			
	Southbound		Northbound	
COMPONENT	**Shoulder**	**Lane**	**Lane**	**Shoulder**
Topsoil	-	-	-	25
Asphalt	-	150	130	-
Granular Base	560	840	890	355
Subbase/Sand & Gravel Fill	250	-	-	-
Buried Topsoil	-	-	-	380
Subgrade	Upper Granular Deposit	Upper Granular Deposit	Upper Granular Deposit	Upper Granular Deposit

5.3.5 Windsor-Essex Parkway Alignments Outside of Existing Road Pavements

The following sections provide a summary of the shallow subsurface conditions encountered along the new alignments within the project limits but outside of the paved areas described in the report sections and tables above.

Proposed Highway 3 Realignment, Windsor Essex Parkway, and Outer Drive Realignment

Thirty eight boreholes were drilled in the area south of existing Highway 3 from approximately 150 m southeast of Outer Drive to approximately 240 m east of Howard Avenue, within the general area of the proposed Highway 401, Highway 3, and Outer Drive interchange (Figure 5.1I). Surficial topsoil was encountered at all of the borehole locations. The surficial topsoil thickness ranged between 200 mm to 380 mm at the borehole locations. Soils of the Upper Granular Deposits were encountered beneath the topsoil at two locations and ranged from about 300 mm to about 340 mm thick. Beneath the topsoil and Upper Granular Deposits, all of the boreholes encountered and were terminated in the native Silty Clay to Clayey Silt Deposit. Figure 5.2E provides the baseline shallow subsurface conditions for the Highway 3 and Outer Drive realignments. For baseline purposes, the surficial topsoil thickness (50th percentile) in this area is to be assumed to be 300 mm.

Proposed Windsor-Essex Parkway and Service Roads, Highway 3/Talbot Road Corridor

The shallow subsurface conditions along proposed Windsor-Essex Parkway and service roads in the present Highway 3/Talbot Road corridor alignment were investigated with 25 boreholes south of the existing Highway 3/Talbot Road and the results are indicated on Figure 5.2F. Surficial topsoil was encountered at the ground surface at 18 of these locations. The surficial topsoil ranged from 80 mm to 610 mm thick at these locations. Buried topsoil was encountered beneath fill at one location. The buried topsoil was encountered at about 300 mm depth and was about 600 mm thick. Fill materials were encountered at the ground surface at the remaining six locations. The fill materials consisted of granular materials mixed with varying amounts of topsoil or clayey materials. The fill materials ranged from about 200 mm to greater than 1.5 m thick with an average thickness of about 600 mm. Fill materials were also encountered beneath the surficial topsoil at four locations. The fill in these areas ranged from about 500 mm to 1.4 m thick with an average thickness of about 1.0 m. One borehole encountered silty sand at the ground surface to a depth of 1.4 m. Native upper granular deposits about 300 mm thick were encountered beneath the surficial topsoil at one location. Twenty two boreholes encountered the native silty clay to clayey silt deposits beneath the surficial layers. For baseline purposes, it is to be assumed that the topsoil thickness (50th percentile) through the new alignment in areas not occupied by pavements or built structures is 300 mm. The baseline existing fill thickness, including buried topsoil is to be assumed to be to 600 mm (50th percentile) over a total of 30 per cent of the area to be occupied by the new roadway and associated works. For construction planning purposes, it is to further be assumed that the existing fill thickness will vary by -75% to +150% of the 600 mm baseline 50th percentile thickness in localized areas.

Proposed Windsor-Essex Parkway and Service Roads, Huron Church Road Corridor

Seventeen boreholes were drilled immediately west of the existing Huron Church Road. The conditions encountered in these boreholes are summarized on Figure 5.2G and were variable but generally consisted of surficial topsoil and fill underlain by the native upper granular deposits and silty clay to clayey silt deposits. Surficial topsoil was encountered in eight of these boreholes. The surficial topsoil ranged from 120 mm to 1.4 m thick at these locations with an average thickness of about 485 mm. Buried topsoil was encountered beneath about 460 mm of sandy fill in one borehole. The buried topsoil was about 230 mm thick at the borehole location. Variable fill materials consisting of granular materials, sands and silty sands were encountered in eight boreholes. The total fill thickness in these boreholes ranged from about 300 mm to 1.2 m with an average thickness of about 705 mm. For baseline purposes, the surficial topsoil thickness (50th percentile) is to be assumed equal to 500 mm. The baseline existing fill depth including buried topsoil is to be assumed equal to 800 mm (50th percentile) throughout this section of the alignment.

Proposed Windsor-Essex Parkway Adjacent to E.C. Row Expressway

Forty seven boreholes were drilled south of the existing E.C. Row Expressway in this section of the Windsor-Essex Parkway. The conditions encountered in these boreholes were variable and are illustrated on Figure 5.2H. In general, the boreholes encountered surficial topsoil and fill at ground surface overlying the native upper granular deposits and silty clay to clayey silt deposits. Surficial topsoil was encountered in 44 of these boreholes. The topsoil thickness ranged between 75 to 910 mm. Buried topsoil was encountered beneath fill materials at three locations. The buried topsoil was encountered at depths of 200 mm to 500 mm and ranged from 150 mm to 700 mm thick. Variable fill materials, ranging from predominantly topsoil to predominantly a material resembling granular base were encountered at five locations. The fill materials were 100 mm to 760 mm thick. For baseline purposes, the average surficial topsoil thickness is to be assumed equal to 350 mm (50^{th} percentile), and the existing fill depth, including buried topsoil, is to be assumed equal to 500 mm (50^{th} percentile) for 20 per cent of the alignment.

Proposed Windsor-Essex Parkway Alignment at Proposed Ojibway Parkway Interchange

Five boreholes were drilled in the area south of the existing E.C. Row Expressway and east of Ojibway Parkway. Surficial topsoil was encountered in four of these boreholes and a thin layer of pavement granular materials was encountered in one borehole. The topsoil thicknesses were variable and ranged from about 230 mm to 810 mm. The existing pavement granular materials at the one borehole location were 80 mm thick. The surficial layers were underlain by sands, silts and sand and gravel materials. One borehole encountered clayey silt to silty clay. For baseline purposes, the topsoil thickness (50^{th} percentile) in this area is to be assumed equal to 500 mm.

5.4 Native Soil Stratigraphy

This section of the report provides baseline soil classification parameters to be used for design of temporary and permanent works and for selection of equipment and construction methods as required. Within this section, baseline values are provided consistent with the 10^{th}, 50^{th}, and 90^{th} percentiles. The baseline 10^{th}, 50^{th}, and 90^{th} percentile values are provided as a means for quantitatively describing the statistical distribution of the values. It should be noted that the values provided in the tables within this section represent statistical characterization of the classification criteria and cannot necessarily be considered in combination. For example, the 90^{th} percentile values related to the per cent (by weight) composition of a soil for the gravel, sand, silt and clay fractions will not necessarily add to 100 per cent. Likewise, the difference between the 50^{th} percentile values for liquid and plastic limits will not necessarily be equal to the 50^{th} percentile plasticity index. While the baseline 50^{th} percentile values can be used for some design purposes, if the designs are sensitive to minimum or maximum values, then the range must be accounted for based on the successful proponent’s level of acceptable risk. Likewise, with respect to selection of equipment and methods, the range of properties must also be considered as variability in physical properties is intrinsic to the nature of earth materials.

5.4.1 Upper Granular Deposits

Silty sand to sandy silt was encountered at the location of 24 boreholes completed for this project (not including pavement boreholes) to depths of as much as about 2.4 m, with the majority of these located along the alignment west of Cabana Road. In some instances, classification of this material was based only on auger cuttings and visual and textural evidence. A summary of the grain size distribution determinations is provided on Figure 5.3; however, it is noted that gravel sizes larger than about 40 mm maximum dimension were not recovered by the sampling methods used. Therefore, Figure 5.3 and the table below are considered representative of the fraction of the deposit smaller than 25 mm in maximum dimension. The thickness of the Upper Granular Deposits ranged between about 0.2 m and 2.0 m and exhibited Standard Penetration Test (SPT) “N” values between 4 and 19 blows per 0.3 m penetration, with a typical value of about 9 blows per 0.3 m penetration, indicating a loose to compact relative density. Measured CPT tip resistance values within the Upper Granular Deposits were typically about 2 megapscals (MPa) to 3 MPa also indicating a loose to compact relative density.

Table 5.12: Baseline Classification Characteristics – Upper Granular Deposits

Parameter	10th Percentile	50th Percentile	90th Percentile
Natural Water Content (%)	11	13	15
Unit Weight, γ (kN/m^3) [1]	19	21	23
Gravel (%)[2]	2	5	14
Sand (%)	5	40	55
Silt (%)	40	50	80
Clay (%)[3]	7	15	23
Percent Passing Standard 75 µm Sieve	45	55	97

Notes:
1. Values based on saturated water content.
2. The samplers used in the geotechnical investigations limit the maximum particle size that can be sampled to about 40 mm and larger particles are known to exist in the deposit as described in the text of this report.
3. Percentage Clay as noted above represents clay-size fraction (less than or equal to 2 µm) of the sample (i.e., “rock-flour” particles as well as clay minerals) and does not necessarily represent the fraction of clay minerals.

5.4.2 Clayey Silt to Silty Clay Deposit

A thick deposit of clayey silt to silty clay was found in all boreholes completed for this project that penetrated deeper than 2 m. Boreholes and CPT test results indicated that seams or interbeds of silty sand to sandy silt are embedded within the clayey silt to silty clay deposit. The subsurface data indicate that these seams or interbeds typically range in between 0.1 m and 1.5 m thick above Elevation 155 m. These interbeds are not described in further detail and, for baseline purposes, the classification characteristics of the interbeds are to be assumed identical to those described in Section 5.4.1, above.

The clayey silt to silty clay deposit is generally mottled grey and brown within and near the frost-depth (upper 1.2 m to 2 m), brown below this level, and grey below the static groundwater level. The upper mottled zone and brown zone, and a transition zone within the grey portion of the deposit represent a “crust” in which weathering processes during and following deposition have resulted in this material being generally stronger than the underlying deposit. For baseline purposes, the thickness of the “crust” was estimated using the average of:

- the depth to the interface between the brown and mottled deposits and the underlying grey materials; and
- the depth at which the uncorrected piezocone penetration tip resistance, q_c, profiles exhibited a marked change in the pattern of decreasing q_c with increasing depth, typically exhibited at a value of q_c equal to between 1.3 MPa and 1.5 MPa.

In general, the crust thickness is on the order of 4 m to 6 m near the eastern end of the project, and decreases to near 2 m near the western end of the project. A baseline profile of the average crust thickness is illustrated on Figures 5.1A to 5.1I. This profile represents the average crust thickness and for baseline, design, and planning purposes the crust thickness is to be considered to be greater than and less than the indicated profile by about 1 m.

Weathering processes including seasonal freezing, drying and wetting, have produced natural fissures within the clayey silt to silty clay crust (Hanna 1966, Soderman and Kim 1970, Quigley and Ogunbadejo 1974 and 1976, Dittrich 2000, Lo and Hinchberger 2006). Exposures in the region shown that the fissures form irregular polygons in plan view, ranging in shape from nearly square to roughly octagonal, with the largest horizontal dimensions between fissures ranging from about 0.2 to 1 m (see Figure 5.5). For baseline purposes, vertical fissures within the crust are to be anticipated at the horizontal spacing intervals noted below. Between these depth intervals, the natural fissure spacing can be assumed to be linearly transitional. Vertical spacing of horizontal fissures can be assumed to be equal to the spacing of vertical fissures, such that block-like structures of irregular plan shape are formed with vertical and horizontal aspect (largest dimension divided by smallest dimension) on the order of 1 to 3.

Table 5.13: Baseline Fissure Characteristics in Weathered Crust

Depth Below Ground Surface (m)	Spacing Between Natural Vertical Fissures (m)
1	0.02±0.010
2	0.05±0.025
3	0.10±0.05
4	0.30±0.15
5	1.0±0.5

Evidence of root penetration to depths on the order of 1 m to 3 m is common, though root penetration has been evident to unusual depths of 7 m to 9 m (Hanna 1966). Weathering and root penetration have been shown to affect the overall hydraulic conductivity and strength properties of this soil mass - the baseline geotechnical engineering properties are discussed in Section 6.0 of this report.

In general, the deposit consists mainly of low plasticity clayey silt to intermediate plasticity silty clay. The measured clay-size particle content of this deposit ranged between about 25 and 70 percent (by weight). Clay-size particles are predominantly illite, though swelling minerals of the smectite group and chlorite compose up to 15 per cent of the clay-size fraction in the weathered crust. Within the unweathered deposit, swelling minerals represent 2 per cent or less of the clay-size fraction (Quigley and Ogunbadejo, 1974, 1976). Total carbonate content ranged between about 19 and 32 percent in the unweathered materials. Carbonate content in the weathered crust is expected to vary between 0 and 60 per cent as the carbonate leaches from the near-surface materials and is redeposited through downward groundwater flow in zones near the crust and unweathered soil boundary.

Gravel sized particles constituted between about 0 and 5 percent (by weight) of the tested materials. A summary of grain size distribution data for this deposit is provided on Figure 5.4. Results of Atterberg Limits determinations are summarized and illustrated on Figure 5.6. The plasticity index ranged between less than 5 and 31 per cent with an average of about 16 per cent for the entire deposit. The natural water content measured on selected samples of this deposit ranged between about 10 and 30 percent but was typically between 20 and 25 percent. The higher water contents are typically associated with the middle portion of the deposit. Table 5.14, below, summarizes the baseline classification characteristics of this deposit overall. However, because spatial variation of the water content in this deposit will be important for design and construction, Figures 5.7A to 5.7I present baseline water content profiles for geographic locations along the project alignment. Where the geographic sections abut, the baseline water content profile is to be taken as the average of the two adjacent profiles. These profiles are considered representative of the 50th percentile values below the crust boundary as defined above, with the 10th and 90th percentile values taken as minus or plus a water content of 5 per cent from these given profiles, respectively. Above the crust boundary, the baseline water content values are provided in the table below.

Within the soft to hard mottled brown and grey soils, the SPT "N" values ranged between about 4 and 36 blows per 0.3 m of penetration. The soft to hard brown clayey silt and silty clay exhibited SPT "N" values from about 4 to 59 blows per 0.3 m of penetration. Standard Penetration Test "N" values typically ranged between about 1 and 32 blows per 0.3 m of penetration in the grey silty clay below the groundwater level. In numerous boreholes the deposits became very stiff to hard near the bedrock surface at depths ranging from about 17.7 m to about 32.7 m. Within the very stiff to hard lower part of the silty clay deposits, SPT "N" values ranged between 15 and 71 blows per 0.3 m of penetration.

The unweathered soils of the Silt and Clay Deposits are characterised as a low-sensitivity (undisturbed divided by remoulded field vane shear strength) materials with an average sensitivity of 2.6. Minimum and maximum sensitivity values ranged from 1.0 to about 8.2, with only three values above 5.0.

Table 5.14: Baseline Classification Characteristics - Clayey Silt to Silty Clay Deposit

Parameter	10th Percentile	50th Percentile	90th Percentile
Natural Water Content, Brown and Mottled Brown/Grey Soils ("Crust") (%)	12	17	24
Natural Water Content Grey Soils Below Static Water Level (%)	13	21	28
Liquid Limit (%)	23	29	37
Plastic Limit (%)	13	15	18
Plasticity Index	9	14	21
Unit Weight, γ (kN/m^3) [1]	19.5	20.5	22
Gravel (%)	0	3	6
Sand (%)	12	29	33
Silt (%)	35	40	53
Clay (%)[2]	23	38	67

Notes:
1. Values based on saturated water content measurements and measured density of solids.
2. The samplers used in the geotechnical investigations limit the maximum particle size that can be sampled to about 40 mm and larger particles are known to exist in the deposit as described in the text of this report.
3. Percentage Clay as noted above represents clay-size fraction (less than or equal to 2 μm) of the sample (i.e., "rock-flour" particles as well as clay minerals) and does not necessarily represent the fraction of clay minerals.

5.4.3 Lower Granular Deposits

Deposits of loose to very dense silt, sandy silt, silty sand, silty sand and gravel, and sand and gravel were encountered beneath the silty clay to clayey silt in multiple boreholes along the alignment. This deposit typically exhibited "N" values of between 22 blows per 0.3 m penetration and more than 100 blows per 0.3 m penetration, though lower "N" values were recorded in some localized areas with these low values considered to reflect disturbance during drilling and sampling. A summary of grain size distribution data is presented in Figure 5.8, although it is noted that gravel larger than about 40 mm maximum dimension was not recovered by the sampling methods used. Therefore, Figure 5.8 and the table below are considered representative of the fraction of the deposit smaller than 25 mm in maximum dimension. This deposit also includes zones or interbeds of clayey silt and silty clay, similar in composition to the overlying Silt and Clay Deposits. These materials are considered representative of the complex depositional environment near the contact between glacial ice and the bedrock. The consistency of these interbeds typically varies from firm to hard. The baseline classification parameters provided below are representative only of the granular fraction of these deposits. The thickness of this deposit, where present, varied up to about 10 m as illustrated on Figures 5.1A to 5.1I.

Table 5.15: Baseline Classification Characteristics – Lower Granular Deposits (Granular Fraction)

Parameter	10^{th} Percentile	50^{th} Percentile	90^{th} Percentile
Natural Water Content (%)	8	14	22
Unit Weight, γ (kN/m^3)[1]	20	22	24
Gravel (%)	0	16	39
Sand (%)	8	38	74
Silt (%)	8	30	81
Clay (%)[2]	4	7	9
Percent Passing 75 µm Sieve	14	35	88

Notes: 1. Values based on saturated water content.

2. Percentage Clay as noted above represents clay-size fraction (less than or equal to 2 µm) of sample and does not necessarily represent fraction of clay minerals (i.e., “rock-flour” particles as well as clay minerals).

5.4.4 Bedrock

Limestone and dolostone bedrock of the Hamilton Group (Dundee Formation) or Detroit River Group (Lucas Formation) were encountered in all boreholes that included rock coring for this project at depths as identified on Figures 5.1A to 5.1I. Based on the cores recovered from the boreholes, this project is in an area characterised by a transition in bedrock formations at the bedrock surface. Such transitions in the bedrock formations encountered at the rock-soil interface may be expected in the general vicinity based on available mapping. In some boreholes, the rock encountered consisted of a light grey limestone and in other boreholes the bedrock was composed of brown dolostone. In some boreholes, both rock types were encountered. Some portions of the rock exhibited a hydrocarbon odour. For baseline purposes, it is considered that the hydrocarbon odour is from natural sources since these formations are known to contain natural bitumen. The rock encountered ranged from slightly weathered to fresh.

5.5 Groundwater Conditions

The Essex Region/Chatham-Kent (ECK) Regional Groundwater Study (Dillon and Golder 2004), states that groundwater is not widely utilized for public water supply within the study region. Further, it is anticipated that within the Windsor metropolitan area, groundwater is not used for public water supplies and is at most used on limited basis for private water supplies. Based on the MOE water well database, there are eight mapped wells in the immediate vicinity of the proposed project between Highway 401 and the plaza location near Ojibway Parkway.

Measured groundwater levels indicate that in the eastern part of the project area, near Howard Avenue and North Talbot Road, the groundwater exhibits a downward pressure gradient. This condition is consistent with the generally low-permeability clayey silt to silty clay soils that will inhibit downward seepage of water from the ground surface to the static groundwater level. The upper soils within the "crust" are fissured and of higher mass permeability than the native soils below the groundwater level. Within this weathered crust, there will be transitions in soil saturation from near-surface soils that become wetted with stormwater, down through the fissured, unsaturated soils (that exhibit mottled colouring), to the fully saturated soils below (grey in colour). Near-surface clayey silt and silty clay soils will tend to pool storm water in local surface depressions. Within the overburden soil, static groundwater levels were measured near Elevations 179.5 m near Ojibway Parkway to about 184.5 m near Howard Avenue and North Talbot Road. In these same areas, however, measured groundwater levels within the bedrock were close to about Elevation 180.5 m to 177.5 m, respectively. There is a trend of increasing piezometric water levels within the bedrock from south and east to north and west, opposite the trend indicated for piezometers within the overburden. Near the western end of the project, flowing artesian conditions were encountered indicating upward hydraulic gradients through the overburden. Two baseline groundwater pressure elevation lines are illustrated on Figure 5.1A to 5.1I, showing the conditions expected near the top of the saturated soils (i.e. within the top 10 m) and near the soil/bedrock interface. Figure 5.9 schematically illustrates the upward and downward gradients that such conditions cause.

A suite of analytical tests were carried out on water samples obtained from the groundwater observation wells installed in Boreholes BH-1 through BH-160. These analytical tests were completed solely for the purpose of identifying the concentration magnitude of a selected group of minerals and chemicals that may be found in the local natural groundwater. This testing was not carried out for the purposes of identifying man-made chemicals that may or may not have affected soil and groundwater chemistry from past discharges to the environment. Baseline values for the remaining groundwater parameters are provided below that consider both the analytical results of testing carried out for this project as well as published values.

Table 5.16: Baseline Natural Groundwater Chemistry Characteristics

Parameter	10^{th} Percentile	50^{th} Percentile	90^{th} Percentile
Hardness as C_aCO_3 (ppm)	110	700	2500
Calcium (ppm)	30	150	500
Magnesium (ppm)	5	90	270
Sulphate (ppm)	20	500	1900
Iron (ppm)	0.08	1.75	5.5
Total Dissolved Solids	690	2300	3500
pH	6.4	7.4	7.9
Conductivity (mS/cm)	0.2	1.7	4.5

5.6 Subsurface Gases

The groundwater in the project area contains dissolved hydrogen sulphide (H_2S) that is liberated from the water on exposure to atmospheric pressure. Hydrogen sulphide gas was noted by its characteristic "rotten egg" odour during drilling of Boreholes BH-23 and BH-160 when bedrock and flowing artesian water pressures were encountered. Hydrogen sulphide gas can frequently be detected by smell at concentrations on the order of 0.5 parts per million (ppm) and can be corrosive at concentrations of about 2 ppm to 3 ppm (Powers et al. 2007) as measured in the groundwater. During drilling for investigations between Highway 401 and Ojibway Parkway, H_2S concentrations in the air surrounding the boreholes did not exceed the health and safety trigger levels of personnel monitoring equipment set to alarm at 10 ppm atmospheric concentrations. Other investigations carried out near Ojibway Parkway and Sandwich Street encountered hydrogen sulphide gas during and immediately following drilling to install 340 mm diameter steel casings through the overburden to the bedrock surface and during later deep drilling to explore bedrock mass characteristics. The hydrogen sulphide gas concentrations were sufficient to trigger personnel health and safety monitoring equipment on several occasions.

Hydrogen sulphide concentrations measured in 28 water samples taken from the observation wells and boreholes completed for this project (Boreholes BH-1 through BH-160) ranged from a minimum value less than detection limits to a maximum value of 238 ppm. For samples in which H_2S was detected, excluding the maximum value from Borehole BH-160 and non-detection values, the maximum and minimum values were 5.54 ppm and 0.03 ppm, respectively. No trends in the data were observable with respect to the geographic observation well locations. It is considered that the presence, absence, or concentration of H_2S were and will be directly related to the variability in the local bedrock composition (including the presence of natural petroleum hydrocarbons), flow of groundwater and gasses through the bedrock fracture systems, and whether or not investigation drilling or future construction activities intersect these fracture systems. The concentration in air of H_2S released to the atmosphere will be dependent upon the local geologic and groundwater conditions as well as construction and subsurface gas management methods. Therefore, the design and construction must account for the presence of hydrogen sulphide up to the maximum concentration encountered.

Dissolved methane, CH_4, was also detected within the groundwater. Dissolved methane concentrations in the water ranged from less than 5 parts per billion (ppb) to a maximum measured value of 485 ppb. No trends in the data were observable with respect to the geographic observation well locations. It is considered that the presence, absence, or concentration of methane in the groundwater were and will be directly related to the variability in the local bedrock composition (including the presence of natural petroleum hydrocarbons), flow of groundwater and gasses through the bedrock fracture systems, and whether investigation drilling or future construction activities intersect these fracture systems. The concentration in air of methane released to the atmosphere will be dependent upon the local geologic and groundwater conditions as well as construction and subsurface gas management methods. Therefore, the design and construction must account for the presence of methane up to the maximum concentration encountered.

Methane will form an explosive mixture with air while hydrogen sulphide is toxic. These gasses are a potential hazard for deep excavation and construction work. Based on the geologic information for the area and the test results obtained from the recent investigations, it has been interpreted that these gasses originate from bacterial action and naturally occurring substances within the bedrock and the groundwater within bedrock and close to the bedrock surface. The Windsor vicinity is characteristically underlain by a relatively thick deposit of clayey silt and silty clay as discussed in Section 5.2 of this report. These materials will tend to trap subsurface gasses in underlying zones of granular soil or within the underlying bedrock. It is anticipated that construction within the top 10 m of the overburden soil deposits will not encounter such gasses. For baseline purposes it is, however, anticipated that construction (excavation, dewatering or depressurization wells, or drilling) that penetrates deeper and into isolated or continuous zones of granular materials (silt, sand and gravel) or bedrock will encounter groundwater that includes dissolved hydrogen sulphide and methane gasses. The current absence of gas in a particular area is not to be construed to indicate that there is no risk of its presence in the future. Changes in groundwater pressure that may be caused by dewatering or seepage into underground spaces can lead to migration of gaseous or dissolved methane or hydrogen sulphide. Therefore, air monitoring and adequate ventilation will be required during construction.

5.7 Boulders and Other Obstructions

All the deposits through which construction (including drilling, pile driving, excavation, etc.) will be completed are glacially derived and therefore will contain cobbles and boulders. During exploratory drilling for the Windsor-Essex Parkway, cobbles and boulders were inferred by coring or difficult drilling behaviour in boreholes BH-112, BH-116, BH-119, and BH-127. A boulder was encountered and cored in borehole BH-158. Cobbles are defined as particles that cannot pass through a screen with 75 mm square openings and less than 300 mm in maximum dimension. Boulders are defined as particles with their maximum dimension being equal to or greater than 300 mm.

Based on experience elsewhere in Ontario, a convenient means to quantify the potential for encountering boulders can be defined based on the volume of material to be directly excavated or encountered during construction. The total volume of all boulders, V_{bT}, can be calculated as the sum of individual boulder volumes (i.e. $V_{bT} = \Sigma V_b$) and then this volume can be compared to the volume of earth material involved in the construction of drilled piles, diaphragm walls, drilling, tunnelling, pile driving, or mass excavation. The ratio of the total volume of all boulders to the total volume of a particular deposit that is involved in construction is then defined as the boulder volume ratio (BVR). In addition, the number of boulders per unit cubic metre of rock (V_{bT}) can be identified through a Boulder Number Ratio (BNR) to assist in providing an estimate of the number of boulder obstructions that could be encountered.

It is known that boulders have been encountered in Windsor in both the Clayey Silt to Silty Clay Deposit as well as the underlying Lower Granular Deposits (silty sand to sand and gravel) encountered near the bedrock interface. Based on boulder data from work completed elsewhere and published data from the Detroit, Michigan region for deposits of similar geologic origin, baseline BVR, BNR, and boulder size distribution data have been developed as summarized in Table 5.17, below.

Table 5.17: Baseline Numbers and Sizes of Boulders

Parameter	Clayey Silt to Silty Clay Deposit	Lower Granular Deposits
300 mm ≤ Maximum Diameter ≤ 1.0 m		
Boulder Volume Ratio (BVR)	0.13%	0.60%
Boulder Number Ratio (BNR)	14.3	14.3
1.0 m < Maximum Diameter		
Boulder Volume Ratio (BVR)	0.04%	0.10%
Boulder Number Ratio (BNR)	1.8	1.8

For baseline purposes, where these calculations result in a fractional number of boulders that may be encountered, the number is to be rounded to the nearest integer. Furthermore, while boulders may protrude into the neat volume of the construction (e.g., within the dimensions defined by the outside diameter of drilled holes, or outside dimensions of driven piles), the neat volume calculated based upon the construction dimensions is to be utilized for the purposes of estimating the baseline number of boulders to be encountered. It is to be assumed for baseline purposes that the boulders will be composed of dolomitic limestone or dolostone with engineering properties consistent with those provided in Section 6.5.1 of this report.

For example, if 1,000 H-piles with outside dimensions of 300 mm by 300 mm are to be each driven 30 m through the Clayey Silt to Silty Clay Deposit, a total of 2,700 m^3 of earth will be directly in the path of the pile driving. Therefore, a total of approximately 3.5 m^3 of boulders measuring 300 mm to 1.0 m can be assumed to be encountered, resulting in a total of 50 boulders of this size hit by the pile driving. For this same example, a total of 1.08 m^3 of boulders measuring 1 m or greater can be assumed to be encountered, resulting in 2 boulders of this size hit during pile driving.

Other subsurface obstructions associated with former structures or utilities will be encountered during construction. These are not identified in this Subsurface Conditions Baseline Report and are identified elsewhere in the Contract Documents.

6.0 BASELINE GEOTECHNICAL ENGINEERING PARAMETERS

This section of the report provides baseline geotechnical engineering parameters to be used for design of temporary and permanent works and for selection of equipment and construction methods, as required. Within this section, baseline values are provided consistent with the 10th, 50th, and 90th percentiles. The baseline 10th, 50th, and 90th percentile values are provided as a means for quantitatively describing the statistical distribution of the parameter values. While the baseline 50th percentile value can be used for some design purposes, if the designs are sensitive to minimum or maximum values, then the range must be taken into account. Likewise, with respect to selection of equipment and methods, the range of properties must also be considered as variability in physical properties is intrinsic to the nature of earth materials. Baseline classification and composition characteristics of the soil and rock materials are provided in Section 5.4 of this report.

Discussions related to the methods used for determination of the baseline geotechnical parameters are also provided in this section of the report. As part of this section, summary graphs of interpreted geotechnical engineering parameters are provided as supporting documentation of the variability and character of the subsurface materials, however, these summary graphs do not constitute the baseline values. Baseline values are provided in specifically identified figures and within the text of this section. Should it be necessary to ascertain potential differences between actual field conditions and this baseline report, these same methods for test interpretation will be applied to new data so as to determine whether or not there exists a material difference in the subsurface conditions. The baseline geotechnical engineering parameter values were based on the interpretation of the test results compiled in the Geotechnical Data Report supplemented by published and unpublished information where relevant and necessary. Where the selection of a parameter is dependent upon the stress path experienced by the soil, methods for determination of this stress-path dependency are also discussed. The parameters as provided in this report are considered appropriate for the in situ condition of the ground. The influence of construction methods, equipment, materials and sequencing on the engineering performance of the soil, water, and rock are to be evaluated by the successful proponent.

The field and laboratory testing completed for the Windsor-Essex Parkway (Geotechnical Data Report, Windsor-Essex Parkway, 2009) was planned such that, at sixteen locations, multiple testing methods were used to develop profiles of geotechnical parameters that could be readily compared in which the spatial variability would be limited. The data developed at these locations are summarized on Figures 6.1A to 6.1P for general information purposes only. Limitations related to the test data shown on these figures and baseline parameters are provided in subsequent sections of this report.

6.1 Clayey Silt to Silty Clay

6.1.1 Undrained Shear Strength

Determination of the undrained shear strength of the clayey silt to silty clay was achieved during investigations carried out during fall 2006 and through 2008 and early 2009 for this project using two types of field vane shear test, three types of laboratory triaxial test, and the piezocone penetration test (CPT). Historical data from other projects were also reviewed and incorporated into this report, where applicable. This historical data included field vane shear testing, unconfined compression tests, and direct shear tests. This report section summarizes the methods by which the undrained shear strength for various testing methods and modes of failure were evaluated.

Two types of field vane shear tests were completed at this site. A vane shear testing device conventional to MTO practice in Ontario was used as well as the Nilcon Field Vane Borer testing device (see Geotechnical Data Report, Windsor-Essex Parkway, 2009). The conventional vane was used in each borehole and the Nilcon device was used adjacent to selected boreholes. The conventional vane device was turned with a calibrated torque wrench at shear rates such that times to failure ranged from about 10 to 30 seconds. The Nilcon field vane device uses a torque head turned by a worm gear and crank and provides a continuous record of angular rotation and torque to interpret rod friction, peak, post-peak, and remoulded shear strengths while allowing close control of shear rates (see Geotechnical Data Report). The Nilcon vane was advanced without drilling through much of the soil profile at the project site, except where the surface crust was too strong to allow direct pushing of the device. In such cases, a hole was drilled through the crust without removing the soil to permit direct pushing of the vane while also supporting the vane rods. Time to failure using the Nilcon field vane device typically ranged from about 2 to about 4 minutes. The differences in undrained shear strength indicated by the two tests are considered to be the result of differing strain rates during this testing in slightly to moderately overconsolidated soils as reflected by the times to failure noted above. Based on the range of plasticity index values the correction factor to be applied to field vane shear tests (Bjerrum 1972, 1973) ranges between about 1.0 and 1.1 and, therefore, a correction factor was not applied to the field vane shear test results.

Piezocone penetration tests (CPT) were carried out to assist with profiling the geotechnical characteristics of the Silt and Clay Deposit. The CPT was used because of the relatively constant rate of strain during the test, its repeatability among operators and CPT systems, and since it also provides a nearly continuous profile of data through the test. A site-specific correlation between the corrected CPT tip resistance (q_c) and undrained shear strength was developed considering the field vane shear test results as well as the laboratory testing. The undrained shear strength from the relevant CPT data was interpreted using the following equation:

$$s_{u(CPT)} = q_c / N_c$$

where:	$s_{u(CPT)}$	=	undrained shear strength as derived from the CPT (kPa)
	q_c	=	tip resistance (kPa)
	N_c	=	cone factor

While other published correlations between undrained shear strength and corrected tip resistance were examined, it was determined that the above relationship provided the most suitable estimates for baseline purposes. The "cone factor" was chosen such that the calculated undrained shear strength was in reasonable agreement with the typical range of the field vane shear tests. Figure 6.2 illustrates a comparison between undrained shear strength values determined using the above relationship and each of the other testing methods. Based on the field vane shear tests and laboratory testing data, the baseline cone factor has been defined to be $N_c = 16$. The undrained shear strength, as derived above from the cone penetration test, will be the basis for judging differences in subsurface conditions between this baseline report an actual conditions.

Laboratory tests were also carried out to estimate the undrained shear strength of the overburden soils. A total of 46 tests were consolidated isotropically and sheared in undrained compression with porewater pressure measurements (CI^*UC). Another 10 tests were consolidated isotropically and sheared in undrained extension while obtaining porewater pressure measurements (CI^*UE). These isotropically consolidated samples were consolidated to an all-around confining pressure of about one-quarter to one-half the estimated existing vertical effective stress (σ'_{vo}) so as not to stress the soils past their one-dimensional vertical yield stress point ("preconsolidation pressure", σ'_p) or estimated in situ K_o conditions (K_o = in situ ratio between horizontal and vertical effective stresses), that might otherwise destroy or disturb the sample structure either in horizontal (radial) or vertical stress directions. For testing purposes the lower bound value of K_o was approximated as K_o =

1-sinϕ', or about 0.5. This procedure contrasts with typical practice (e.g. Donaghe and Townsend 1975, Mayne 1985) in which consolidation pressures are often chosen to be equal to the estimated in situ vertical effective stress (σ'_{vo}), noted in this report as CIUC tests. Several tests were also carried out during the testing program completed for this project using conventional CIUC tests as a comparison.

Seven compression and extension tests were completed by consolidating the soils anisotropically where the radial confining stress was set equal to $0.5\sigma'_{vo}$ and the vertical consolidation pressures were chosen to provide values both above and below the estimated preconsolidation pressure, σ'_p. These are noted as CAUC and CAUE tests.

A third group of tests was completed in which the pressures during consolidation and the shear phases of the test were chosen to further define properties as they may relate to the overconsolidation ratio (OCR) when the OCR is associated with mechanical precompression. Following the consolidation phase of each of these tests, the consolidation pressures were reduced to match the approximate in situ vertical and horizontal stresses, equilibrium was attained, and the samples were then sheared in either compression or extension. This group included tests using both anisotropic and isotropic consolidation pressure approaches. Comparisons of the ratio of peak undrained shear strength and maximum consolidation stress from the laboratory tests discussed above are presented on Figures 6.3.

The undrained shear strength measured by any test will be stress-path and strain-rate dependent. Therefore, a series of relationships are provided below to address stress-path dependency of undrained shear strength with reference to specific test types and conditions. These relationships are based on the guidance of Mesri (1989), Kulhawy and Mayne (1990), and Woo and Moh (1990), a review of all laboratory testing data, and considering that $S_{u(CPT)}$ represents the undrained shear strength determined based on the correlation between CPT tip resistance and vane shear test results developed for this project. This reference undrained shear strength must be modified, however, for design conditions where the shear plane passes through the weathered crust as discussed below. For the purposes of this baseline report, it was considered that the relationship between test methods (and their respective stress-path and mode of shear) and reference in situ undrained shear strength, $S_{u(ref)}$, for the low-plasticity clayey silt and silty can be expressed by the following equations:

$S_{u(ref)} = [S_{u(CKoUC)} + S_{u(DSS)} + S_{u(CKoUE)}]/3$

$S_{u(ref)} = S_{u(CI*UC)}$

$S_{u(ref)} = 1.33S_{u(CI*UE)}$

$S_{u(ref)} = 0.63S_{u(CIUC)}$

$S_{u(ref)} = 0.84S_{u(CIUE)}$

$S_{u(ref)} = 0.78S_{u(CAUC)}$

$S_{u(ref)} = 1.09S_{u(CAUE)}$

$S_{u(ref)} = 1.07S_{u(DSS)}$

$S_{u(ref)} = 0.72S_{u(CKoUC)}$

$S_{u(ref)} = 1.48S_{u(CKoUE)}$

$S_{u(ref)} = 0.69S_{u(PSC)}$

$S_{u(ref)} = 1.13S_{u(PSE)}$

$S_{u(ref)} = S_{u(FVT)}$

$S_{u(ref)} = S_{u(CPT)}$

Where:

$S_{u(CI*UC)}$ and $S_{u(CI*UE)}$ = undrained shear strength derived from isotropically consolidated, undrained triaxial compression (CI*UC) or extension (CI*UE) test with pore water pressure measurements, with the consolidation pressure equal to between ¼ and ½ the estimated in situ vertical effective stress;

$S_{u(CIUC)}$ and $S_{u(CIUE)}$ = undrained shear strength derived from isotropically consolidated undrained triaxial compression (CIUC) or extension (CIUE) test with pore water pressure measurements, with the consolidation pressure approximately equal to the estimated in situ vertical effective stress;

$S_{u(CAUC)}$ and $S_{u(CAUE)}$ = undrained shear strength derived from anisotropically consolidated undrained triaxial compression (CAUC) or extension (CAUE) test with pore water pressure measurements, with the vertical consolidation pressure equal the estimated in situ vertical effective stress and the radial consolidation stress equal to ½ of the estimated in situ vertical confining stress;

$S_{u(CKoUC)}$ and $S_{u(CKoUE)}$ = undrained shear strength derived from samples consolidated under K_o conditions (zero radial strain) and sheared in undrained triaxial compression (CK_oUC) or extension (CK_oUE) with pore water pressure measurements, with the vertical consolidation pressure equal the estimated in situ vertical effective stress and the radial consolidation stress equal to the estimated in situ vertical stress times the in situ horizontal to vertical stress ratio, K_o;

$S_{u(DSS)}$ = undrained shear strength in direct simple shear shear mode;

$S_{u(PSC)}$ = undrained shear strength in plane-strain compression shear mode;

$S_{u(DSS)}$ = undrained shear strength in plane-strain extension shear mode; and

$S_{u(FVT)}$ = undrained shear strength determined by field vane shear test provided that strain rates are maintained to between two and four minutes for each test.

Of the 52 triaxial compression tests completed on unweathered clayey silt to silty clay soils from this site, 65 per cent exhibited strain softening behaviour at large strains (strains of 10 per cent to 20 per cent). The strength values for these tests decreased by as much as 17 per cent, with an average decrease of 8 per cent and a standard deviation of 6 per cent. Of the 11 triaxial extension tests completed on unweathered clayey silt and silty clay soils from this site, many exhibited strain softening behaviour at large strains (strains on the order of 10 per cent to 20 per cent) with strength decreases of as much as 73 per cent, with an average and standard deviation of strength decrease of about 40 per cent and 14 per cent.

Within the upper silty clay "crust" the field tests indicated relatively high peak undrained shear strength values. The laboratory compression tests indicated variable strength properties depending on whether or not the sample specimen exhibited natural fissuring. It has been shown, for construction of embankments on soft ground in particular, that the operative (reference) shear strength of the ground mass in such crusts is less than measured peak strengths yet greater than remoulded strengths. The approaches of Lefebvre et al. (1987) and Tavenas and Leroueil (1980) as well as the measured post-peak values were considered in defining a means for determining baseline undrained shear strength values in the weathered crust.

For design of embankments, the baseline undrained shear strength within the weathered crust is to be considered uniformly equal to the value of the reference undrained shear strength immediately below the weathered crust at the boundary between the weathered crust and underlying deposit as illustrated on Figures 5.1A to 5.1I. For design of cut slopes, the baseline profile of undrained shear strength in the crust is to be considered uniformly equal to the value of the reference undrained shear strength immediately below the weathered crust at the boundary between the weathered crust and underlying deposit as illustrated on Figures 5.1A to 5.1I.

Figures 6.4A to 6.4F provide baseline reference undrained shear strength profiles for sections along the Windsor-Essex Parkway. Where the sections abut, the undrained shear strength profile is to be taken as the average of the two adjacent profiles. Modification to these profiles are to be made as described above based on the local baseline crust thickness.

Areas with comparatively low undrained shear strength were identified near CPT-106, CPT-9 and CPT-13 and CPT-134. Separate baseline profiles are presented for these locations on Figures 6.4J and 6.4I, 6.4G and 6.4E, respectively. In these areas the baseline undrained shear strength at any point between the identified lower strength CPT location and the next adjacent CPT is to be linearly interpolated between the given 50th percentile profiles at the same elevation, based on the horizontal distance between the test locations. The 10th and 90th percentile values of undrained shear strength is to be considered 80 per cent and 125 per cent of the provided 50th percentile profile values for CPT-106, CPT-9, CPT-13 and CPT-134 only.

6.1.2 Preconsolidation Pressure

For baseline purposes, the "preconsolidation pressure" is to be determined based on the reference undrained shear strength values, as determined from calibration to the vane shear test results as described above, using the approach as follows (after Mesri 1975):

$S_{u(ref)} = 0.22\sigma'_p$ or for the preconsolidation pressure, $\sigma'_p = S_{u(ref)} / 0.22$

where:	$S_{u(ref)}$	=	reference undrained shear strength (kPa)
	σ'_p	=	preconsolidation pressure

The "preconsolidation" pressure of a clay soil can be influenced by weathering (wetting and drying cycles) and cementation and it is known that the soils in southwestern Ontario can be lightly cemented (e.g. Brown 1970, Quigley and Ogunbadejo 1974 and 1976, De Lory and Salvas 1970, Boone and Lutenegger 1997). Interpretation of oedometer tests in the till-like soft soils in southwestern Ontario can be problematic as the nature of the soils tends to produce curves that do not have a distinct change in behaviour that clearly demarcates the "preconsolidation" pressure. Settlement calculations based on such ambiguous determinations of "preconsolidation pressure" typically overestimate field settlements. The oedometer tests completed for this project were interpreted using a slope-intercept method in which the following steps are carried out:

1) if the oedometer test does not include a load increment equal to σ'_{vo} so as to allow direct determination of the value of the voids ratio at this stress, e_{vo}, determine the slope of the line between load increments that passes through σ'_{vo}, noted as C_{cvo}, so that the value of e_{vo} can be mathematically interpolated;

2) determine the load increment at which C_c is a maximum, C_{cmax}, where the ordinates of the corresponding ending stress and voids ratio are thus noted σ'_{vmax} and e_{min}, respectively ;

3) determine the recompression index, C_r, defined as the average slope of an unload-reload cycle preferably conducted at stresses above and below σ'_{vo} and less than σ'_p;

4) calculate the intercept values along the voids ratio axis for the compression and recompression index lines using:

$$e_c = C_{cmax}\log\sigma'_{vmax} + e_{min}$$

$$e_r = C_r\log\sigma'_{vo} + e_{vo}$$

5) calculate the intersection of these two lines, the first defined by the C_{cmax} line and the second defined by a line parallel to the C_r line passing through the in situ vertical effective stress, solving for the voids ratio at the preconsolidation pressure, e_p, by:

$$e_p = (e_c/C_{cmax} - e_r/C_r)/(1/C_{cmax} - 1/C_r)$$

6) and the "preconsolidation pressure", may then be found by:

$$\sigma'_p = 10^{\{(e_c - e_p)/C_{cmax}\}}$$

Figure 6.5 summarizes this approach to determining a unique preconsolidation pressure based on the oedometer test. The interpretation method described above was utilized for this project because the method and the parameters derived from the testing program provided an excellent correlation with measured embankment settlements in the region where oedometer data was available.

A comparison of estimated preconsolidation pressure using the CPT and oedometer tests is provided on Figure 6.6. For baseline purposes, the preconsolidation pressure is to be derived based on the above correlation and the undrained shear strength profiles provided in Figures 6.4A to 6.4F. Where the sections identified in Figures 6.4A to 6.4F abut, the baseline preconsolidation pressure is to be taken as the average of the preconsolidation pressure derived from the adjacent baseline profiles. The horizontal one-dimensional yield stress can, for baseline purposes, be taken as 0.7 to 1.0 times the vertical effective one-dimensional yield stress (preconsolidation pressure).

6.1.3 Stress-Strain Properties

Determination of the stress-strain properties of the soils was accomplished using the laboratory oedometer and triaxial tests conducted for this project, examination of other laboratory test results from Golder files, and comparison to published correlations and theoretical relationships. Correlations among oedometer test results developed for this project are illustrated in the summary on Figure 6.7. Figure 6.8 summarizes data interpreted from the triaxial testing.

One-dimensional consolidation properties were determined based on the results of oedometer tests completed for this project and others in the vicinity. "Virgin" compression index, C_c, values were defined based on the maximum slope of the oedometer compression curve. The "recompression" index C_r was taken to be representative of the average slope of an unload-reload cycle conducted at pressures equal to or less than the preconsolidation pressure. The "swelling index", C_s, typically defined by the unloading phase following the maximum load in the oedometer test is not to be considered indicative of the unload-reload compression index, C_r. These parameters were found to be readily related to the natural water content of the specimens, w_n. Oedometer test data were also used to define the coefficient of consolidation, c_v, that is related to the time-rate of settlement. The secondary compression index, C_α, was also compared to the compression index, C_c, for two different stress levels as illustrated on Figure 6.7. The results of data evaluation provided the following correlations:

C_c = $0.0086w_n - 0.0086$

C_r = $0.11C_c$

C_s = $0.25C_c$

C_α = $0.028C_c$ for all stress ranges

w_n = natural water content expressed as a percent

Baseline values for these parameters are to be derived from the baseline water content profiles as discussed in Section 5.4.2 and the correlations provided above.

Table 6.1: Baseline Vertical Coefficient of Consolidation, c_v (m^2/s)

Stress Condition	10th Percentile	50th Percentile	90th Percentile
Recompression	4.5×10^{-7}	1.8×10^{-6}	5.5×10^{-6}
Virgin Compression	2.2×10^{-7}	7.2×10^{-7}	2.9×10^{-6}

Based on laboratory testing of horizontally-oriented samples, the horizontal coefficient of consolidation, c_h, is to be assumed to be 2 times the vertical coefficient of consolidation as identified in the table above for baseline purposes.

These correlations are consistent with published correlations for similar soil types (e.g. Holtz and Kovacs 1981, Mesri and Godlewski 1977, Kulhawy and Mayne 1990) and experience with back-analysis of embankments and foundations in the region that are supported on similar soils (Becker et al. 1984, Crooks et al. 1984).

During triaxial testing (CI^*UC tests), each sample was subjected to unloading and reloading at a fraction of the failure stress, with the start of the cycle typically close to the estimated in situ horizontal stress. Non-linear stress strain properties were defined consistent with the hyperbolic constitutive model (e.g. Duncan and Chang 1970). Deformation moduli were developed for three positions within the stress strain curve: (1) an approximate of the initial undrained tangent modulus, E_{uit} consistent with strains in the range of about 0.1 per cent to 0.2 per cent; (2) secant undrained modulus at 50 percent failure stress, E_{us50}, corresponding to a strain range of 1 per cent to 3 per cent; and (3) unload-reload modulus, E_{ur}, assessed based on an unload-reload cycle typically carried out between these strain levels. The initial tangent modulus was considered equivalent to a secant modulus defined by a linear best fit of the first several points on the stress-strain curve following any obviously disturbed early portions of the test curve. The unload-reload modulus was generally defined using a linear fit line between the minimum and maximum stress-strain coordinates of unload-reload cycle, again, after accounting for any obvious disturbance or inconsistencies in the data. Data from tests that exhibited a volumetric strain during consolidation of about 7 per cent or more were excluded from evaluations of stress-strain properties as volumetric strains above this value were considered as representative of sample disturbance and lower sample quality (Lunne et al. 1997, DeGroot et al. 2005). Drained initial secant deformation moduli were also derived using the results of the oedometer test via the coefficient of volume compressibility, m_v, consistent with the unload-reload cycle and an assumed drained Poisson's ratio, ν', of 0.35. These data are also presented on

Figure 6.8 where similarity between the data sets is evident. The data evaluation resulted in the correlations below. These values are generally consistent with, though somewhat lower than, published correlations for similar soil types (e.g. Becker Kulhawy and Mayne 1990).

E_{uit} = [150, 290, 500] $S_{u(ref)}$ for the 10th, 50th, and 90th percentiles, respectively, where the 50th percentile $S_{u(ref)}$ value is used as identified in Section 6.1.1, above

E_{ur} = $1.65E_{uit}$

E_{us50} = $0.44E_{uit}$

E' = $0.9E_u$, where E' represents the drained deformation modulus and E_u represents the undrained deformation modulus for any of the strain levels identified above

For baseline purposes, the deformation moduli for the relevant strain ranges are to be derived based on the above correlations and the reference undrained shear strength profiles provided in Figures 6.4A to 6.4F. Where the sections identified in Figures 6.4A to 6.4F abut, the deformation moduli are to be taken as the average of the moduli derived from the adjacent baseline profiles. Furthermore, where the design is sensitive to the deformation moduli, the full range is to be considered and the more critical case used for design. If the design is sensitive to high values of deformation moduli within the "crust", the above correlations are to be applied to the 50th percentile reference undrained shear strength values within the "crust" rather than to the reduced baseline profile that considers the effects of fissuring.

The baseline deformation moduli as provided above represent the stress-strain response of the soils as related to strain rates typical for the laboratory testing methods used to derive these parameters (e.g., typical average rate of strain of approximately 0.5% per hour for triaxial tests). These deformation moduli, therefore, do not represent the long-term, time-dependent low strain-rate behaviour. Displacements estimated using the above deformation moduli are not to be considered applicable to long-term creep behaviour of retaining structures, slopes, or foundations. Two CI*UC triaxial compression tests and two CI*UE triaxial extension tests were performed to as part of this project to measure the rate-sensitivity of these soils. The strain rate parameter, $\alpha = 1/n$ (Hinchberger 1996, Hinchberger and Rowe 1998, Hinchberger and Qu 2009), was evaluated using these tests as well as a comparison to the secondary compression index, C_α. For baseline purposes, the strain rate parameter for evaluating time-dependent creep behaviour in the clayey silt to silty clay within elasto-viscoplastic models as referenced above may be derived using the following relationship and the baseline parameters for C_c, C_r, and C_α as provided above:

$$\alpha = 1/n = C_\alpha/(C_c - C_r)$$

6.1.4 Effective Stress Strength Parameters

Estimation of the Mohr-Coulomb strength parameters of effective internal angle of soil friction, ϕ', and effective cohesion intercept, c', was based on the results of the laboratory triaxial testing. Figure 6.9 summarizes data used to interpret the effective angle of internal friction. The corresponding effective angle of internal friction for an assumption of an effective cohesion intercept of zero was determined to be about 30 degrees. These values are generally consistent with published correlations for similar soil types (e.g. Kulhawy and Mayne 1990). For baseline purposes, the effective cohesion intercept is to be assumed equal to zero and the peak effective angle of internal friction is to be assumed equal to 30 degrees and the residual angle of internal friction is to be assumed equal to 27 degrees.

Measurements of the porewater pressure parameter at failure in triaxial compression, A_f, are summarized in Figure 6.9. A relationship developed by Mayne and Stewart (1988) was found to be a reasonable basis on which to estimate A_f provided that upper and lower bounds were also defined as shown below and on Figure 6.9:

$$A_f = \frac{0.5\left(OCR^{0.5} - 0.67OCR\right) + \dfrac{OCR}{0.58}}{0.5OCR^{0.5} + 0.67OCR - 1} \pm 0.2$$

The relationship above is to be used to define baseline porewater pressure behaviour at failure during undrained compressive shear. The baseline OCR profile is to be determined using the methods for determining the "preconsolidation pressure" profile as identified in Section 6.1.2, above, and, by dividing the "preconsolidation pressure" values by the corresponding values of in situ vertical effective stress to arrive at the OCR profile. Where the design is sensitive to the porewater pressure parameter at failure, the more adverse of the upper or lower bound values are to be used for baseline design purposes.

6.1.5 In Situ Horizontal Stress

There is no evidence for significant mechanical preconsolidation of the native soils in the area of the project resulting from mass erosion of overburden or past glacial overriding stresses except, possibly, toward the eastern end of the project area. It is considered that no single test method (field or laboratory) or empirical approach based on stress history is capable of accurately deducing the in situ horizontal stress state. Geologic complexities including cementation, weathering, depositional rate and environment, past direct stresses (from pre-existing overburden since removed, or ice stresses), and stresses induced by multiple groundwater level fluctuations all render the use of empirical relationships based on simple mechanical stress history problematic. Therefore, the baseline value for the ratio of in situ horizontal to vertical stresses, K_o, is to be taken as the average of the two relationships of $K_o = (1-\sin\phi')$ and $K_o = (1-\sin\phi')OCR^{\sin\phi'}$ with a maximum $K_o = 1$ for soils below the crust, and a maximum value in the crust equal to the lower of either maximum value calculated for the soils immediately below the crust or in any event, not greater than 1.5.

6.1.6 Permeability/Hydraulic Conductivity

The coefficient of permeability or hydraulic conductivity, k, of the clayey silt to silty clay materials was inferred from oedometer testing and measured during laboratory testing using a flexible wall permeameter. All measurements derived from laboratory tests are summarized in Figure 6.10. Figure 6.10 illustrates that the measured permeability based on the flexible wall permeameter results are approximately one order of magnitude smaller than those obtained through interpretation of the oedometer tests. The laboratory oedometer test measurements of permeability, however, are only considered appropriate for the small specimens of the Clayey Silt to Silty Clay Deposit. In addition, rising or falling head tests were completed in selected piezometers or observation wells within the silty clay to clayey silt deposits. For baseline purposes, the in situ mass permeability in the vertical direction for the Clayey Silt to Silty Clay Deposit is provided in the table below and the baseline value in the horizontal direction is to be assumed to be twice the values provided below.

Table 6.2: Baseline In Situ Mass Permeability in Vertical Direction for Clayey Silt to Silty Clay Deposit, k (m/s)

Deposit Type	10th Percentile	50th Percentile	90th Percentile
"Crust" Soils	1.1×10^{-9}	3.4×10^{-9}	1.5×10^{-8}
Unweathered Soils	1.6×10^{-10}	5.3×10^{-10}	2.3×10^{-9}

6.1.7 Maximum Dry Density and Optimum Water Content for Compaction

The maximum dry density and optimum water content for compaction were determined on a total of 18 samples for this project using the "Standard Proctor Compaction" test (ASTM D698). These tests were conducted on samples obtained from the boreholes using thin-wall tube sampling methods. In addition, these data were supplemented with 16 tests from surrounding project sites. A comparison of the optimum water content for compaction and the maximum dry density for these test data is provided on Figure 6.11. Baseline natural water content values for the native soils is provided in Section 5.4 of this report. Baseline optimum compaction water content and maximum dry density values for the native cohesive soils are provided in Table 6.3, below. Figure 6.11 presents a probability histogram illustrating a comparison between the distribution of optimum compaction water content for the natural clayey silt/silty clay soils and the natural water content for the brown and mottled "crust" materials and the underlying grey soils.

Table 6.3: Optimum Compaction Water Content and Maximum Dry Density for Clayey Silt to Silty Clay

Parameter	10th Percentile	50th Percentile, Average	90th Percentile
Optimum Compaction Water Content (%)	12.6	14.8	17.5
Maximum Dry Density (kN/m^3)	17.3	18.2	19.0

The native clayey silt to silty clay soils have been separated into two categories with respect to re-use of these materials for fill, these being:

1) the "crust", consisting primarily of clayey silt to silty clay that is characteristically brown to mottled brown and grey within the zone subject to seasonal wetting, drying, and freezing (generally the top 1 to 1.2 m) and brown below this depth; and

2) the underlying grey clayey silt to silty clay that is always saturated and above 8 m depth.

Furthermore, the clayey silt to silty clay soils west of the Huron Church Road and E.C. Row Expressway intersection have been found to be generally unsuitable for use as compacted fill materials; for baseline purposes, native soils excavated from the Windsor-Essex Parkway project west of Station 12+260 (Figure 5.1C) are to be considered unsuitable for re-use as compacted fill materials.

A comparison has been made between the natural water content of the native materials, the laboratory optimum water content for compaction (w_{oc}) for each test, and the range of water contents to permit achieving 95% of the material's maximum dry density during compaction for each test. MTO specifications require cohesive embankment materials to be compacted to at least 95% of the Standard Proctor maximum dry density for new construction. Figure 6.12 illustrates an example Standard Proctor compaction test along with cumulative probability (percentile) distributions comparing the in situ water content with the laboratory optimum water content for all tests. The example illustrates the method by which the bounds of water content were determined for achieving 95% compaction for each individual test.

Figure 6.12 illustrates a probabilistic comparison between the range of water contents necessary to achieve 95% of the Standard Proctor maximum dry density and measured water content at the time of testing. The curves on Figure 6.12 (lower figure) represent percentiles for the measured water content of all samples tested, expressed relative to the optimum water content. The grey shaded area on the percentile curves illustrate the range of upper and lower bound water contents (wet or dry) of optimum 95% of the maximum dry density 85% of the time. In other words, if the water content at the time of the compaction is less than 4% wet of optimum or 4% dry of optimum, there is an 85% probability that the material can be compacted to 95% of the maximum dry density without further wetting or drying the soils. Referring to Figure 6.12, the analysis indicates that there is an approximately 30% chance (i.e., 30th percentile) that the crust materials (blue line on Figure 6.12) will have a natural water content equal to or less than the optimum compaction water content (green line on Figure 6.12). For baseline purposes, therefore:

- the water content at the time of placement (w_p) can be no more than 4% wet of the laboratory optimum compaction water content to achieve 95% of the maximum dry density;
- the water content at the time of placement (w_p) can be no less than 4% dry of the laboratory optimum compaction water content to achieve 95% of the maximum dry density;
- approximately 43% of the excavated crust soils will have an in situ water content within the water content bounds suitable for achieving 95% of the maximum dry density during compaction (adequate compaction);
- approximately 45% of crust will require drying from the in situ water content to achieve adequate compaction and approximately 10% of the crust materials will require wetting from the in situ water content to achieve adequate compaction; and
- the native grey clayey silt to silty clay materials (above 8 m depth) exhibit natural water content values that indicate that less than approximately 30% of these materials will have in situ water contents that fall within the water content bounds suitable for achieving 95% of the maximum dry density during compaction without modification of placement water content. Because of the spatial variability in the water content and the relatively low probability of materials falling within the upper and lower bounds for compaction, selectively identifying native grey clayey silt to silty clay soils in the field for re-use as fill materials is not considered practical.

The baseline conditions summarized above represent an analysis based on the measured water content of the native materials at the time of sampling and testing. Excavation, transportation, spreading, and exposure to weather as controlled by methods of construction selected by the proponent will all have an effect on the water content at the time of placement and compaction. Drying of materials that are too wet to achieve adequate compaction will be problematic. Materials that are too wet will foul and impede equipment. If left exposed to dry, windy, and hot weather for too long, the soils will also dry and form hard, brick-like lumps. Successful re-use of the in situ soils will require that the earthworks be appropriately staged and planned according to prevailing weather conditions, selective excavation to delineate which materials are to be protected and used for new fill areas and which are to be sent off site for disposal, and use of other appropriate modification methods as selected by the proponent.

6.1.8 Subgrade Moduli for Pavement Design

The subgrade resilient modulus for use in flexible pavement design and the modulus of subgrade reaction for rigid pavement design have been assessed based on the results of the field and laboratory testing. A baseline subgrade resilient modulus of 25 megapascals (MPa) and a modulus of vertical subgrade reaction of 30 MPa per metre (MPa/m) are to be assumed for the crust soils of the Silt and Clay Deposits. These values are consistent with the recommendations for low to medium plasticity clays as indicated in Tables 8.6 and 8.9 of the "Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions" document in use by MTO and are also consistent with the typical pavement thicknesses constructed by MTO and others in this area.

As indicated in previous sections of this report, the undrained shear strength beneath the upper weathered crust decreases significantly. While undrained shear strength cannot necessarily be directly correlated to resilient modulus and modulus of subgrade reaction, the shear strength data clearly indicates a reduction in support characteristics for the pavements. Therefore, for baseline purposes, the resilient moduli and moduli of subgrade reaction to be used are provided in relation to the baseline undrained shear strength profiles as described in previous sections of this report.

BASELINE UNDRAINED SHEAR STRENGTH (kPa)	MODULUS OF VERTICAL SUBGRADE REACTION (MPa)	SUBGRADE RESILIENT MODULUS (MPa/m)
greater than 100	25	30
75 to 100	20	20
less than 75	15	10

6.2 Fill

All existing fill materials are to be considered unsuitable for support of structures or for use as engineered fill due to their compositional heterogeneity. Where existing fill is to remain in place, such as behind in situ retaining structures, baseline geotechnical engineering parameters are provided in the table below for use in design and construction planning. Low and High values are provided for these parameters and no statistical evaluation data are provided. The full range is to be considered for baseline purposes. Where the design is sensitive to these parameters, the more adverse condition is to be applied.

Table 6.4: Baseline Geotechnical Engineering Parameters for Existing Fill

Parameter	Range of Values
Saturated Unit Weight, γ_{sat} (kN/m^3)	19 to 23
Effective Angle of Internal Friction, ϕ' (degrees)	20 to 26
Effective Cohesion Intercept, c' (kPa)	0
Undrained Shear Strength, S_u (kPa)	0 to 50
Undrained Deformation Modulus (at 50 per cent failure stress), E_{s50} (MPa)	5 to 25

6.3 Upper Granular Deposits

Baseline geotechnical engineering parameters are provided in the table below for use in design and construction planning. Low and High values are provided for these parameters and no statistical evaluation data are provided. The full range is to be considered for baseline purposes. Where the design is sensitive to these parameters, the more adverse condition is to be applied.

Table 6.5: Baseline Geotechnical Engineering Parameters for Upper Granular Deposits

Parameter	Range of Values
Effective Angle of Internal Friction, ϕ' (degrees)	28 to 35
Effective Cohesion Intercept, c' (kPa)	0
Deformation Modulus (at 50% failure stress), E_{s50} (MPa)	5 to 15
Hydraulic Conductivity, k (m/s)	$1x10^{-6}$ to $1x10^{-4}$

6.4 Lower Granular Deposits

Baseline geotechnical engineering parameters are provided in the table below for use in design and construction planning. Low and High values are provided for these parameters and no statistical evaluation data are provided. The full range is to be considered for baseline purposes. Where the design is sensitive to these parameters, the more adverse condition is to be applied. Dewatering of the Lower Granular Deposits is not anticipated. Therefore, baseline permeability or hydraulic conductivity parameters are not provided.

Table 6.6: Baseline Geotechnical Engineering Parameters for Lower Granular Deposits

Parameter	Range of Values
Effective Angle of Internal Friction, ϕ' (degrees)	32 to 35
Effective Cohesion Intercept, c' (kPa)	0 to 200
Deformation Modulus (at 50 per cent failure stress), E_{s50} (MPa)	25 to 50

6.5 Bedrock

6.5.1 Strength and Stress-Strain Parameters

Figure 6.13 presents a summary of data related to recovery of rock cores as well as rock quality designation (RQD) values. In these data summaries, there are values in excess of 100% and these represent cores in which rock was recovered from the previous run. Baseline parameters derived from the drilling character and sample recovery including Total Core Recovery (TCR), Solid Core Recovery (SCR), and Rock Quality Designation (RQD) are summarized in Table 6.7 below. Bedrock samples were tested in uniaxial compression to determine both the uniaxial compression strength (UCS) as well as the compression deformation modulus, E. Figure 6.14 presents summaries of the resulting strength and modulus data. No clear trends were observed with respect to geographic location and rock strength or between compression strength and deformation modulus. These baseline parameters, however, are considered representative of intact rock strength and no down-grading (reduction) has been applied for rock mass characteristics to account for rock quality, fractures, bedding, or other characteristics. All data have been evaluated together and Table 6.8, below, summarizes baseline values for the strength and deformation modulus parameters.

Table 6.7: Baseline Intact Rock Properties

Parameter	10th Percentile	50th Percentile	90th Percentile
Unit Weight, γ (kN/m^3)	22	24	25
UCS (MPa)	20	28	78
E (GPa)	0.8	1.8	3.2

Table 6.8: Baseline Rock Engineering Parameters

Parameter	10th Percentile	50th Percentile	90th Percentile
Top 2 m of Bedrock			
TCR (%)	27	90	100
SCR (%)	0	68	90
RQD (%)	0	58	90
2 m to 4 m Penetration			
TCR (%)	60	98	100
SCR (%)	38	90	100
RQD (%)	48	84	100
More than 4 m Penetration			
TCR (%)	93	98	100
SCR (%)	87	93	100
RQD (%)	74	84	98

6.5.2 Permeability/Hydraulic Conductivity of Bedrock and Bedrock/Soil Interface

Dewatering or depressurization of the Bedrock is not anticipated. Therefore, baseline permeability or hydraulic conductivity parameters are not provided.

6.6 Frost Penetration Depth

For baseline purposes, the maximum depth of frost penetration is to be considered equal to 1.2 m below the lowest overlying or adjacent ground surface.

7.0 INSTRUMENTATION AND MONITORING

The Contract Documents identify minimum requirements for installation of instrumentation, reading frequencies, response thresholds, and other monitoring and reporting criteria.

8.0 MANAGEMENT OF SOIL AND GROUNDWATER

Baseline parameters related to the environmental chemistry of subsurface materials affected by chemical discharges to the environment (anthropogenic degradation) are not addressed as part of this report. Anticipated subsurface conditions as related to anthropogenic degradation are identified elsewhere within the Contract Documents. For management of excess soils generated during construction of this project, the following baseline conditions are provided:

- native soils not subject to anthropogenic degradation will not exhibit parameter concentrations in excess of risk based standards as provided in Tables 2 or 3 from the Ministry of the Environment (MOE 2004, "Ministry Standards"); and

- all native soils not subject to anthropogenic degradation will routinely exhibit at least one parameter concentration in excess of "background" levels as provided in Table 1 from the Ministry of the Environment (MOE 2004, "Ministry Standards").

Because there are no established regulatory standards for use in the management of excess soils, some receivers (such as aggregate pits) have adopted the Table 1 standards as acceptance criteria when receiving excess soils from construction projects. Some receivers are governed by the Ministry of Natural Resources policies and under these policies materials exceeding Table 1 cannot be accepted. The Contract Documents specify that disposal areas for excess soill materials must be identified and environmental quality criteria are to be specifically identified for the particular receiving site by the proponent.

Should dewatering or depressurization of bedrock aquifers be required, the Contract Documents require that the proponent undertake a survey of surrounding wells within a 3 km radius of the dewatering site, document all active groundwater wells, and complete the necessary work to secure water supply for these wells for the duration of dewatering activities. In addition, Provincial regulations require that a Permit to Take Water (PTTW) be obtained from the Ministry of the Environment Ontario in all cases in which the expected groundwater extraction rate is 50,000 litres per day or greater.

9.0 CLOSURE

This Subsurface Conditions Baseline Report was prepared by Golder Associates Ltd. on behalf of the Ministry of Transportation Ontario for the Windsor Essex Parkway project. This report was prepared by Mr. Mrinmoy Kanungo and Mr. Tyson Pitt under the direction of Dr. Storer Boone, P.Eng., and was reviewed by Mr. Philip R. Bedell, P.Eng., Mr. Murty Devata, P.Eng., and Mr. John Westland, P.Eng. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact carried out a quality control audit for this project.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. J. BOONE
90559733
PROVINCE OF ONTARIO

Storer J. Boone, P.Eng.
Associate

John Westland, P.Eng.
Principal

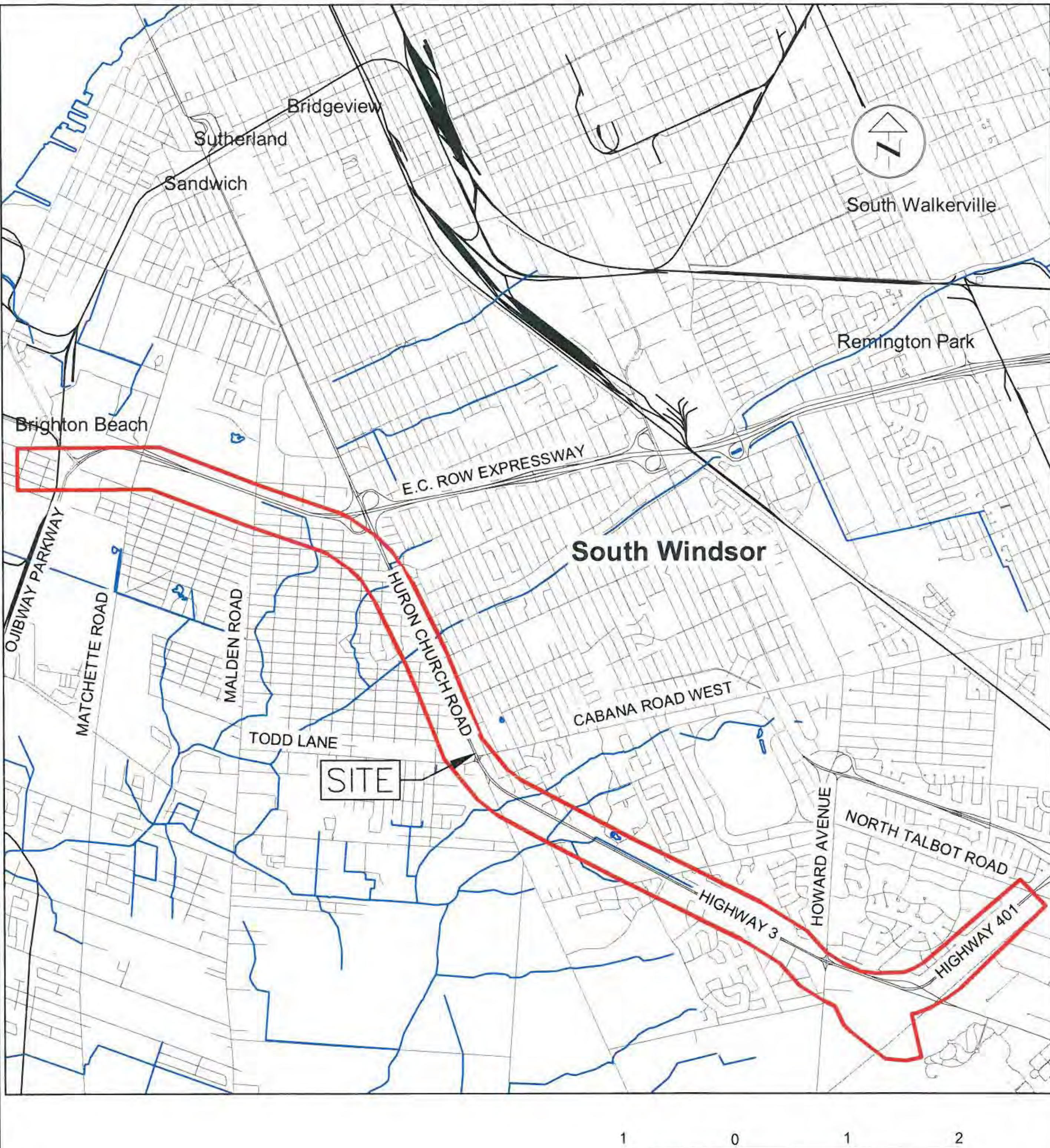
REGISTERED PROFESSIONAL ENGINEER
F.J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.
Designated MTO Contact

Philip R. Bedell, P.Eng.
Senior Consultant

SJB/JW/FJH/PRB/cr

n:\active\2007\1130 - geotechnical\1130-2000\07-1130-207-0 urs - dric approach gbr - windsor\reports\0711302070-r02 (final)\0711302070-r02 jun 26 09 - subsurface conditions baseline rpt.docx



REFERENCES

1) DRAWING BASED ON CANMAP STREETFILES V2005.4.

NOTES

1) THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

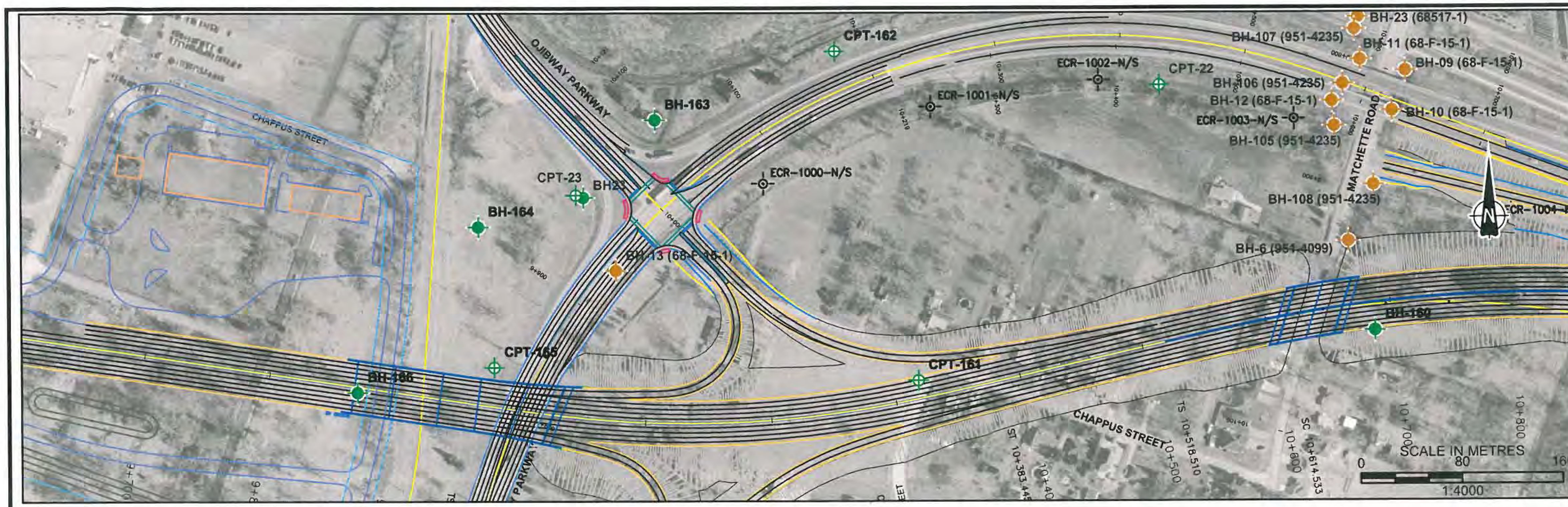
TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02001
			SCALE AS SHOWN	REV. 0
CADD	WDF	June 26/09	**FIGURE 1.1**	
CHECK	SJB	Jun 26/09		

Drawing file: 0711302070-R02001.dwg Jun 26, 2009 - 9:43am

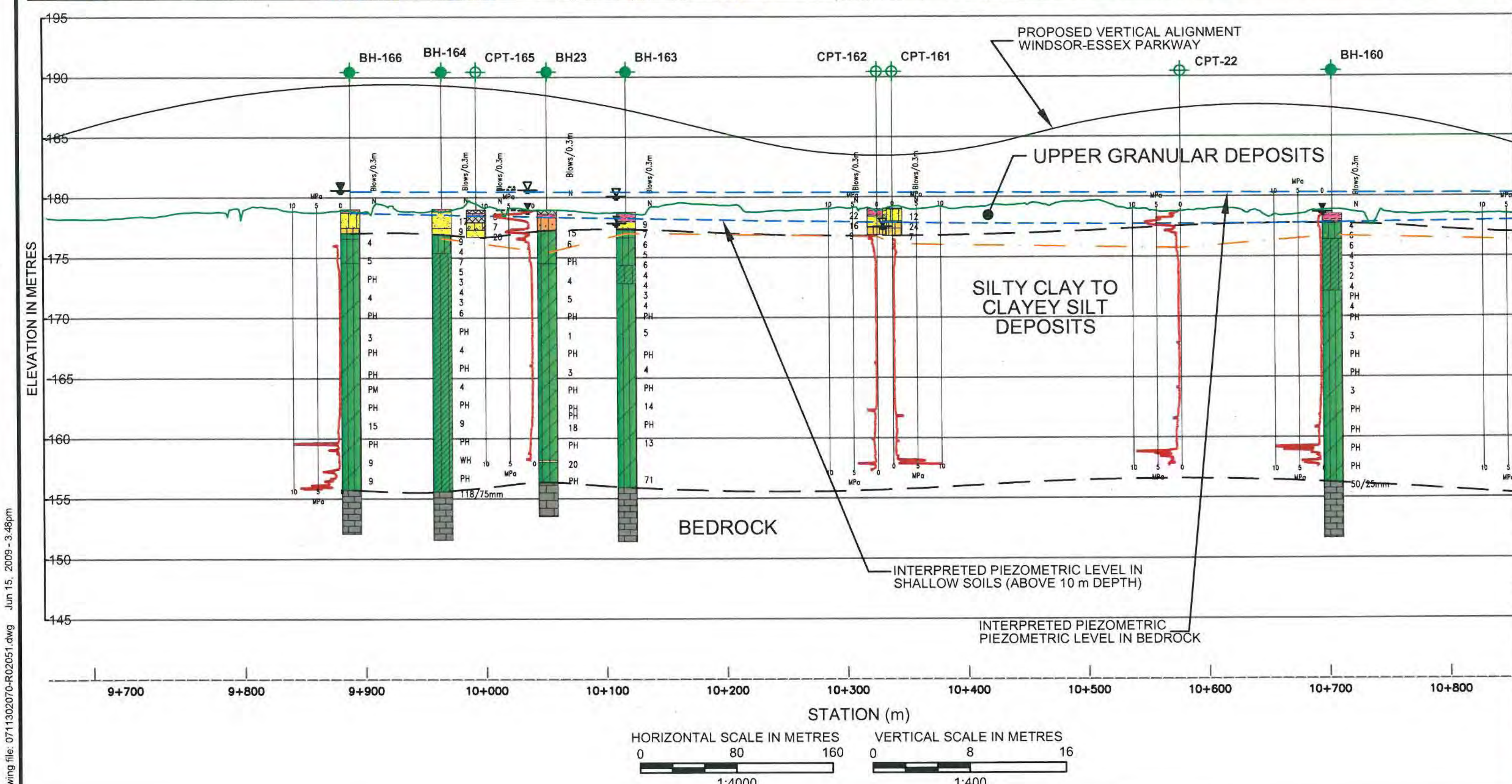


LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE



NOTES

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7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE STRATIGRAPHIC PROFILE
STN 9+800 TO 10+800

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0	FILE No.	0711302070-R02051
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		REV.	0
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CHECK	SSB	June 24/09	5.1A

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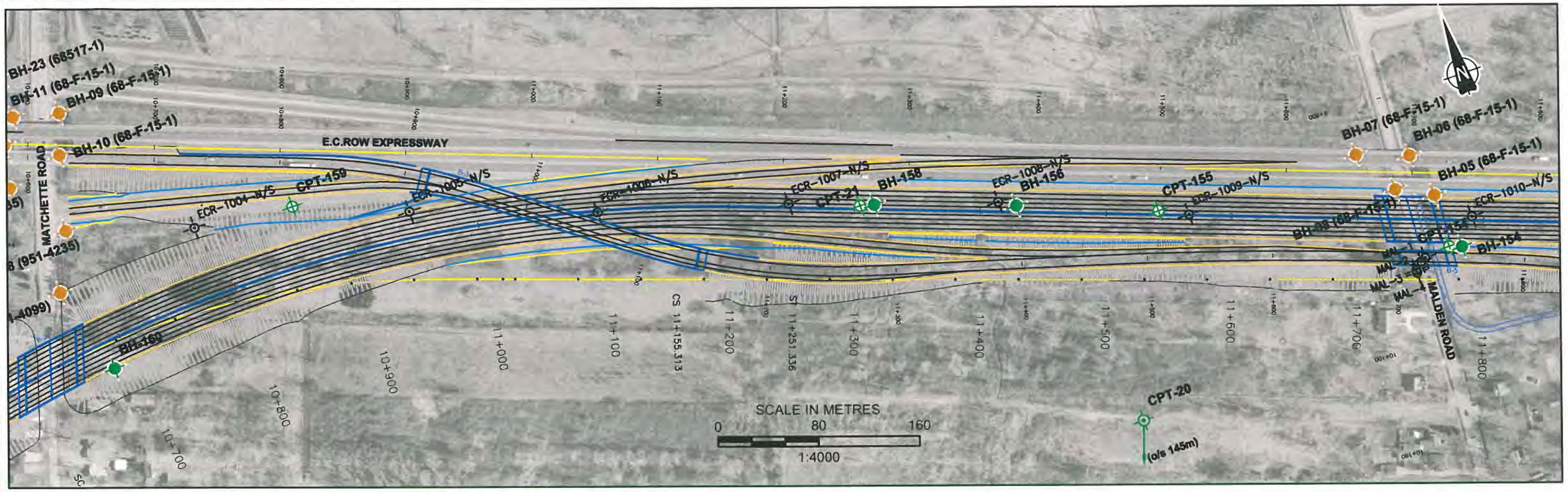
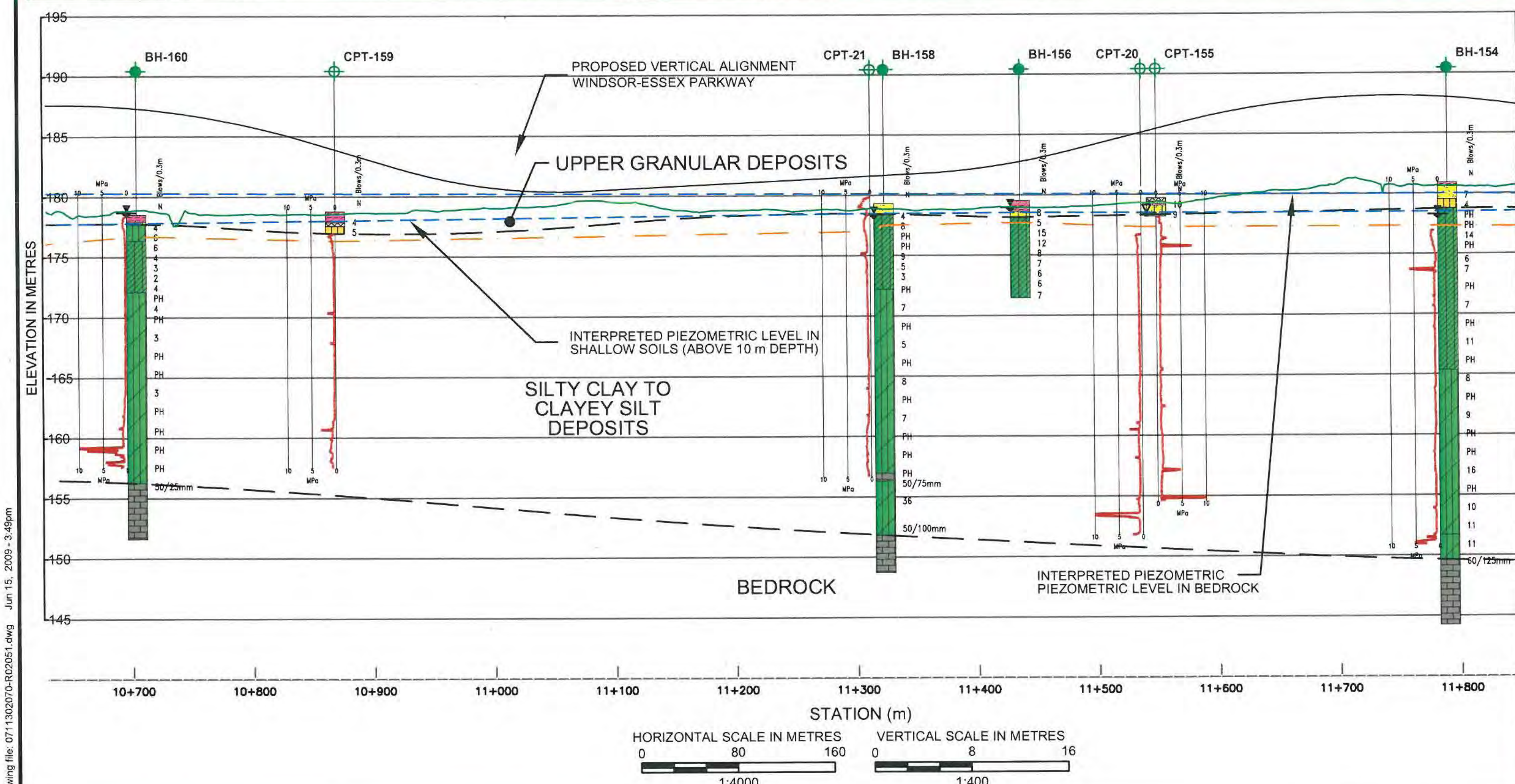


LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE



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7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE STRATIGRAPHIC PROFILE
STN 10+800 TO 11+800

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02051
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CHECK	SJB	Jun 26/09		

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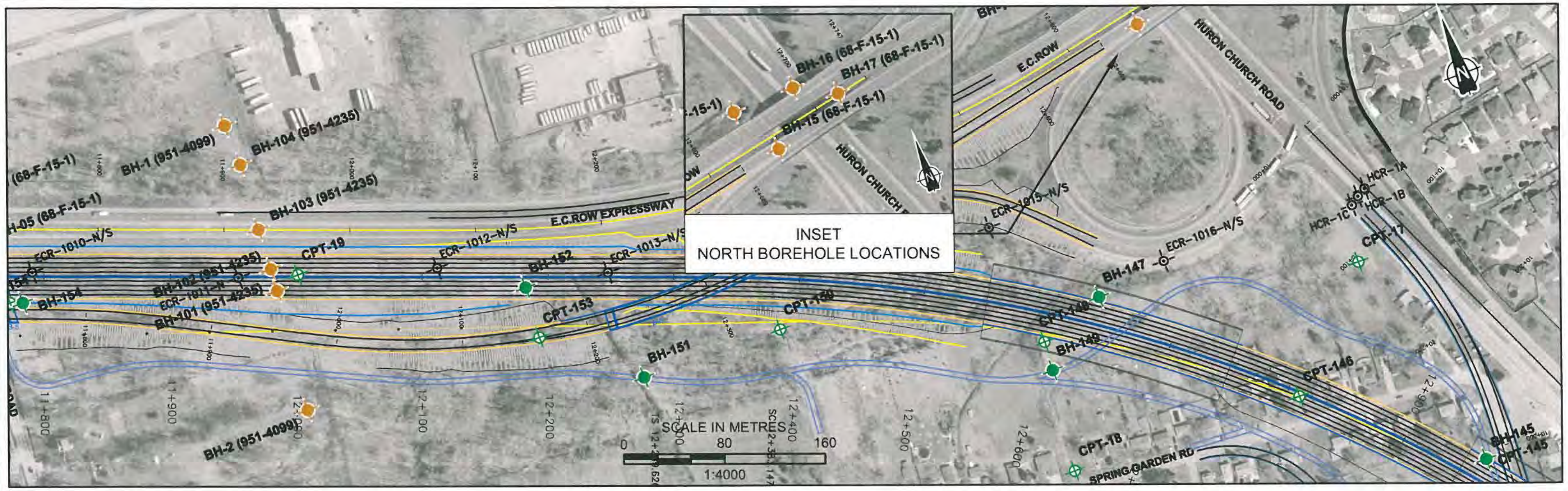

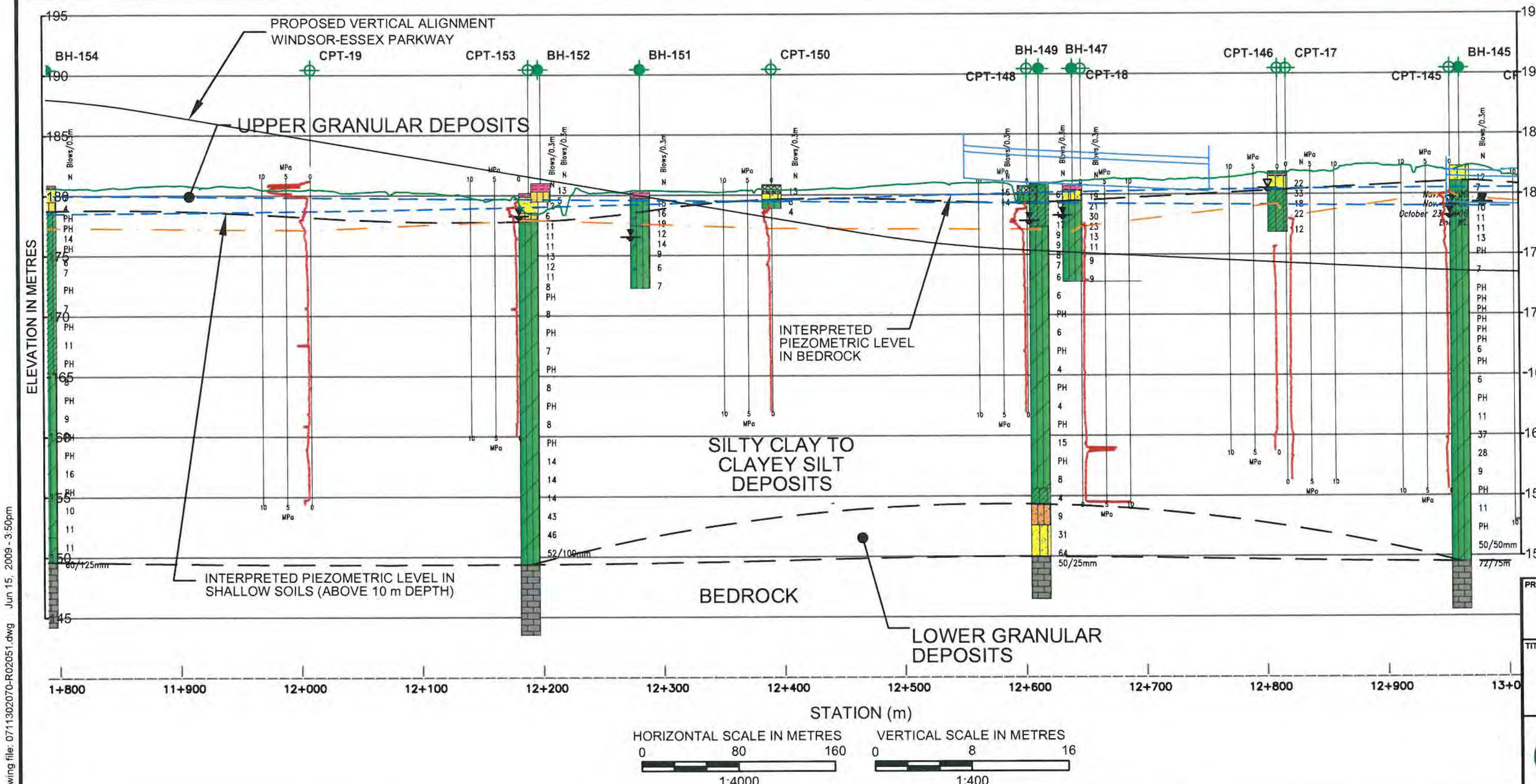

LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE

NOTES

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7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE STRATIGRAPHIC PROFILE
STN 11+800 TO 12+900

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02051	
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CADD	JDR/WDF	MAY 12/09			
CHECK	SJB	June 26/09		5.1C	

Golder Associates
LONDON, ONTARIO

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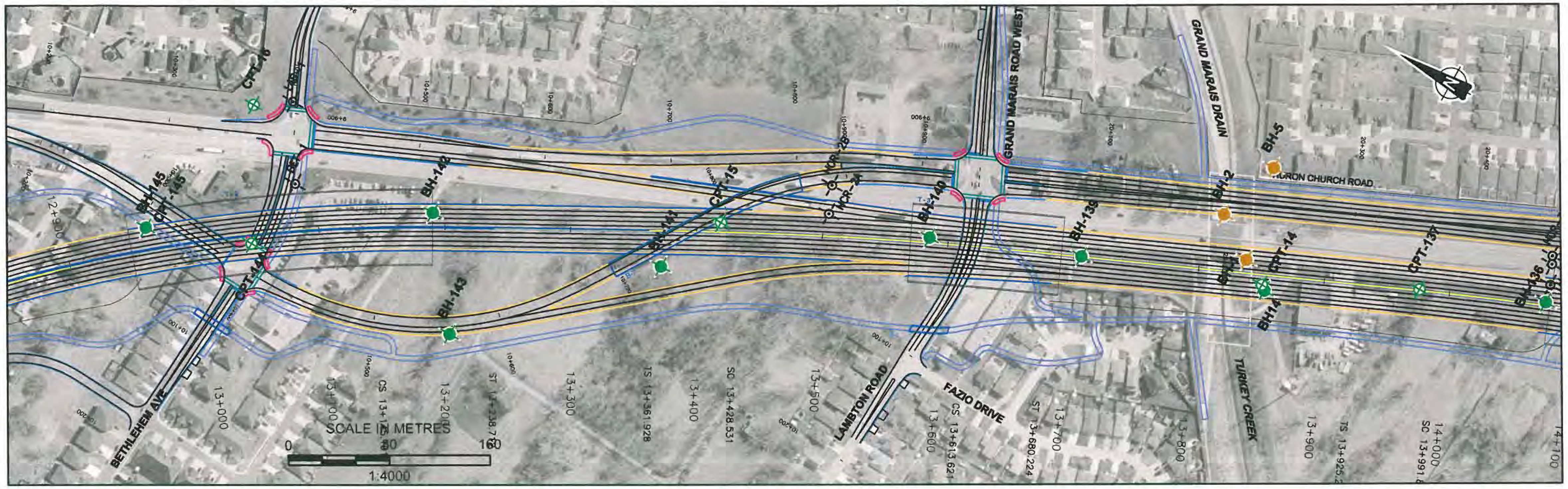

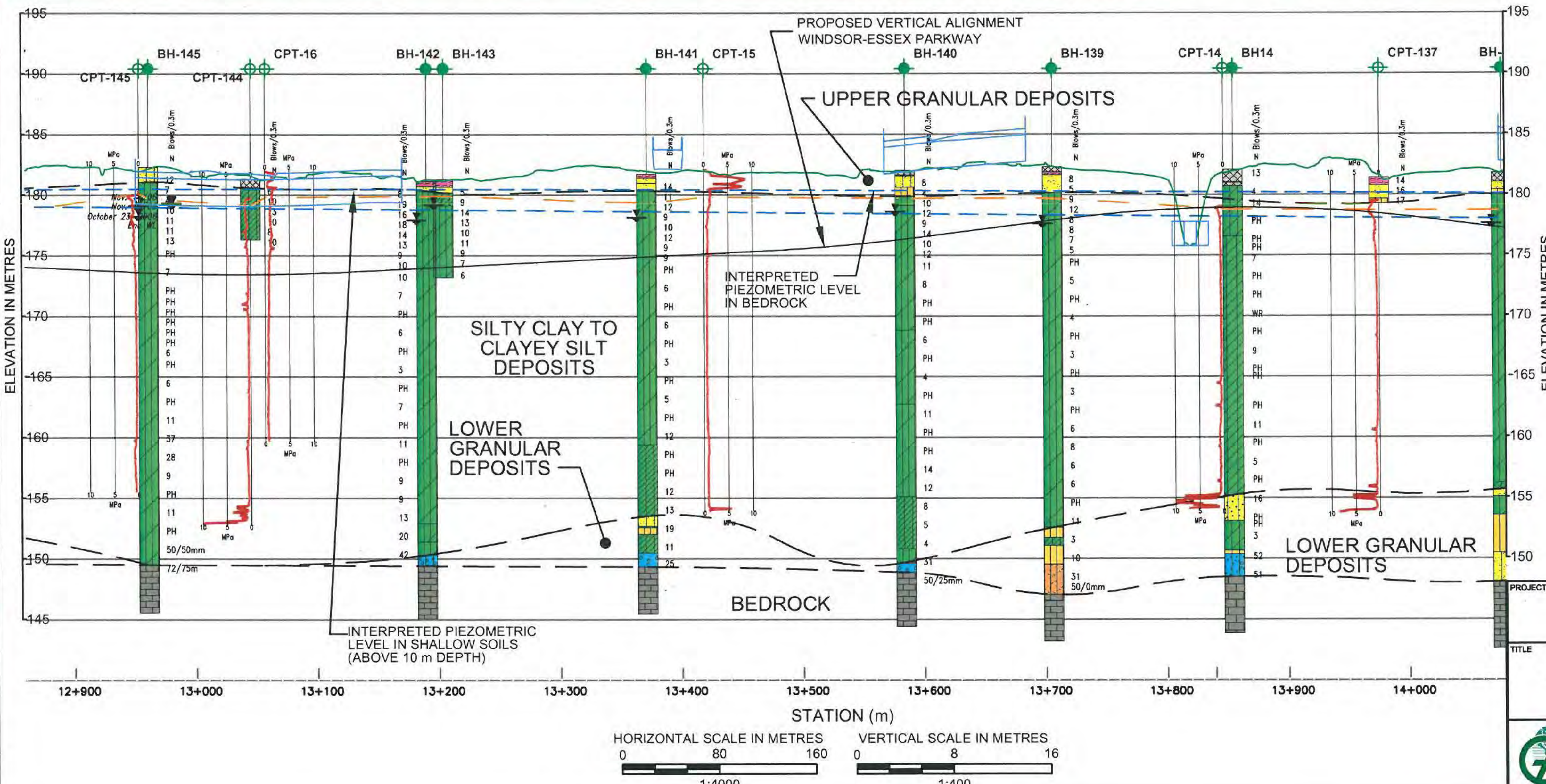

LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT

FOR DETAILS REFER TO RECORD OF BOREHOLES

- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY

CONE PENETRATION TIP RESISTANCE

MPa

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE

NOTES

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7. Borehole width in profile is not to scale.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE STRATIGRAPHIC PROFILE
STN 12+900 TO 14+000

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02051
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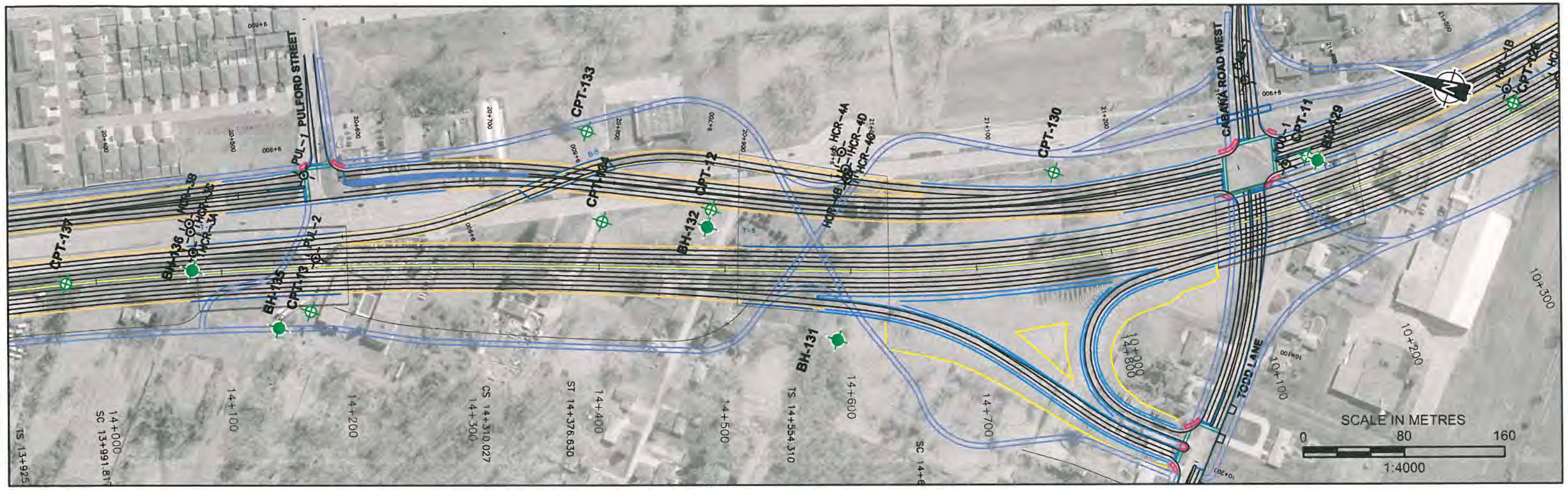
CHECK	SJB	Jun 26/09		

Drawing file: 0711302070-R02051.dwg Jun 15, 2009 - 3:51pm

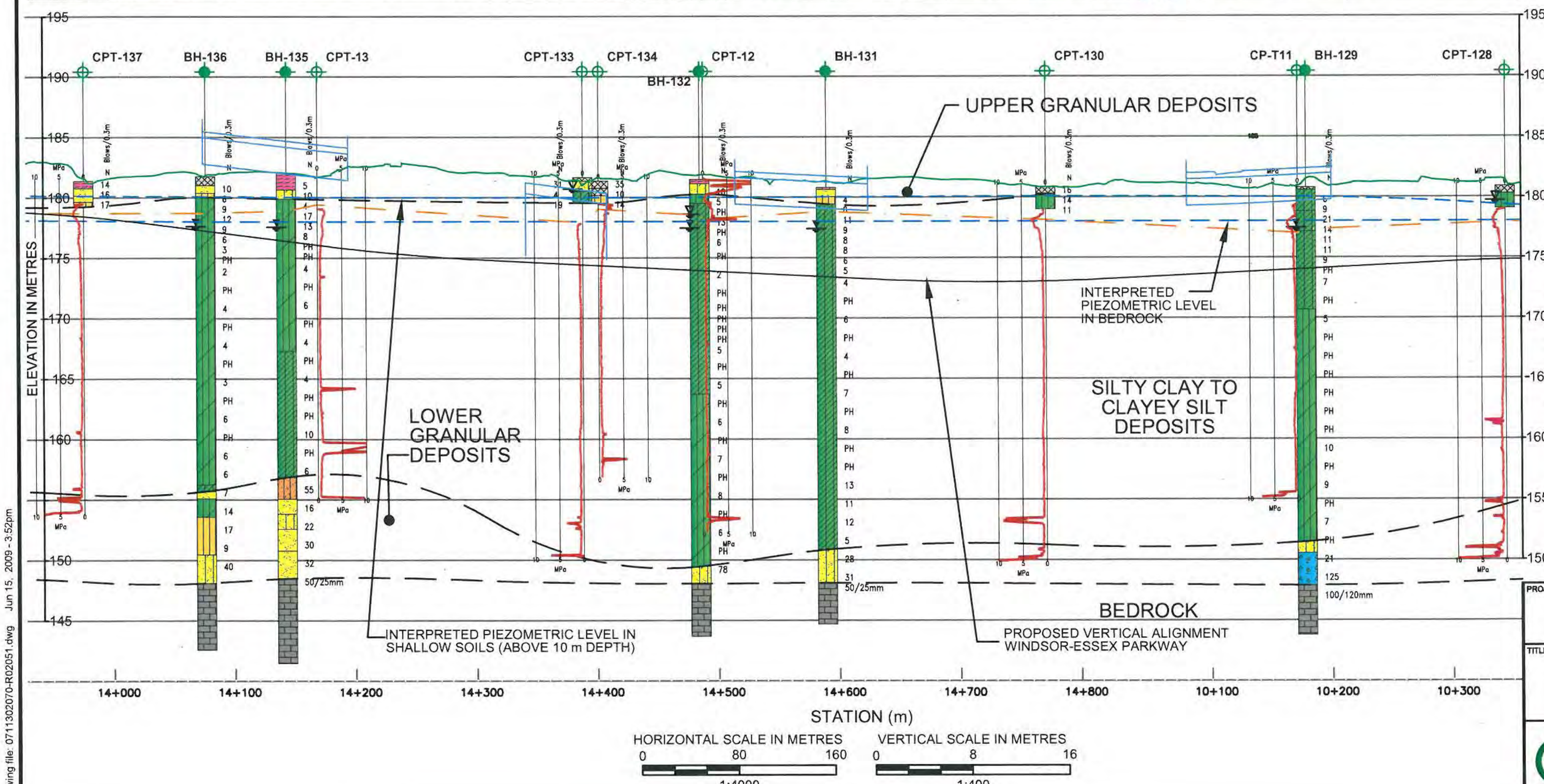


LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- SILT
- FILL
- SANDY SILT
- SAND
- CLAYEY SILT
- SILTY CLAY
- SAND AND GRAVEL
- SILTY SAND
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE



NOTES

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7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE **BASELINE STRATIGRAPHIC PROFILE**
STN 14+000 TO 14+800
STN 10+000 TO 10+300

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0	FILE No.	0711302070-R02051
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CHECK	SSB	Jun 26/09	5.1E

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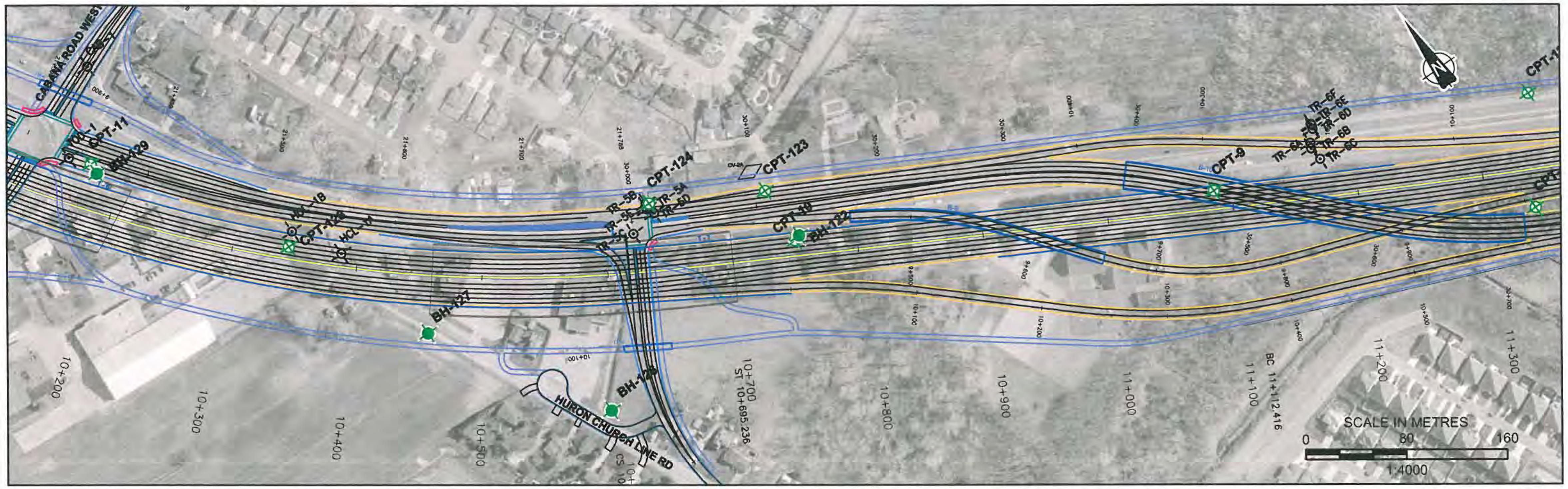


LEGEND

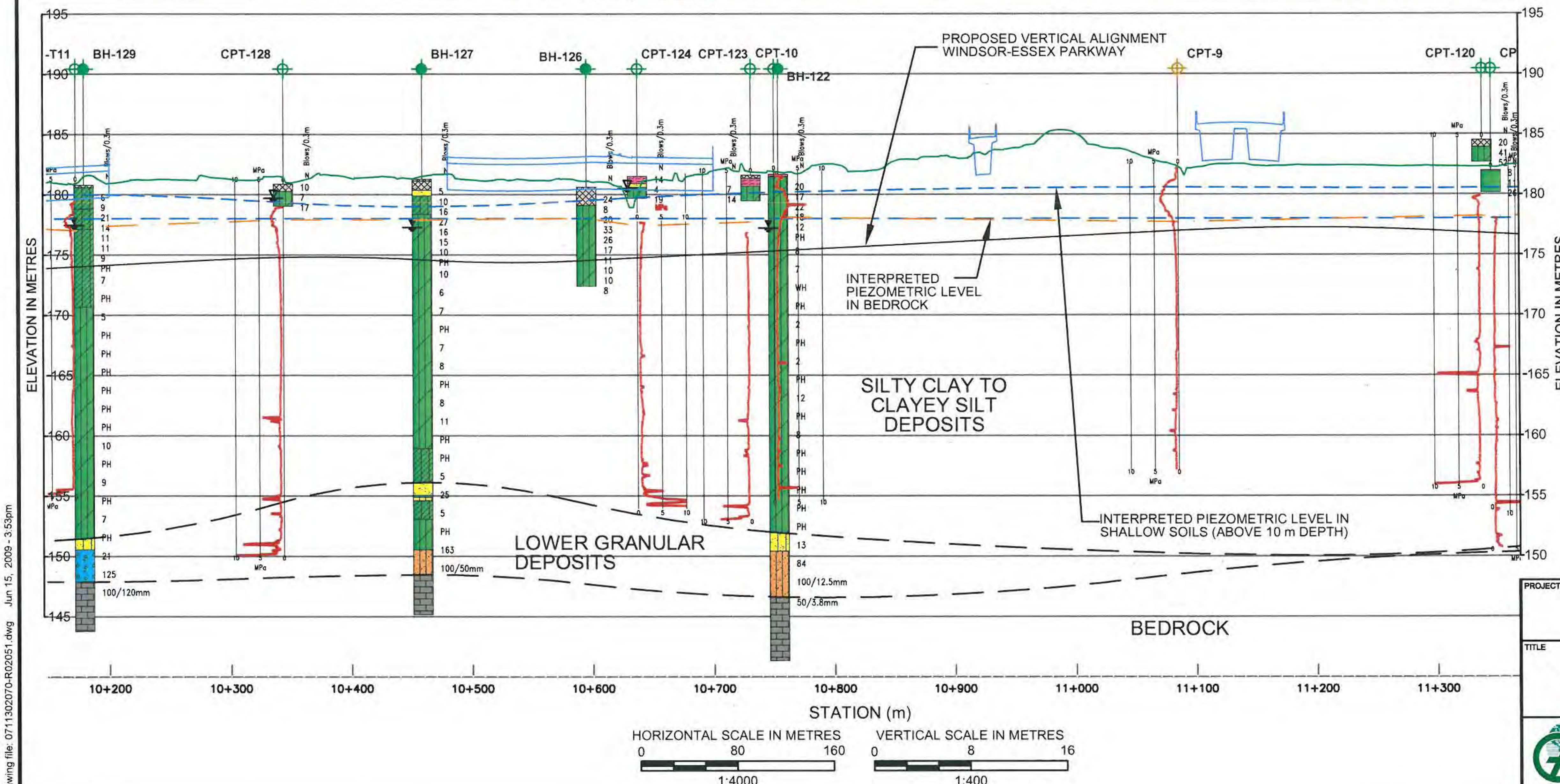
- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT

FOR DETAILS REFER TO RECORD OF BOREHOLES

- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

TOPSOIL/ ORGANICS	SILT
FILL	SANDY SILT
SAND	CLAYEY SILT
SILTY CLAY	SAND AND GRAVEL
SILTY SAND	SILTY SAND AND GRAVEL
	LIMESTONE/ DOLOSTONE



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7. Borehole width in profile is not to scale.

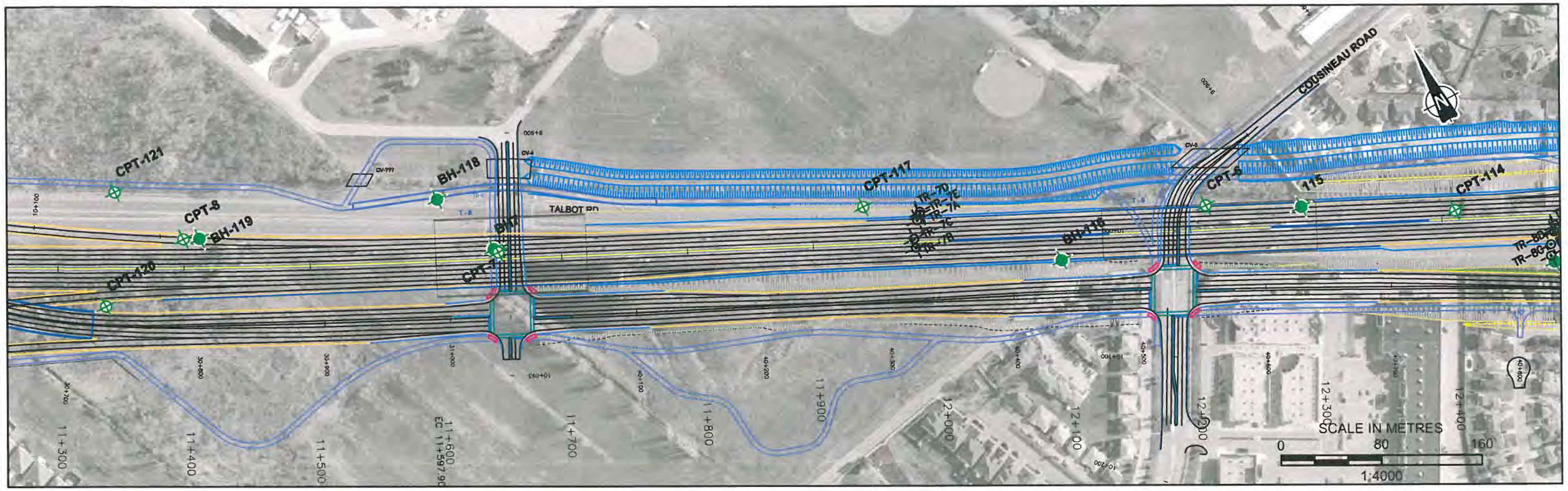
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

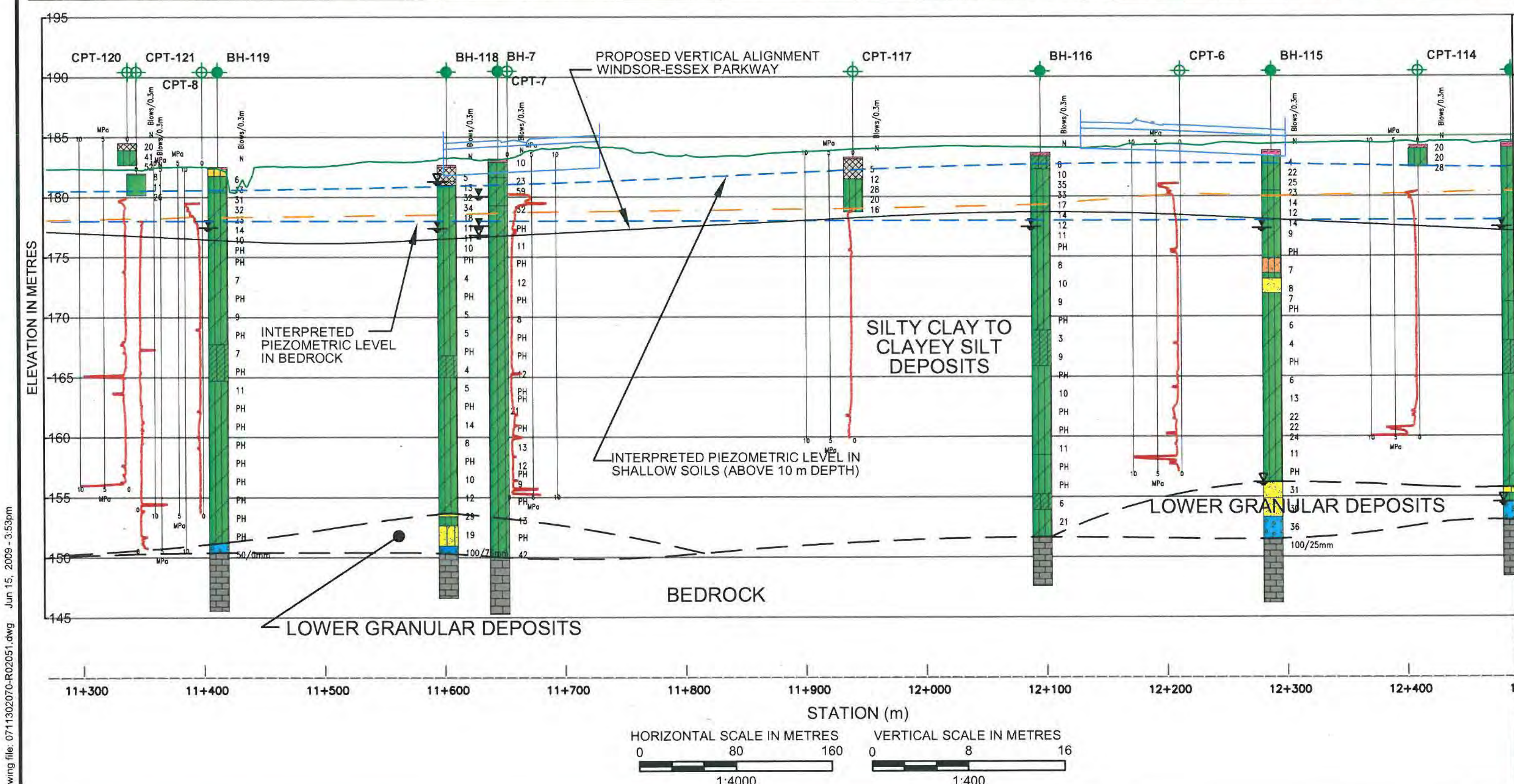
TITLE
BASELINE STRATIGRAPHIC PROFILE
STN 10+300 TO 11+300

PROJECT No. 07-1130-207-0		FILE No.	0711302070-R02051
		SCALE AS SHOWN	REV. 0
CADD	JDR/WDF	MAY 12/09	5.1F
CHECK	SJB	Jun 26/09	

Golder Associates
LONDON, ONTARIO

Drawing file: 0711302070-R02051.dwg Jun 15, 2009 - 3:53pm





LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE

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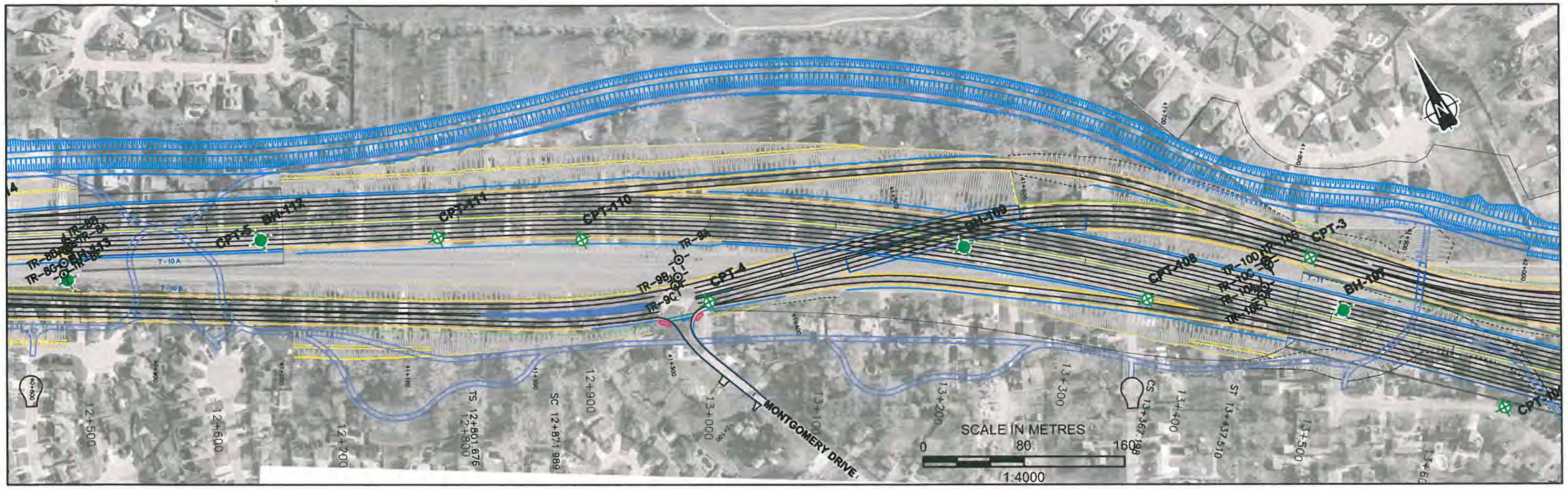
PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE STRATIGRAPHIC PROFILE STN 11+300 TO 12+500

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0	FILE No.	0711302070-R02051
		SCALE	AS SHOWN — REV. 0
CADD	JDR\WDF MAY 12/09		
CHECK	SJB Jun 26/09		5.1G

Drawing file: 0711302070-R02051.dwg Jun 15, 2009 - 3:53pm

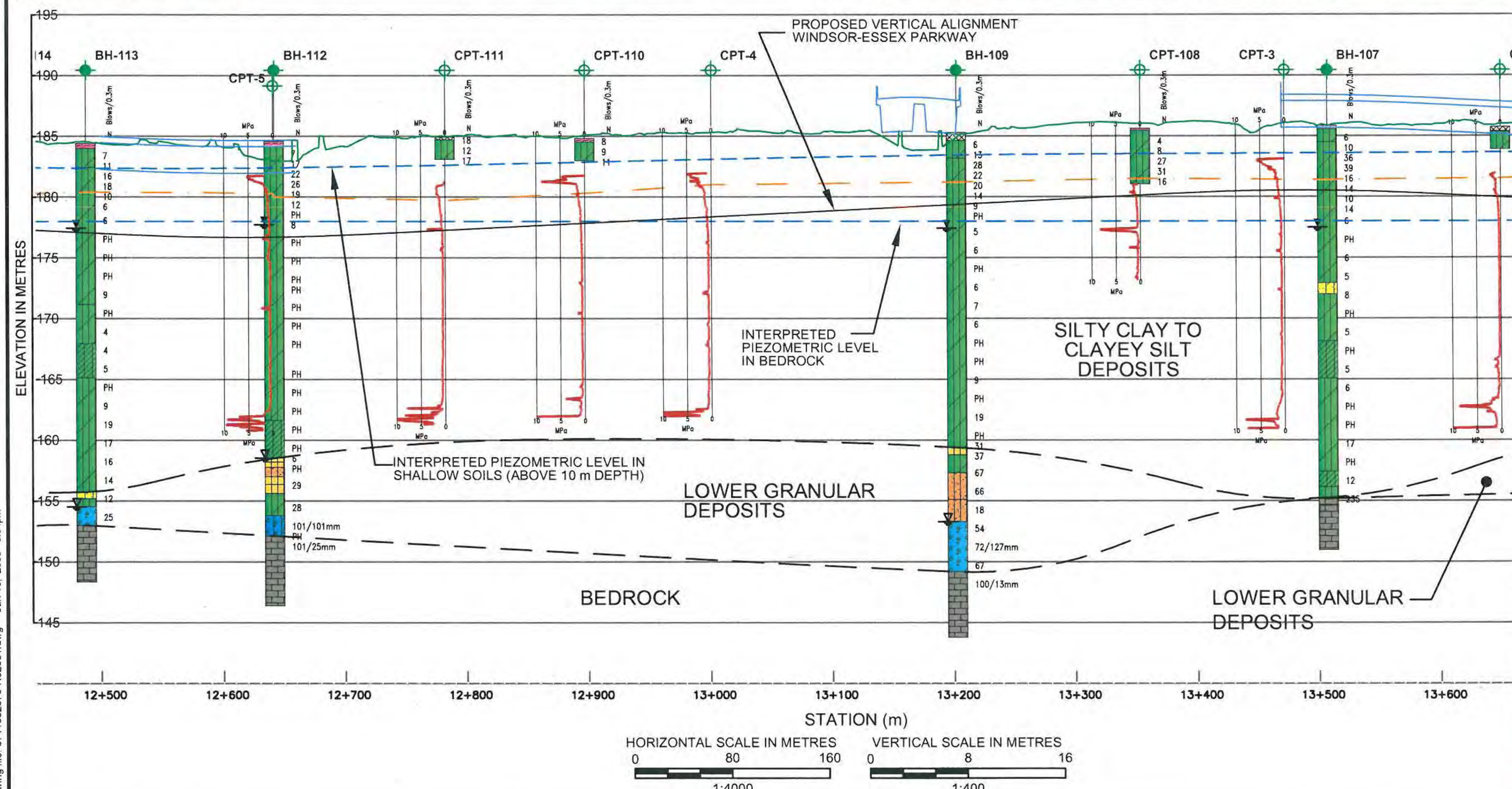


LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE



NOTES

1. This drawing is to be read in conjunction with the report titled "Subsurface Conditions Baseline Report, Windsor-Essex Parkway".
2. This interpreted stratigraphy is a simplification of the subsurface conditions. Detailed descriptions of the conditions encountered at the borehole locations are found on the records of boreholes in the geotechnical data and investigation reports referenced in this report.
3. Major soil deposit and rock formations are delineated by the boundary line identified above. The boundary established represents the baseline ground condition; however, variation in the boundaries from those illustrated must be anticipated both parallel and perpendicular to the section line.
4. The characteristics and variability anticipated within the major soil deposits are described in the text of this report. Significant layers, interlayers, and lenses within the major deposits are illustrated where identified on the borehole logs. The boundaries so illustrated are intended to highlight the variability within the deposits that will exhibit gradual transitions from one soil type to another. In addition, lenses and interlayers not detected by the subsurface investigation will be present between boreholes.
5. Construction equipment and procedures must be selected to accommodate variation in the deposit boundaries as well as variations within the deposits as described in the report text. Where precise determination of deposit boundaries and deposit variability are critical for safety and stability they should be verified by investigation during design and construction.
6. The ground surface profile and plan and profile of proposed construction are approximate and shown for illustrative purposes only. Refer to contract drawings for dimensions and limits of the work.
7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

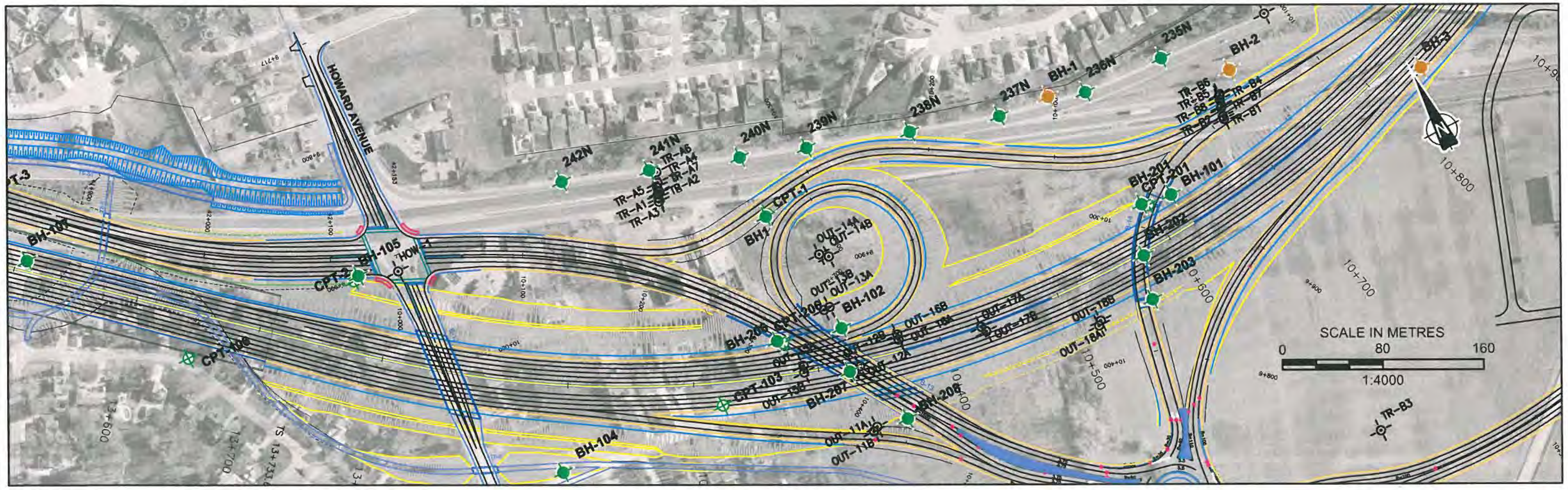
TITLE

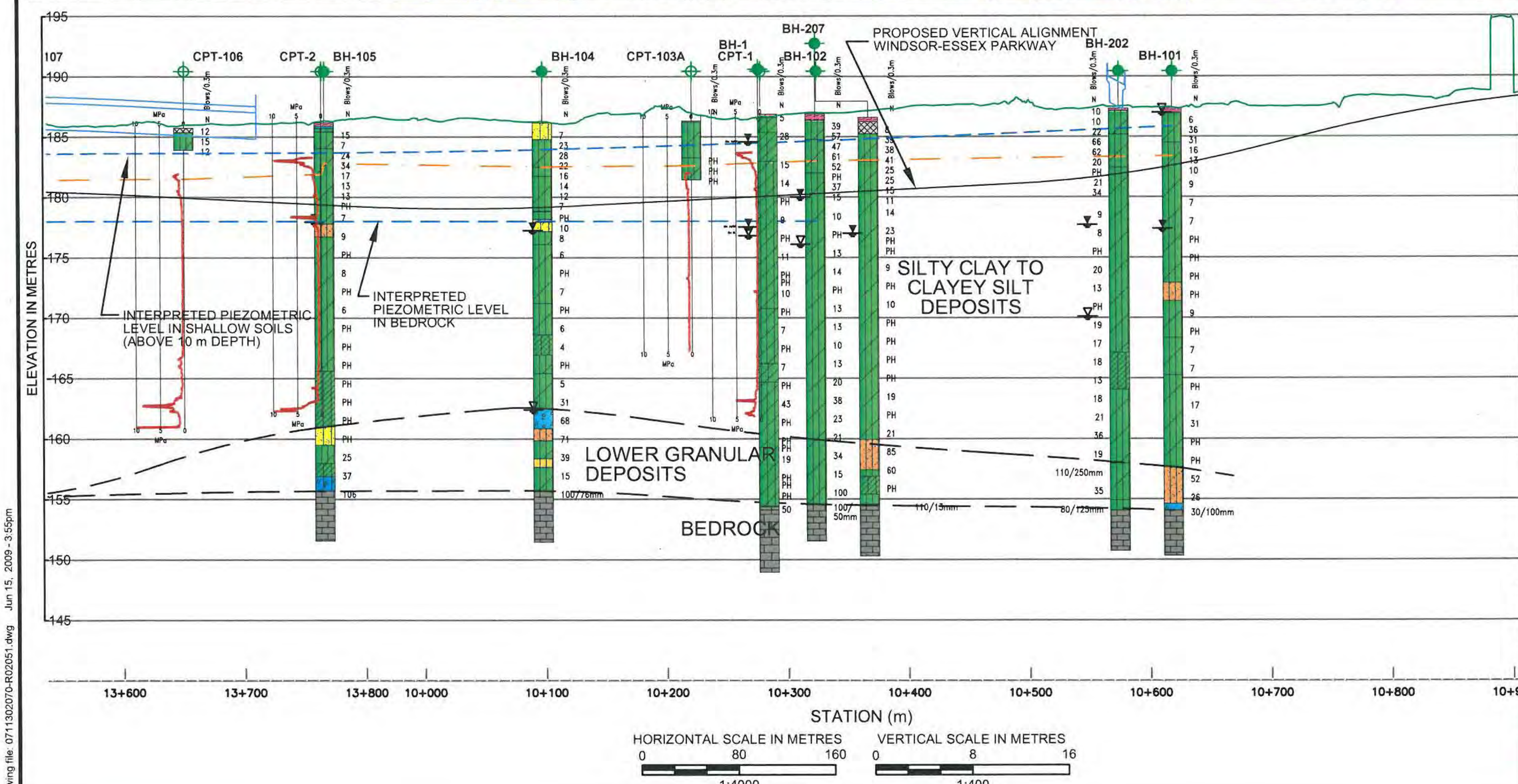
BASELINE STRATIGRAPHIC PROFILE
STN 12+500 TO 13+600

PROJECT No. 07-1130-207-0		FILE No. 0711302070-R02051	
CADD JDR/WDF	MAY 12/09	SCALE AS SHOWN	REV. 0
CHECK SJB	Jun 26/09	5.1H	

Golder Associates
LONDON, ONTARIO

Drawing file: 0711302070-R02051.dwg Jun 15, 2009 - 3:54pm





LEGEND

- BOREHOLE (Previous Investigation)
- PAVEMENT BOREHOLE (Current Investigation)
- CPT HOLE (Current Investigation)
- BOREHOLE (Current Investigation)
- PENETRATION RESISTANCE (Blows/0.3m)
- WATER LEVEL ENCOUNTERED
- WATER LEVEL
- STRATA PLOT
- FOR DETAILS REFER TO RECORD OF BOREHOLES
- DRY — BOREHOLE DRY DURING DRILLING
- BASELINE MAJOR DEPOSIT OR BEDROCK BOUNDARY
- GROUND SURFACE
- INTERPRETED PIEZOMETRIC WATER LEVEL
- BASELINE AVERAGE CRUST BOUNDARY
- CONE PENETRATION TIP RESISTANCE (MPa)

SIMPLIFIED STRATIGRAPHY

- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE/ DOLOSTONE

NOTES

1. This drawing is to be read in conjunction with the report titled "Subsurface Conditions Baseline Report, Windsor-Essex Parkway".
2. This interpreted stratigraphy is a simplification of the subsurface conditions. Detailed descriptions of the conditions encountered at the borehole locations are found on the records of boreholes in the geotechnical data and investigation reports referenced in this report.
3. Major soil deposit and rock formations are delineated by the boundary line identified above. The boundary established represents the baseline ground condition; however, variation in the boundaries from those illustrated must be anticipated both parallel and perpendicular to the section line.
4. The characteristics and variability anticipated within the major soil deposits are described in the text of this report. Significant layers, interlayers, and lenses within the major deposits are illustrated where identified on the borehole logs. The boundaries so illustrated are intended to highlight the variability within the deposits that will exhibit gradual transitions from one soil type to another. In addition, lenses and interlayers not detected by the subsurface investigation will be present between boreholes.
5. Construction equipment and procedures must be selected to accommodate variation in the deposit boundaries as well as variations within the deposits as described in the report text. Where precise determination of deposit boundaries and deposit variability are critical for safety and stability they should be verified by investigation during design and construction.
6. The ground surface profile and plan and profile of proposed construction are approximate and shown for illustrative purposes only. Refer to contract drawings for dimensions and limits of the work.
7. Borehole width in profile is not to scale.

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

BASELINE STRATIGRAPHIC PROFILE
STN 13+600 TO 13+848
STN 10+000 TO 10+900

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02051
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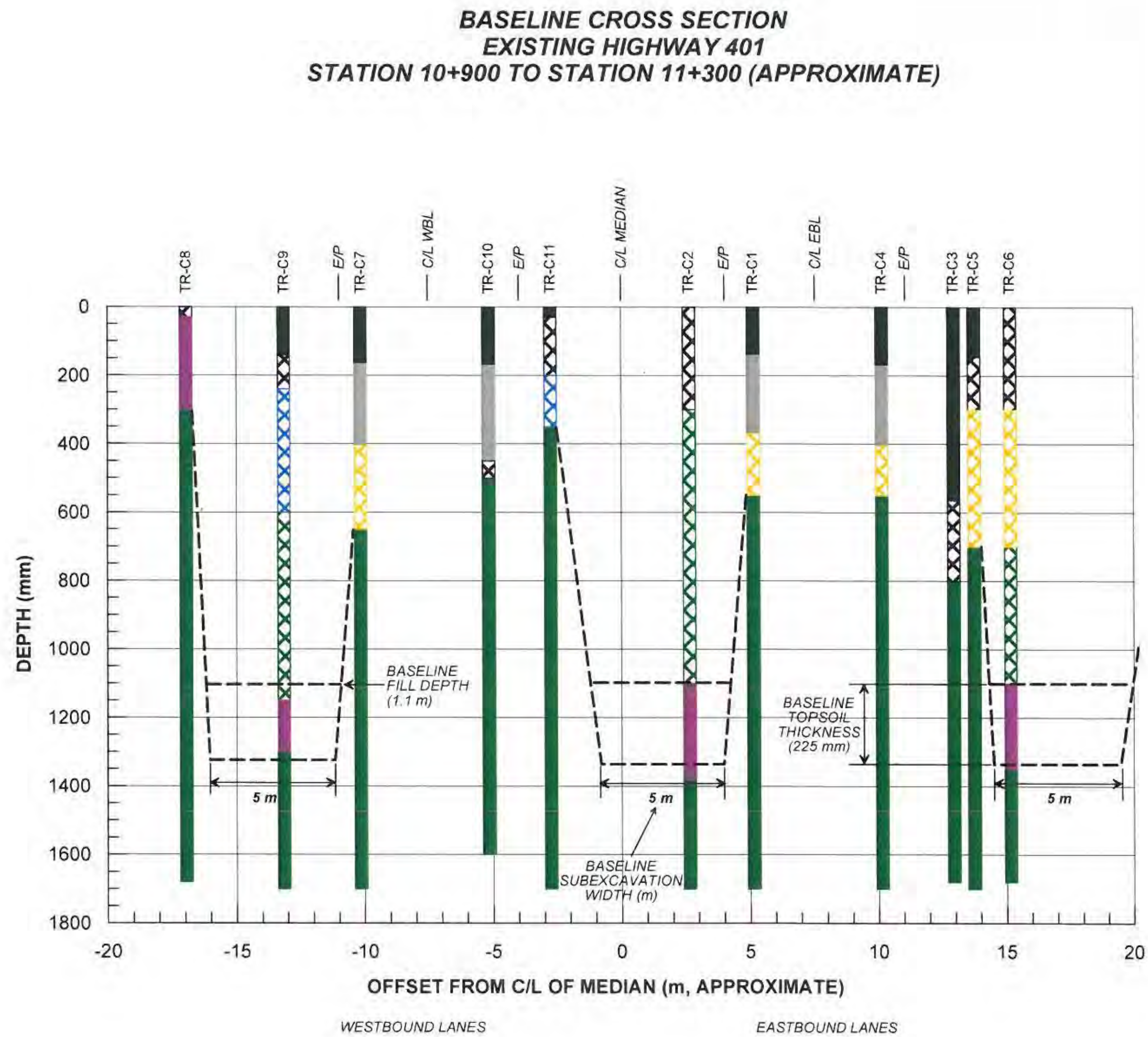
CADD	JDR\WDF	MAY 12/09	5.1I	
CHECK	SDR	Jun 24/09		

Drawing file: 0711302070-R02051.dwg Jun 15, 2009 - 3:55pm



NOTES:
1. E/P - EDGE OF PAVEMENT.
2. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
3. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

- ASPHALT
- CONCRETE
- GRANULAR BASE
- SAND FILL
- SAND & GRAVEL FILL
- CLAYEY FILL
- CLAYEY SILT/SILTY CLAY
- TOPSOIL

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE **BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS EXISTING HIGHWAY 401**

Golder Associates LONDON, ONTARIO

PROJECT No.		05-1140-003	FILE No.	DRIC PAVT 5-2-A
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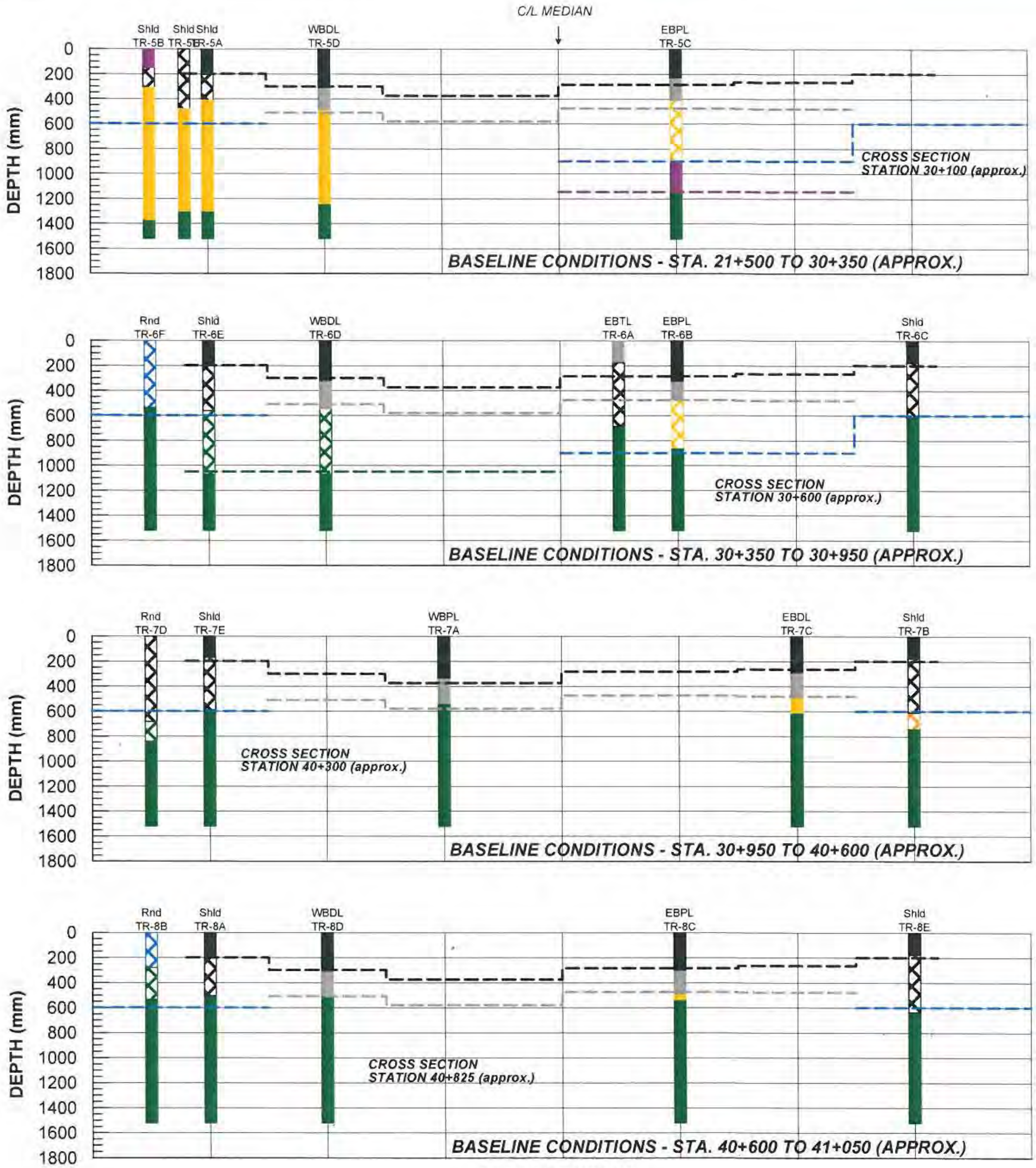
DRAWN	MEB	FEB 8-09		
CHECK	SJB	Jun 26/09	FIGURE 5.2A	



CROSS SECTIONS

WESTBOUND ***EASTBOUND***

- ASPHALT
- CONCRETE
- GRANULAR BASE
- SAND FILL
- TOPSOIL
- SAND & GRAVEL FILL
- CLAYEY FILL
- SAND
- CLAYEY SILT/SILTY CLAY

BASELINE DEPTHS

- ASPHALT
- CONCRETE
- GRANULAR FILL
- CLAYEY FILL
- BURIED TOPSOIL

EB	EASTBOUND
WB	WESTBOUND
DL	DRIVING LANE
PL	PASSING LANE
TL	TURNING LANE
Rnd	ROUNDING
Shld	SHOULDER

NOTES:
1. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
2. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS EXISTING HIGHWAY 3/TALBOT ROAD

Golder Associates
LONDON, ONTARIO

PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-B	
		SCALE	AS SHOWN	REV. 0
DRAWN	MEB	JUN 17-09		
CHECK	SJB	Jun 26/09	**FIGURE 5.2B**	

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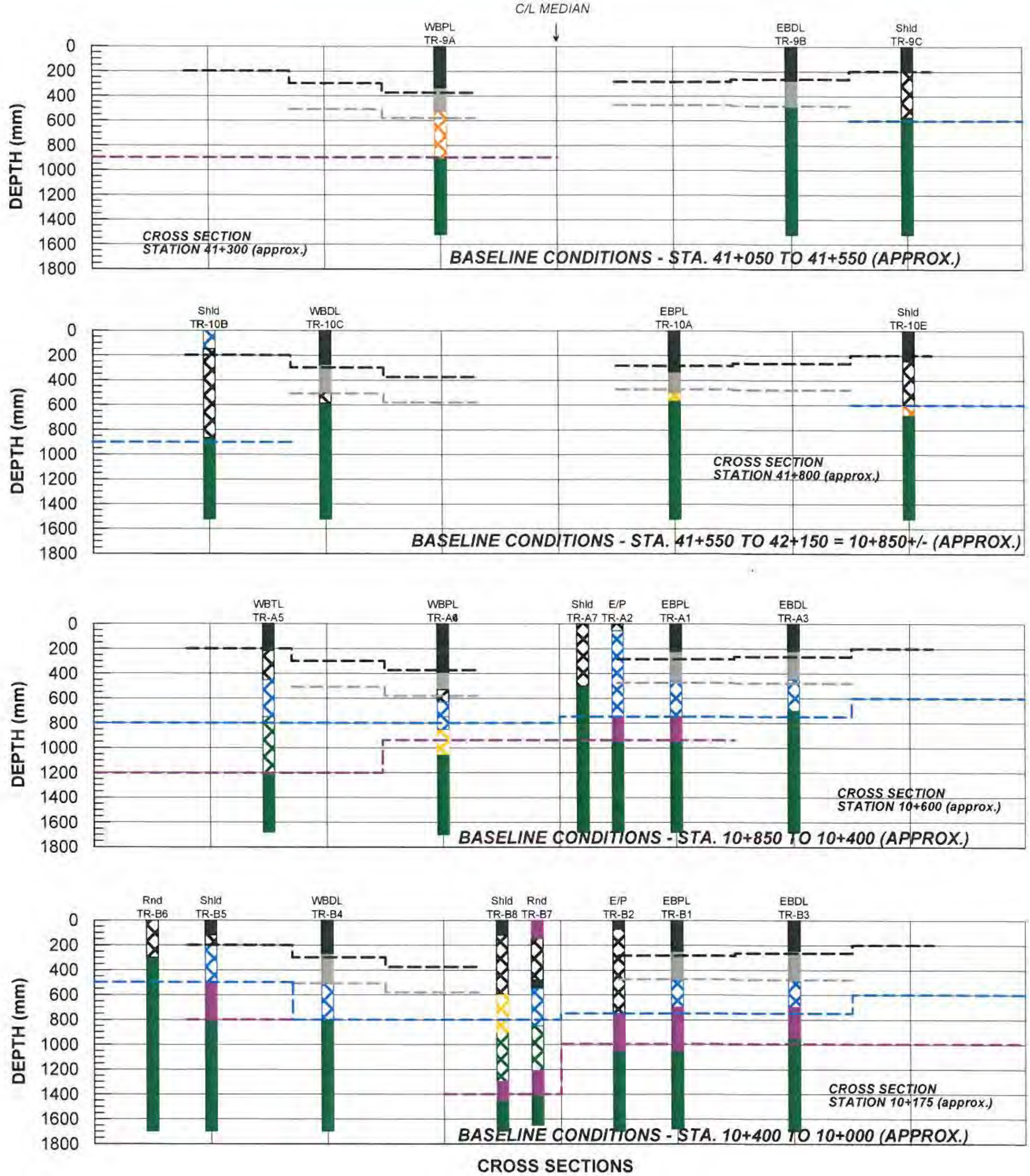

CROSS SECTIONS

WESTBOUND ***EASTBOUND***

NOTES:
1. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
2. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

ASPHALT
CONCRETE
GRANULAR BASE
SILTY SAND FILL
SAND & GRAVEL FILL
SAND FILL
CLAYEY FILL
CLAYEY SILT/SILTY CLAY
TOPSOIL

BASELINE DEPTHS

ASPHALT
CONCRETE
GRANULAR FILL
BURIED TOPSOIL AND FILL

EB	EASTBOUND
WB	WESTBOUND
DL	DRIVING LANE
PL	PASSING LANE
TL	TURNING LANE
Rnd	ROUNDING
Shld	SHOULDER

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE PAVEMENT AND SHALLOW SUBSURACE CONDITIONS EXISTING HIGHWAY 3/TALBOT ROAD

Golder Associates
LONDON, ONTARIO

PROJECT No.		05-1140-003	FILE No.	DRIC PAVT 5-2-C
			SCALE AS SHOWN	REV. 0
DRAWN	MEB	JUN 17-09	**FIGURE 5.2C**	
CHECK	SJB	Jun 26/09		

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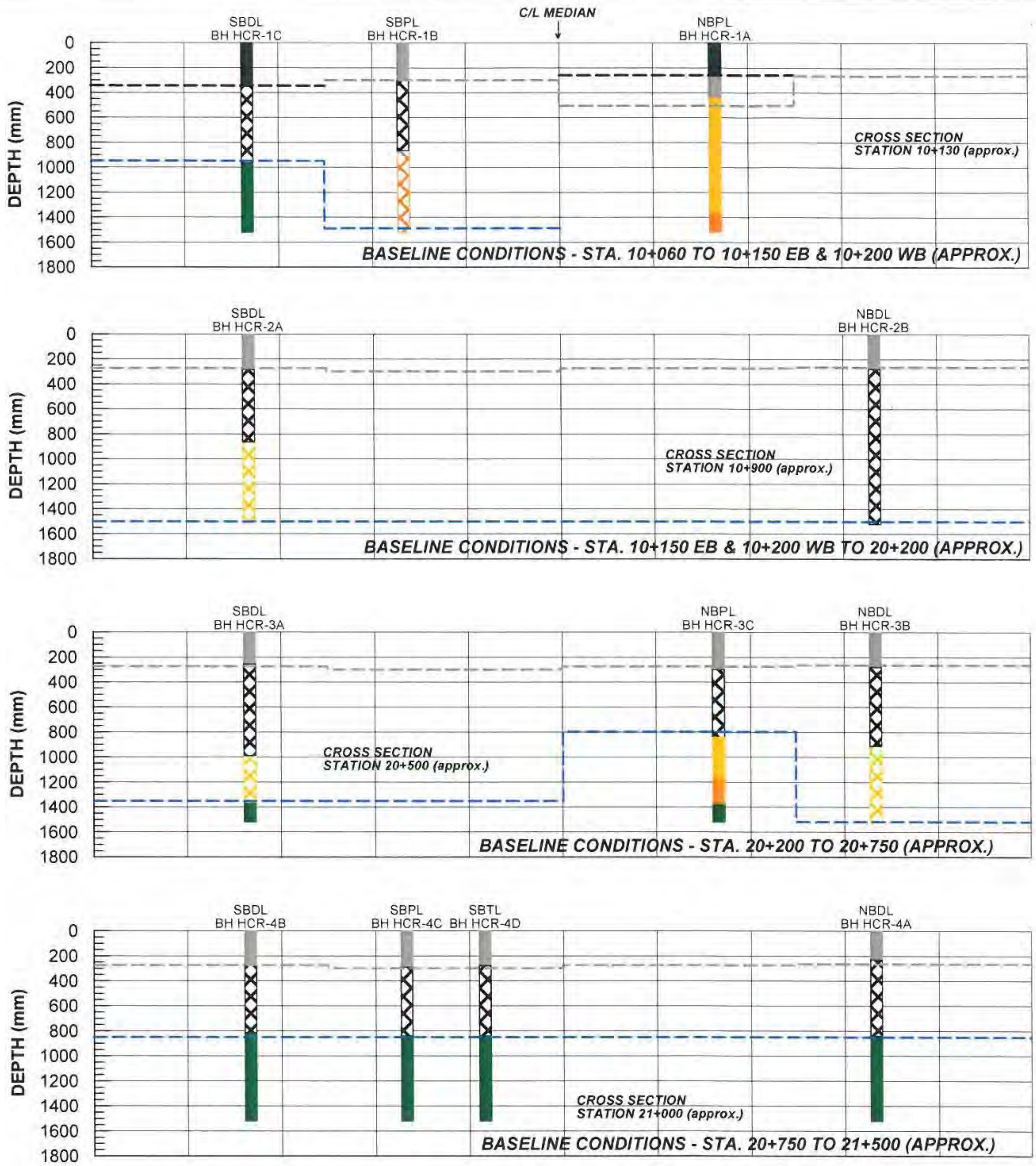

CROSS SECTIONS

SOUTHBOUND

NORTHBOUND

NOTES:
1. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
2. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

ASPHALT
CONCRETE
GRANULAR BASE
SILTY SAND FILL
SAND FILL
SAND
SILTY SAND
CLAYEY SILT/SILTY CLAY

BASELINE DEPTHS
ASPHALT
CONCRETE
GRANULAR FILL

SB SOUTHBOUND
ND NORTHBOUND
DL DRIVING LANE
PL PASSING LANE
TL TURNING LANE

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS EXISTING HURON CHURCH ROAD

Golder Associates
LONDON, ONTARIO

PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-D
		SCALE AS SHOWN	REV. 0
DRAWN MEB	JUN 17-09	**FIGURE 5.2D**	
CHECK SJB	Jun 26/09		

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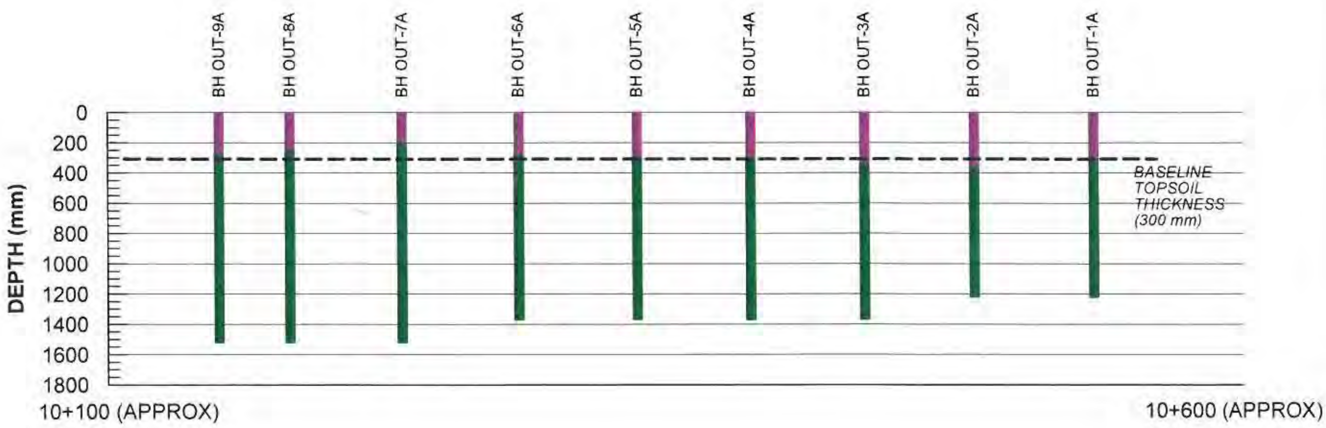
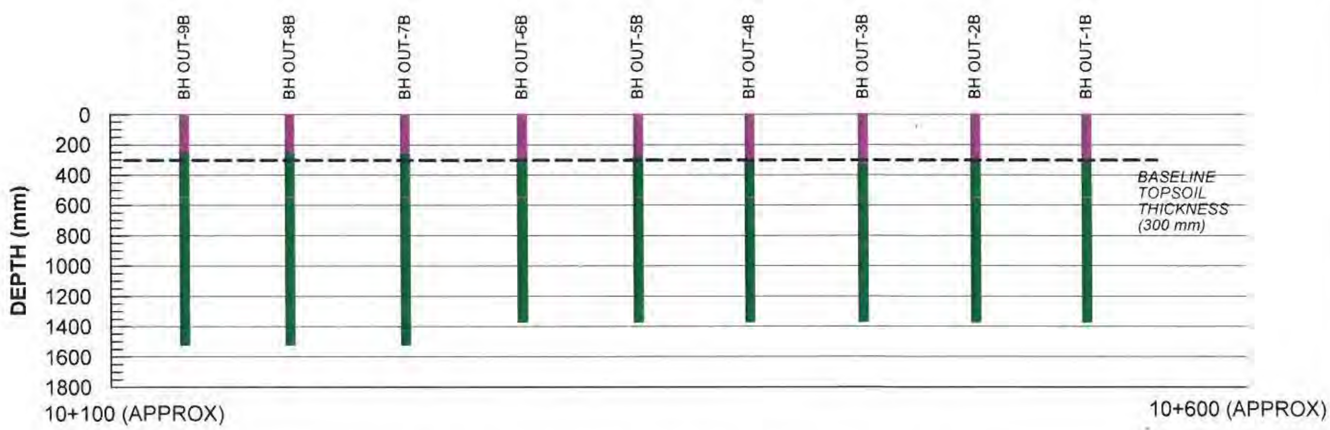
BASELINE PROFILE
PROPOSED HIGHWAY 3 REALIGNMENT
STATION 10+100 TO STATION 10+600 (APPROXIMATE)





FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
TWO PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION

NOTES:
1. TWO PLOTS SHOWN FOR CLARITY.
2. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
3. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

TOPSOIL
CLAYEY SILT/SILTY CLAY

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE **BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED HIGHWAY 3 REALIGNMENT**

Golder Associates
LONDON, ONTARIO

PROJECT No.		05-1140-003	FILE No.	DRIC PAVT 5-2-E
			SCALE AS SHOWN	REV. 0
DRAWN	MEB	JUN 17-09	**FIGURE 5.2E**	
CHECK	SSB	Jun 26/09		

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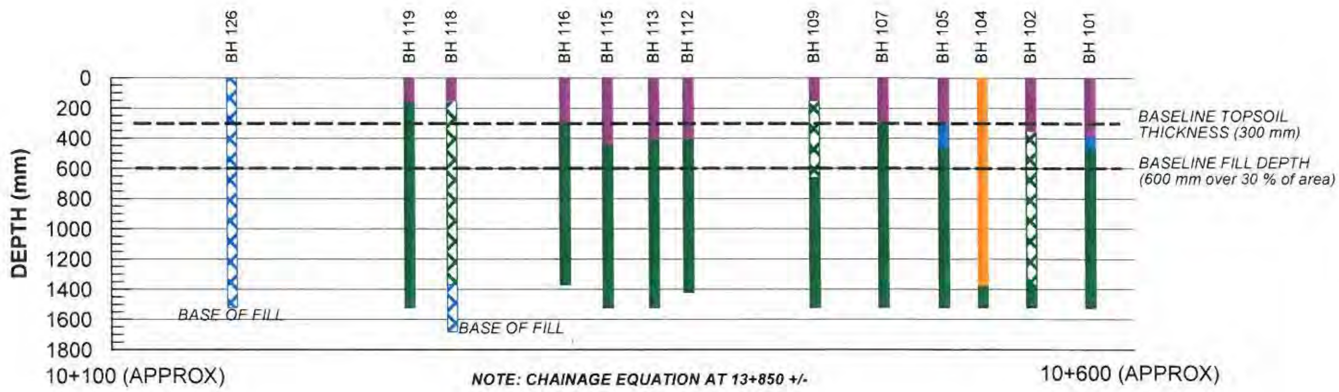
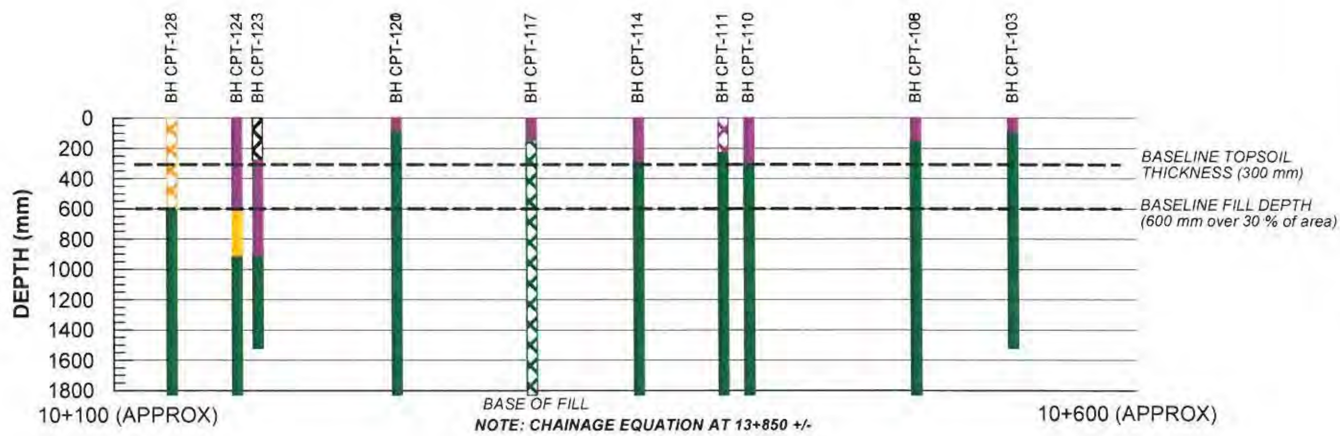
BASELINE PROFILE
PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS
HIGHWAY 3/TALBOT ROAD CORRIDOR





FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
TWO PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION

NOTES:
1. TWO PLOTS SHOWN FOR CLARITY.
2. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
3. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

SILTY SAND FILL
TOPSOIL FILL
GRANULAR FILL
TOPSOIL
SAND
CLAYEY FILL
SAND & GRAVEL FILL
CLAYEY SILT/SILTY CLAY
SAND & GRAVEL
SILTY SAND

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE **BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS HIGHWAY 3/TALBOT ROAD CORRIDOR**

Golder Associates
LONDON, ONTARIO

PROJECT No.		05-1140-003	FILE No.	DRIC PAVT 5-2-F
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DRAWN	MEB	JUN 17-09	**FIGURE 5.2F**	
CHECK	SJB	Jn 26/09		

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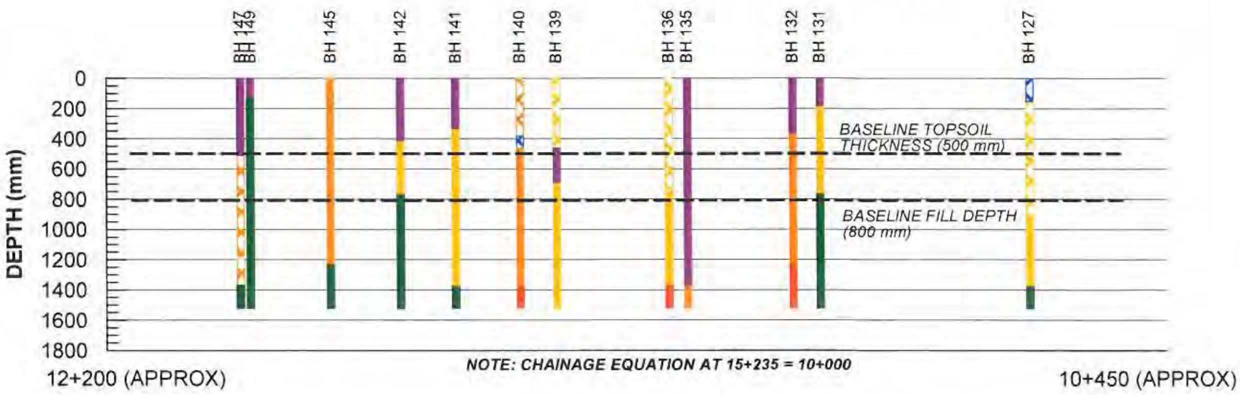
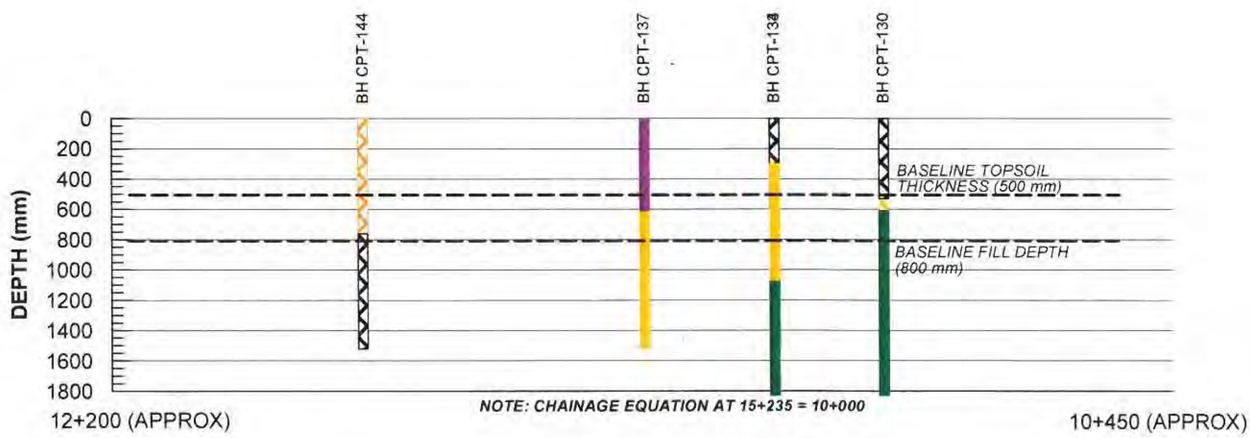
BASELIINE PROFILE
PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS
HURON CHURCH ROAD CORRIDOR





FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
TWO PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION

NOTES:
1. TWO PLOTS SHOWN FOR CLARITY.
2. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
3. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

- SAND FILL
- SILTY SAND FILL
- GRANULAR FILL
- TOPSOIL
- SAND
- CLAYEY SILT/SILTY CLAY
- SAND & GRAVEL FILL
- SILTY SAND/SANDY SILT
- SILT

PROJECT SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE **BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS HURON CHURCH ROAD CORRIDOR**

Golder Associates
LONDON, ONTARIO

PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-G
		SCALE AS SHOWN	REV. 0
DRAWN MEB	JUN 17-09		
CHECK SAMS	Jun 24/09	**FIGURE 5.2G**	

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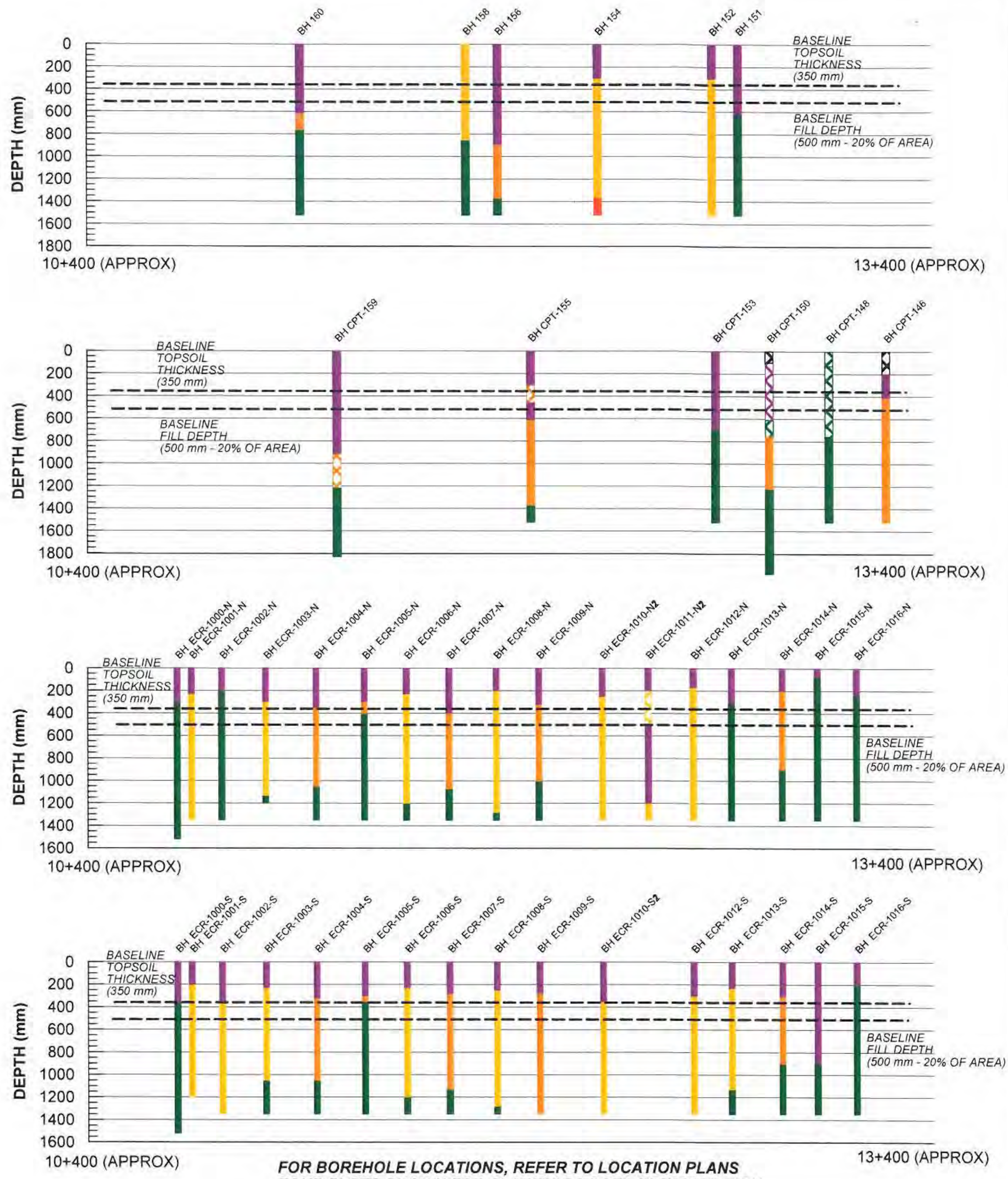

FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
FOUR PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION

TOPSOIL FILL	TOPSOIL
CLAYEY FILL	SAND
SILTY SAND FILL	SILT
GRANULAR FILL	CLAYEY SILT/SILTY CLAY
SAND FILL	SILTY SAND/SANDY SILT

NOTES:
1. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
2. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS E.C. ROW EXPRESSWAY CORRIDOR

Golder Associates
LONDON, ONTARIO

PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-H
		SCALE	AS SHOWN REV. 0
DRAWN MEB	JUN 17-09	**FIGURE 5.2H**	
CHECK SSB	Jun 26/09		

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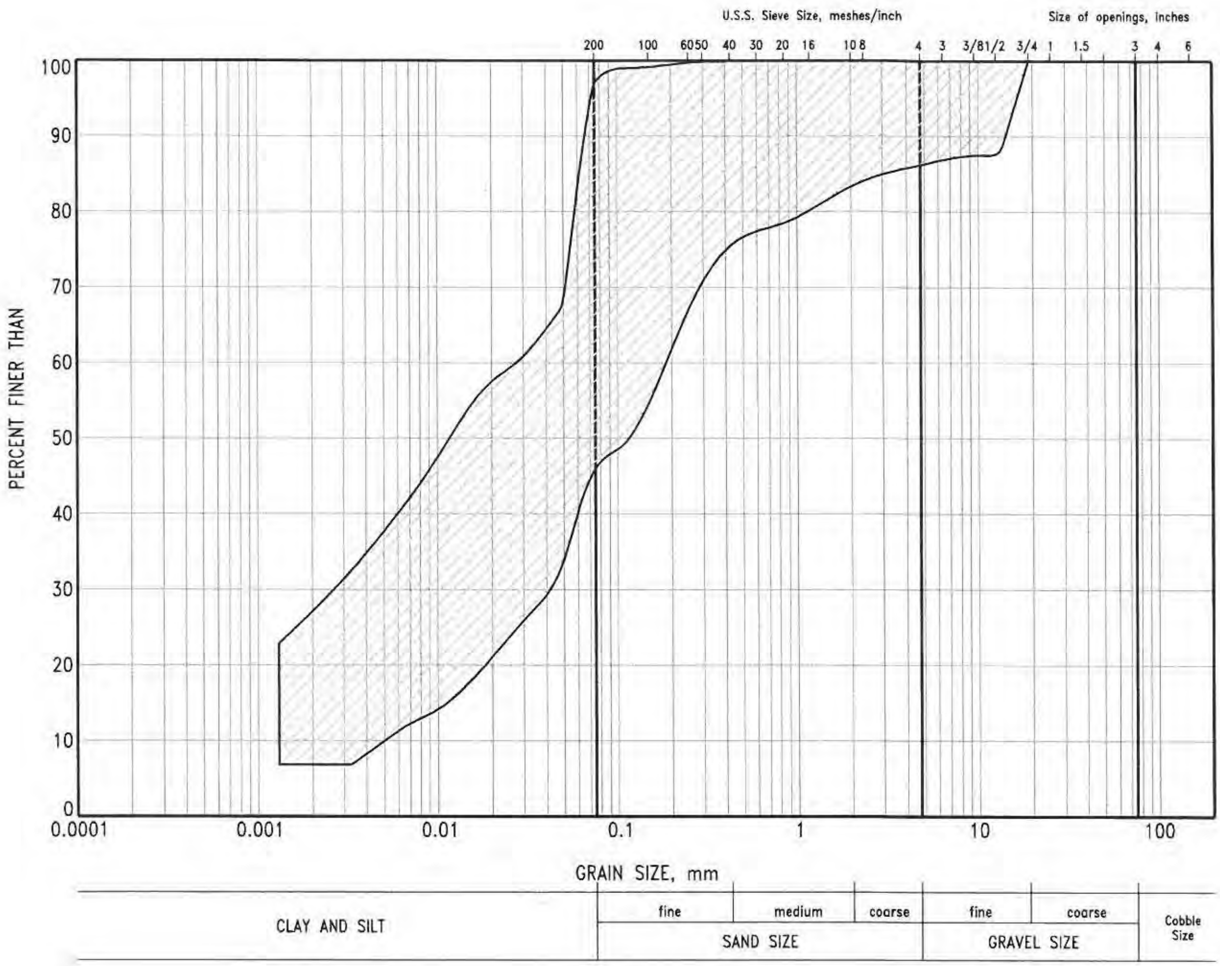


The grain size distribution envelope shown above is based on all grain size distribution analyses conducted on samples obtained from the Upper Granular Deposits. For individual test results refer to the Geotechnical Data Report referenced in section 3.1. The samplers used for the explorations limit the maximum particle size that can be sampled and tested to about 40mm. Larger particles are known to be present in the deposit as discussed in the report text.

PROJECT

SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

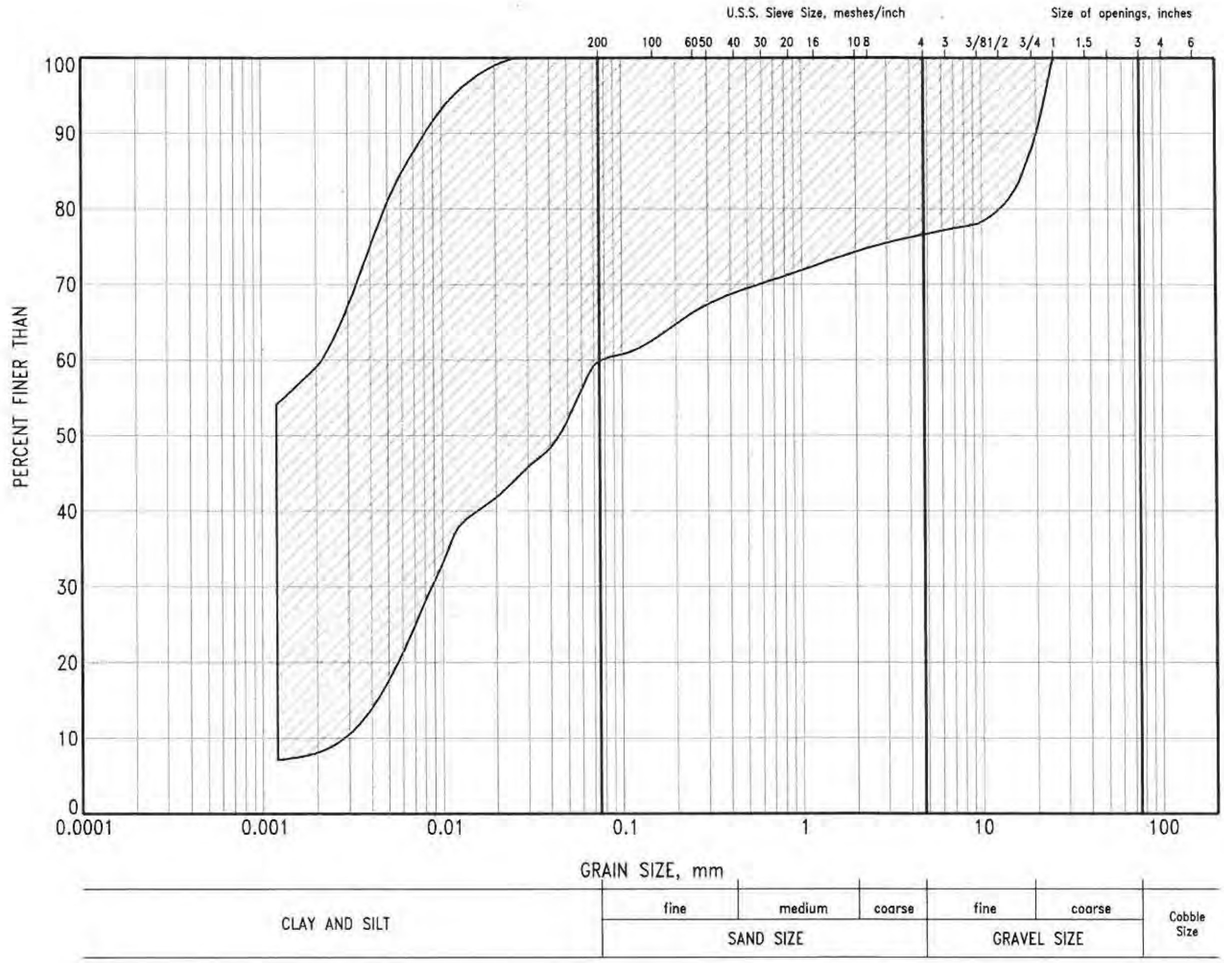
TITLE

GRAIN SIZE DISTRIBUTION ENVELOPE
UPPER GRANULAR DEPOSITS

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02053
			SCALE AS SHOWN	REV. 0
CADD	LMK	June 17/09		
CHECK	SJB	Jun 24/09	FIGURE 5.3	

Drawing file: 0711302070-R02053.dwg Jun 18, 2009 - 5:28pm



The grain size distribution envelope shown above is based on all grain size distribution analyses conducted on samples obtained from the Silt and Clay Deposits. For individual test results refer to the Geotechnical Data Report referenced in section 3.1. The samplers used for the explorations limit the maximum particle size that can be sampled and tested to about 40mm. Larger particles are known to be present in the deposit as discussed in the report text.

PROJECT

SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

GRAIN SIZE DISTRIBUTION ENVELOPE
CLAYEY SILT TO SILTY CLAY DEPOSITS

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02054
			SCALE AS SHOWN	REV. 0
CADD	LMK	June 17/09		
CHECK	SJB	Jun 26/09	FIGURE 5.4	

Drawing file: 0711302070-R02054.dwg Jun 18, 2009 - 5:16pm

Natural fissuring of silty clay to clayey silt deposit exposed during excavation in Sarnia, Ontario.

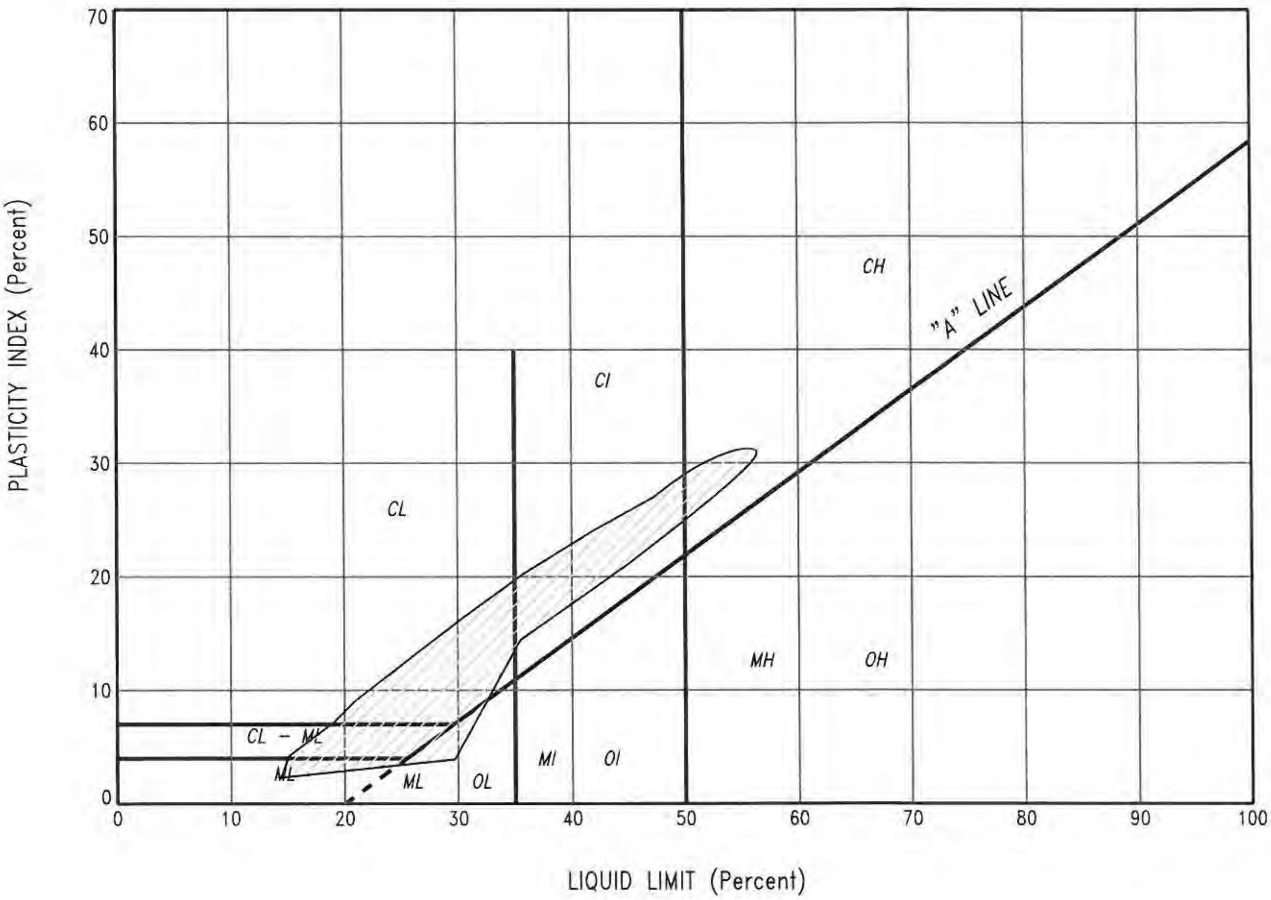
NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
FISSURING OF CLAYEY SILT TO SILTY CLAY DEPOSITS

Golder Associates	PROJECT No. 07-1130-2070/05-1140-003		FILE No.	0711302070-R02055
			SCALE	NA REV.
	CADD	SJB	MAY 2009	
	CHECK			5.5



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

Drawing file: 0711302070-R02056.dwg Jun 18, 2009 - 5:29pm

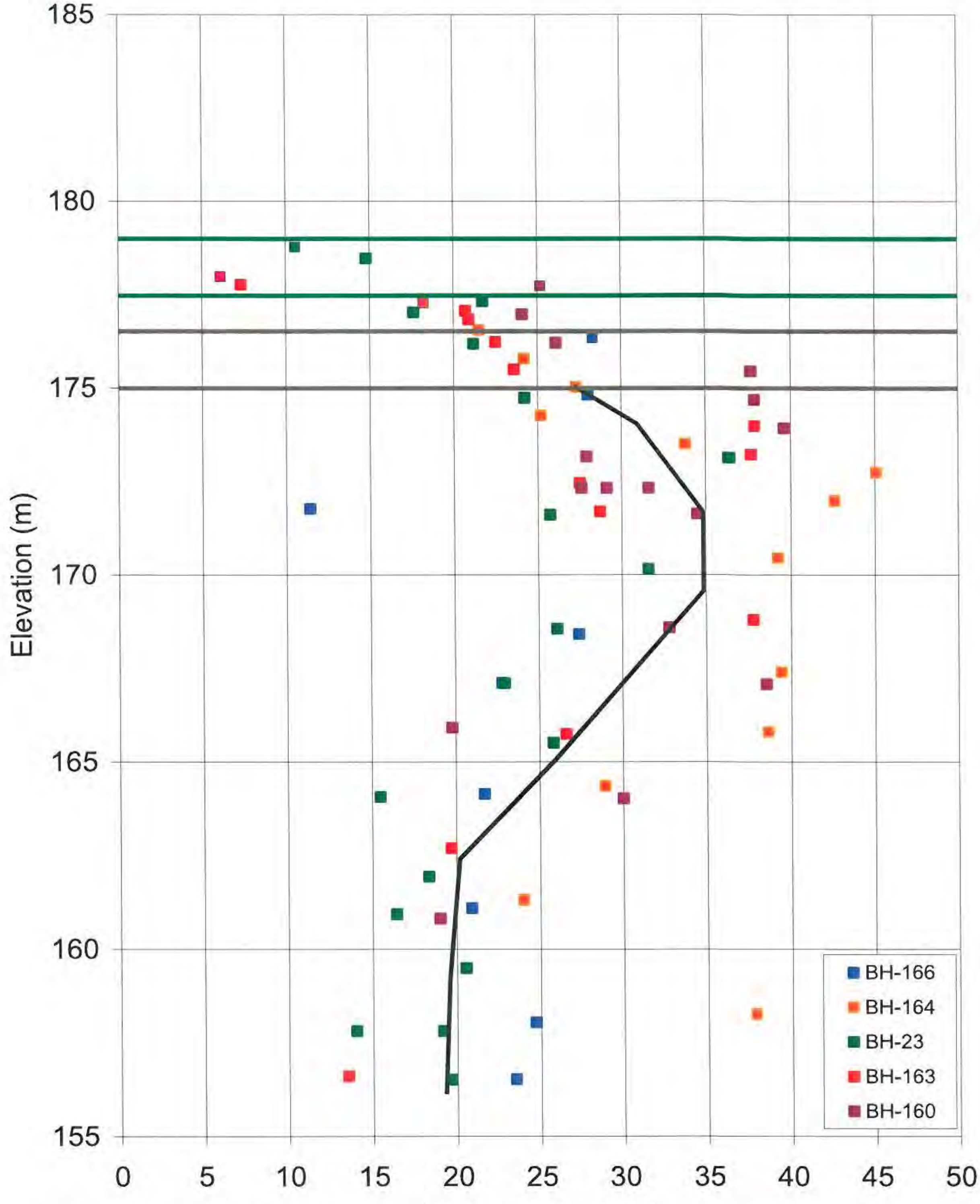
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

ATTERBERG LIMITS ENVELOPE

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02056		
DESIGN			SCALE	AS SHOWN	REV.	0
CADD	LMK	June 17/09				
CHECK	SJB	Jun 26/09	FIGURE 5.6			
REVIEW						



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

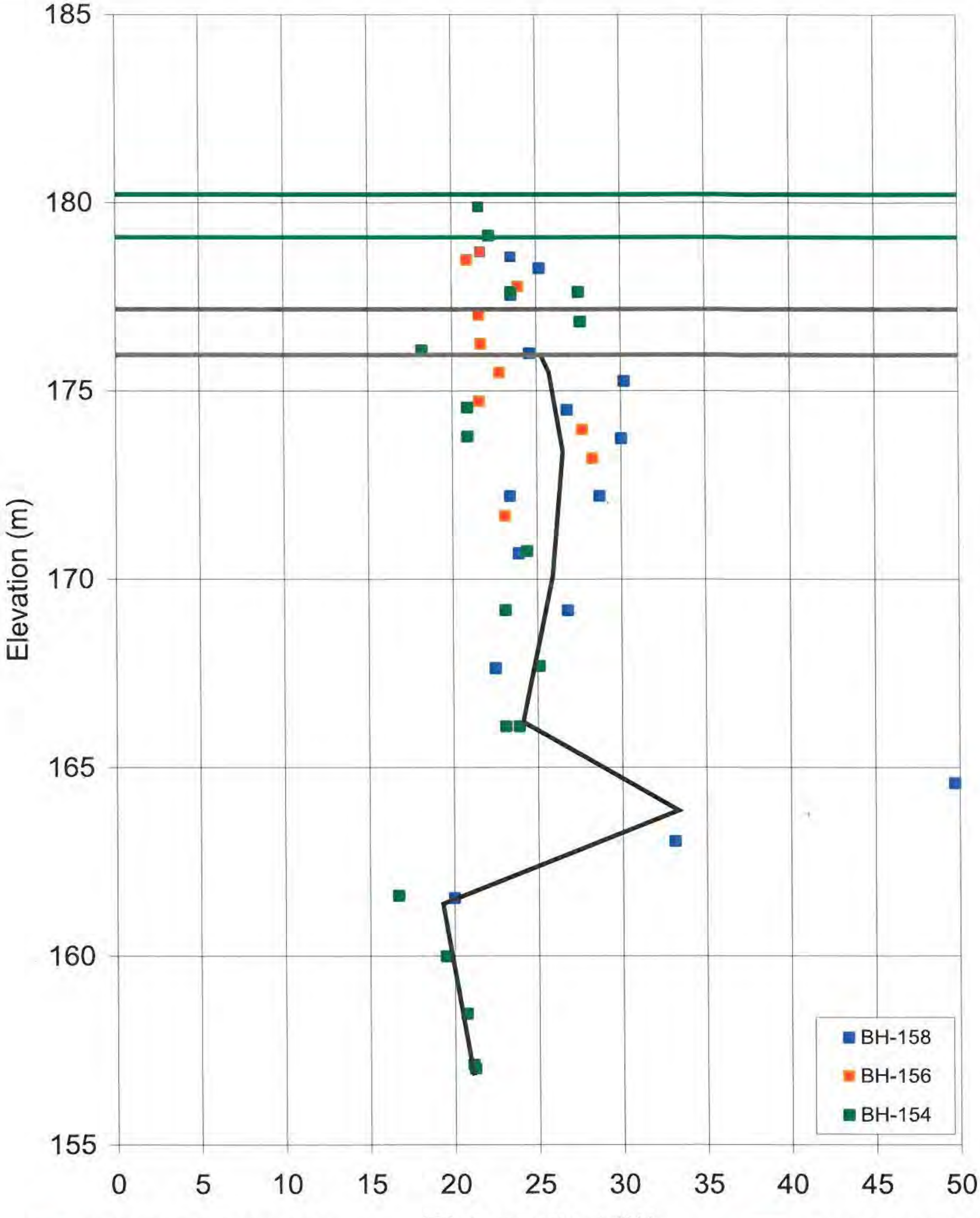
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE PROFILE PROVIDED ABOVE IS CONSIDERED TO BE REPRESENTATIVE OF THE 50TH PERCENTILE VALUES AND THE 90TH AND 10TH PERCENTILE VALUES ARE TO BE TAKEN AS THE 50TH PERCENTILE WATER CONTENT PLUS OR MINUS A WATER CONTENT OF 5%, RESPECTIVELY

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 9+900 to 11+100

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
			SCALE AS SHOWN REV. 01
CADD	SJB	MAY 09	
CHECK			5.7A



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

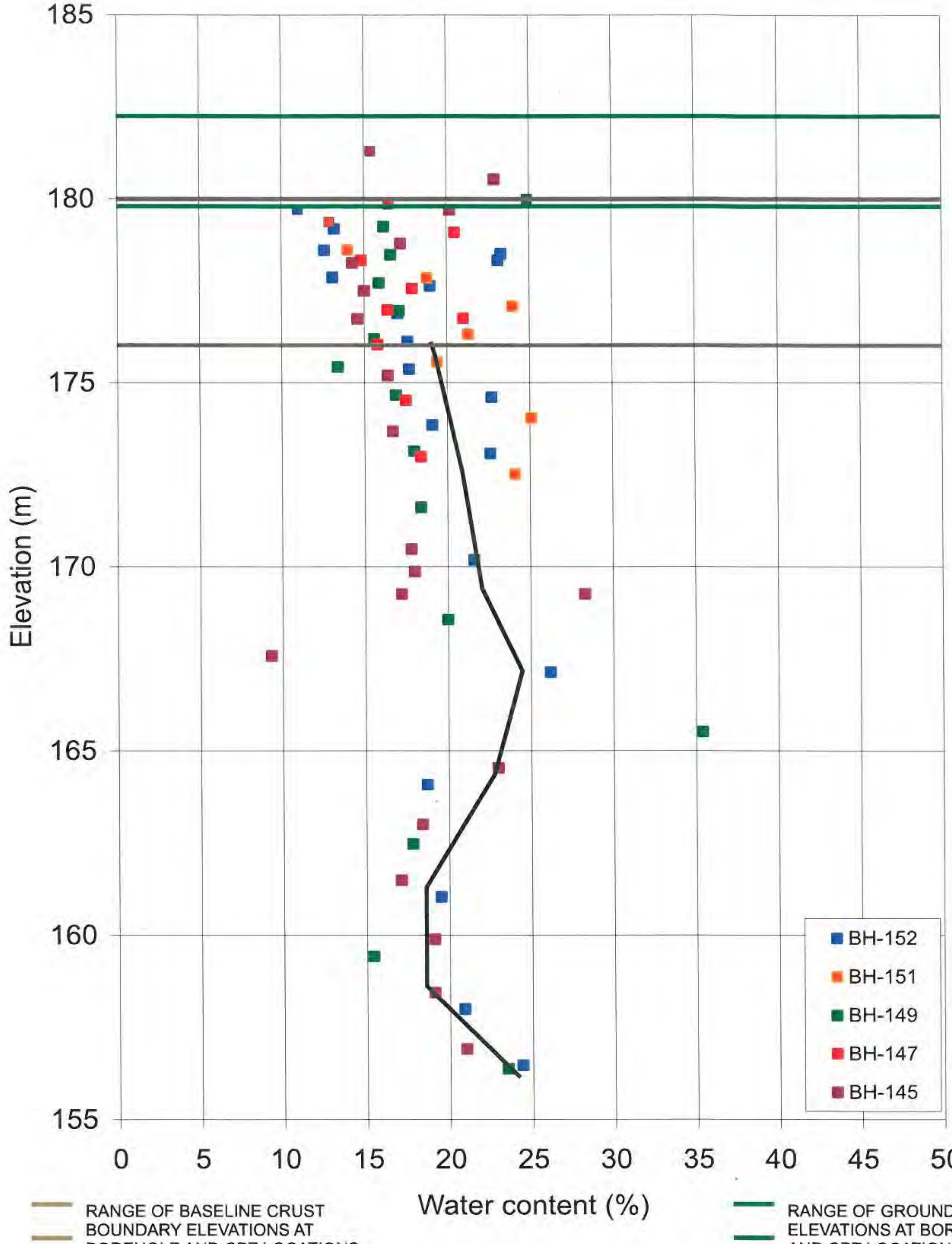
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE PROFILE PROVIDED ABOVE IS CONSIDERED TO BE REPRESENTATIVE OF THE 50TH PERCENTILE VALUES AND THE 90TH AND 10TH PERCENTILE VALUES ARE TO BE TAKEN AS THE 50TH PERCENTILE WATER CONTENT PLUS OR MINUS A WATER CONTENT OF 5%, RESPECTIVELY

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 11+100 to 12+100

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003		FILE No.	0711302070-R02057
		SCALE	AS SHOWN REV. 01
CADD	SJB	MAY 09	
CHECK		26 Jun 09	**5.7B**



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 12+100 to 13+000

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003		FILE No.	0711302070-R02057
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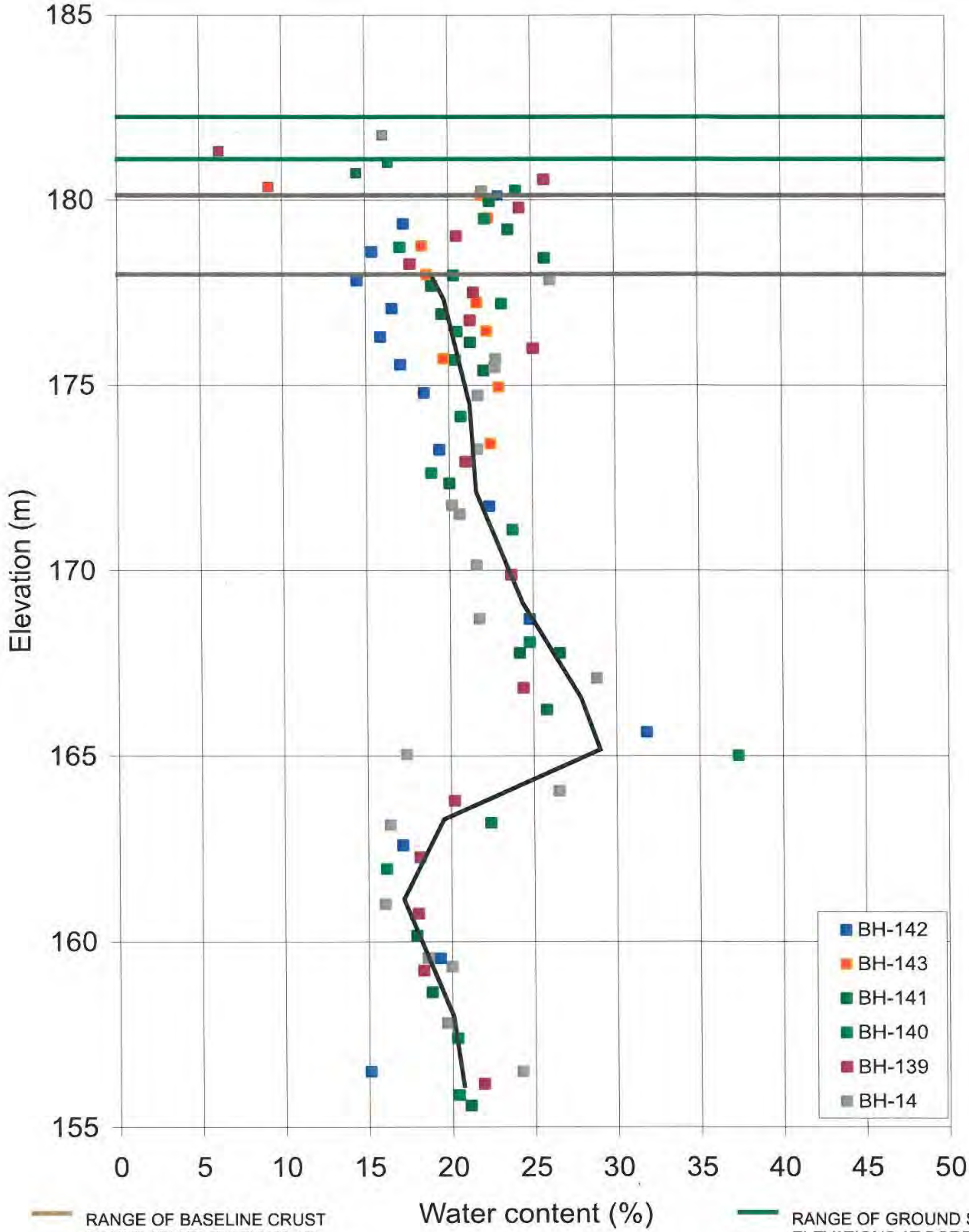
CADD	SJB	MAY 09	
CHECK			5.7C



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 13+000 to 13+900

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
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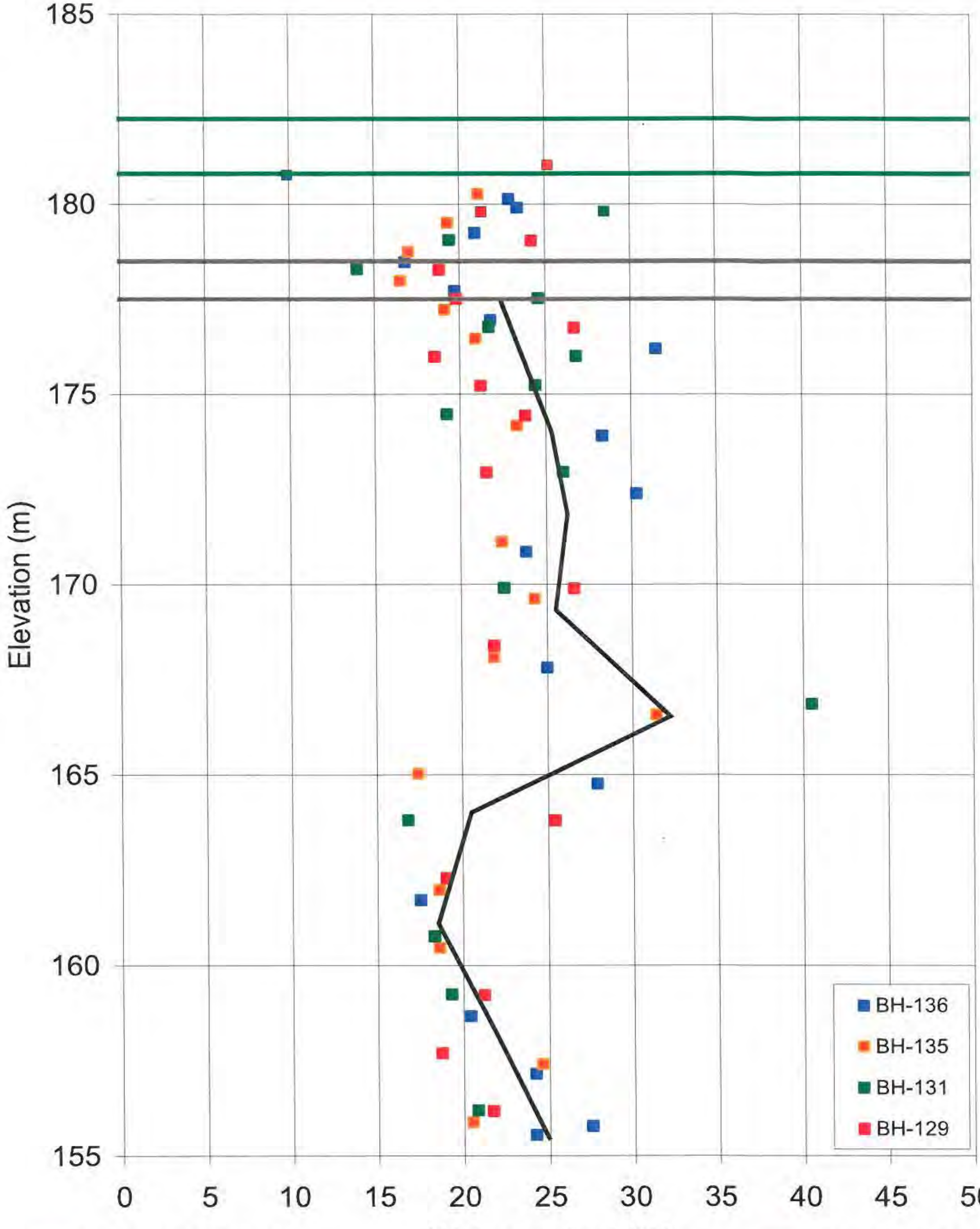
CADD	SAV	MAY 09	
CHECK		26 Jun 09	5.7D



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 13+900 to 10+300

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003		FILE No.	0711302070-R02057
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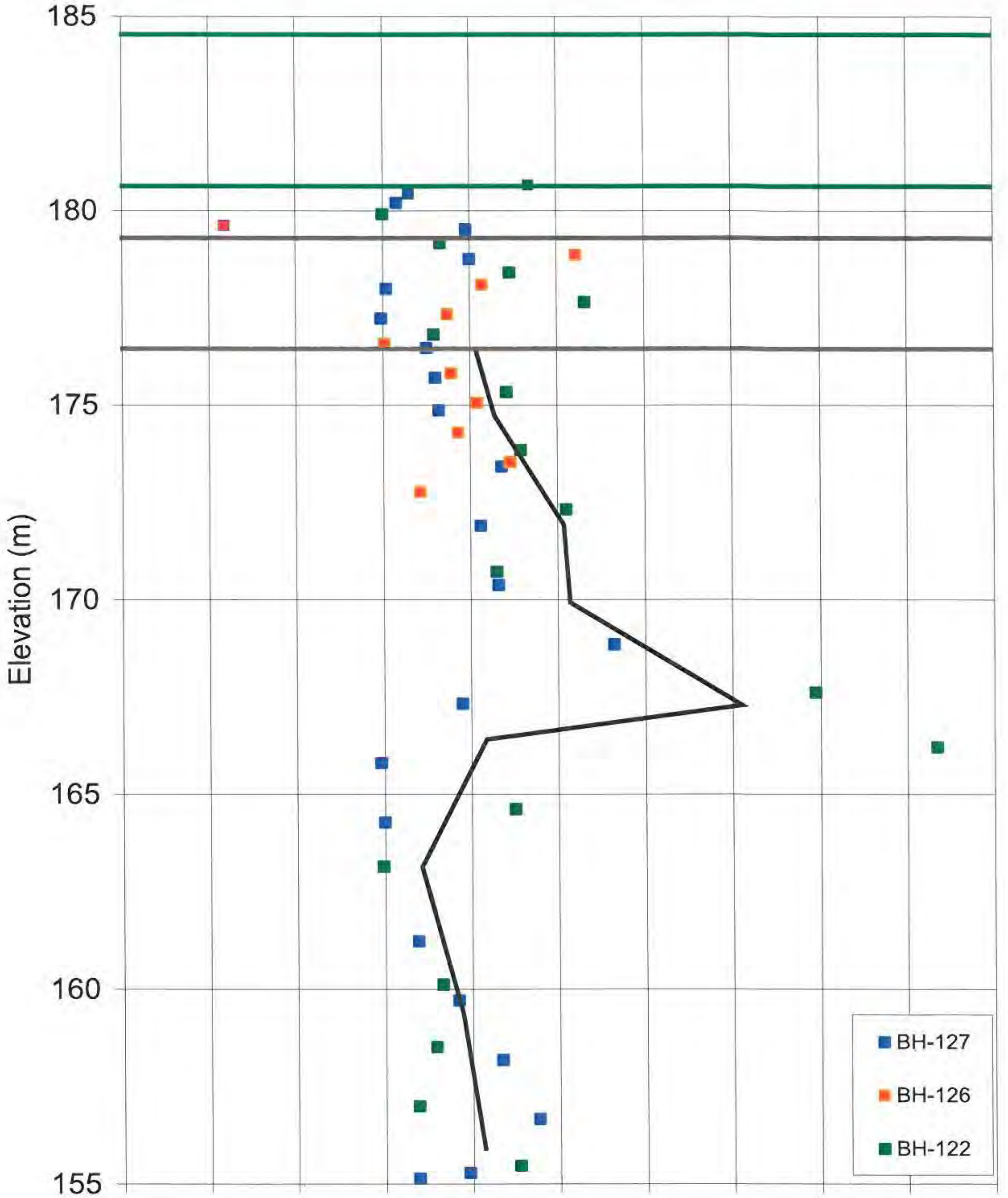
CADD	S.J.B.	MAY 09	5.7E
CHECK		26 Jun 09	



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 10+300 to 11+300

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
			SCALE AS SHOWN REV 01
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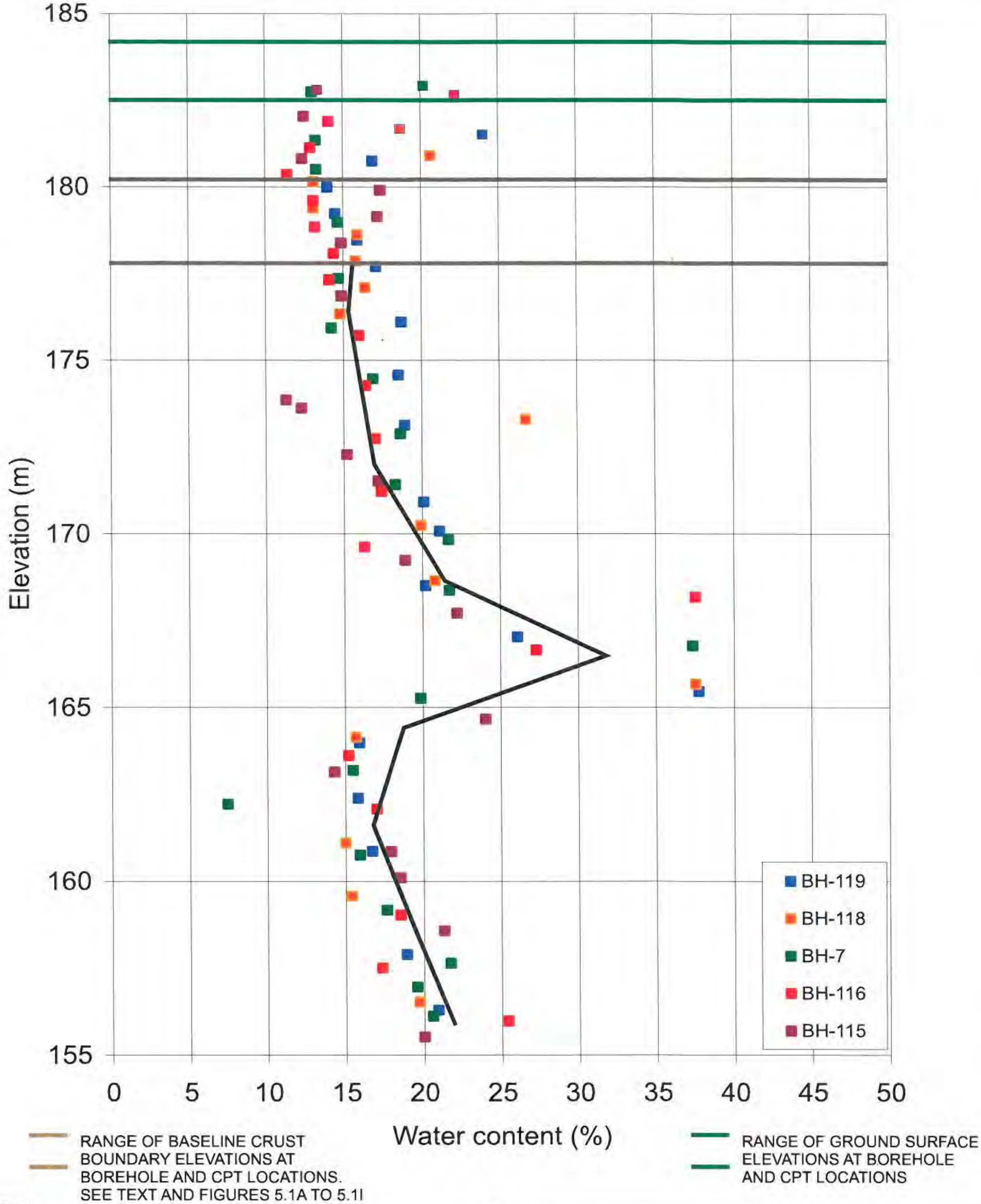
CHECK		26 Jun 09	5.7F



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
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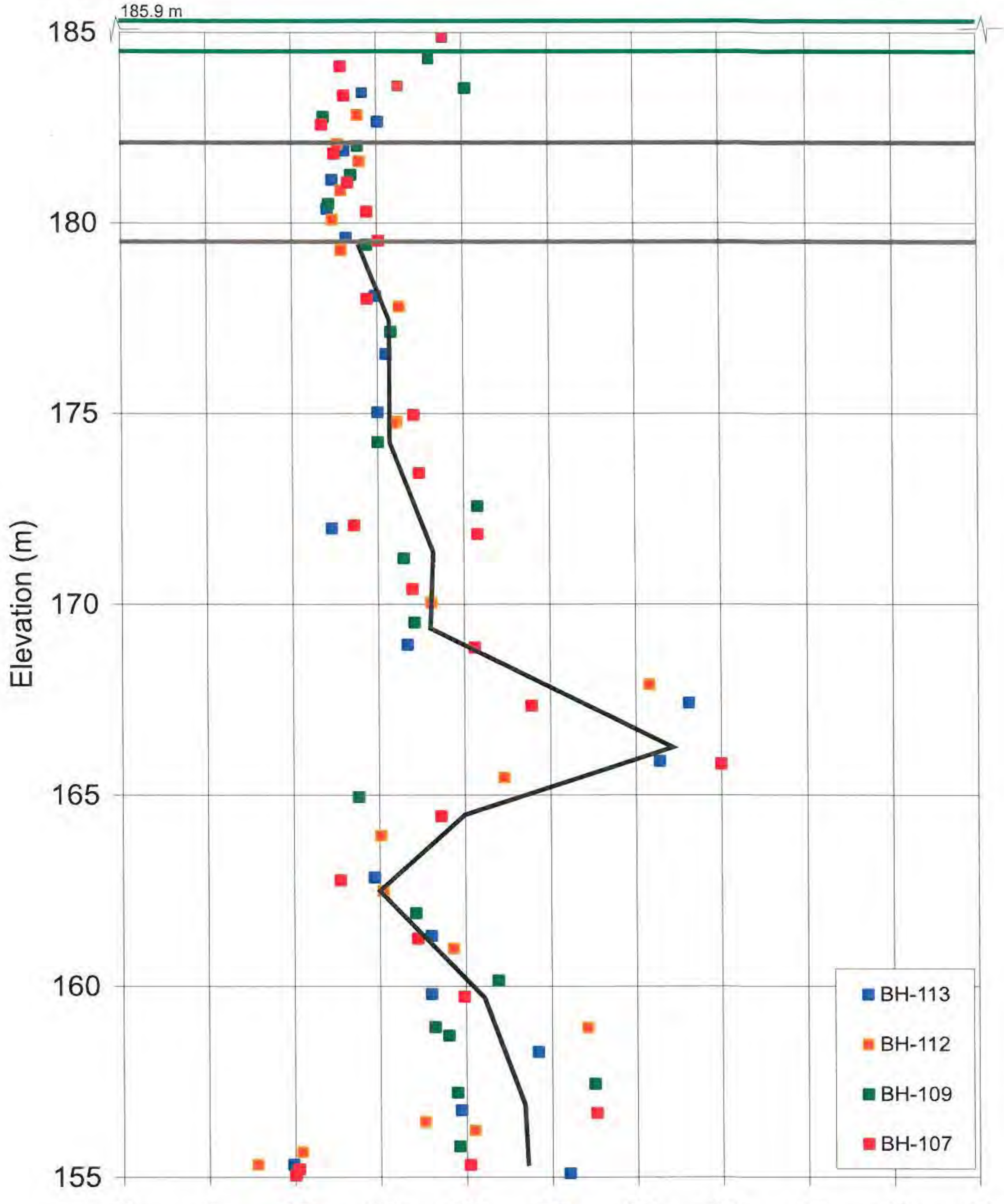
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 11+300 to 12+600

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
			SCALE AS SHOWN REV. 01
CADD	SJB	MAY 09	
CHECK			5.7G



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

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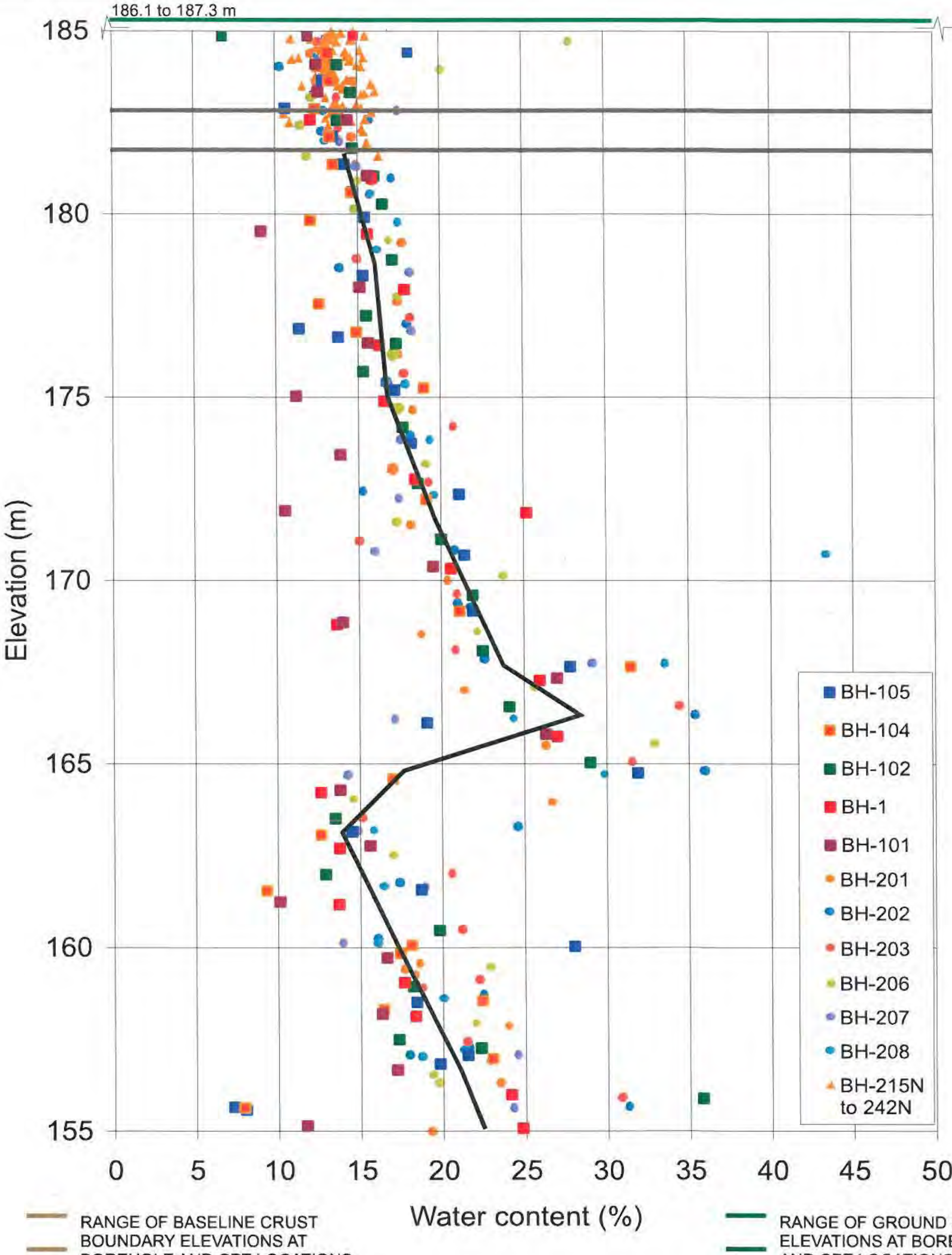
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 12+600 to 13+600

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
			SCALE AS SHOWN REV. 01
CADD	SJB	MAY 09	
CHECK	MM	26 Jun 09	5.7H



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT BOREHOLE AND CPT LOCATIONS

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE BASELINE WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
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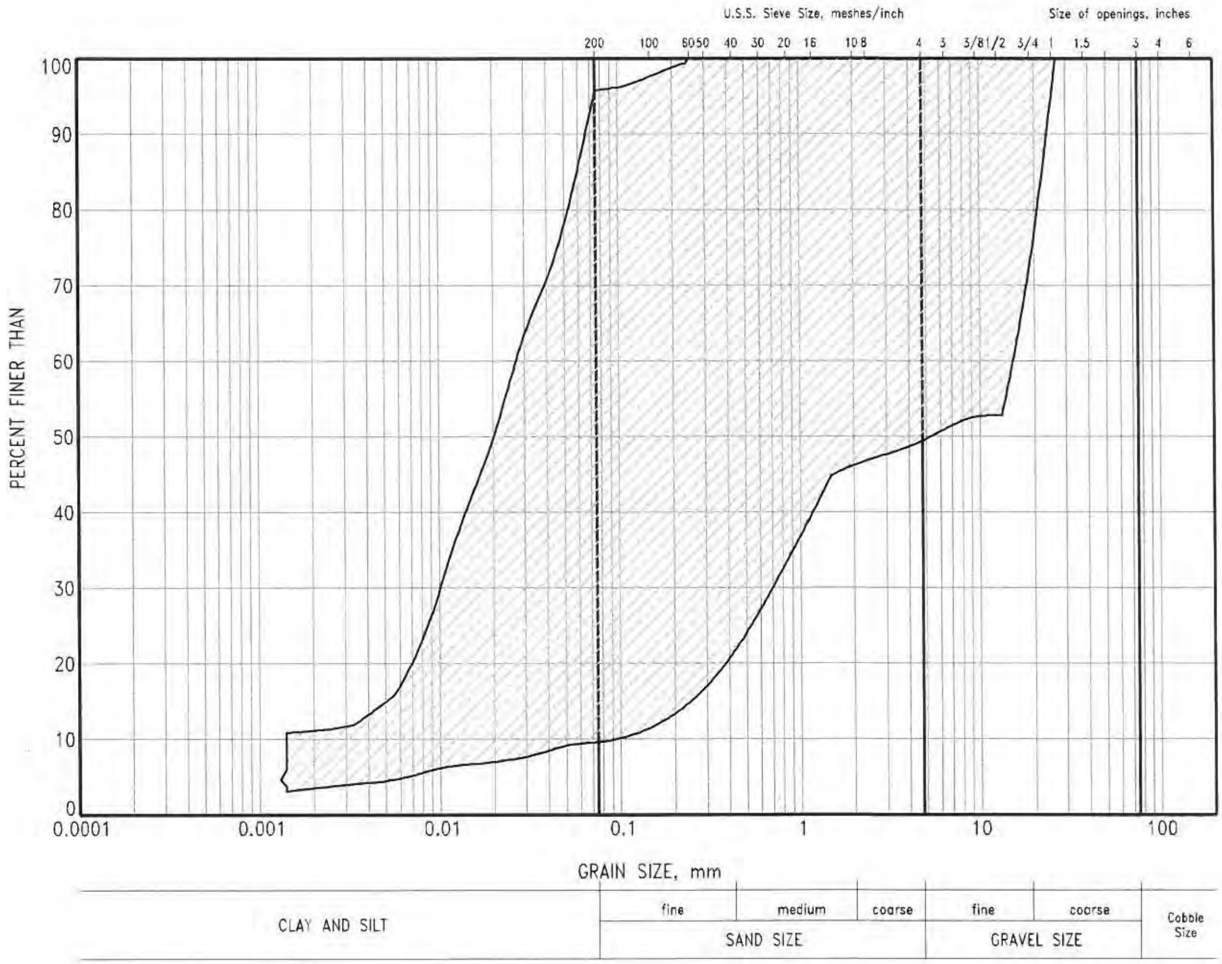
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE WATER CONTENT PROFILE
STATION 13+600 to 10+900

Golder Associates

PROJECT No. 07-1130-2070/05-1140-003			FILE No. 0711302070-R02057
			SCALE AS SHOWN REV. 01
CADD	SJB	MAY 09	
CHECK	MY	26 Jun 09	5.7I



The grain size distribution envelope shown above is based on all grain size distribution analyses conducted on samples obtained from the Lower Granular Deposits. For individual test results refer to the Geotechnical Data Report referenced in section 3.1. The samplers used for the explorations limit the maximum particle size that can be sampled and tested to about 40mm. Larger particles are known to be present in the deposit as discussed in the report text.

PROJECT

SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

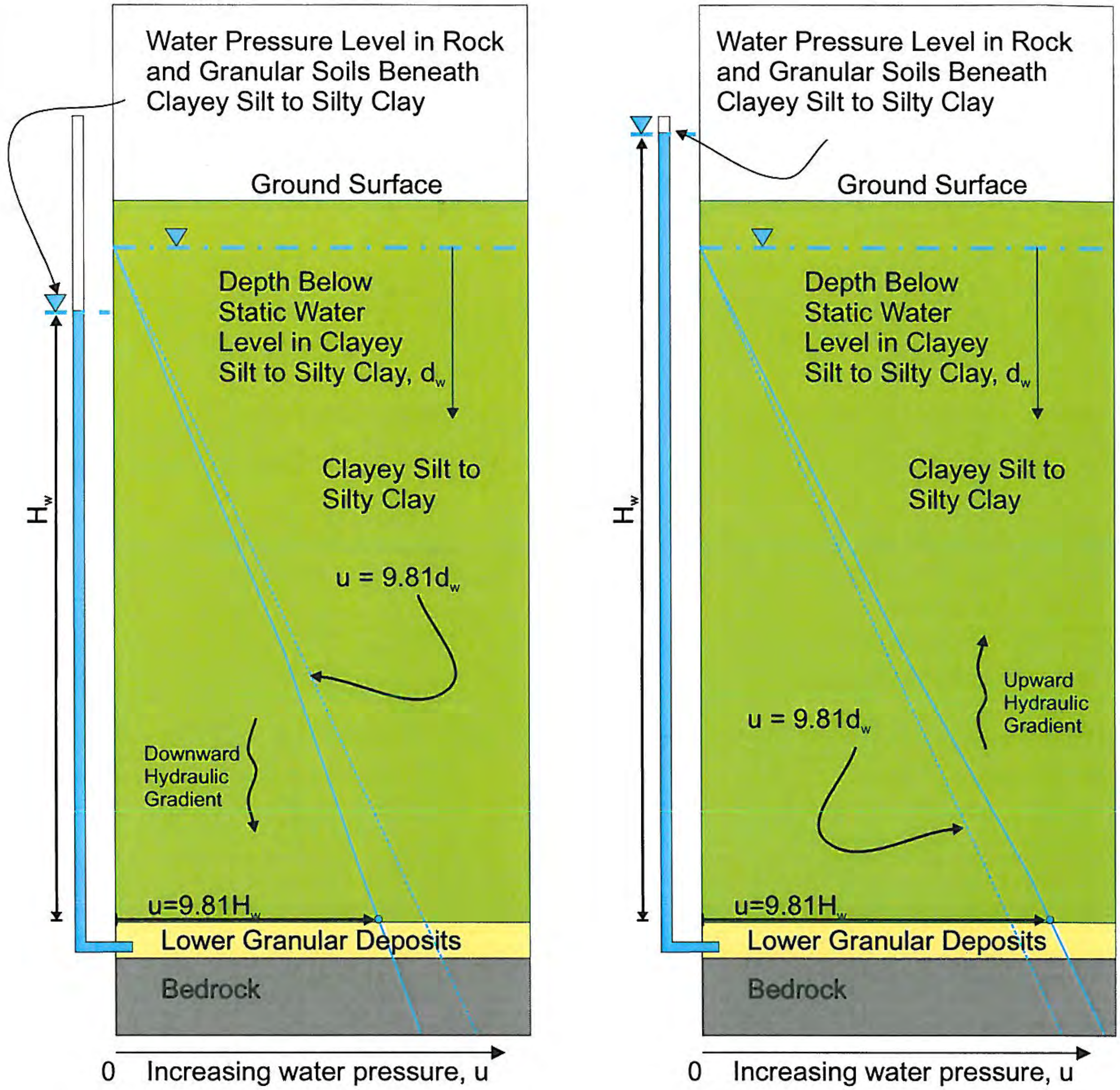
TITLE

GRAIN SIZE DISTRIBUTION ENVELOPE
LOWER GRANULAR DEPOSITS

Golder Associates
LONDON, ONTARIO

PROJECT No.	07-1130-207-0		FILE No.	0711302070-R02058
			SCALE AS SHOWN	REV. 0
CADD	LMK	Feb. 11/09		
CHECK	JAY	26 Jan 09	**FIGURE 5.8**	

Drawing file: 0711302070-R02058.dwg Feb 17, 2009 - 11:46am



u = water pressure
H_w = head of water in bedrock or Lower Granular Deposits
d_w = depth below static water level in Clayey Silt to Silty Clay Deposits
9.81 = unit weight of water, kN/m^3

NOTES

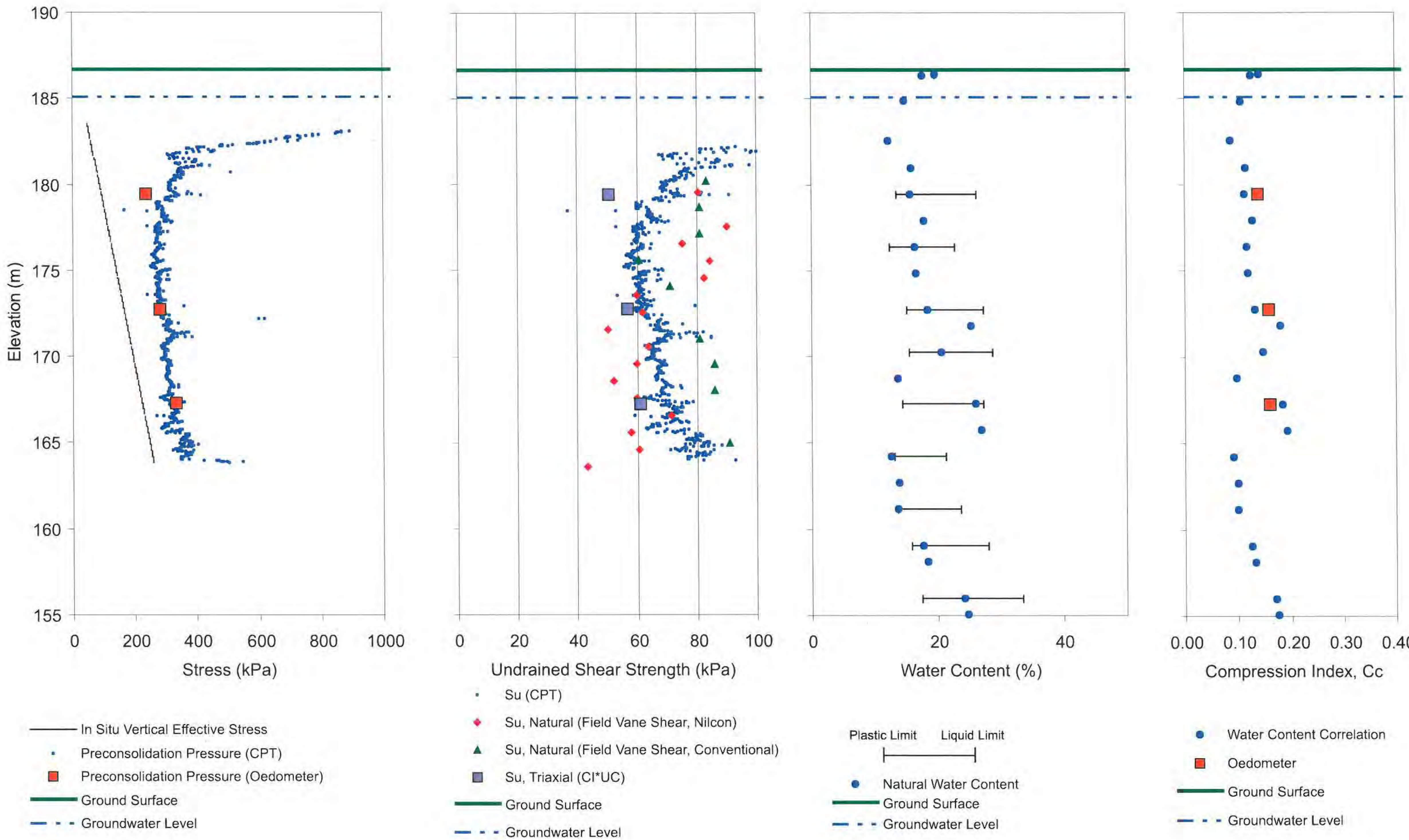
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE IS SCHEMATIC ONLY

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
ILLUSTRATION OF UPWARD AND DOWNWARD HYDRAULIC GRADIENTS

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02059
		SCALE	AS SHOWN REV 01
CADD	SJB	JUN 09	
CHECK	AW	30 Jun 09	5.9



NOTES

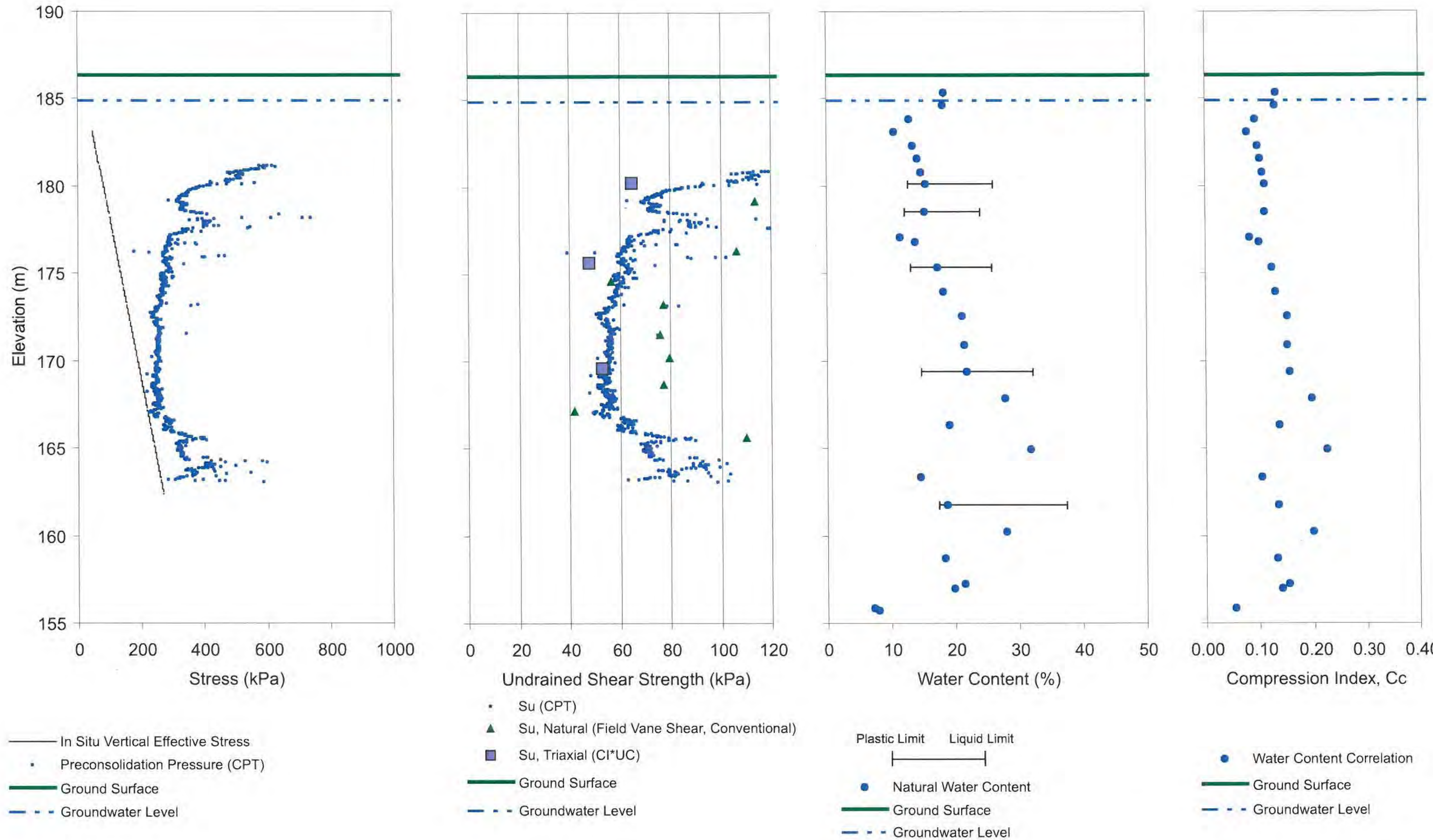
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-1/CPT-1

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
CADD	SJB	JUN 09	SCALE AS SHOWN REV
CHECK	DY	26 Jun 09	6.1A



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT

SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH105/CPT-2

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02061
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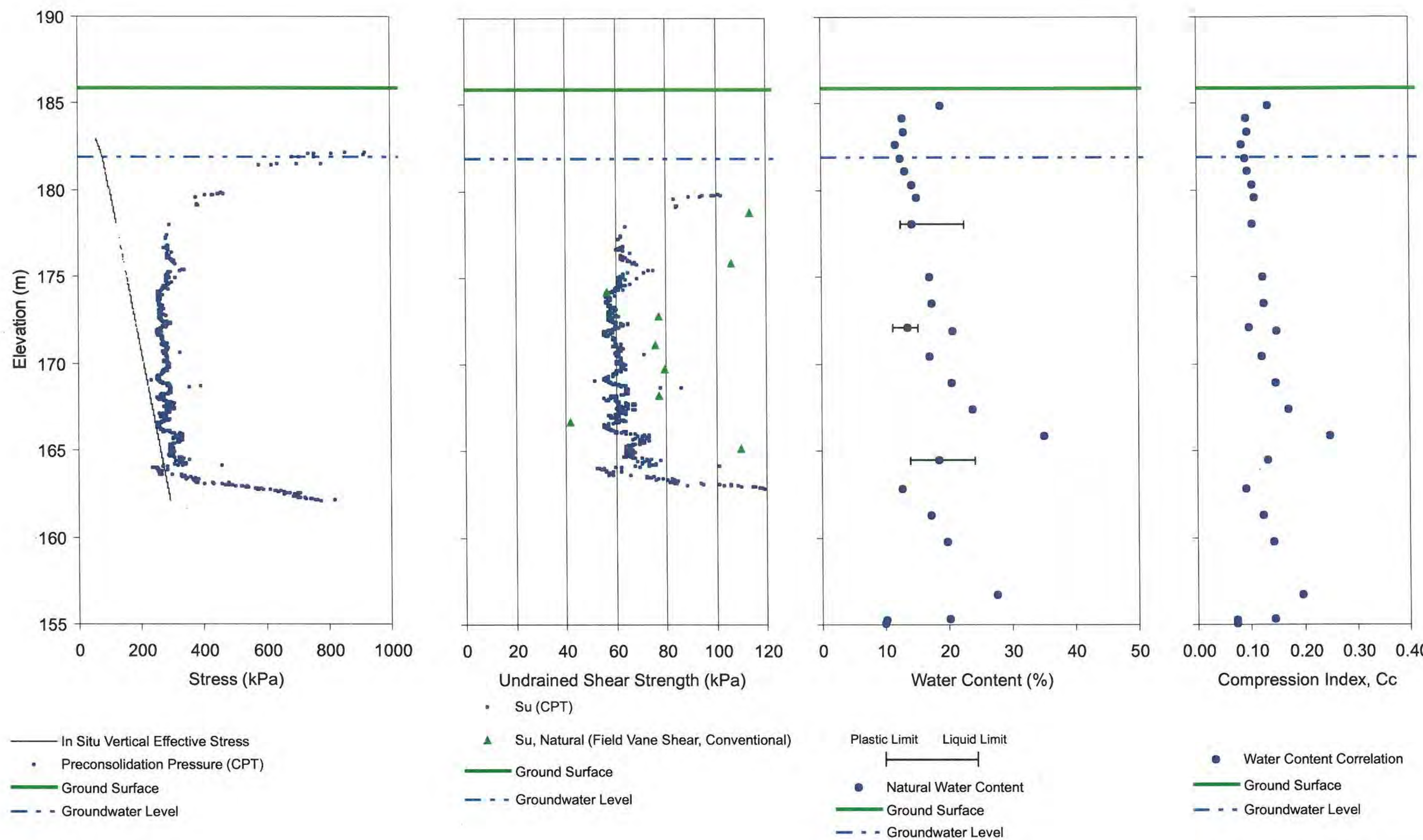

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-107/CPT-3

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02061
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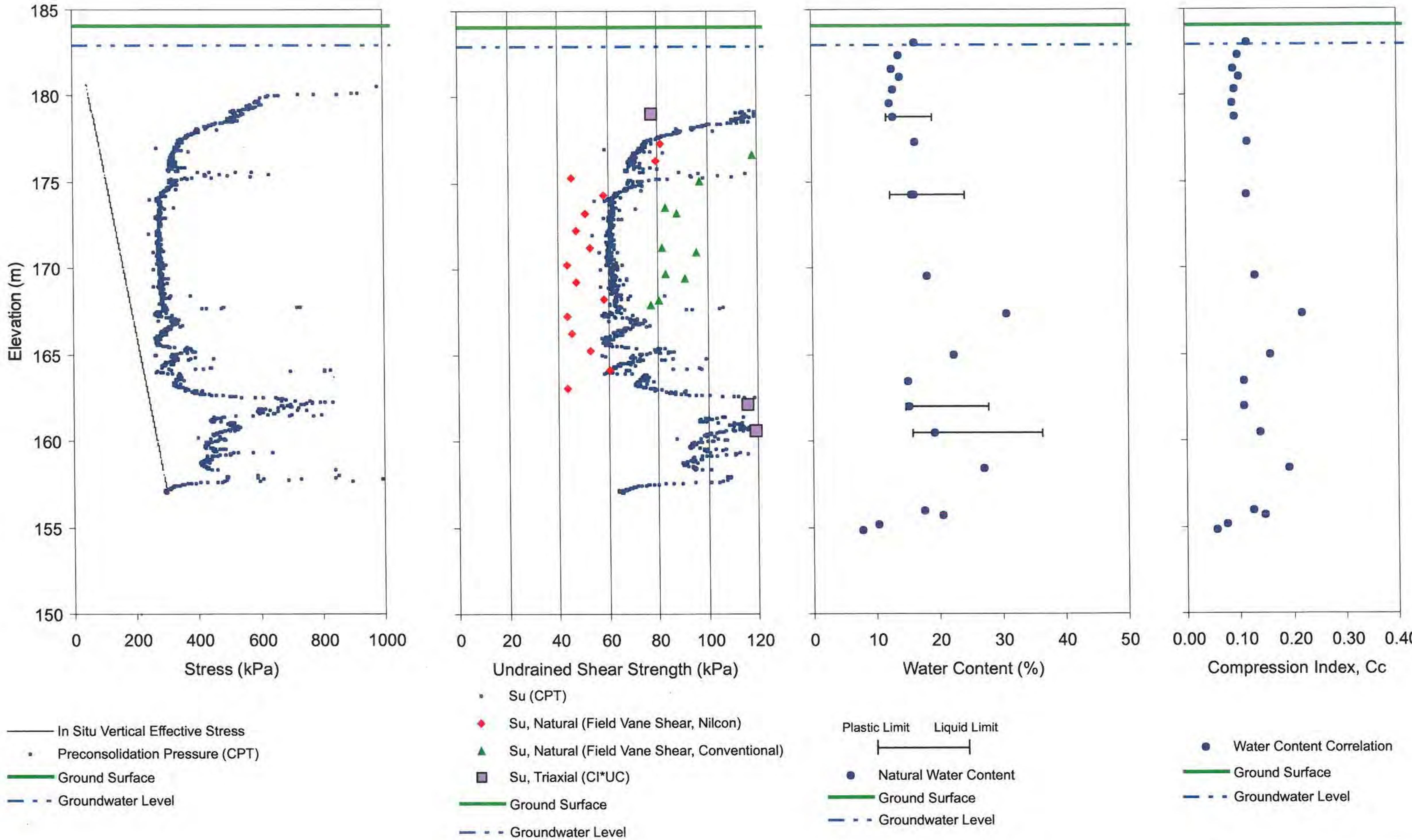
CADD	SJB	JUN 09	6.1C	
CHECK	MT	26 June 09		



NOTES

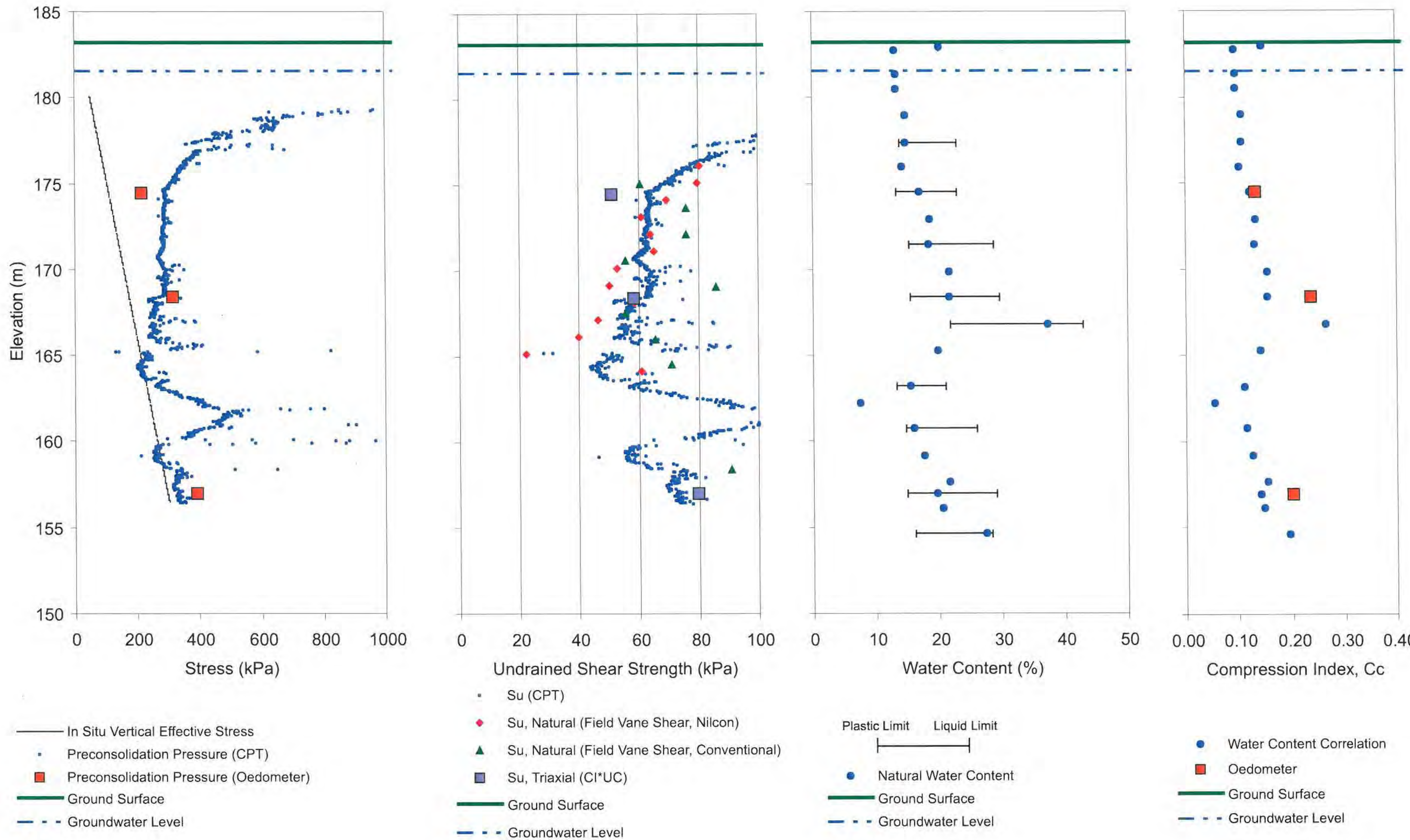
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-112/CPT-5

Golder Associates

PROJECT No. 07-1130-2070		FILE No. 0711302070-R02061	
		SCALE AS SHOWN	REV.
CADD	SJB	JUN 09	6.1D
CHECK			



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-7/CPT-7

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
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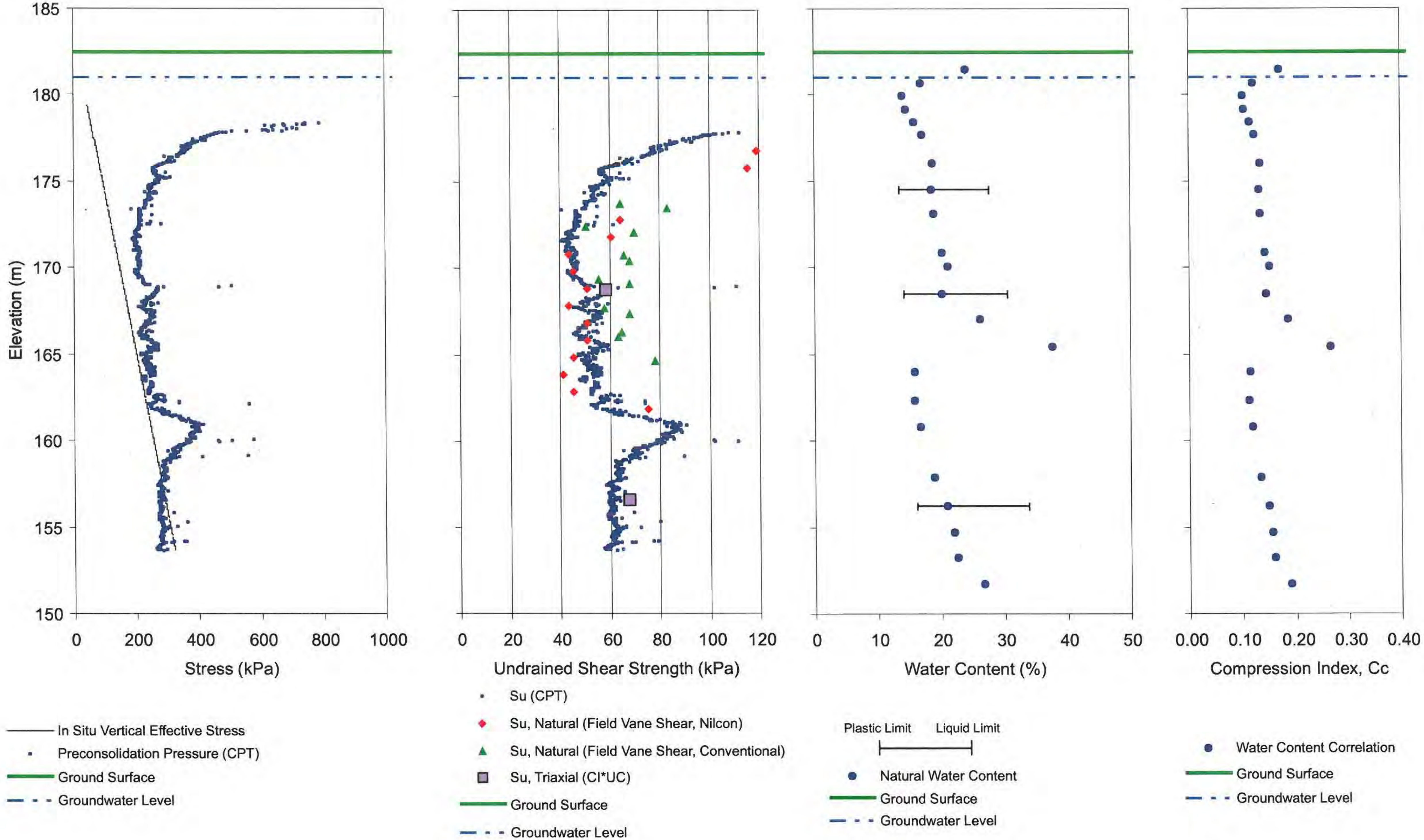

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-119/CPT-8

Golder Associates

PROJECT No.	07-1130-2070		FILE No.	07111302070-R02061
			SCALE	AS SHOWN REV.
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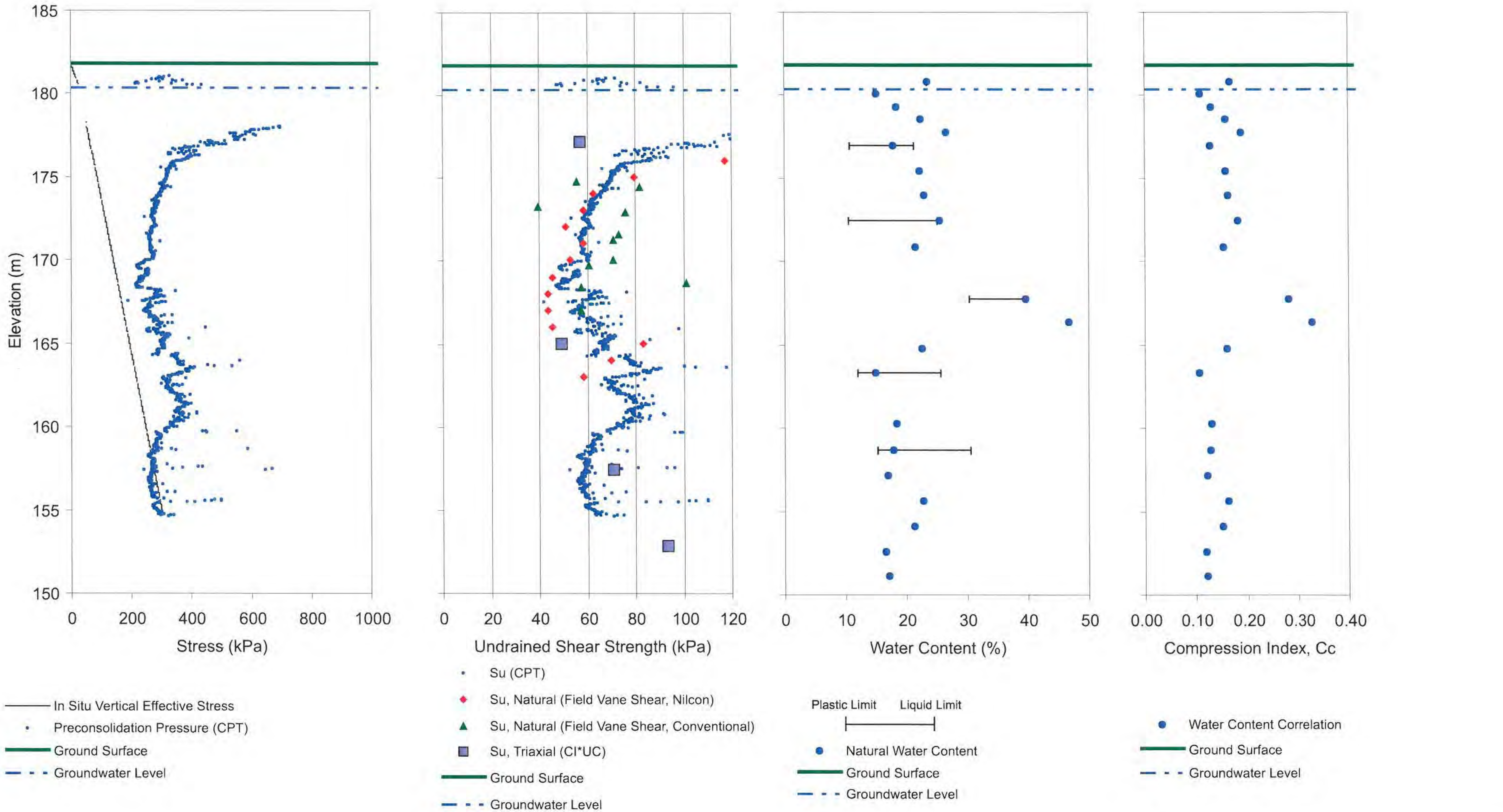
CHECK				6.1F



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-122/CPT-10

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
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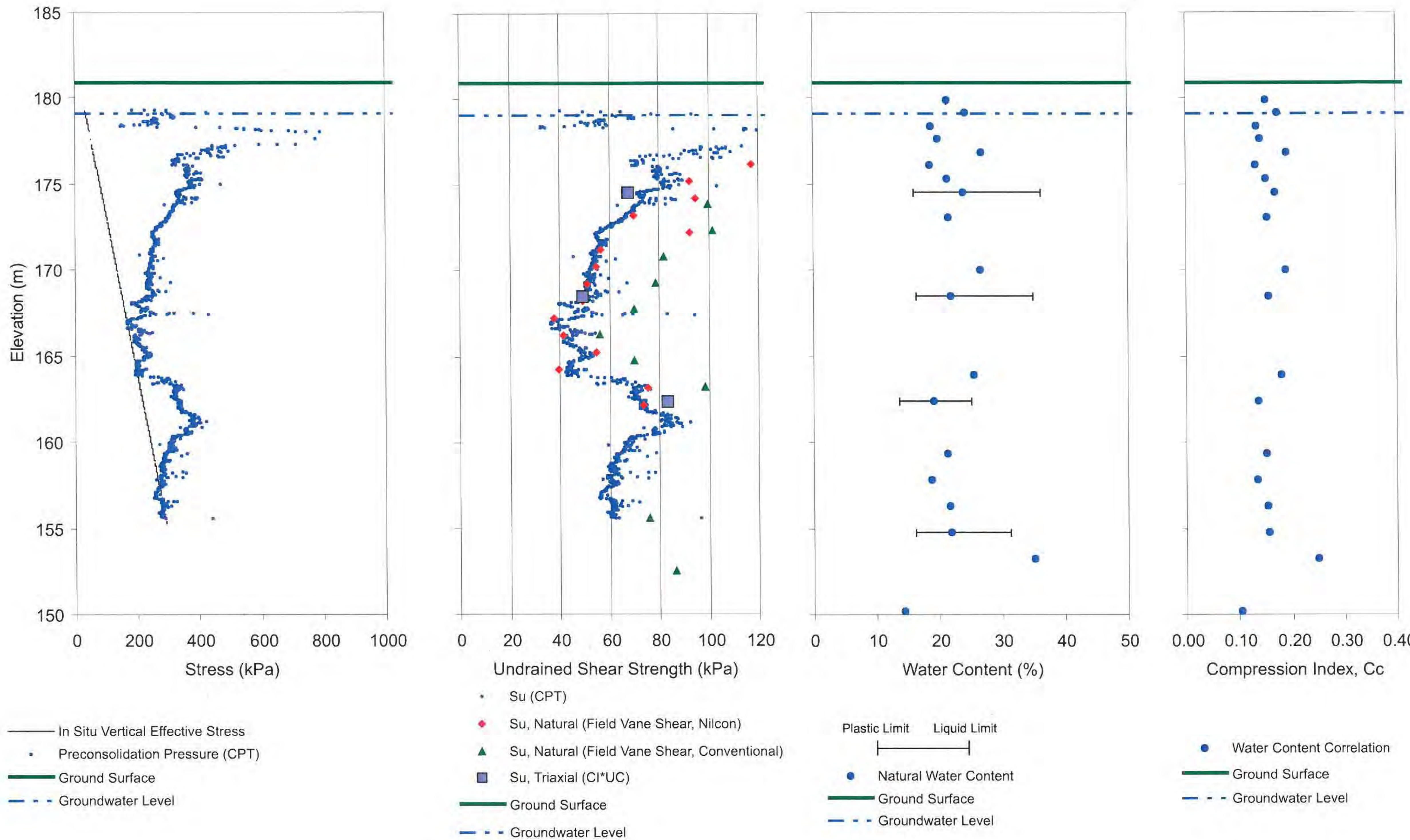
CADD		JUN 09	6.1G
CHECK			



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT

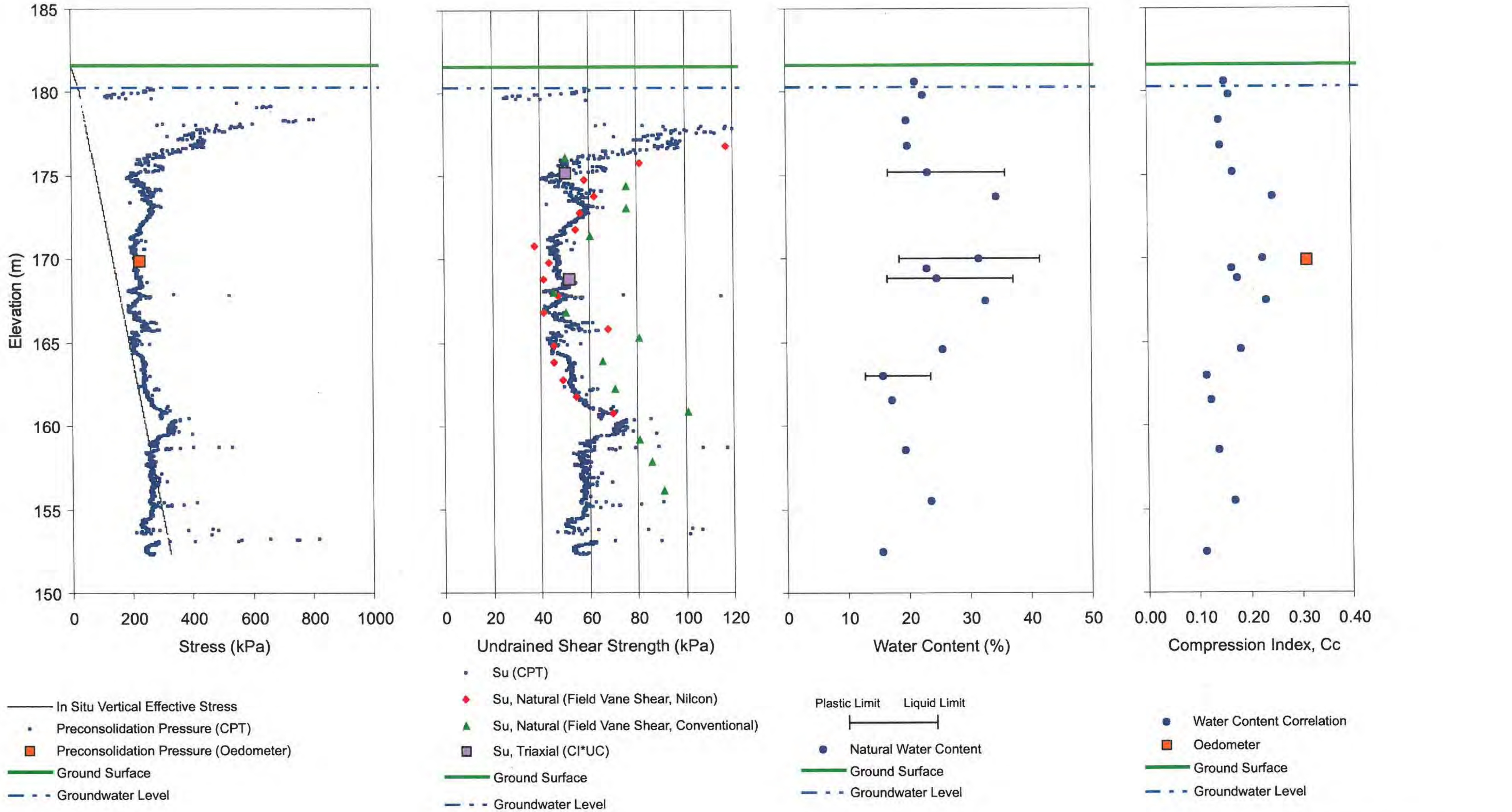
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-129/CPT-11

Golder Associates

PROJECT No	07-1130-2070		FILE No	0711302070-R02061
			SCALE	AS SHOWN REV
CADD	SJB	JUN 09		6.1H
CHECK				



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-132/CPT-12

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
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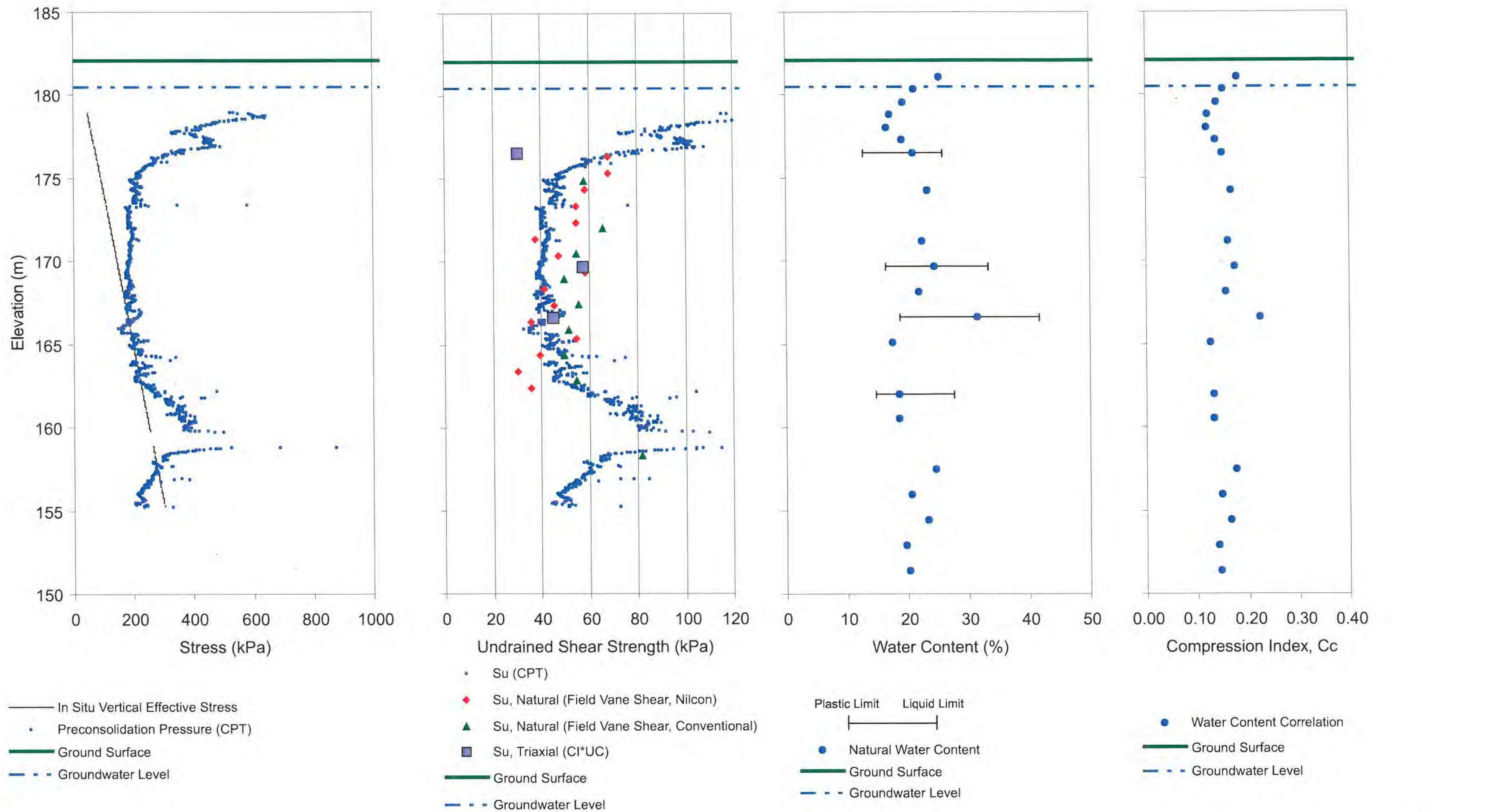
CHECK	[signature]	25 Jun 09	6.1I



NOTES

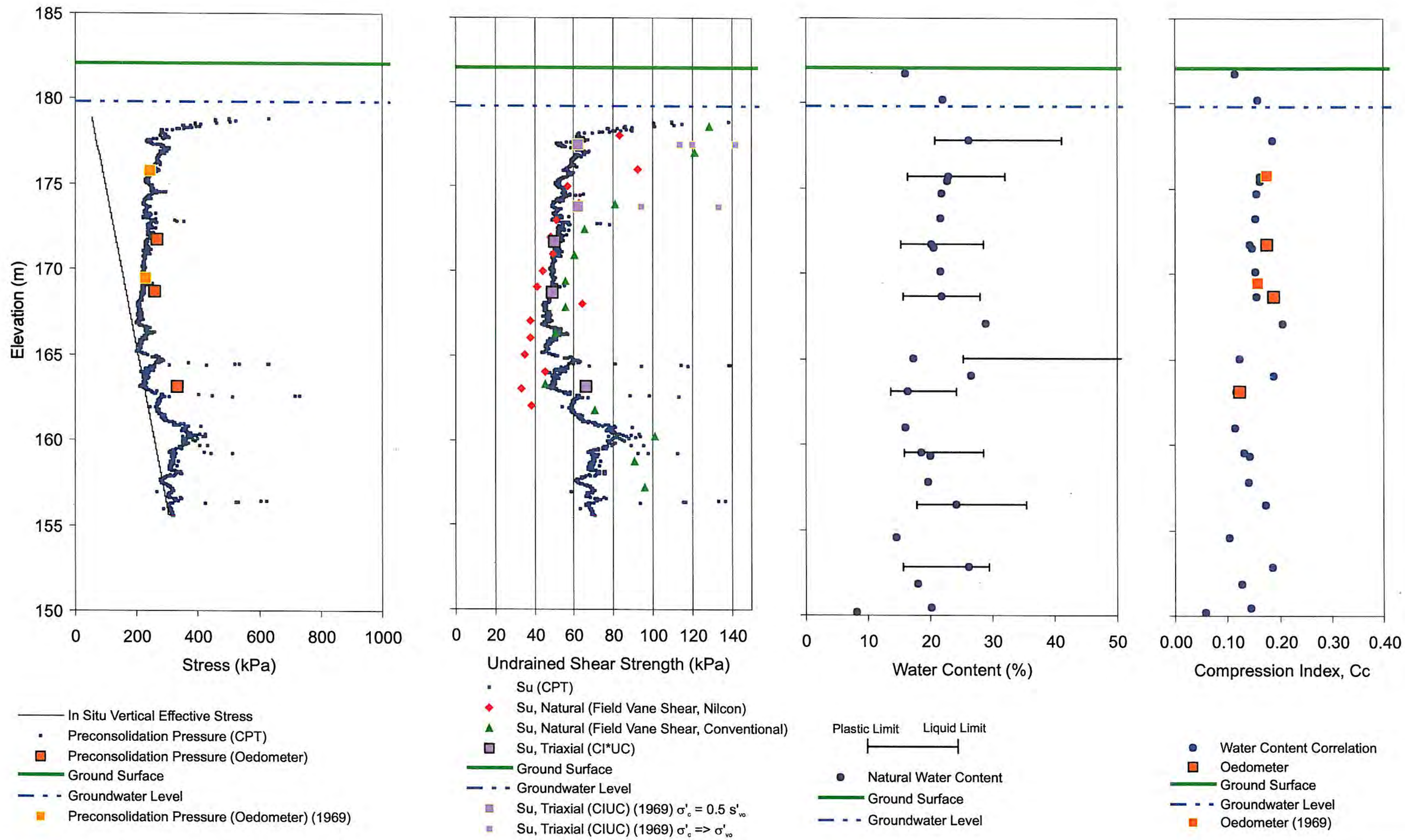
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-135/CPT-13

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
		SCALE	AS SHOWN REV
CADD	SJB	JUN 09	
CHECK			6.1J



NOTES

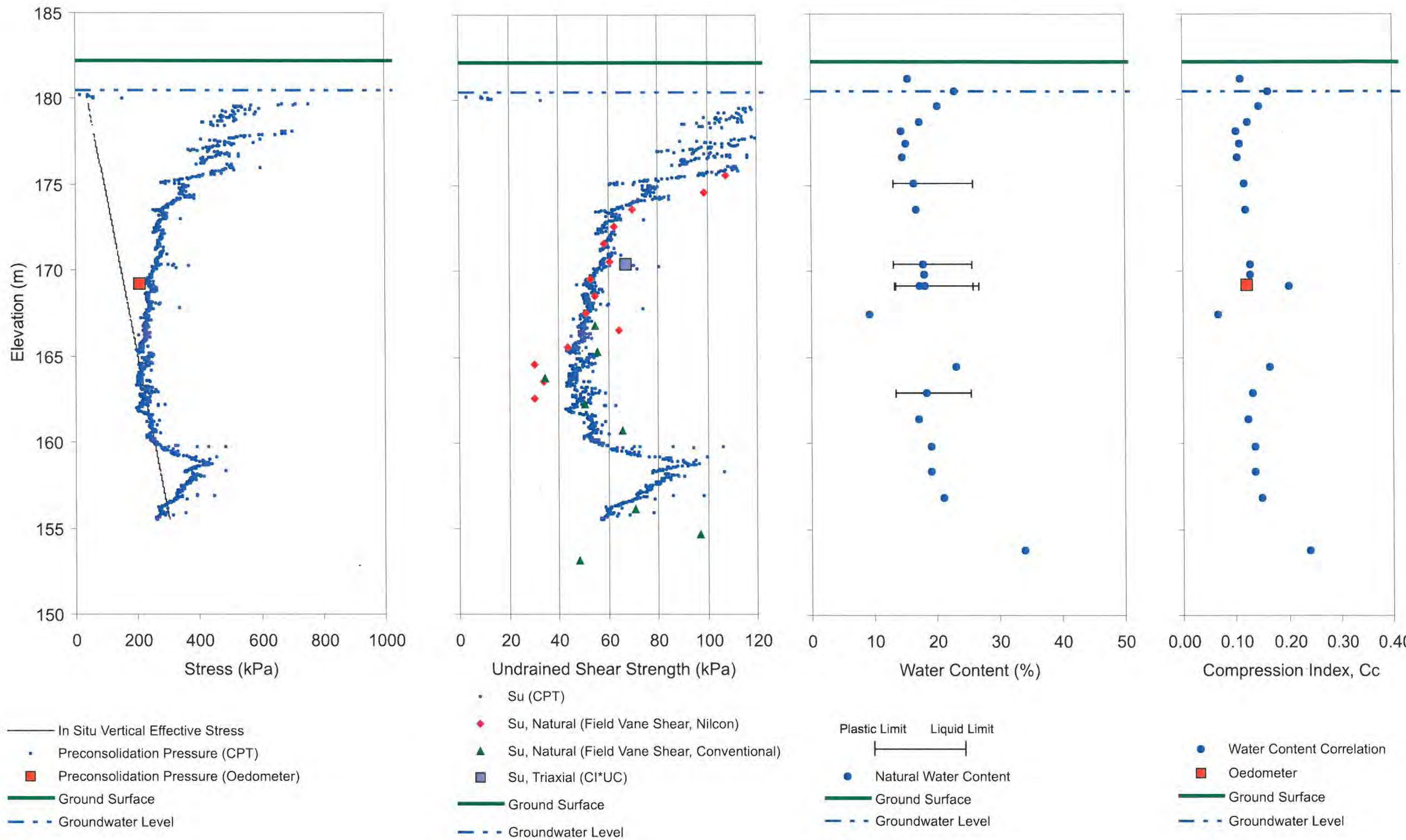
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-14/CPT-14

Golder Associates

PROJECT No. 07-1130-2070	FILE No. 0711302070-R02061
CADD SJB JUN 09	SCALE AS SHOWN REV.
CHECK	6.1K



NOTES

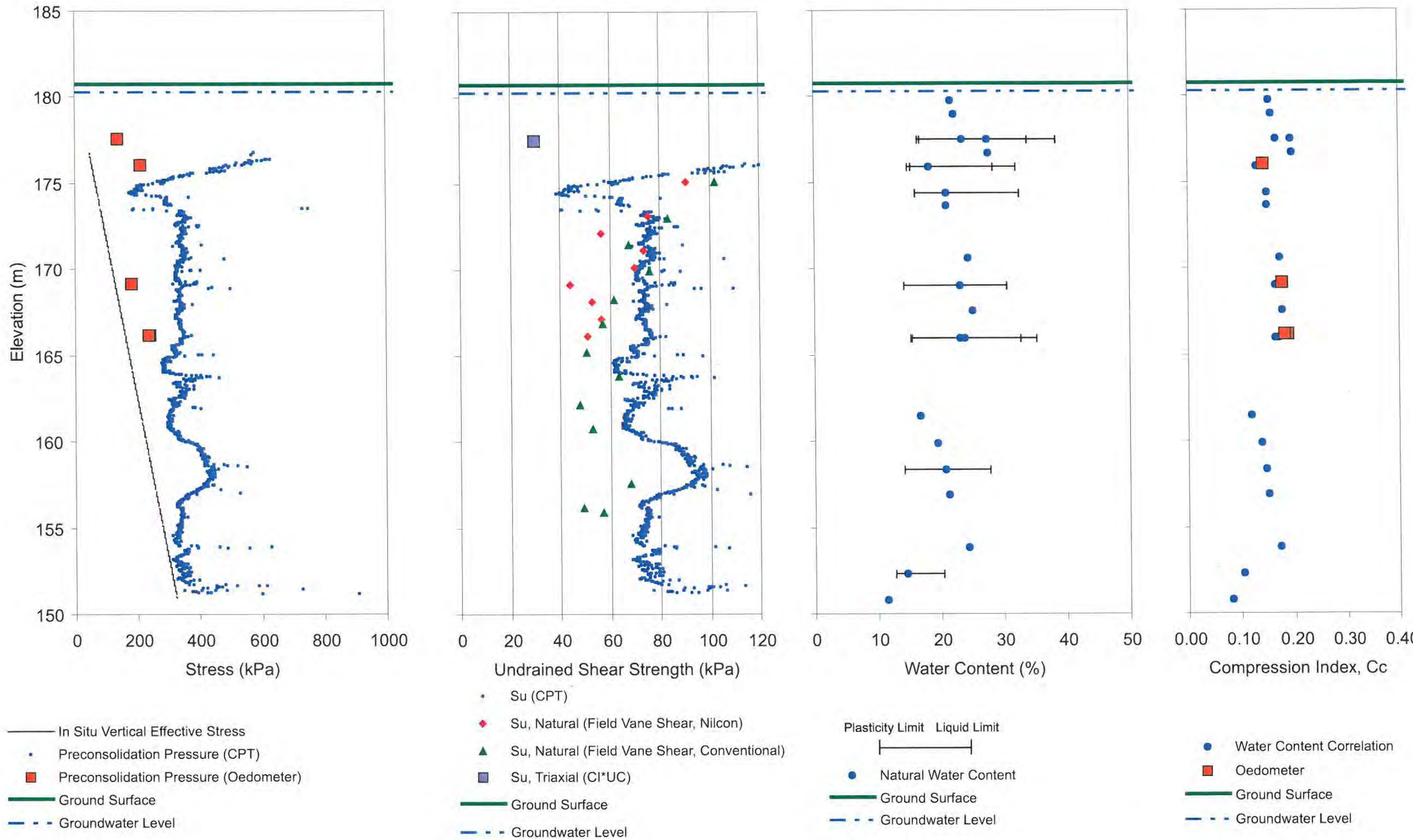
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-145/CPT-145

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
		SCALE	AS SHOWN REV.
CADD	SJB JUN 09		6.1L
CHECK			



NOTES

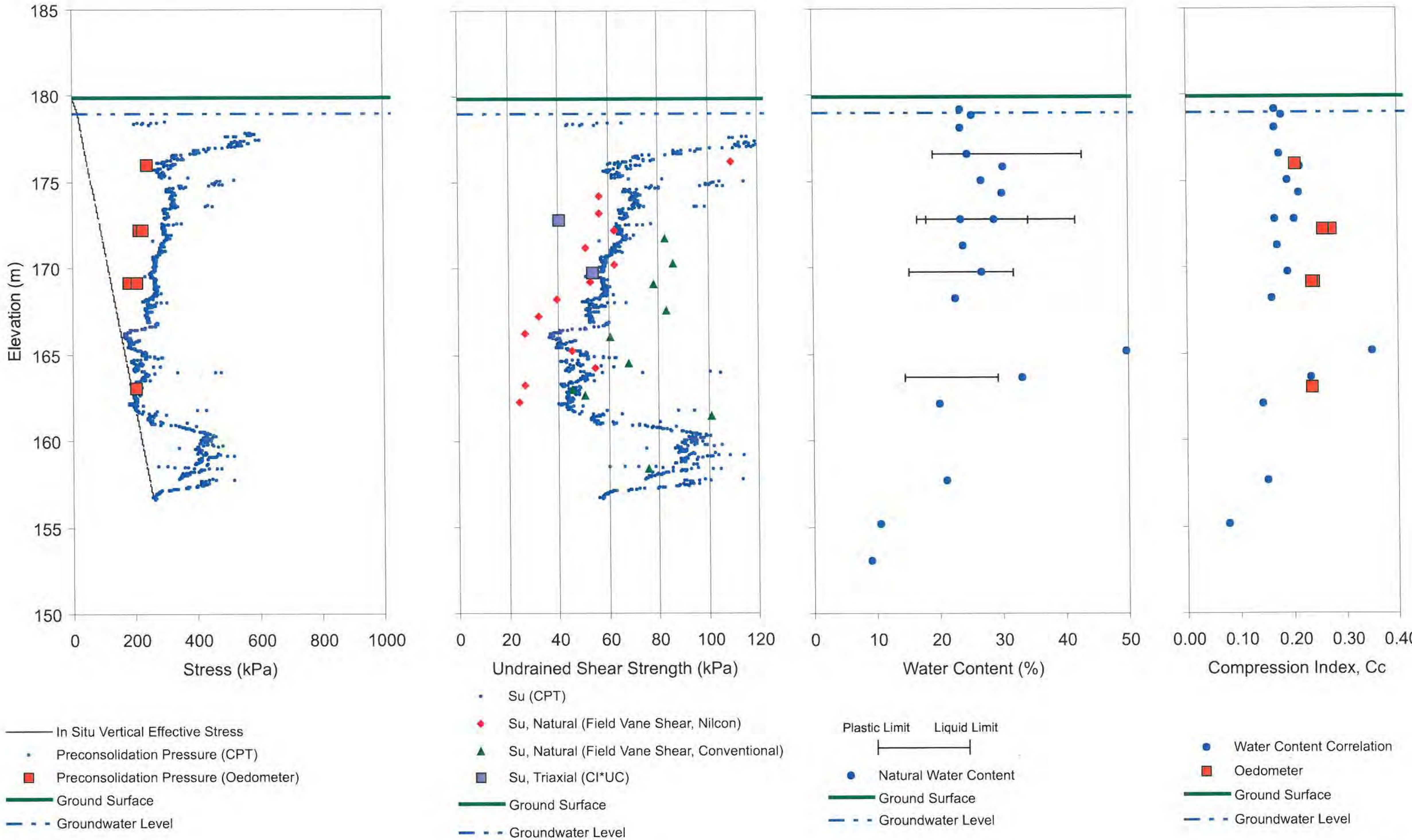
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-154/CPT-154

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
		SCALE	AS SHOWN REV.
CADD	SJB	JUN 09	
CHECK			6.1M



NOTES

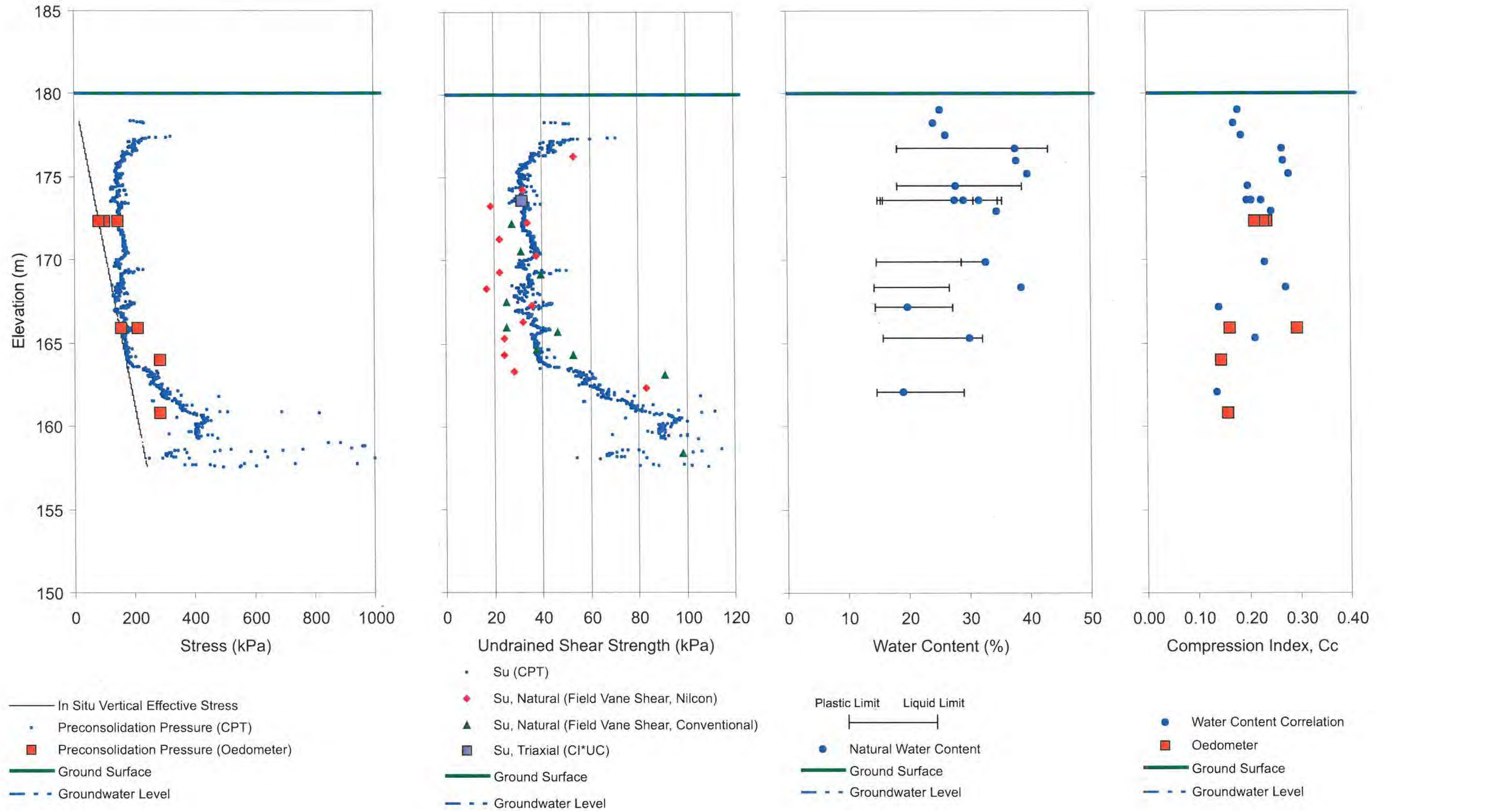
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-158/CPT-21

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
		SCALE	AS SHOWN REV.
CADD	SJB	JUN 09	
CHECK	MY	25 Jun 09	6.1N



NOTES

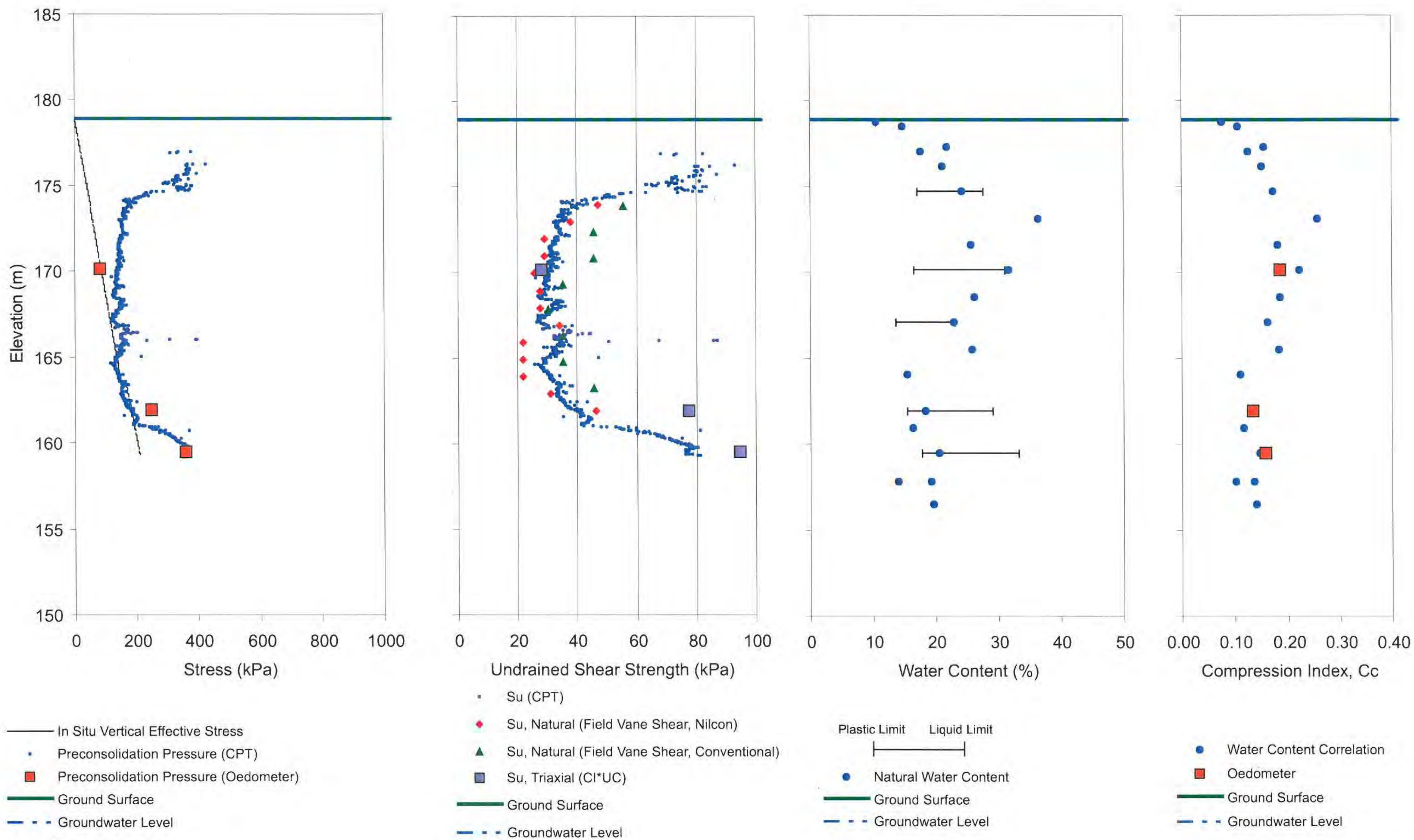
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-160/CPT-160

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02061
		SCALE	AS SHOWN REV
CADD	SJB	JUN 09	
CHECK			6.10



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT

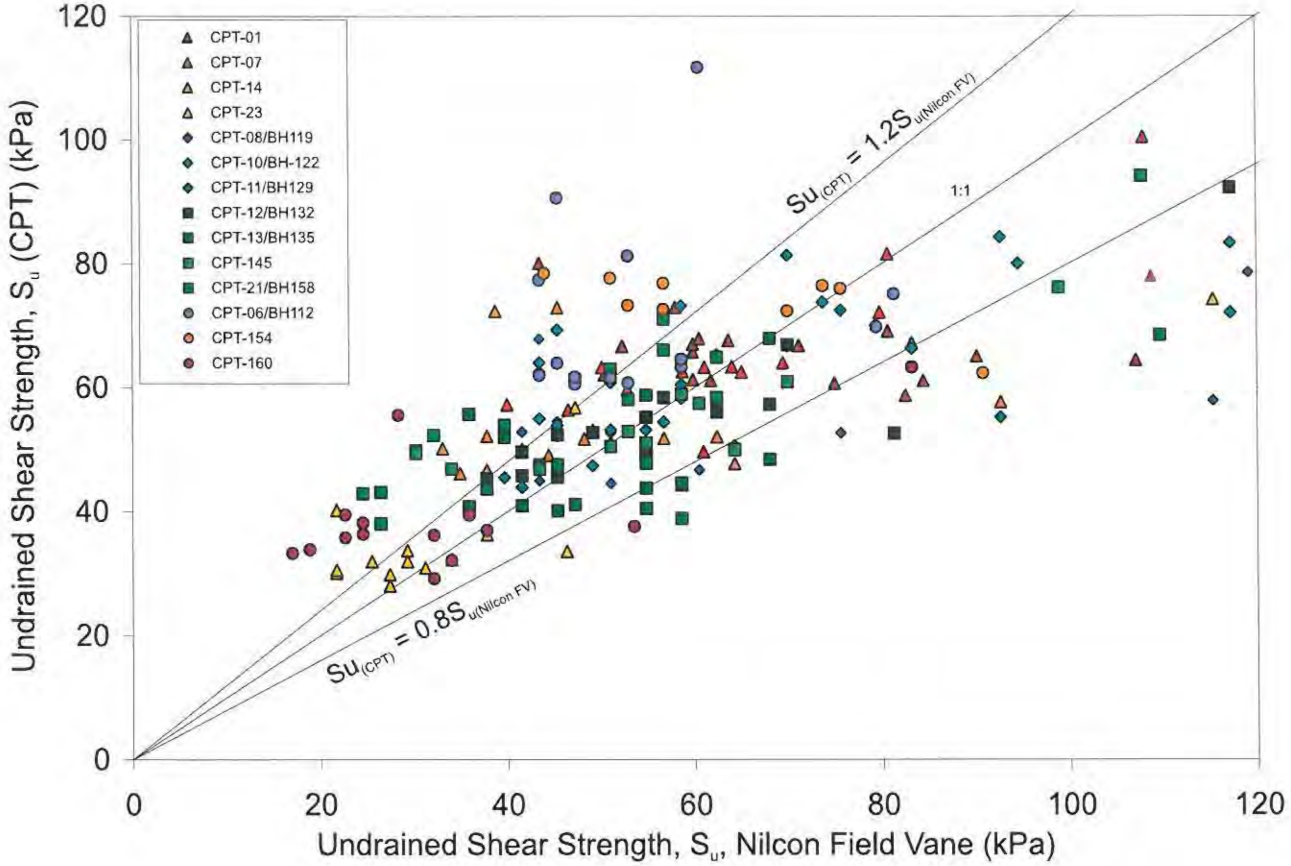
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

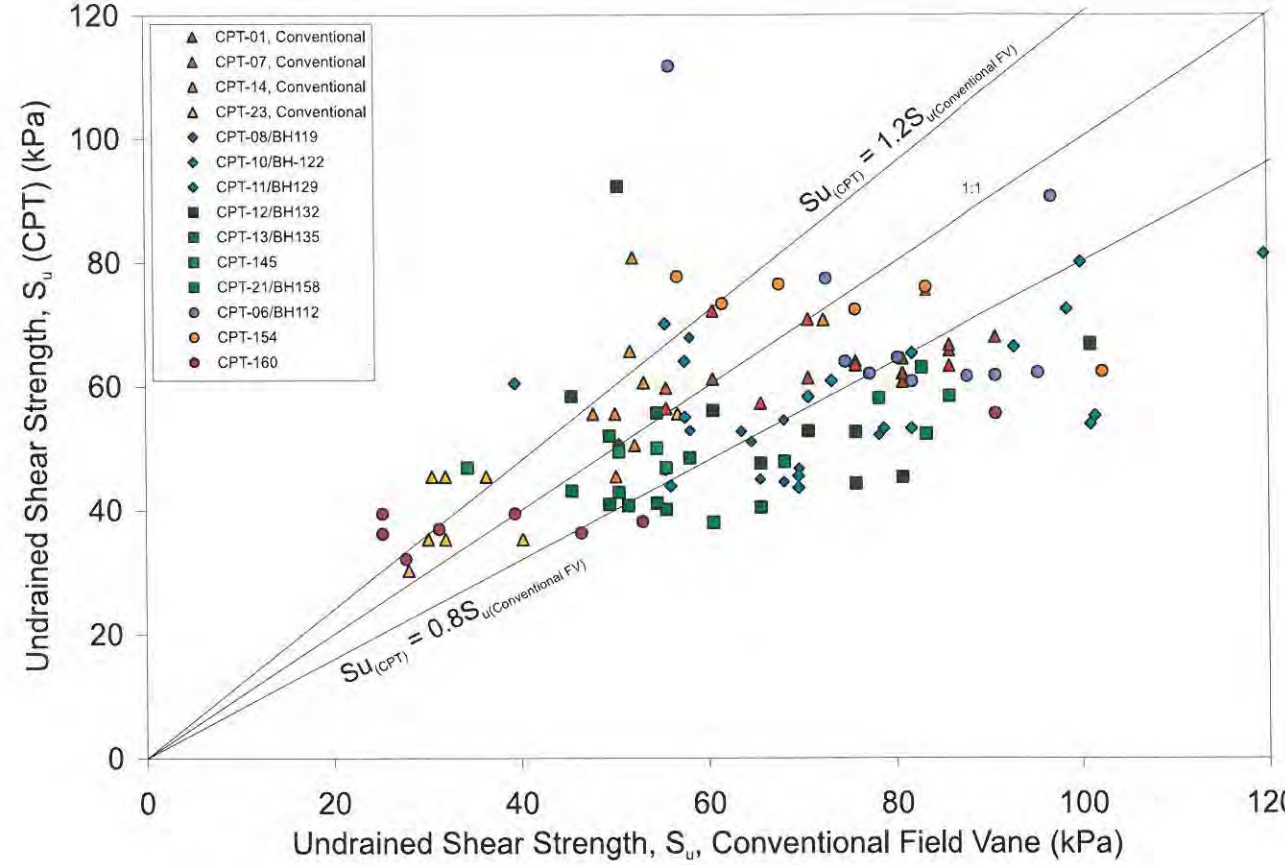
TITLE

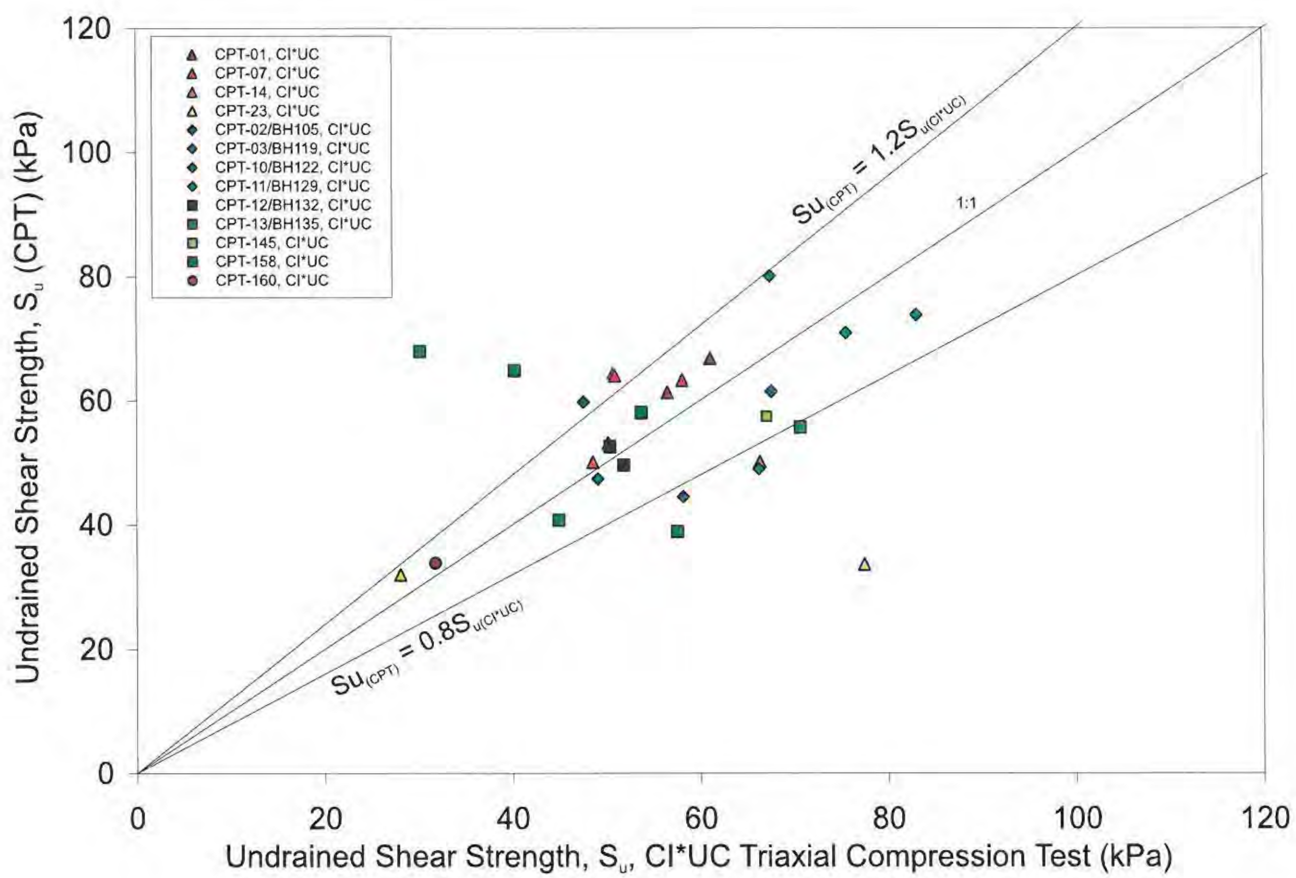
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE BH-23/CPT-23

Golder Associates

PROJECT No		07-1130-2070	FILE No 0711302070-R02061
			SCALE AS SHOWN REV
CADD	SJB	JUN 09	
CHECK	My	16 Jun 09	6.1P







NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT

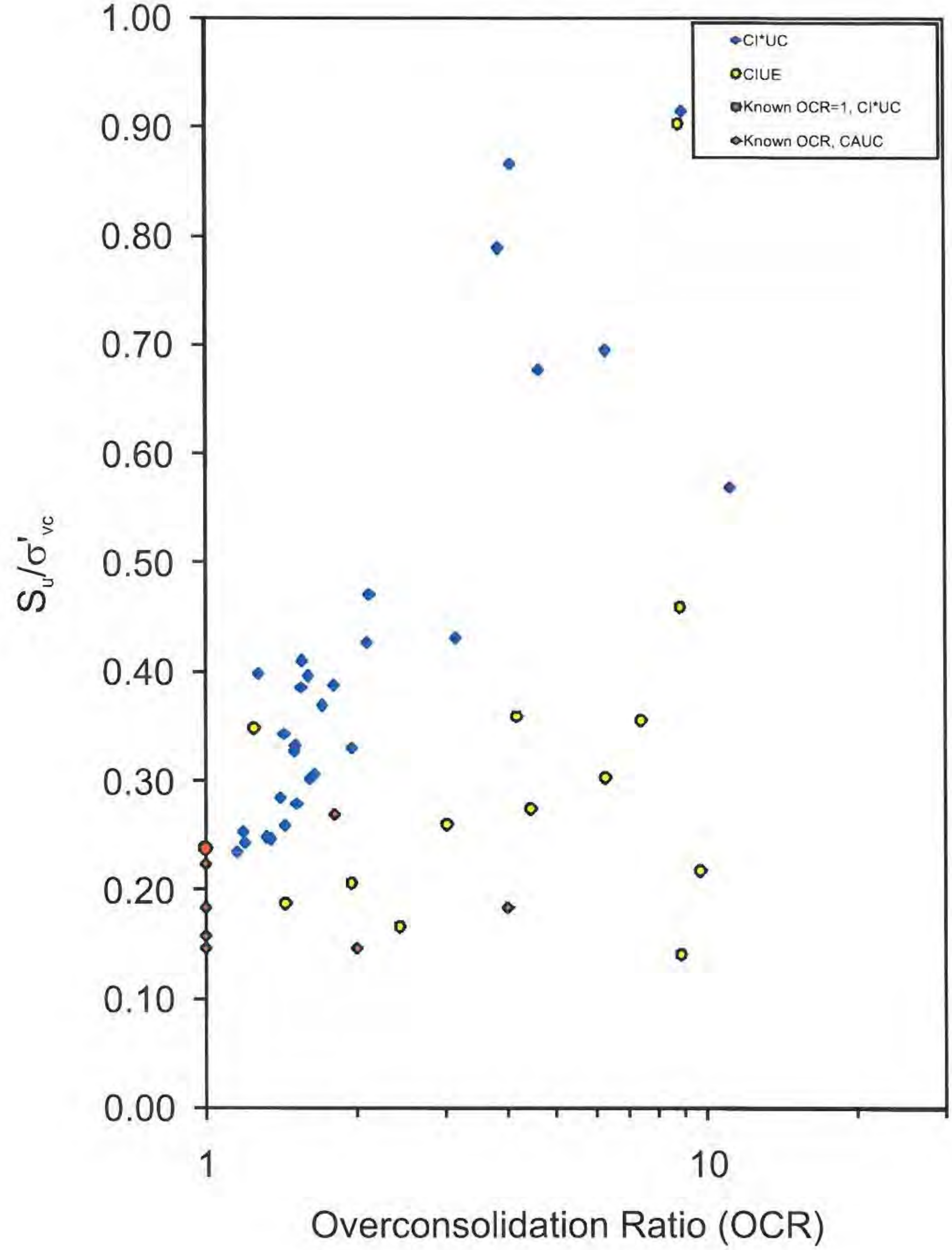
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

DATA SUMMARY
UNDRAINED SHEAR STRENGTH
CPT, LABORATORY AND FIELD VANE TESTS

PROJECT No.	0711302070	FILE No	0711302070-R02062
		SCALE	AS SHOWN
CADD	SJB	JUN 09	6.2
CHECK			

Golder Associates



NOTES

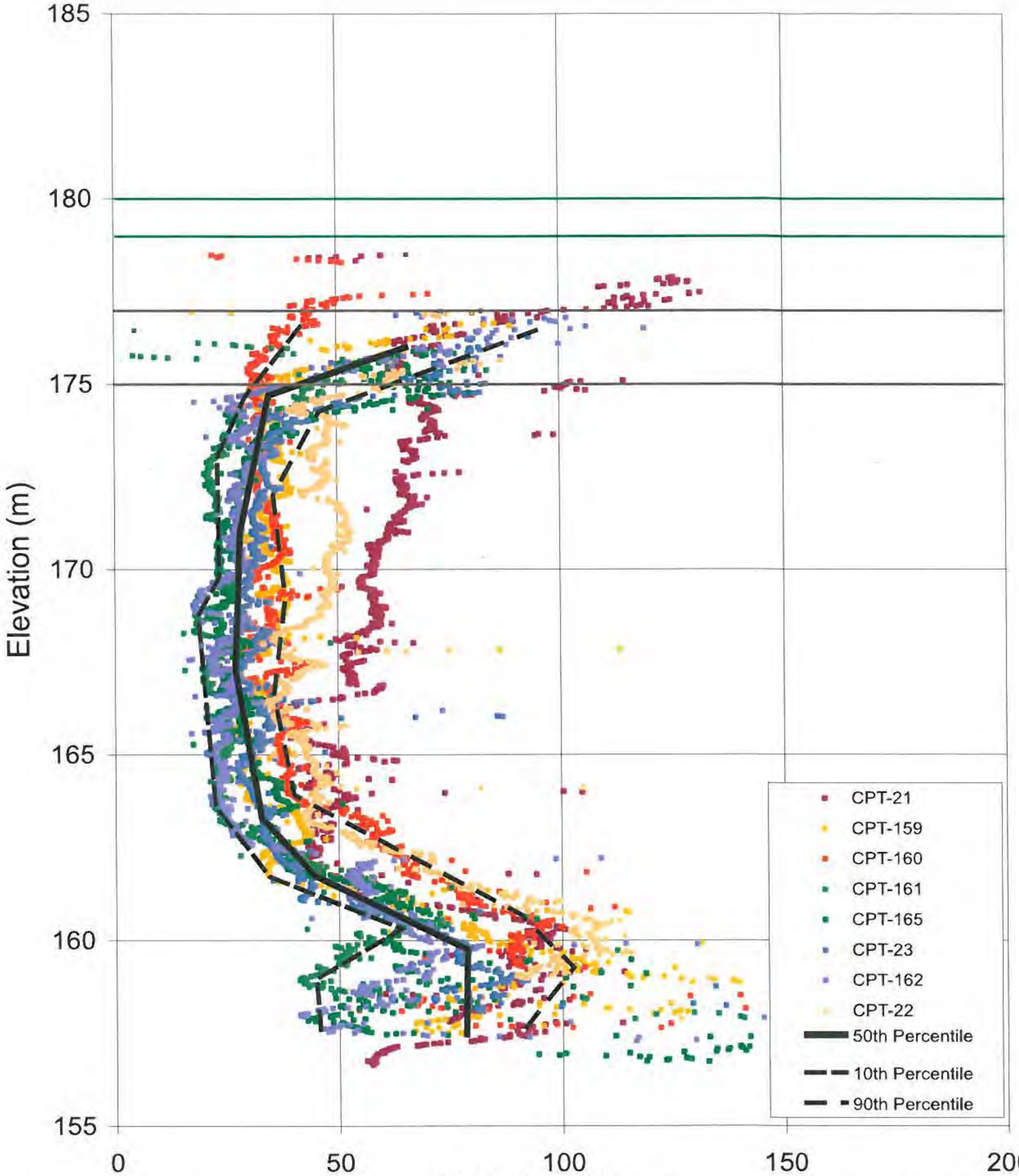
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE AND ON SEPARATE FIGURES.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
LABORATORY UNDRAINED SHEAR STRENGTH
DATA COMPARISON

Golder Associates

PROJECT No	07-1130-2070	FILE No.	0711302070-R02063
		SCALE	AS SHOWN REV 01
CADD	SJB	JUN 09	
CHECK	MY	26 Jun 09	6.3



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT CPT LOCATIONS

NOTES

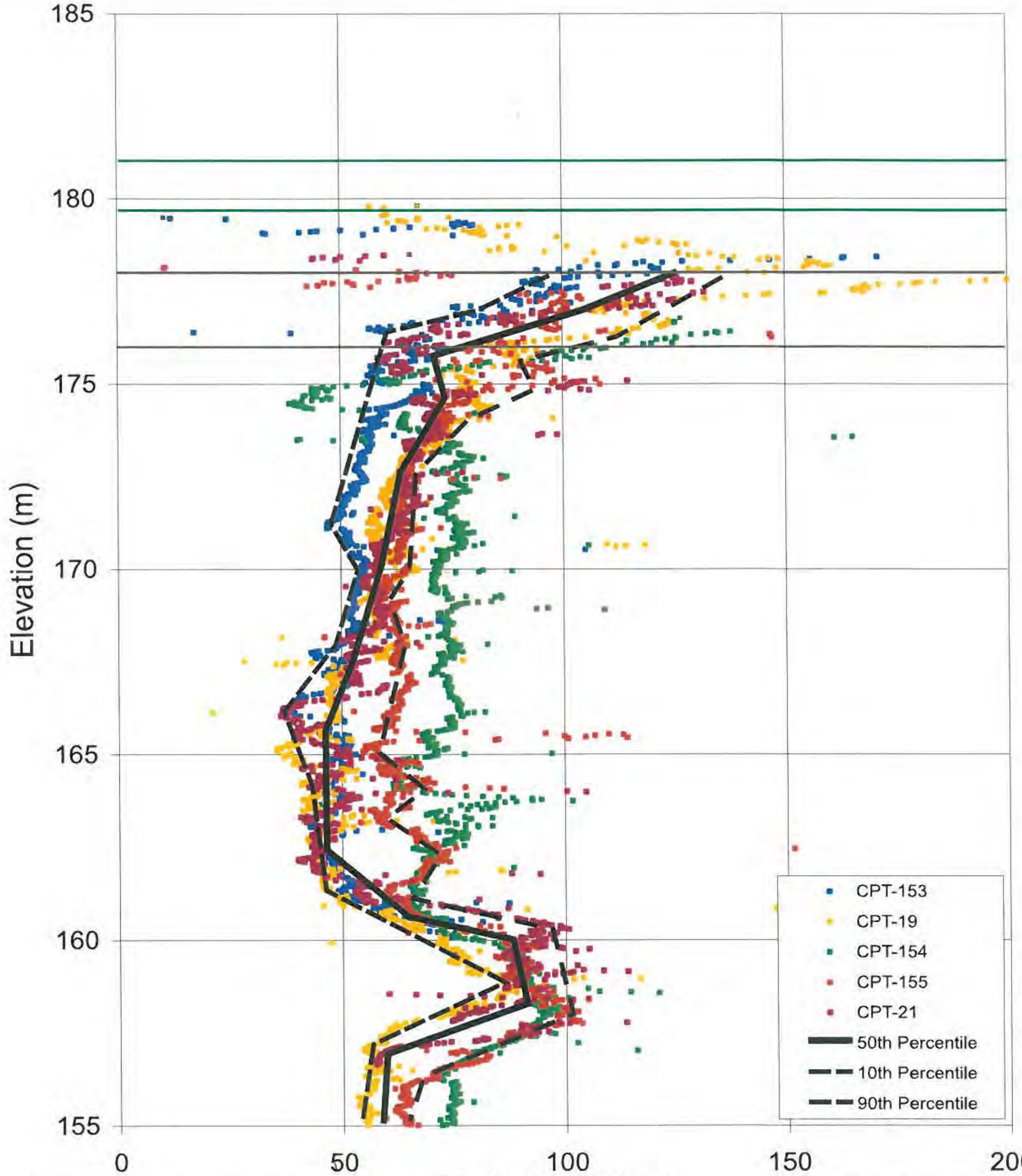
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 9+900 to 11+100

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02064
		SCALE	AS SHOWN REV 01
CADD	SJB	JUN 09	
CHECK	HW	26 Jun 09	**6.4A**



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT CPT LOCATIONS

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

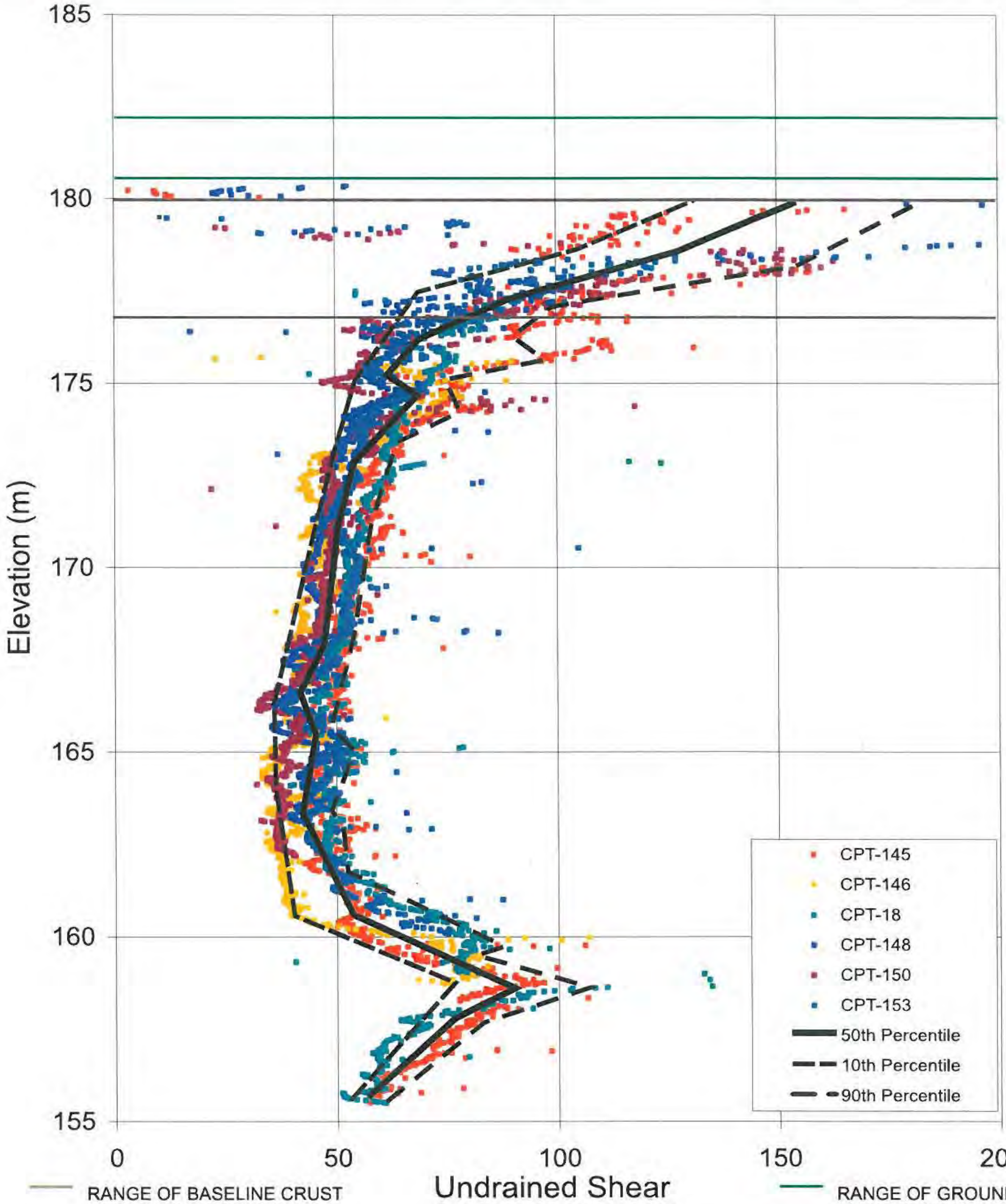
TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 11+100 to 12+100

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	MW	25 Jun 09		6.4B

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT CPT LOCATIONS

NOTES

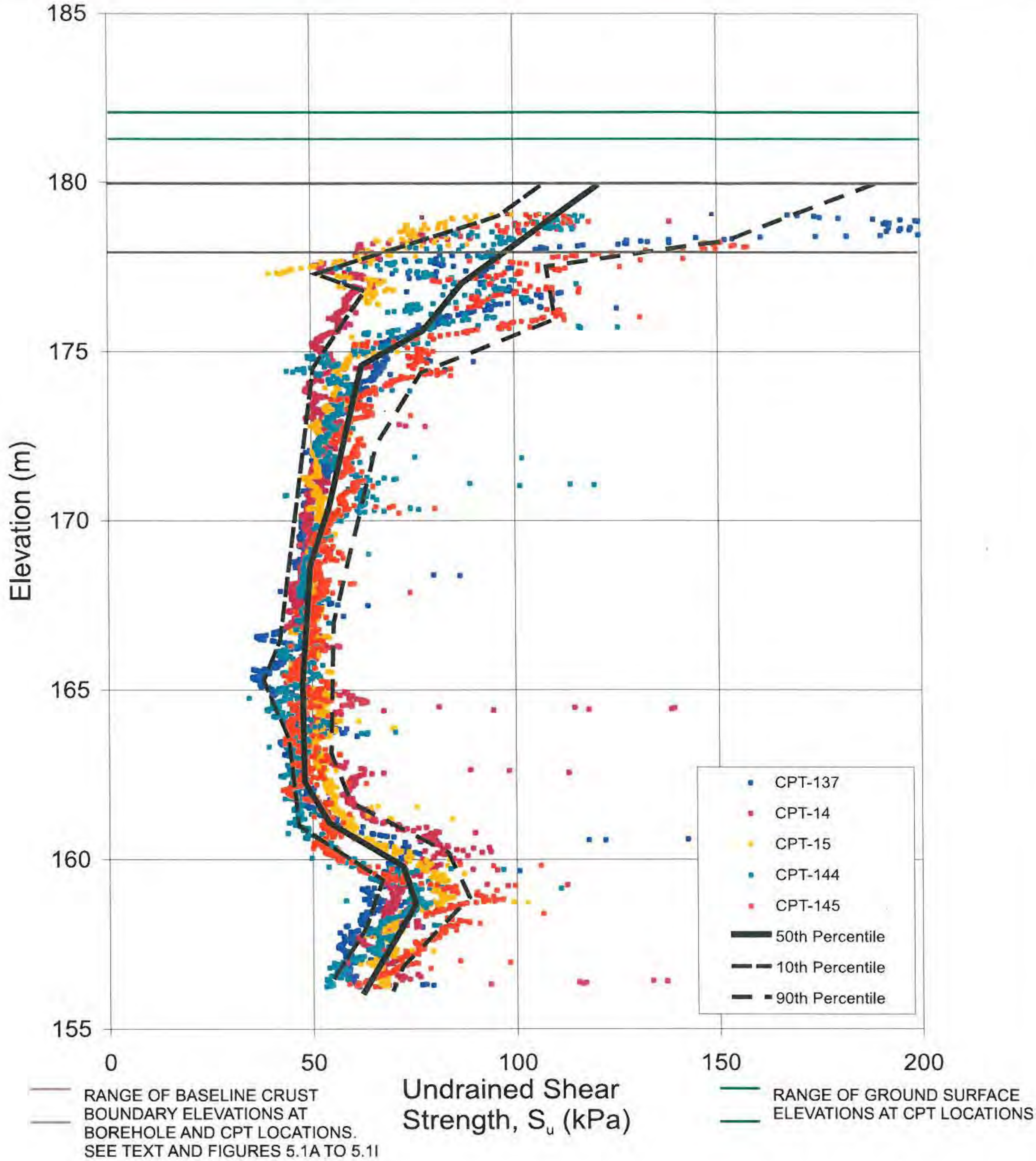
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 12+100 to 13+000

Golder Associates

PROJECT No	07-1130-2070	FILE No	0711302070-R02064
		SCALE	AS SHOWN REV 01
CADD	SJB	JUN 09	
CHECK	[illegible]	26 Jun 09	6.4C



PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

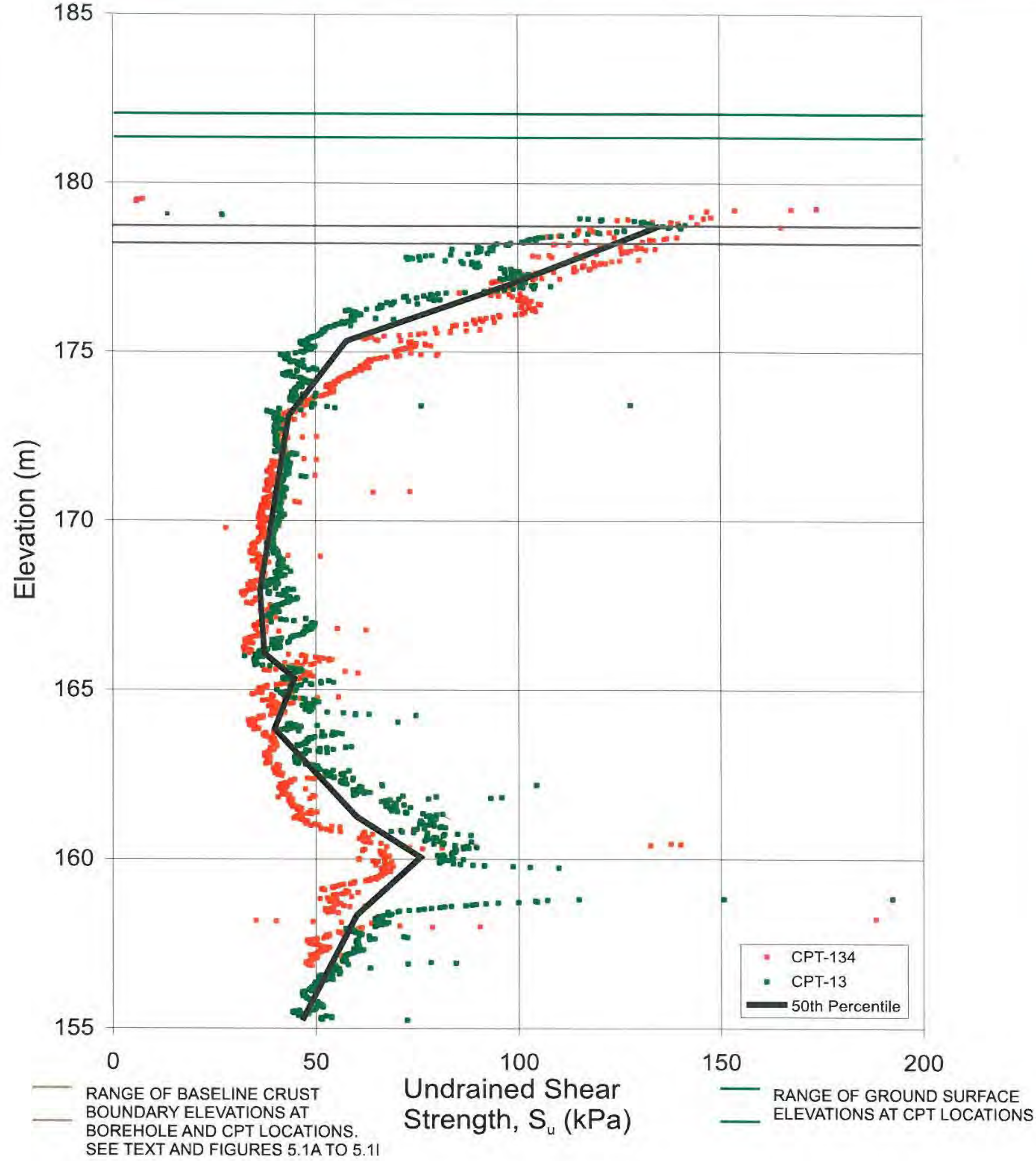
TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 13+000 to 13+900

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

Golder Associates

PROJECT No.	07-1130-2070		FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		6.4D
CHECK				



NOTES

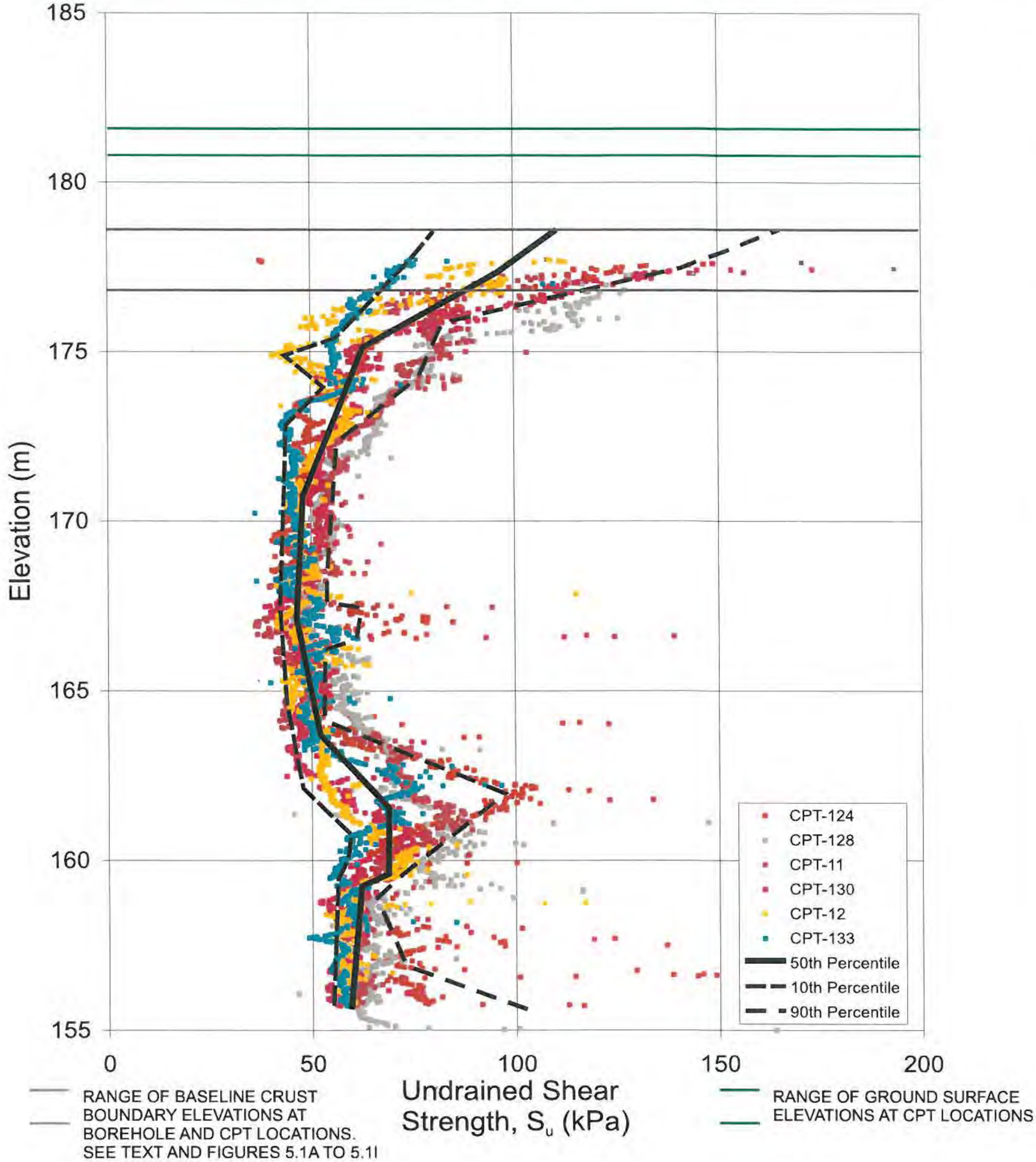
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE 10TH AND 90TH PERCENTILE PROFILES FOR THIS SECTION ARE TO BE TAKEN AS 80% AND 125% OF THE VALUES INDICATED BY THE 50TH PERCENTILE PROFILE

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 13+900 to 14+450

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R02064	
		SCALE	AS SHOWN	REV. 01
CADD	SJB	JUN 09		
CHECK	Mrs	26 Jun 09	6.4E	



PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

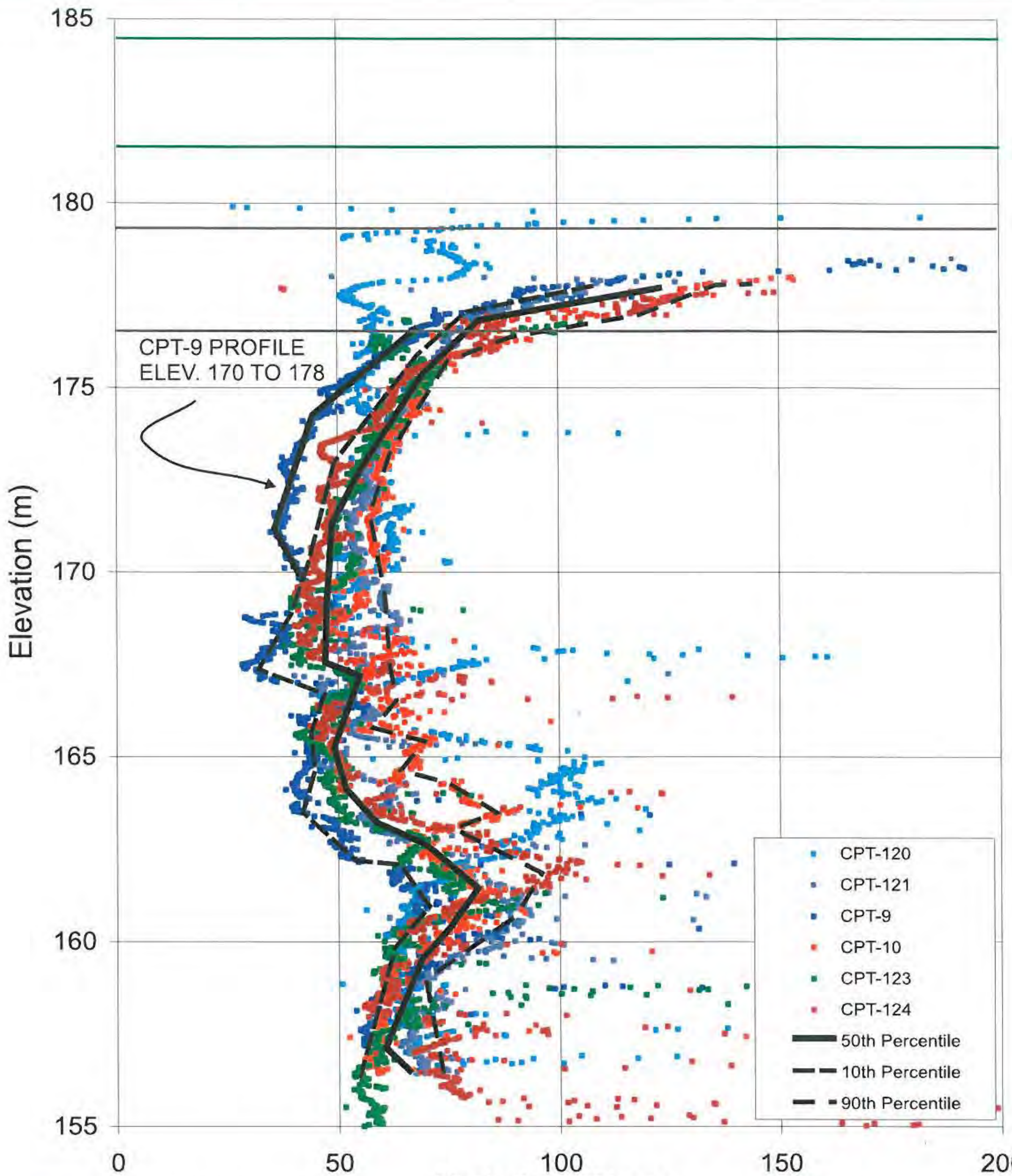
TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 14+450 to 10+300

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	AAM	26 Jun 09		6.4F

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.



RANGE OF BASELINE CRUST BOUNDARY ELEVATIONS AT BOREHOLE AND CPT LOCATIONS. SEE TEXT AND FIGURES 5.1A TO 5.1I

RANGE OF GROUND SURFACE ELEVATIONS AT CPT LOCATIONS

NOTES

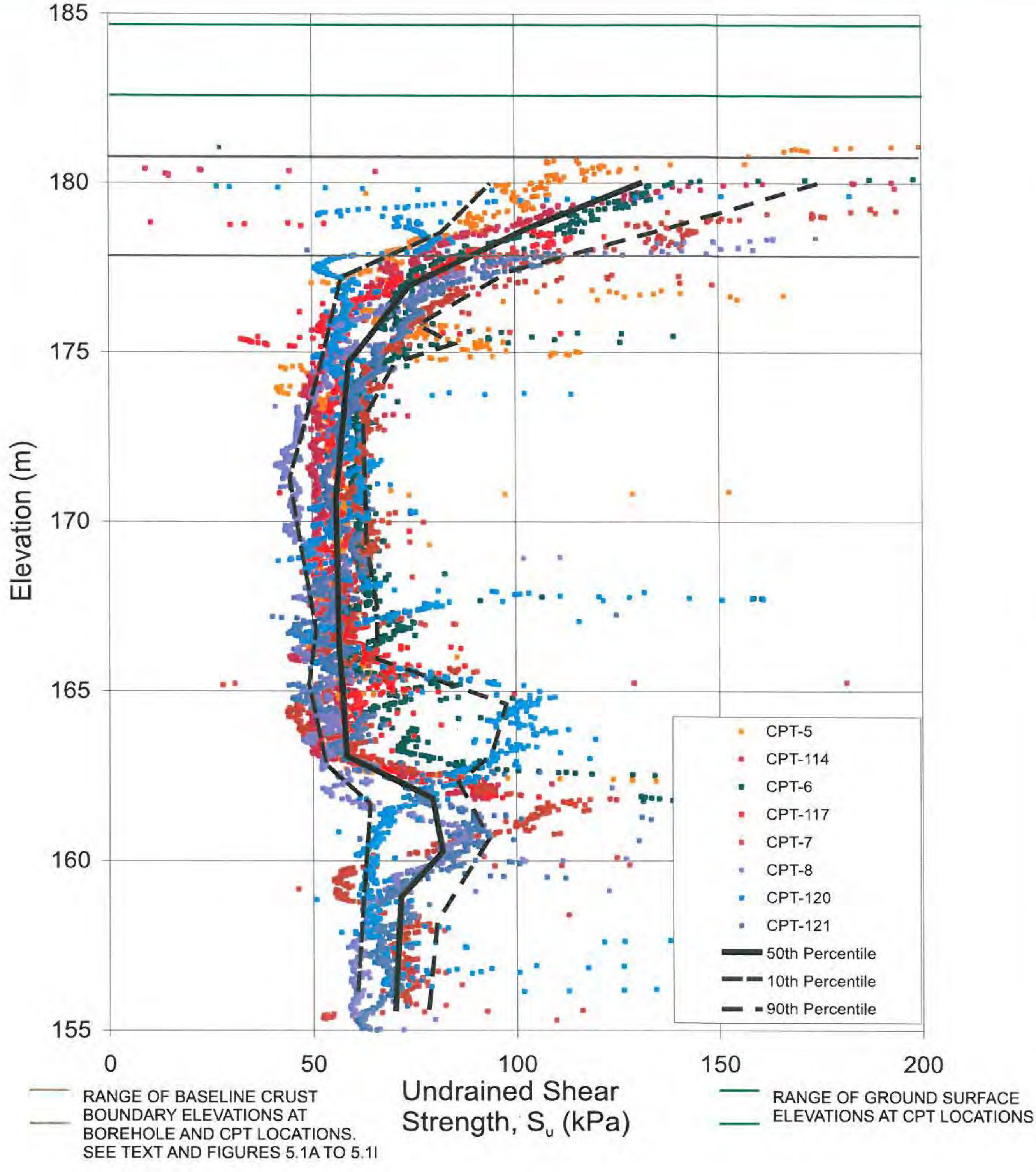
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE BASELINE UNDRAINED SHEAR STRENGTH BETWEEN CPT-123 AND CPT-9 AND CPT-9 AND CPT-120 IS TO BE LINEARLY INTERPOLATED BETWEEN THE PROFILES ILLUSTRATED FOR THE 50TH PERCENTILE AND THE PROFILE FOR CPT-9 BASED ON THE STRAIGHT LINE DISTANCE BETWEEN THE LOCATIONS OF THESE TESTS. THE CPT-9 PROFILE AS INDICATED IS CONSIDERED REPRESENTATIVE OF THE 50TH PERCENTILE PROFILE. THE 10TH AND 90TH PERCENTILE PROFILES AT CPT-106 ARE TO BE TAKEN AS 80% AND 125% OF THE VALUES INDICATED BY THE 50TH PERCENTILE PROFILE AT THIS LOCATION

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 10+300 to 11+300

Golder Associates

PROJECT No.		07-1130-2070	FILE No. 0711302070-R02064
			SCALE AS SHOWN REV 01
CADD	SJB	JUN 09	
CHECK	MY	26 Jun 09	6.4G



PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

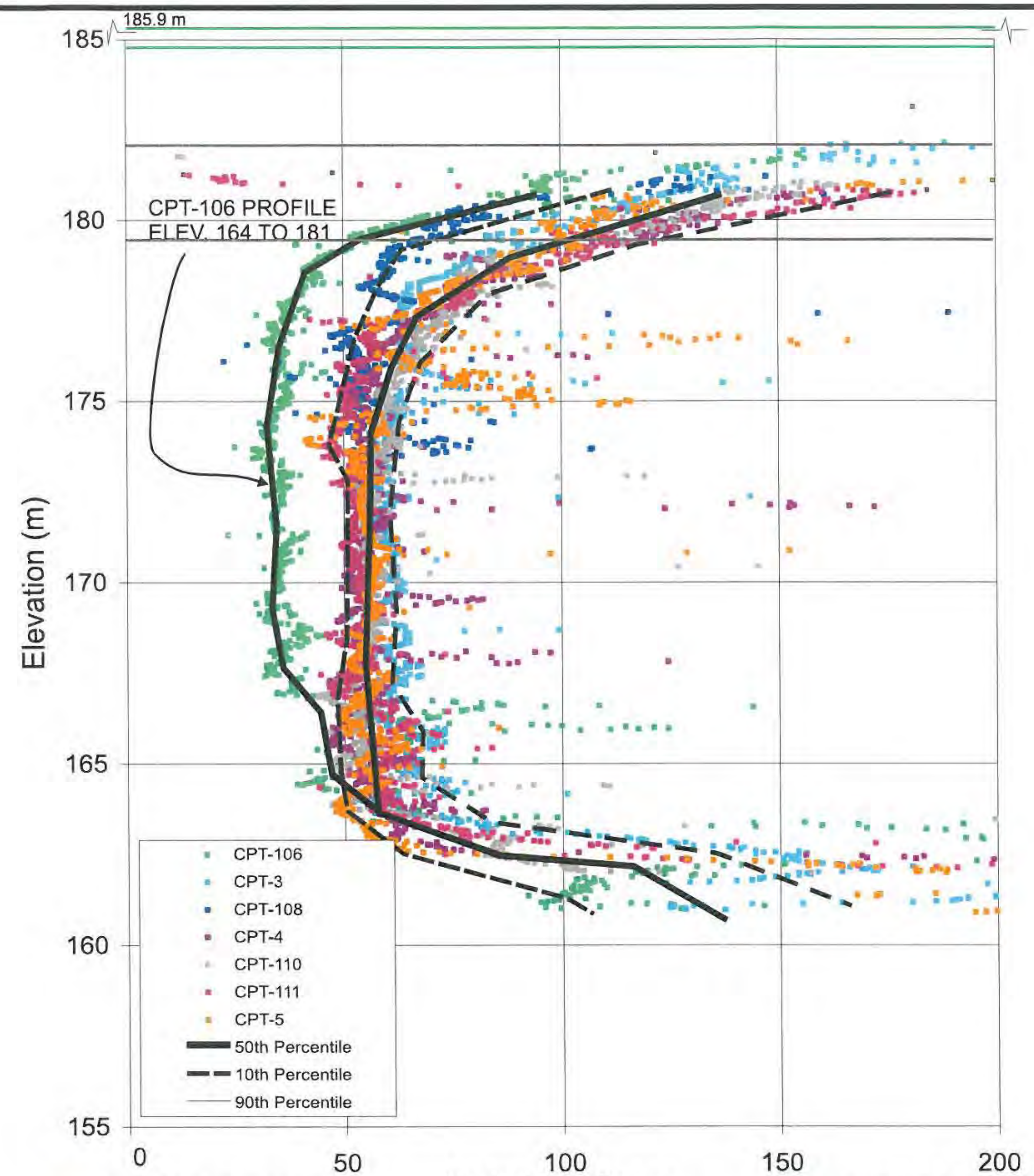
TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 11+300 to 12+600

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	[illegible]	26 Jun 09		6.4H



NOTES

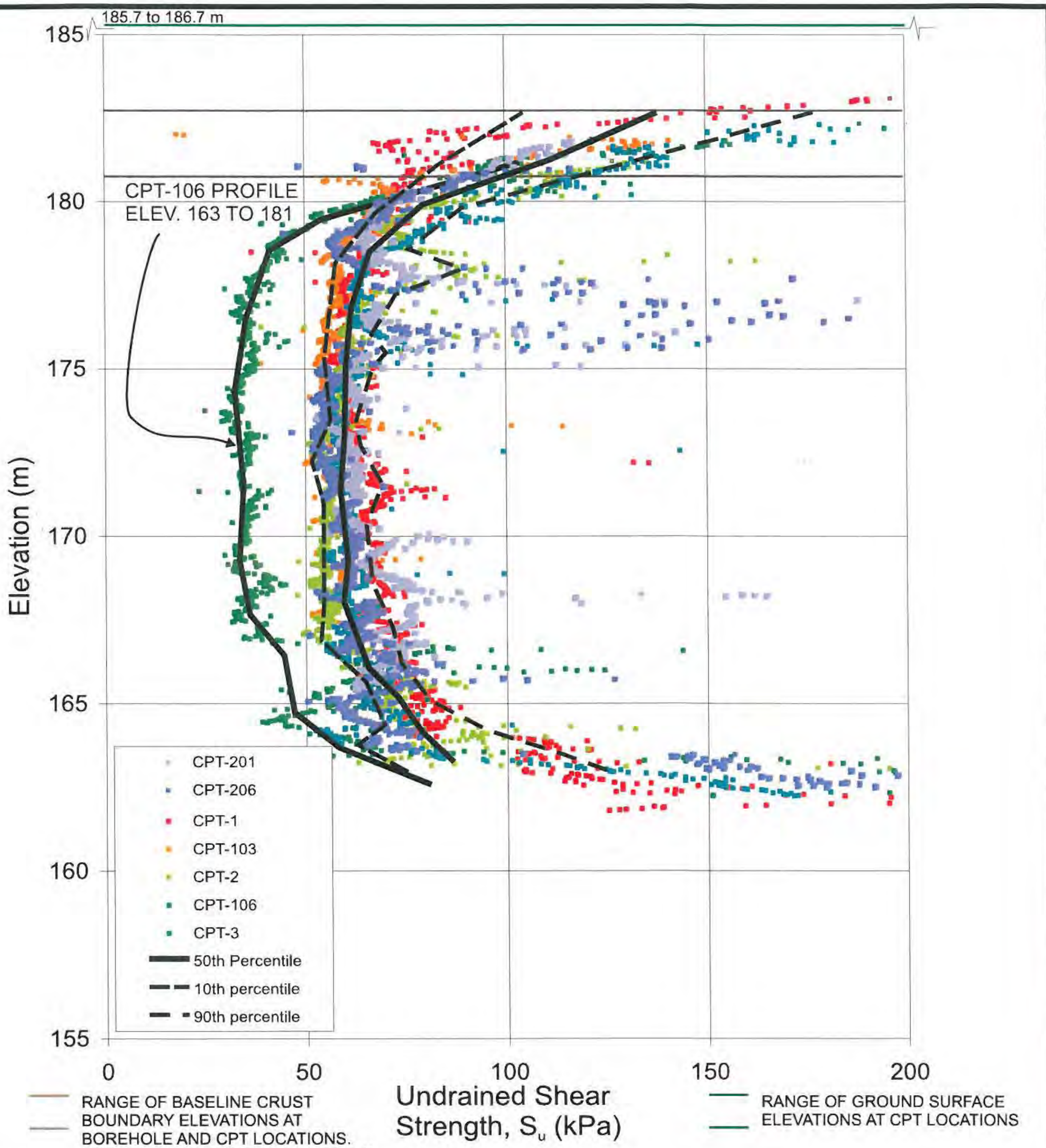
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE BASELINE UNDRAINED SHEAR STRENGTH BETWEEN CPT-106 AND CPT-2 AND CPT-106 AND CPT-3 IS TO BE LINEARLY INTERPOLATED BETWEEN THE PROFILES ILLUSTRATED FOR THE 50TH PERCENTILE AND THE PROFILE FOR CPT-106 BASED ON THE STRAIGHT LINE DISTANCE BETWEEN THE LOCATIONS OF THESE TESTS. THE CPT-106 PROFILE AS INDICATED IS CONSIDERED REPRESENTATIVE OF THE 50TH PERCENTILE PROFILE. THE 10TH AND 90TH PERCENTILE PROFILES AT CPT-106 ARE TO BE TAKEN AS 80% AND 125% OF THE VALUES INDICATED BY THE 50TH PERCENTILE PROFILE AT THIS LOCATION

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 12+600 to 13+600

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	KM	25 Jun 09		6.4I



NOTES

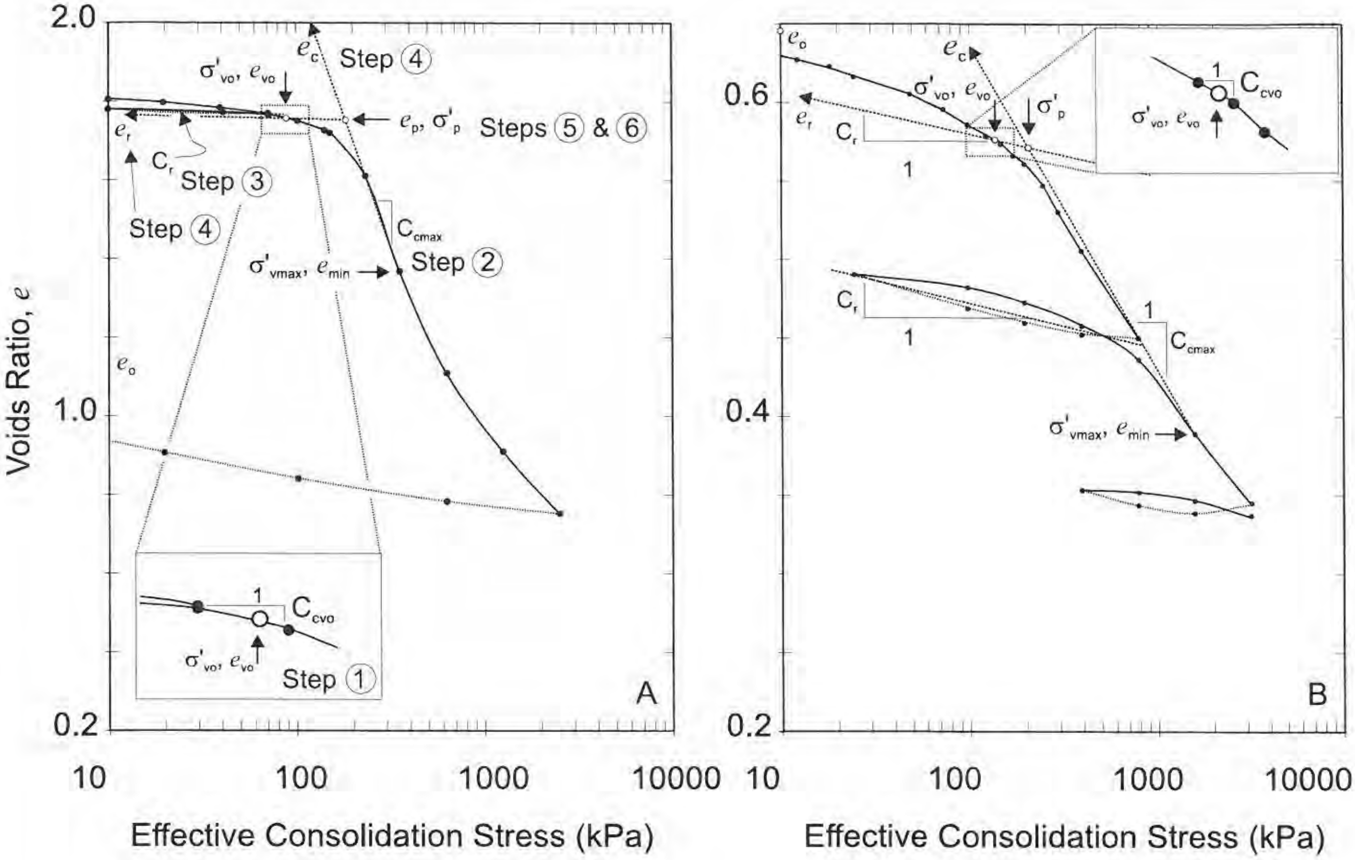
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE BASELINE GEOTECHNICAL UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. THE BASELINE UNDRAINED SHEAR STRENGTH BETWEEN CPT-106 AND CPT-2 AND CPT-106 AND CPT-3 IS TO BE LINEARLY INTERPOLATED BETWEEN THE PROFILES ILLUSTRATED FOR THE 50TH PERCENTILE AND THE PROFILE FOR CPT-106 BASED ON THE STRAIGHT LINE DISTANCE BETWEEN THE LOCATIONS OF THESE TESTS. THE CPT-106 PROFILE AS INDICATED IS CONSIDERED REPRESENTATIVE OF THE 50TH PERCENTILE PROFILE. THE 10TH AND 90TH PERCENTILE PROFILES AT CPT-106 ARE TO BE TAKEN AS 80% AND 125% OF THE VALUES INDICATED BY THE 50TH PERCENTILE PROFILE AT THIS LOCATION

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
BASELINE UNDRAINED SHEAR STRENGTH PROFILE
STATION 13+600 to 10+900

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02064
			SCALE	AS SHOWN REV 01
CADD	SJB	JUN 09		
CHECK	MJ	25 Jun 09		6.4J



C_c = virgin compression index

C_r = recompression index

C_{cvo} = compression index at in situ vertical effective stress

σ'_{vo} = in situ vertical effective stress

σ'_p = preconsolidation pressure

σ'_{vma} = maximum vertical effective stress at end of load increment producing largest value of C_c

e_{min} = minimum voids ratio at end of load increment producing largest value of C_c

e_r = voids ratio axis intercept value at an effective consolidation stress of 1 kPa based on projection of recompression index line

e_c = voids ratio axis intercept value at an effective consolidation stress of 1 kPa based on projection of virgin compression index line

e_p = voids ratio at the preconsolidation pressure

NOTES

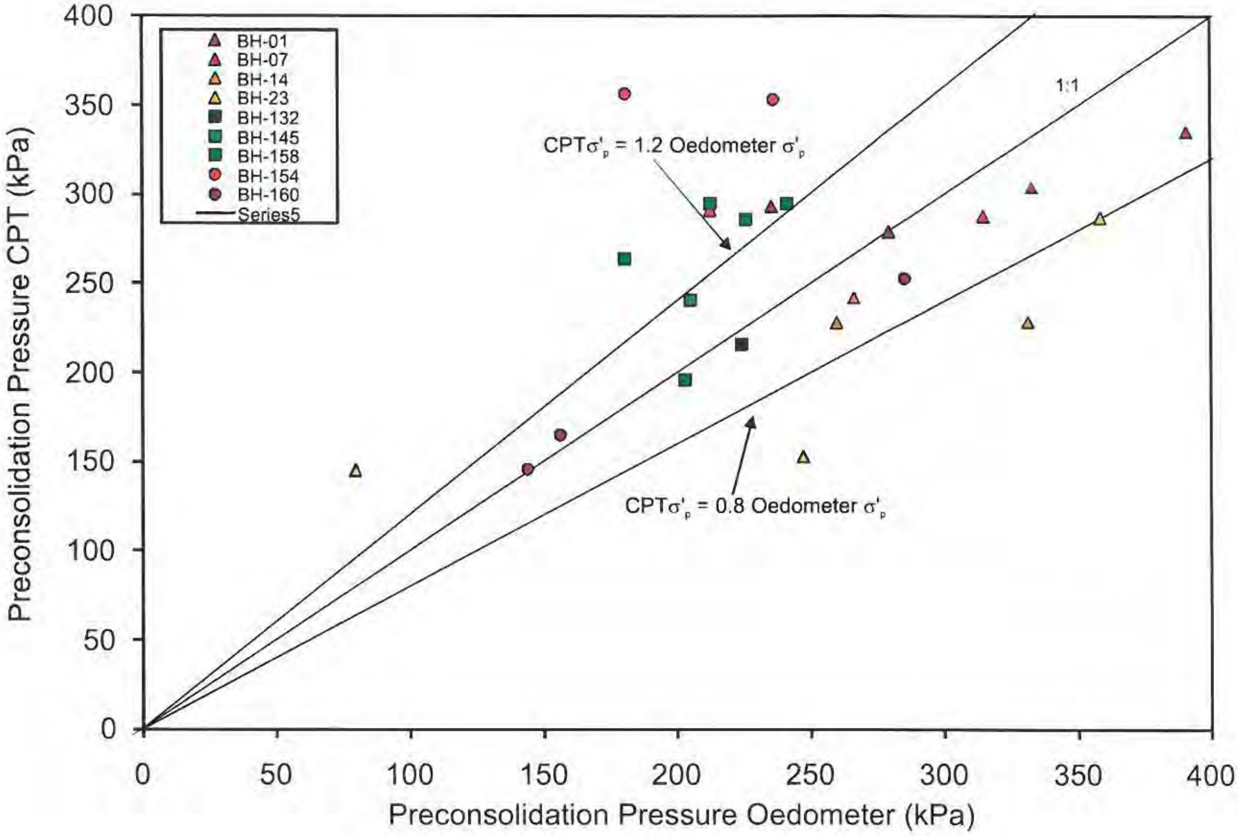
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE SUMMARIZES THE METHODOLOGY USED TO ESTIMATE THE PRECONSOLIDATION PRESSURE FROM OEDOMETER TESTS ON SAMPLES OF THE CLAYEY SILT TO SILTY CLAY DEPOSITS

PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
INTERPRETATION OF OEDOMETER TEST DATA

Golder Associates

PROJECT No	07-1130-2070	FILE No	0711302070-R02065
		SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09	
CHECK	[illegible]	[illegible]	6.5



NOTES

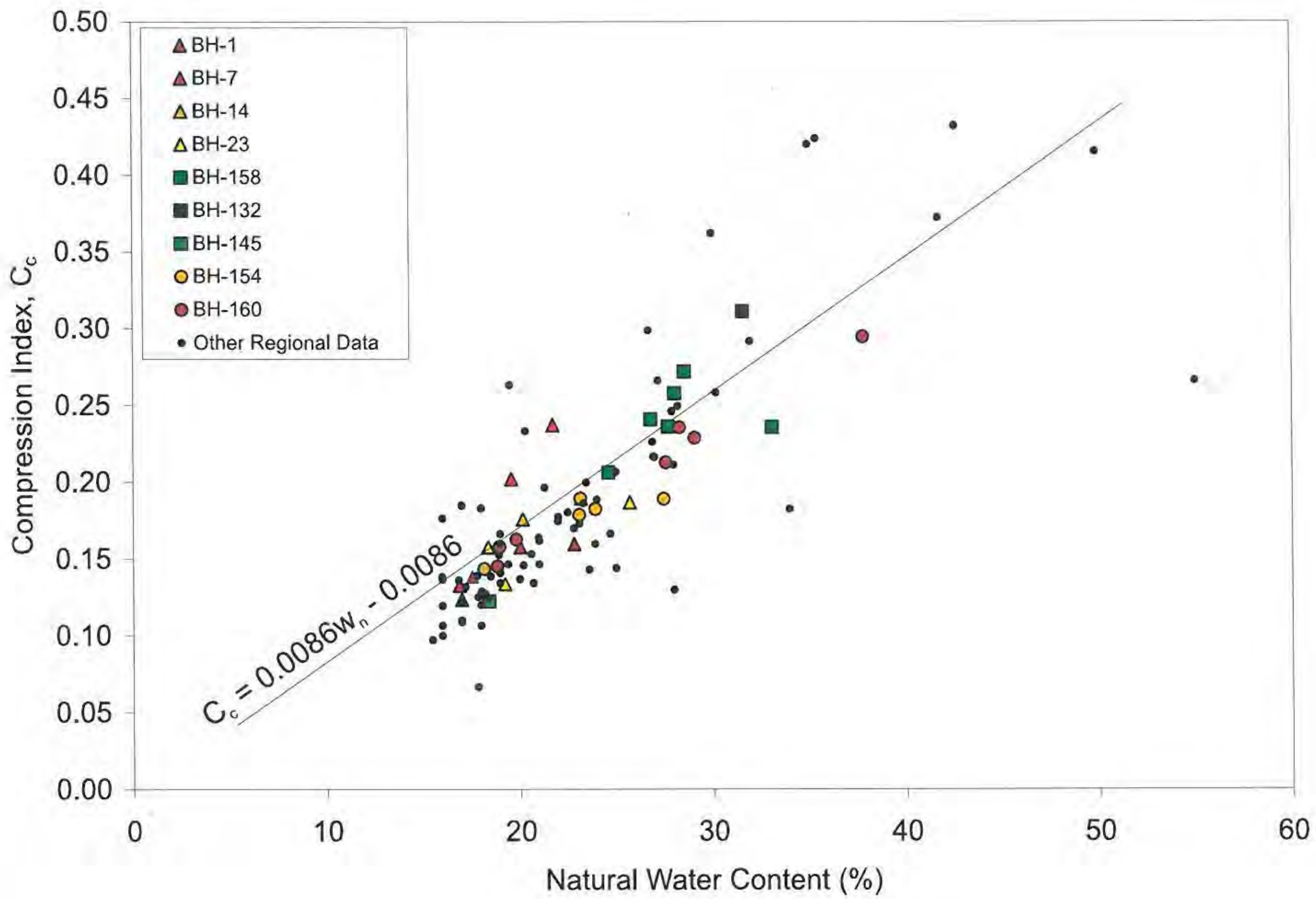
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL ENGINEERING PARAMETERS. BASELINE GEOTECHNICAL ENGINEERING PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE AND ON SEPARATE FIGURES.

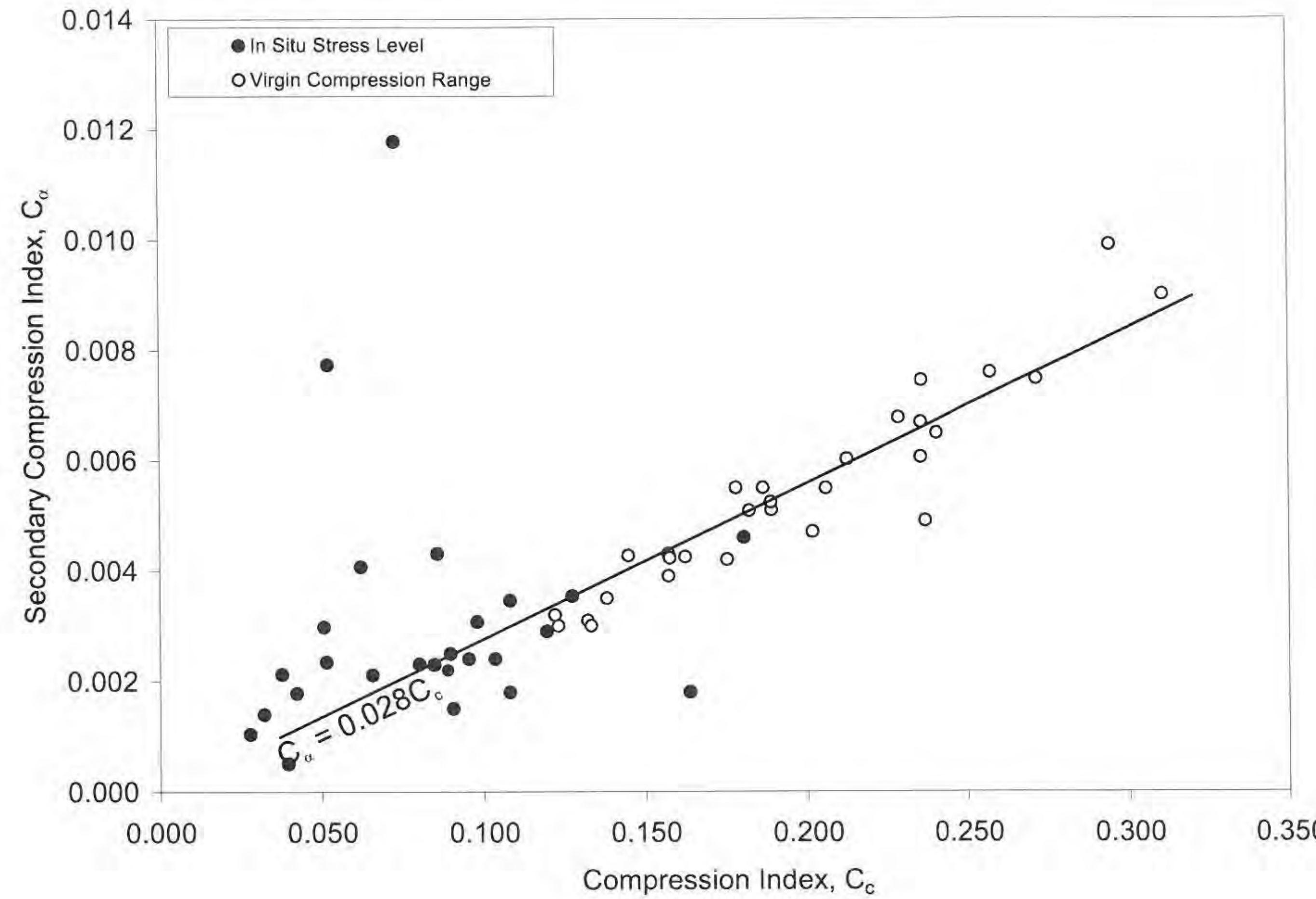
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO

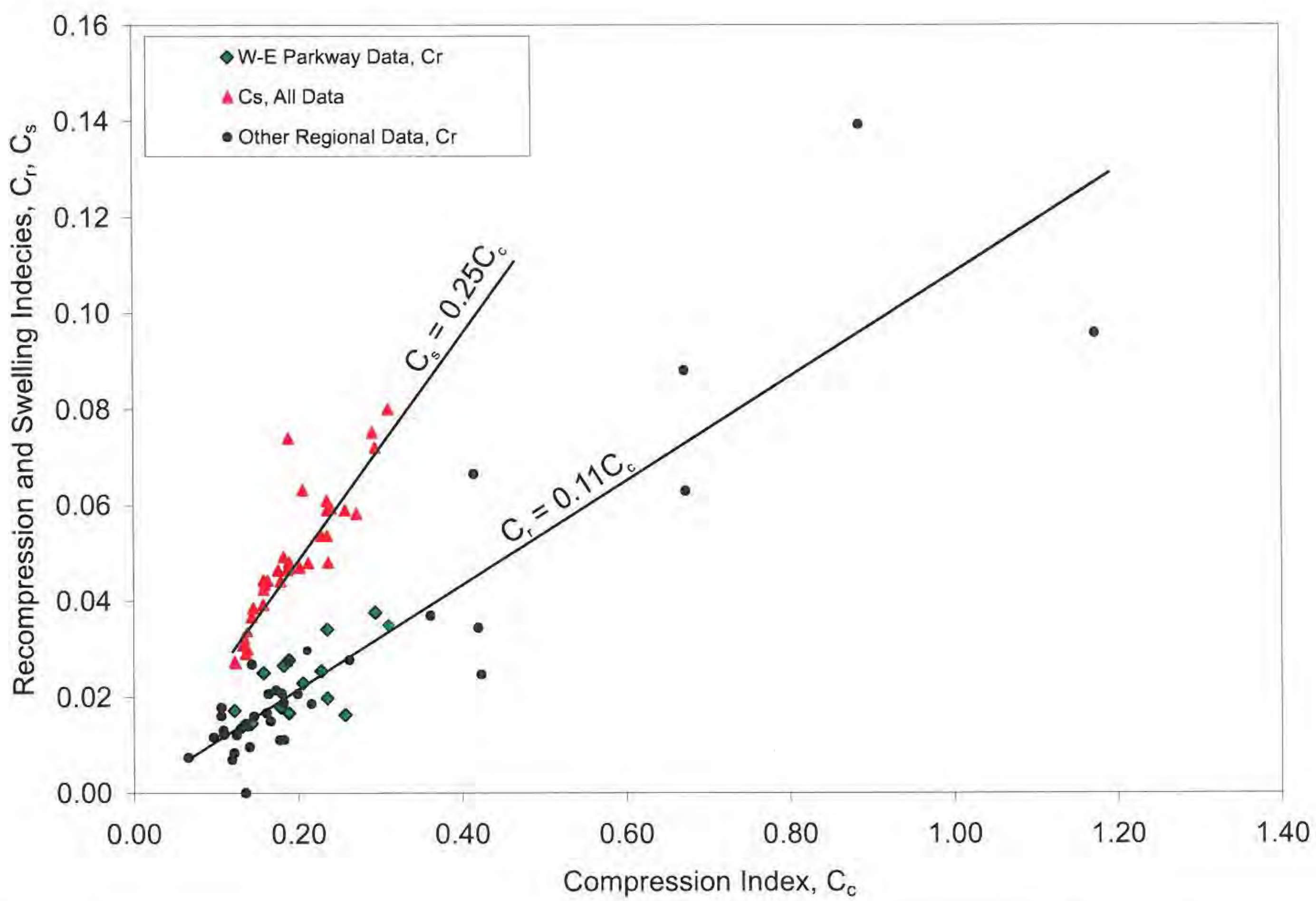
TITLE
DATA SUMMARY PRECONSOLIDATION PRESSURES DERIVED FROM CPT AND OEDOMETER TESTS

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02066
			SCALE	AS SHOWN
CADD	SJB	JUN 09		
CHECK	Wy	26 Jun 09		6.6







NOTES

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PROJECT

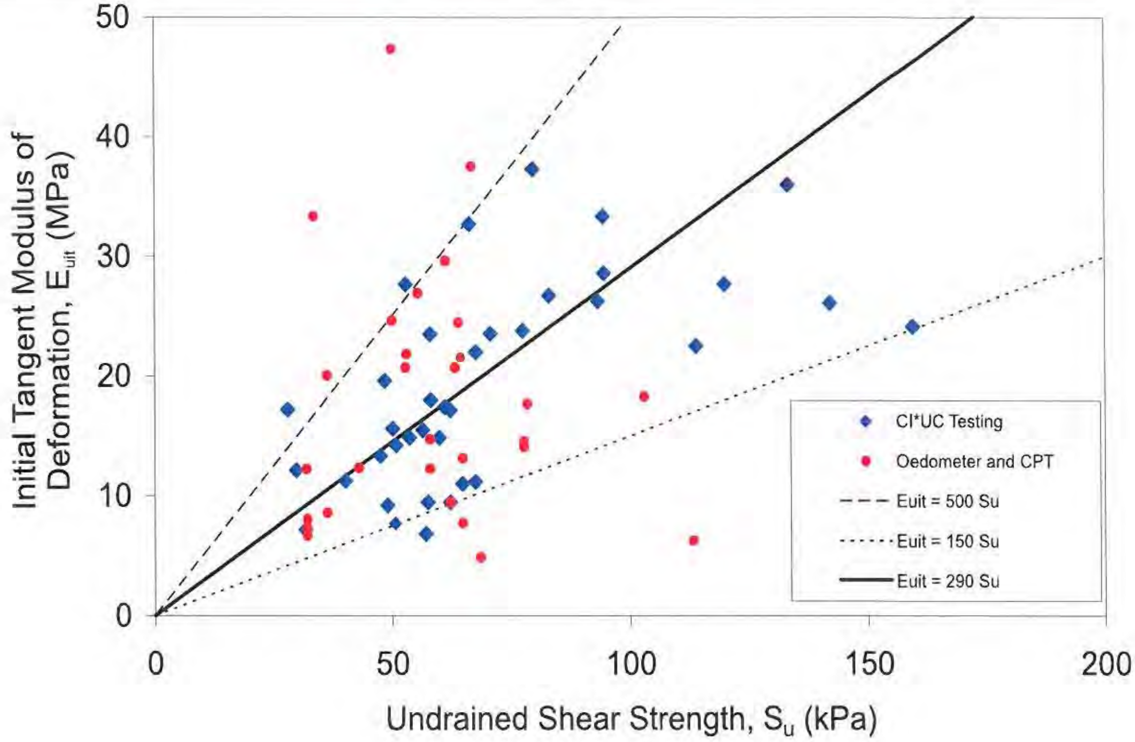
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

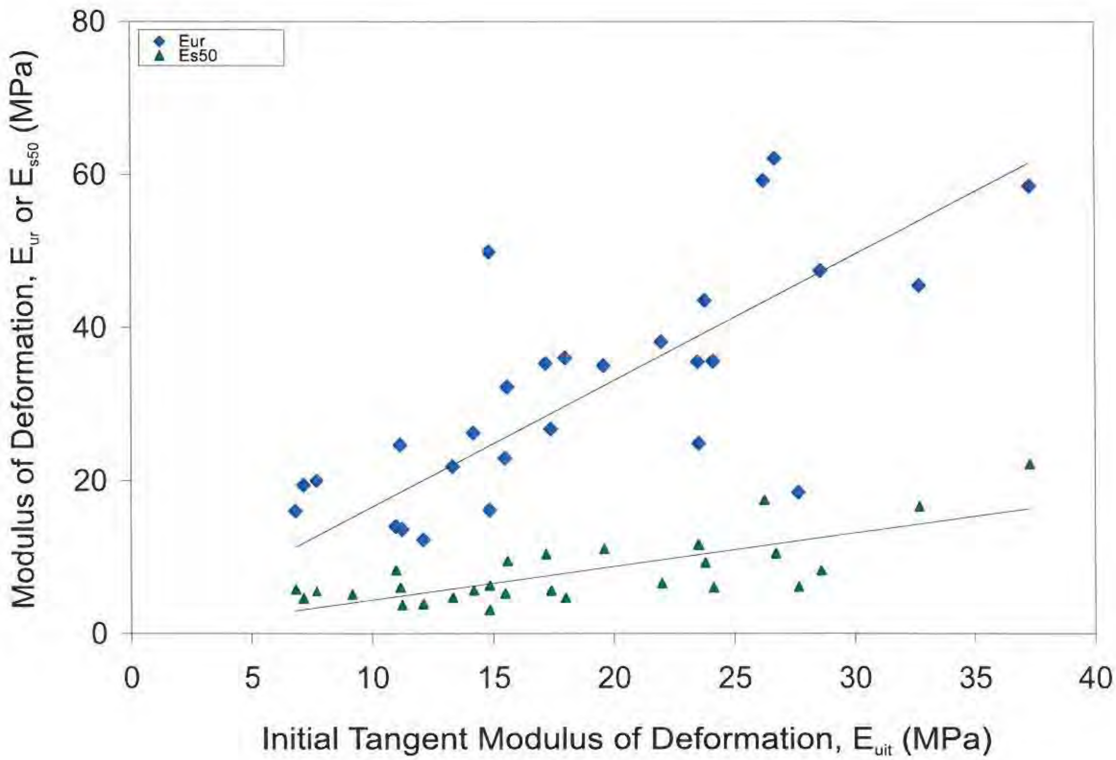
TITLE

DATA SUMMARY
COMPRESSION INDICES

Golder Associates

PROJECT No	07-1130-2070	FILE No	0711302070-R02066
		SCALE	AS SHOWN
CADD	SJB	JUN 09	
CHECK		26 Jun 09	6.7





NOTES

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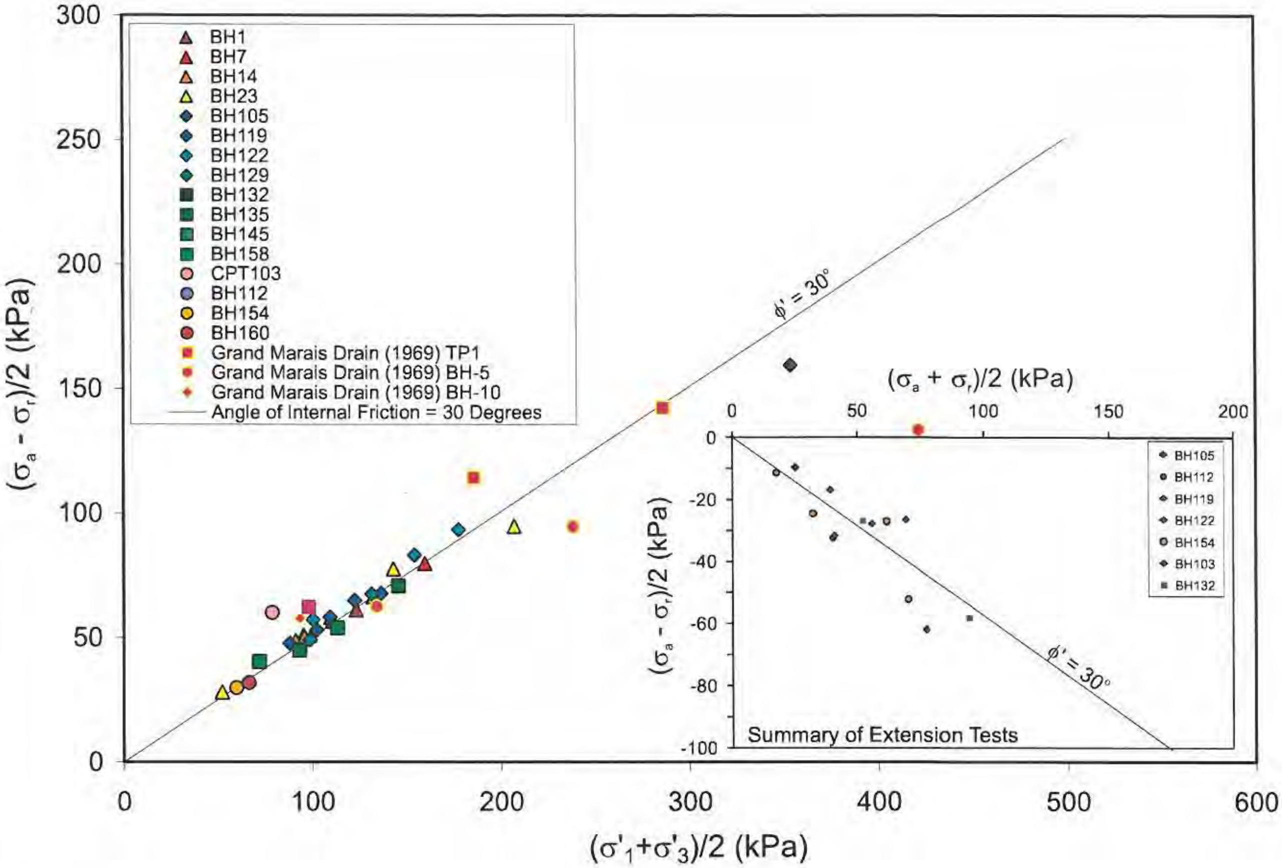
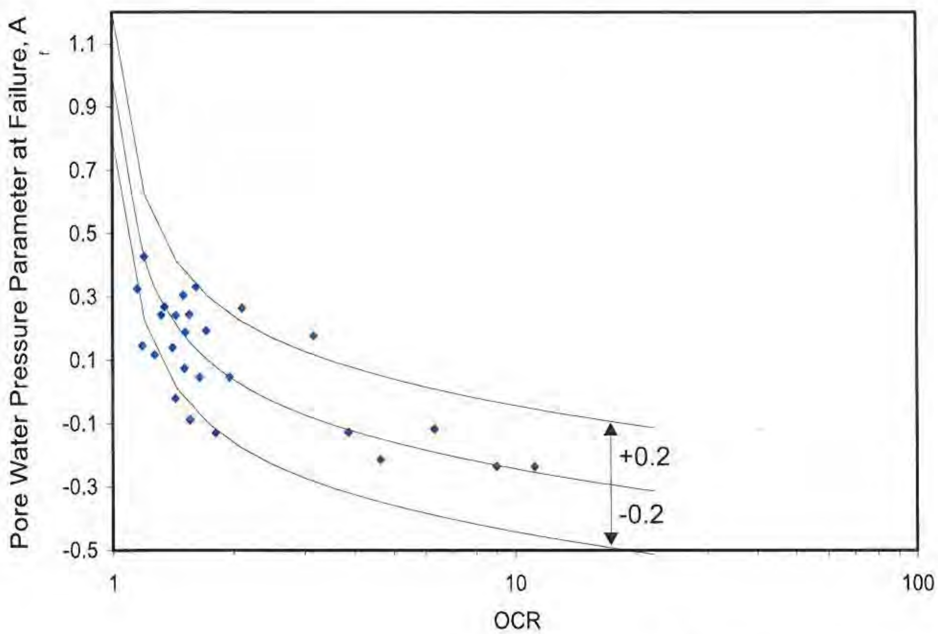
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
STRESS-STRAIN PARAMETERS

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02068
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	MY	26 Jun 09		6.8





σ'_a = axial effective stress at failure
σ'_r = radial effective confining stress at failure
OCR = overconsolidation ratio
A_f = pore water parameter at failure

NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
EFFECTIVE STRESS STRENGTH PARAMETERS

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R02069
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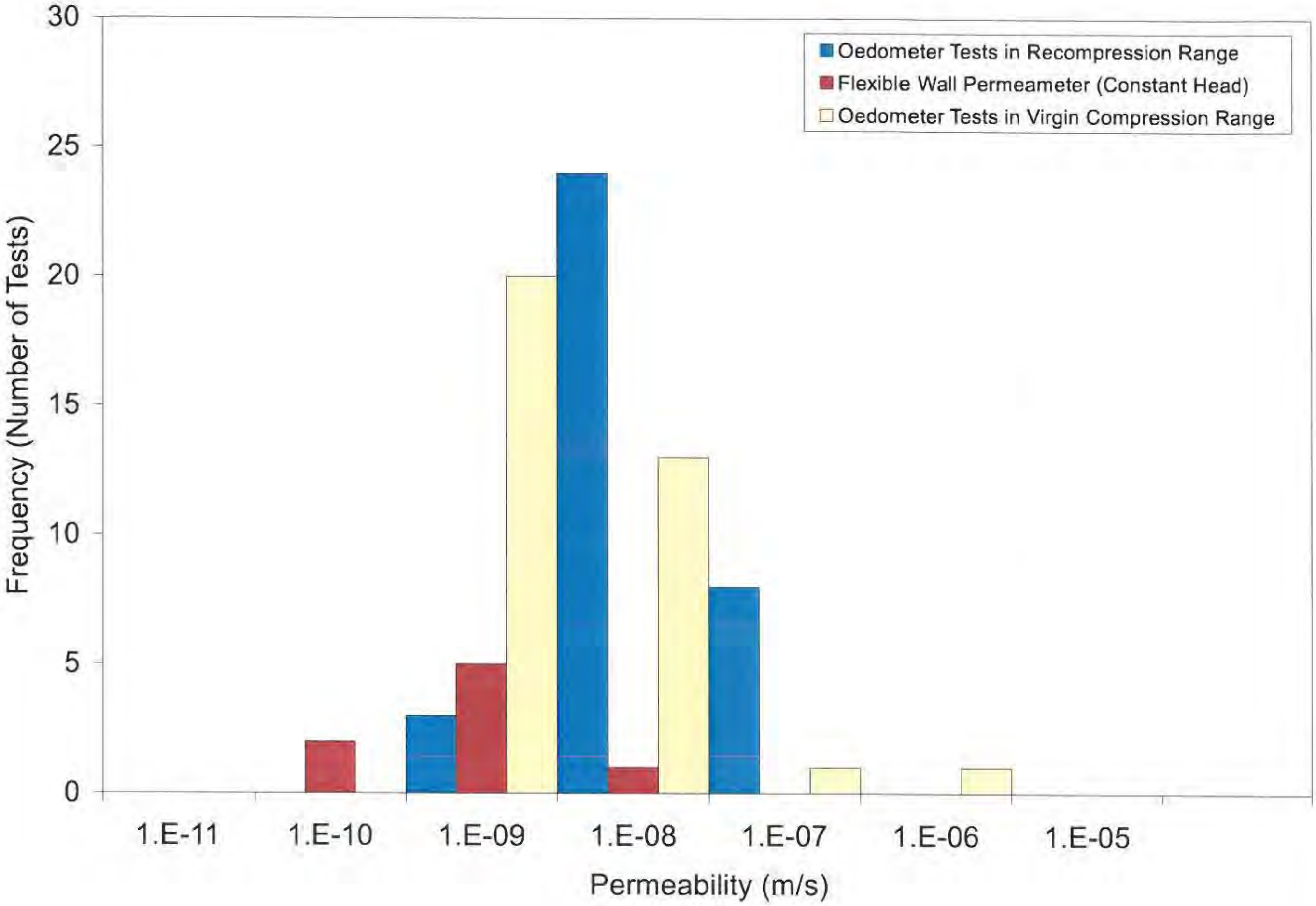
CHECK	MY	26 Jun 09		6.9



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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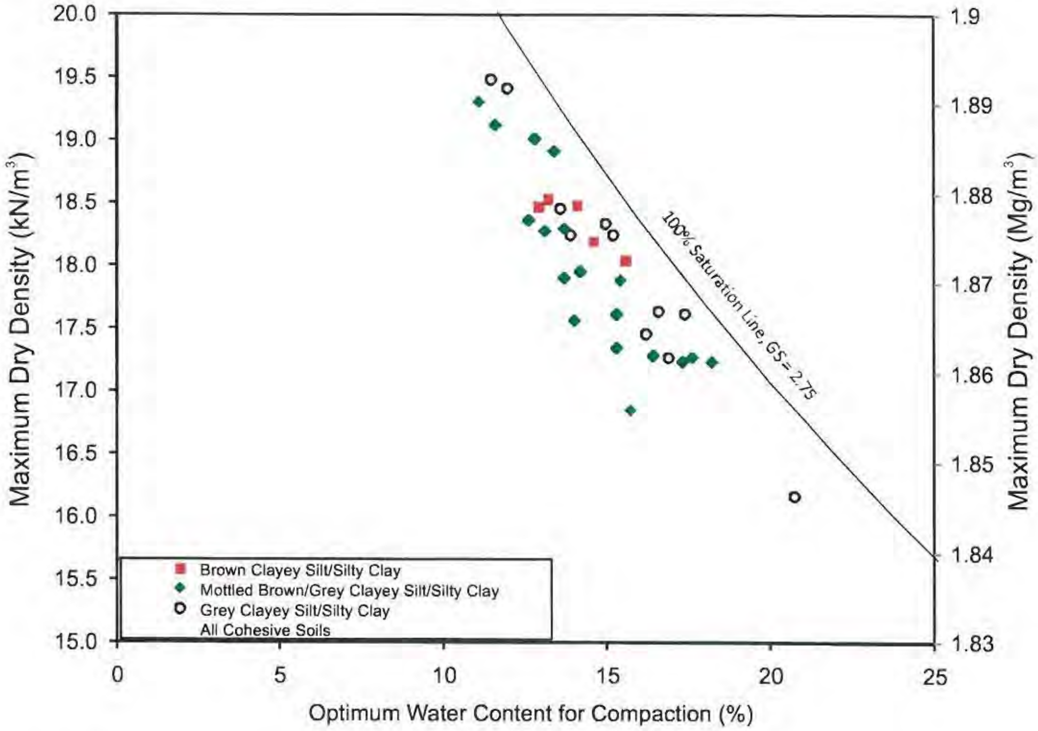
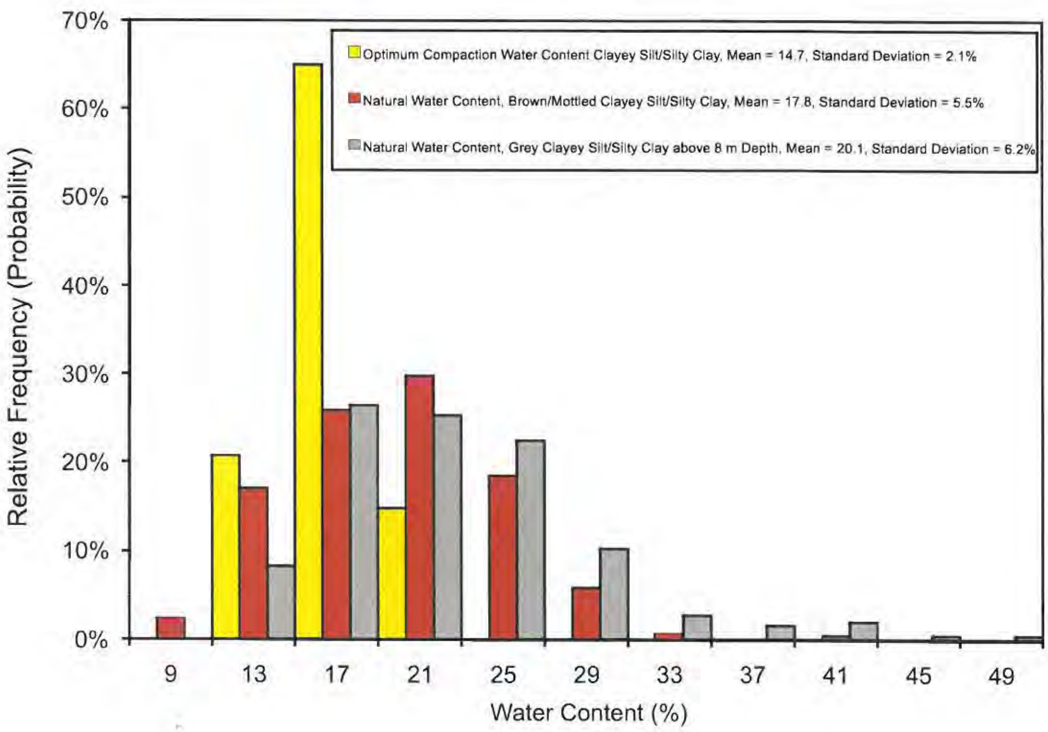
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
MEASURED AND INFERRED PERMEABILITY
FROM LABORATORY TESTS

Golder Associates

PROJECT No		07-1130-2070	FILE No	0711302070-R020610
			SCALE	AS SHOWN REV. 01
CADD	SJB	JUN 09		
CHECK	[illegible]	26 Jun 09		6.10





NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
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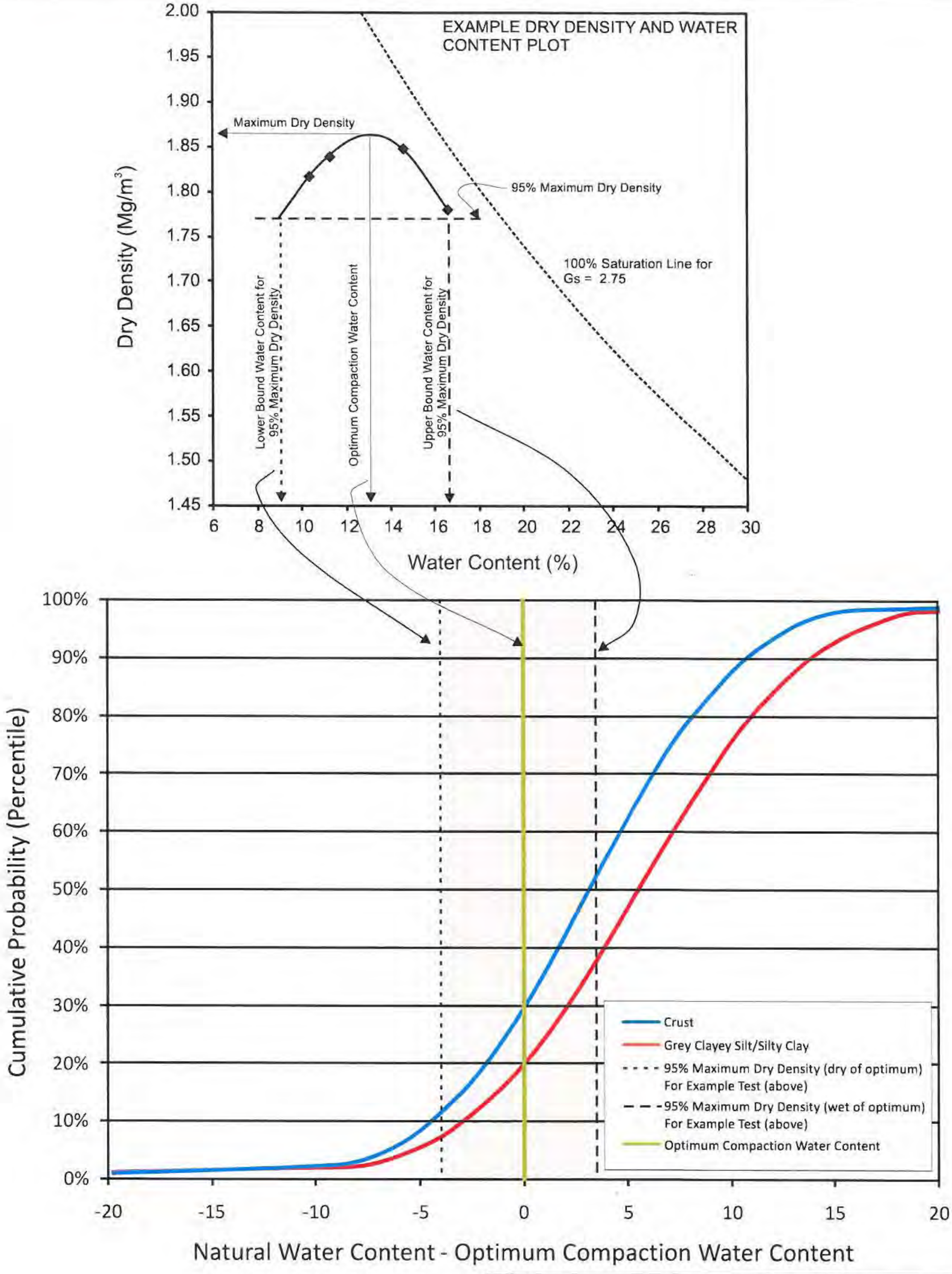
PROJECT **SUBSURFACE CONDITIONS BASELINE REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO**

TITLE **DATA SUMMARY MAXIMUM DRY DENSITY AND OPTIMUM COMPACTION WATER CONTENT**

Golder Associates

PROJECT No.	07-1130-2070	FILE No.	0711302070-R020611
		SCALE	N/A
CADD	SJB	JUN 09	
CHECK		JUN 09	6.11



NOTES

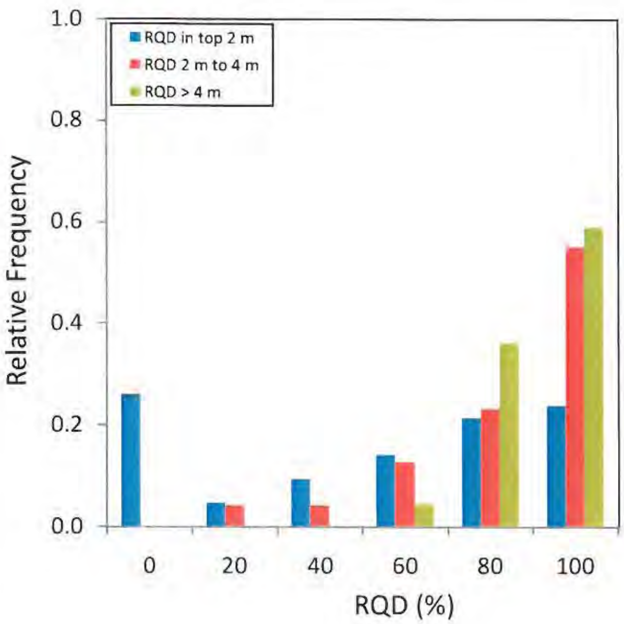
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS BASELINE REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AND DOES NOT REPRESENT THE BASELINE GEOTECHNICAL PARAMETERS. BASELINE GEOTECHNICAL PARAMETERS ARE IDENTIFIED IN THE TEXT OF THE REPORT REFERENCED ABOVE

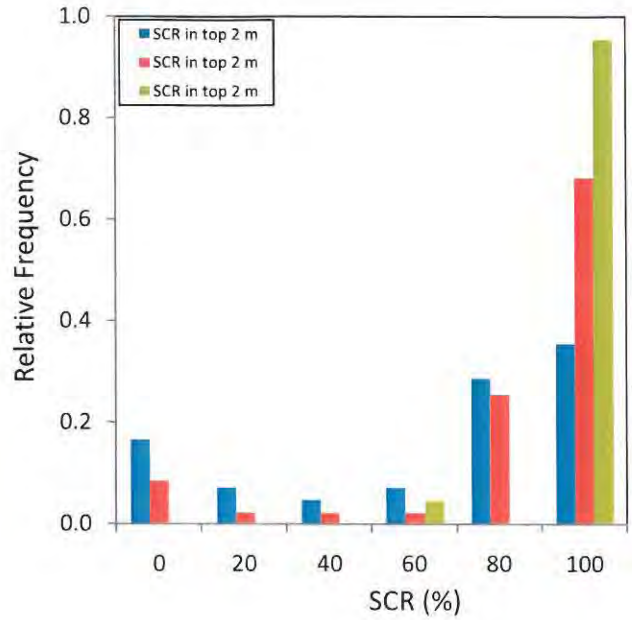
PROJECT **SUBSURFACE CONDITIONS BASELINE REPORT**
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

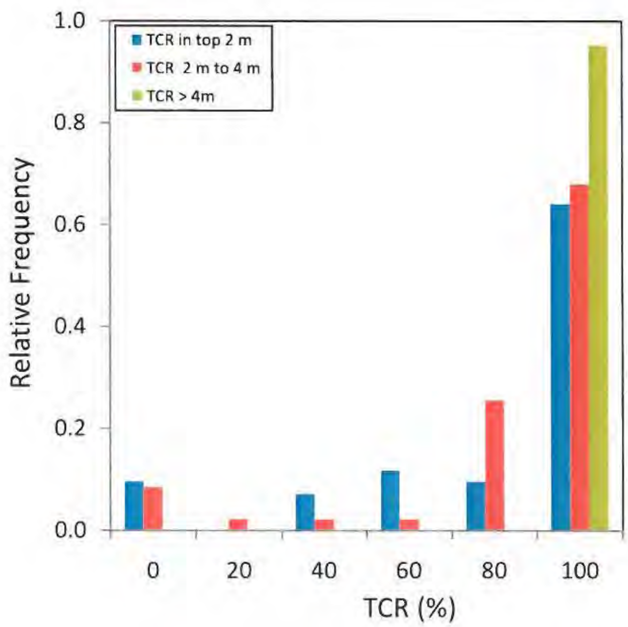
TITLE **DATA SUMMARY**
COMPARISON OF WATER CONTENT AND MAXIMUM DRY DENSITY DATA

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R020611
			SCALE	N/A
CADD	SJB	JUN 09		
CHECK		JUN 09		**6.12**







NOTES

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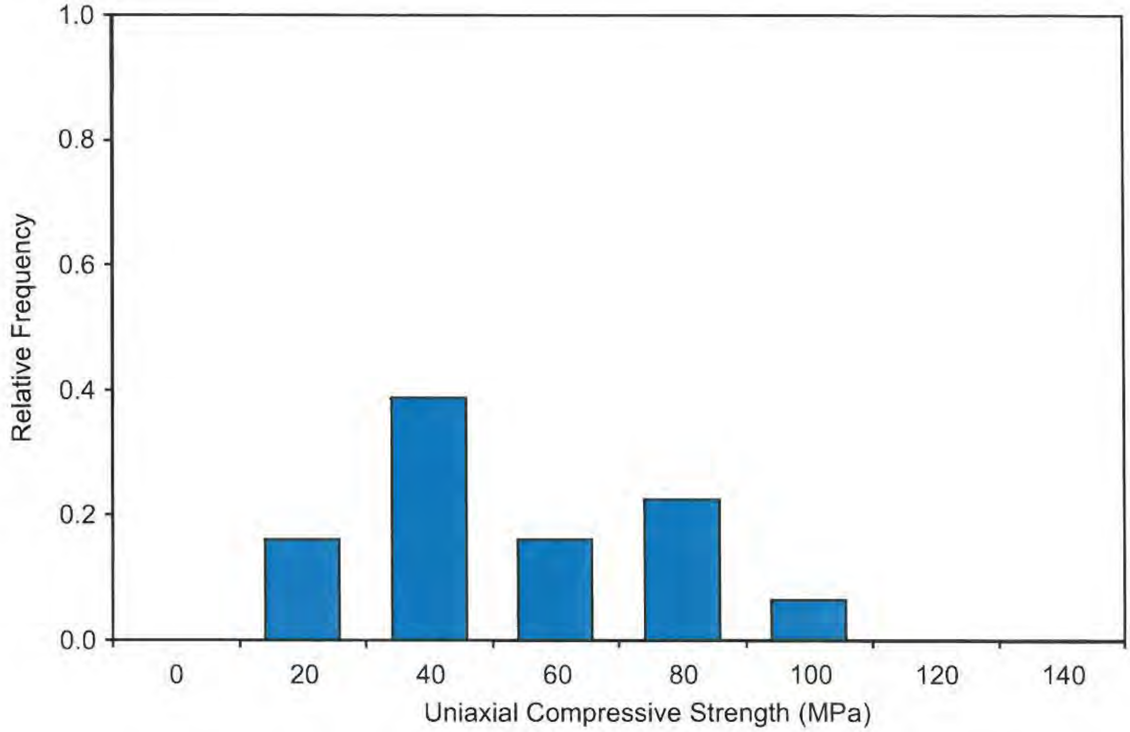
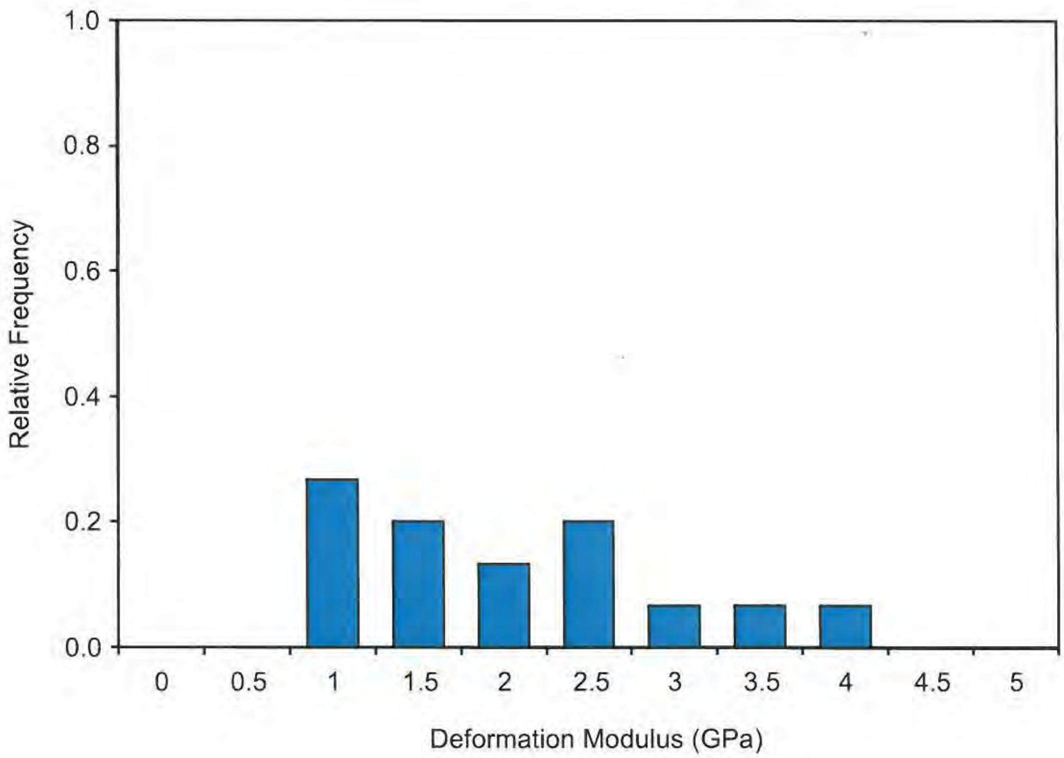
PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
ROCK CORING

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R020613
			SCALE	AS SHOWN
CADD	SJB	JUN 09		
CHECK	MT	26 Jun 09		6.13





NOTES

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PROJECT
SUBSURFACE CONDITIONS BASELINE REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
DATA SUMMARY
ROCK COMPRESSION TESTS

Golder Associates

PROJECT No.		07-1130-2070	FILE No.	0711302070-R020613
			SCALE	AS SHOWN
CADD	SJB	JUN 09		
CHECK				6.14

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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