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WINDSOR-ESSEX PARKWAY

Subsurface Conditions Interpretation Report

Submitted to:

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REVISED REPORT



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1.0 INTRODUCTION

This report presents an updated revision to and supersedes the revised December 2009 Subsurface Conditions Interpretation Report as well as the June 2009 Subsurface Conditions Baseline Report 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 including Addenda 1 through 7 to the Geotechnical Data Report prepared for this project. These addenda documented the supplementary subsurface investigations completed during the tendering period for the Windsor-Essex Parkway.

This report must be read in conjunction with the "Important Information and Limitations of This Report" provided following the text of this report. The reader's attention is specifically drawn to this information as it is essential for the proper use and interpretation of this report.

This report provides a number of tables and figures that summarize data and present interpreted 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. The interpretations of subsurface conditions provided within this report were prepared on behalf of and for the sole use of the Ministry of Transportation, Ontario.



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 with the profile 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 Interpretation Report (SCIR). 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 SCIR represents the most recent interpretation of conditions for the Sponsor's use.

The Geotechnical Data Report (GDR), including all addenda, Geocres No. 40J6-27, referenced below, was used as the primary source of data for development of this SCIR. Where the determination of deposit boundaries or geotechnical engineering parameters is necessary for the design, safety and stability of the works, for other construction concerns, or in instances where specialized subsurface properties or analytical parameters are required, 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 interpretations of engineering properties and parameters for the deposits and the stratigraphy as interpreted between samples as presented in this report are not to be relied upon by the successful Proponent for pricing, design or construction.

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 with Addendums:

Golder Associates Ltd. (2009). Geotechnical Data Report, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010a). Geotechnical Data Report, Addendum No. 1, Soil Chemistry Data, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010b). Geotechnical Data Report, Addendum No. 2, In Situ Cross-Hole and Vertical Seismic Profile Testing, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010c). Geotechnical Data Report, Addendum No. 3, Supplementary Cone Penetration Testing, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010d). Geotechnical Data Report, Addendum No. 4, Supplementary Geotechnical Investigation, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010e). Geotechnical Data Report, Addendum No. 5, Supplementary Laboratory Testing, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010f). Geotechnical Data Report, Addendum No. 6, Supplementary Geotechnical Investigation, Windsor-Essex Parkway. GEOCREs No. 40J6-27.

Golder Associates Ltd. (2010g). Geotechnical Data Report, Addendum No. 7, Supplementary Geotechnical Investigation, Windsor-Essex Parkway. GEOCREs No. 40J6-27.



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

Department of Highways of Ontario (1963). Foundation Investigation Report for Highway #18, Turkey Creek, LaSalle, Ontario, WP#139-60, Job 64-F-212C, GEOCREs No. 40J3-5.

Department of Highways of Ontario (1968). Proposed E.C. Row Expressway, Highway 18 to Dominion Blvd., Windsor, Ontario, WP#260-66-030, Job 68-F-15-1, GEOCREs No. 40J6-03.

Department of Highways Ontario (1968). Foundation Investigation Report for Proposed E.C. Row Expressway, Howard Avenue to Highway #3B, Job 68-F-15-2, WP#257-66-020.

Dillon Consulting Limited and Golder Associates Ltd. (2004). Essex Region/Chatham-Kent Region Groundwater Study, Vol. I, Geologic/Hydrogeologic Evaluation and Vol. II Groundwater Management Principles and Strategies. Essex Region Conservation Authority, Essex, Ontario.

Golder Associates Ltd. (1969). Slope Stability Study for Grand Marais Storm Drain, Windsor, Ontario, 68722.

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Golder Associates Ltd. (1968). Preliminary Subsurface Investigation, Proposed Grand Marais Sanitary Sewerage System, Windsor, Ontario, 68517.

Golder Associates Ltd. (1969). Subsurface Investigation for Proposed Huron Church Line Bridge, Windsor, Ontario, 69305.

Golder Associates Ltd. (1973) Subsurface Investigation, Proposed Bridge Over Turkey Creek, Matchette Road, Township of Sandwich West, Ontario, 73514.

Ministry of Transportation and Communications Ontario (1978). Foundation Investigation Report for Windsor – Highway 3 Overpass, Highway E.C.R., WP# 258-66-02, Site 6-283, GEOCREs No. 40J6-10.

Ministry of Transportation, Ontario (1994). Foundation Investigation Report for Highway 401/Highway 3, High Mast Lighting Foundations. WP#143-91-00, GEOCREs No. 40J-40.

Ministry of Transportation, Ontario (1996). Foundation Investigation Report for Highway 401 & Walker Road, High Mast Lighting Foundations. WO#96-33-002, GEOCREs No. 40J-41.

3.2 Publications

The publications referenced in this document are listed below for information purposes only.

ASCE (2007). Geotechnical Baseline Reports for Construction: Suggested Guidelines. The Technical Committee on Geotechnical Reports of the Underground Technology Research Council, R.J. Essex, chairman, ASCE, Reston, VA, 62 pp.

Becker, D.E. (1981). Settlement analysis of intermittently-loaded structures founded on clay subsoils. Ph.D. Thesis, University of Western Ontario.

Becker, D.E., Crooks, J.H.A., Jefferies, M.G. and McKenzie, K. (1984). Yield behaviour and consolidation 2: strength gain. Proceedings of Symposium on Sedimentation/Consolidation Models: Predictions and Validation, ASCE Geotechnical Engineering Division, San Francisco, California, 381-398.

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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 tube piles were driven 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 (GEOCRE No. 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 (GEOCRE No. 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 (GEOCRE No. 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 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 tube piles were driven 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 (GEOCRE No. 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 horizontal to 1 vertical side slopes.

The foundations investigation completed for this site in 1968 (GEOCRE Nos. 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 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 No. 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 horizontal to 1 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 ranged between 1.5 horizontal to 1 vertical and 2 horizontal to 1 vertical. 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 horizontal to 1 vertical 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 is about 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 unfactored 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 unfactored 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 horizontal to 1 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 unfactored 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 to 1 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 to 1 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 58 boreholes drilled to the bedrock surface or cored into bedrock and 86 cone penetration tests (CPT) have been completed along the alignment for this project. In addition, 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, three



boreholes were drilled to total depths of between 8.1 and 12.7 m for a proposed drainage system, and ten boreholes were drilled at various locations along the alignment to depths of between 8.1 and 8.2 m. Crosshole seismic geophysics was also carried out at three locations along the proposed highway alignment. An additional 158 boreholes were 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 earthwork. Exploration locations are illustrated on Figures 5.1A to 5.1I. Interpreted 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 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 interpreted stratigraphic conditions for the near-surface materials. Subsurface constructed features including remnant foundations and operational or abandoned utilities are not illustrated on these Figures. 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. 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 methods, 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, the 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 Deposit" 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 Deposit" 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 road 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 90 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 interpreted characteristics for the existing pavements, fill, topsoil and shallow subsurface



conditions. Interpreted 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 thicknesses provided in the tables below are to be considered 50th percentile values. It has been assumed that the 10th and 90th percentile thicknesses of the identified materials will vary by 20% below or above the 50th percentile value, respectively, in local areas unless specifically stated otherwise. Interpreted 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 may exist 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 (CPT) locations, unidentified obstructions were encountered within the fill preventing penetration 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 assumed that the fill was placed in an uncontrolled manner and will therefore exhibit great variation in both composition and engineering behaviour and that all existing fill will be unsuitable for reuse as engineered fill. It is also assumed that fill near utilities may 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.

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 interpreted 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.

It is assumed that the 50th percentile thickness of buried topsoil is about 225 mm and it may be found beneath the clayey fill materials in all shoulders at a depth of about 1.1 m, except the eastbound driving lane shoulder where the assumed width of the existing filled subexcavation is about 5 m.



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

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)			
	Westbound Lanes		Eastbound Lanes	
	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: Interpreted Pavement and Shallow Subsurface Conditions, Existing Highway 401 (shoulders) Station 10+900 to 11+300

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)						
	Westbound Lanes			Eastbound Lanes			
	Driving Lane		Passing Lane	Passing Lane	Driving Lane		
	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 interpreted 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. It is assumed that 40 per cent of the existing Highway 3/Talbot Road pavement structures and fill may be underlain by about 250 mm (50th percentile thickness) of buried topsoil.



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

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)			
	Westbound Lanes		Eastbound Lanes	
	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 interpreted 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 interpreted asphalt thickness is assumed to be about 200 mm (50th percentile) in all areas except the westbound speed change lane east of Howard Avenue and, in this case, the interpreted 50th percentile thickness is assumed to be about 530 mm.

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 interpreted 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 shown 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 the Silty Clay to Clayey Silt Deposit.

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

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)			
	Northbound Lanes		Southbound Lanes	
	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



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. Interpreted pavement and shallow subsurface conditions within the project limits for these crossing roads are summarized in the tables below and Figure 5.2D.

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

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)	
	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: Interpreted Pavement and Shallow Subsurface Conditions, Existing Highway 3 West of Highway 401

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (m)			
	West of Highway 401		East of Highway 401	
	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



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

COMPONENT	INTERPRETED 50 th 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: Interpreted Pavement and Shallow Subsurface Conditions, Existing Todd Lane and Existing Cabana Road

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)	
	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: Interpreted Pavement and Shallow Subsurface Conditions, Existing Pulford Street

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)	
	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



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

BETHLEHEM AVENUE		LABELLE STREET	
Component	Interpreted 50 th Percentile Thickness (mm)	Component	Interpreted 50 th 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: Interpreted Pavement and Shallow Subsurface Conditions, Existing Malden Road

COMPONENT	INTERPRETED 50 th PERCENTILE THICKNESS (mm)			
	Southbound		Northbound	
	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 5.11). 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 Clayey Silt to Silty Clay Deposit. Figure 5.2E provides the



interpreted shallow subsurface conditions for the Highway 3 and Outer Drive realignments. The surficial topsoil thickness (50th percentile) in this area is assumed to be about 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 Deposit soils about 300 mm thick were encountered beneath the surficial topsoil at one location. Twenty two boreholes encountered the native Silty Clay to Clayey Silt Deposit beneath the surficial layers. It is assumed that the topsoil thickness (50th percentile) through the new alignment in areas not occupied by pavements or built structures may be about 300 mm. The existing fill thickness, including buried topsoil is assumed to be about 600 mm (50th percentile) and the thickness may vary by about -75% to +150% of the 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 Deposit and Clayey Silt to Silty Clay Deposit. 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. The surficial topsoil thickness (50th percentile) is assumed equal to about 500 mm. The existing fill depth, including buried topsoil, is assumed equal to about 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 the ground surface overlying the native Upper Granular Deposit and Clayey Silt to Silty Clay Deposit. 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 a material



resembling granular base were encountered at five locations. The fill materials were 100 mm to 760 mm thick. The surficial topsoil thickness is assumed equal to about 350 mm (50th percentile) and the existing fill depth, including buried topsoil, is assumed equal to 500 mm (50th percentile).

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. The 50th percentile topsoil thickness in this area is assumed equal to about 500 mm.

5.4 Native Soil Stratigraphy

This section of the report provides interpreted soil classification parameters. Within this section, values are provided consistent with the 10th, 50th and 90th percentiles. The 10th, 50th and 90th percentile values are provided as a means for approximately describing the assumed statistical distribution of the values. It should be noted that the values provided in the tables within this section represent a statistical characterization of the classification criteria and cannot necessarily be considered in combination. For example, the 90th 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 50th percentile values for liquid and plastic limits will not necessarily be equal to the 50th percentile plasticity index. While the 50th percentile values may be suitable for some purposes, if designs or construction means and methods are sensitive to minimum or maximum values, then the range must be accounted for. 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 Deposit

Sand, silty sand, sandy silt and silt were encountered in 36 boreholes (not including pavement boreholes) as well as many of the boreholes at pre-drilled CPT locations with the majority of these located along the alignment west of Cabana Road. These soils were overlain by the surficial topsoil or fill materials and underlain by the extensive Clayey Silt to Silty Clay Deposit. The upper granular layers were found to maximum depths of about 2.9 m. 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 Deposit ranged between about 0.2 m and 2.8 m and exhibited Standard Penetration Test (SPT) "N" values between 3 and 23 blows per 0.3 m penetration indicating a very loose to compact relative density.



Table 5.12: Interpreted Classification Characteristics – Upper Granular Deposit

Parameter	10 th Percentile	50 th Percentile	90 th Percentile
Natural Water Content (%)	7	17	23
Unit Weight, γ (kN/m ³) ¹	19.5	20.5	22.0
Gravel (%) ²	2	5	14
Sand (%)	4	86	90
Silt (%)	8	11	85
Clay (%) ³	3	10	23
Percent Passing Standard 75 μ m Sieve	5	50	96

- 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. The percent clay as noted above represents clay-size fraction (less than or equal to 2 μ m) 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 1.8 m thick. These interbeds are not described in further detail and the classification characteristics of the interbeds are assumed to be similar 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. However, the boreholes at the far western end of the alignment adjacent to Ojibway Parkway encountered only grey cohesive deposits overlain by the Upper Granular Deposit. The upper mottled zone and brown zone (where present) 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. 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. An interpreted profile of the average crust thickness is illustrated on Figures 5.1A to 5.1I. This profile represents the average crust thickness and it has been assumed that the actual crust thickness may be greater than or less than the indicated profile by about 1.5 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). Vertical fissures within the crust may be anticipated at the approximate horizontal spacing intervals noted below. Between these depth intervals, the natural fissure spacing may be assumed to be linearly transitional. Vertical spacing of horizontal fissures may be assumed to be similar to the spacing of vertical fissures, such that block-like structures of irregular shape are formed with vertical and horizontal aspect (largest dimension divided by smallest dimension) on the order of 1 to 3.

Table 5.13: Interpreted 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.

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 about 15 per cent of the clay-size fraction in the weathered crust. Within the unweathered deposit, swelling minerals typically 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 23 percent (by weight) of the tested materials, but was typically less than 5 per cent. 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 36 per cent with an average of about 14 per cent for the entire deposit. The natural water content measured on selected samples of this deposit ranged between about 5 and 62 per cent but was typically between 15 and 25 per cent. The higher water contents are typically associated with the middle portion of the deposit. The area west of borehole 158 exhibits a marked change in the shape and magnitude of the water content profiles. High natural water contents in the range of 30 to 50 per cent are present between about elevations 175 m and 165 m in this region. Table 5.14, below, summarizes the interpreted classification characteristics of this deposit overall and Figures 5.7A to 5.7K present interpreted water content profiles for some geographic locations along the project alignment. These profiles are considered representative of the 50th percentile values below the crust boundary as defined above, with the 10th and 90th percentile values assumed to be minus or plus a water content of 5 per cent from these given profiles, respectively. Above the crust boundary, the interpreted water content values are summarized 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 3 to 59 blows per 0.3 m of penetration. Standard Penetration Test “N” values typically ranged between about 1 and 58 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 over 100 blows per 0.3 m of penetration.

The unweathered soils of the Clayey Silt and Silty Clay Deposit are characterised as a low-sensitivity (undisturbed divided by remoulded field vane shear strength) materials with an average sensitivity of 2.0. Minimum and maximum sensitivity values ranged from 1.0 to about 9.3 with only five values above 5.0.

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

Parameter	10 th Percentile	50 th Percentile	90 th Percentile
Natural Water Content, Brown and Mottled Brown/Grey Soils (“Crust”) (%)	12	15	23
Natural Water Content Grey Soils Below Static Water Level (%)	14	19	29
Liquid Limit (%)	23	29	38
Plastic Limit (%)	13	15	18
Plasticity Index	9	14	21
Unit Weight, γ (kN/m ³) ¹	18.0	20.0	21.0
Gravel (%)	0	1	20
Sand (%)	12	24	31
Silt (%)	36	42	49
Clay (%) ²	23	31	44

- Notes:
1. Values based on measured unit weights of thin-walled tube samples during oedometer consolidation testing.
 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 μ m) 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 Deposit

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 Clayey Silt and Silty 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 classification parameters



provided below are representative only of the granular fraction of these soils. 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: Interpreted Classification Characteristics – Lower Granular Deposit (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	8	35
Sand (%)	8	44	75
Silt (%)	7	33	78
Clay (%) ²	3	7	12
Percent Passing 75 μ m Sieve	13	39	77

Notes: 1. Values based on saturated water content.

2. Percentage Clay as noted above represents clay-size fraction (less than or equal to 2 μ m) 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 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. Based on published geologic information and the recovered cores, 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 Ontario Ministry of the Environment (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 to the underlying bedrock aquifer. 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, piezometric 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 piezometric groundwater levels within the bedrock were close to about Elevations 180.5 m to 177.5 m, respectively. There is a trend of increasing piezometric groundwater levels within the bedrock from south and east to north and west, opposite to the trend indicated for the overburden. Near the western end of the project, flowing artesian conditions were encountered indicating upward hydraulic gradients through the overburden. Two interpreted groundwater pressure elevation lines are illustrated on Figures 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. Groundwater parameters are summarized below that consider both the analytical results of testing carried out for this project as well as published values.

Table 5.16: Interpreted Natural Groundwater Chemistry Characteristics

Parameter	10 th Percentile	50 th Percentile	90 th Percentile
Hardness as CaCO_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, BH-160, BH-166, BH-341, BH-343, BH-345, BH-346 and BH-349 when bedrock and flowing artesian water pressures were encountered. Hydrogen sulphide gas can frequently be detected by odour 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 other geotechnical investigations in the vicinity. The hydrogen



sulphide gas concentrations were sufficient to trigger personnel health and safety monitoring equipment on several occasions. Active ventilation of drilling areas with construction fans and use of controlled density drilling fluids were required to continue drilling at some locations for these nearby explorations.

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 H₂S 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 H₂S 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 H₂S 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.

Dissolved methane, CH₄, 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.

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 should not encounter such gasses. 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, BH-127, BH-346 and BH-349. 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. $V_{bT} = \sum V_b$) 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 Deposit (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, interpreted BVR, BNR and boulder size distribution data have been assumed as summarized in Table 5.17, below, as a means of approximating the numbers of boulders that may be encountered.

Table 5.17: Interpreted 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

It is assumed 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 m³ of earth will be directly in the path of the pile driving. Therefore, a total of approximately 3.5 m³ of boulders measuring 300 mm to 1.0 m may be assumed to be encountered, resulting in a total of 50 boulders of this size hit by the pile driving.

Other subsurface obstructions associated with former structures or utilities may also be encountered during construction. These types of obstructions are outside the scope of this SCIR.



6.0 INTERPRETED GEOTECHNICAL ENGINEERING PARAMETERS

This section of the report summarizes interpreted geotechnical engineering parameters of the native soil deposits. Within this section, values are provided consistent with the 10th, 50th and 90th percentiles. The 10th, 50th and 90th percentile values are used as a means for quantitatively describing the statistical distribution of the parameter values. While the 50th percentile value may be appropriate for some purposes, if the designs are sensitive to minimum or maximum values, then the range should be taken into account. Likewise, the range of values should also be considered since variability in physical properties is intrinsic to the nature of earth materials. 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 geotechnical parameters are also summarized 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. The geotechnical engineering parameter values were based on interpretation of the test results compiled in the Geotechnical Data Report and subsequent addenda supplemented by published and unpublished information, where relevant. 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 summarized in this report are considered relevant 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 and the risks associated by such interpretations are the responsibility of 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 twenty-five locations, multiple testing methods were used to develop profiles of geotechnical parameters that could be readily compared. The data developed at these locations are summarized on Figures 6.1A to 6.1Y for general information purposes only. Limitations related to the test data shown on these figures and interpreted geotechnical parameters are provided in subsequent sections of this report and in the pages immediately following the text of this report.

6.1 Clayey Silt to Silty Clay Deposit

6.1.1 Undrained Shear Strength

Estimation of the undrained shear strength of the Clayey Silt to Silty Clay Deposit 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 tests, three types of laboratory triaxial tests 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 for the majority of the tests conducted in boreholes 1 to 160. Field methods were modified to result in failure times of between 1 and 1.5 minutes for the remaining tests. 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, as such, a correction factor was not applied to the field vane shear test results.

Piezocene 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 uncorrected 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 a suitable estimate for this interpretation. 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 and also ensuring that the preconsolidation pressure profile (derived from the undrained shear strength, see Section 6.1.2) was consistent with the oedometer test results and the in situ effective stress profile. 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 cone factor was defined to be $N_c = 16$ in general. However, for the CPTs west of CPT-159 (inclusive), in an area characterized by higher natural water contents and flowing artesian water pressures near the soil-bedrock interface, a cone factor in the range of 13 to 15 was used to derive the undrained shear strength profiles. Also, a factor of 17 was used for CPT-154 to have a reasonable match with the data from other tests.

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 $K_o = 1 - \sin \phi'$ 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 Figure 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 should be modified, however, for conditions where the shear plane passes through the weathered crust as discussed below. For the purposes of this 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 clay be reasonably expressed by the following equations:

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

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

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

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

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

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

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

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

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

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

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



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

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

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

Where:

$S_{u(\text{CI*UC})}$ and $S_{u(\text{CI*UE})}$ = 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 $\frac{1}{4}$ and $\frac{1}{2}$ the estimated in situ vertical effective stress;

$S_{u(\text{CIUC})}$ and $S_{u(\text{CIUE})}$ = 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;

$S_{u(\text{CAUC})}$ and $S_{u(\text{CAUE})}$ = 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 $\frac{1}{2}$ of the estimated in situ vertical confining stress;

$S_{u(\text{CK}_0\text{UC})}$ and $S_{u(\text{CK}_0\text{UE})}$ = undrained shear strength derived from samples consolidated under K_0 conditions (zero radial strain) and sheared in undrained triaxial compression (CK_0UC) or extension (CK_0UE) with pore water pressure measurements with the vertical consolidation pressure equal to 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_0 ;

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

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

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

$S_{u(\text{FVT})}$ = 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 estimating undrained shear strength values in the weathered crust. The interpreted undrained shear strength within the weathered crust is estimated to be 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 6.4J summarize interpreted reference undrained shear strength profiles for sections along the Windsor-Essex Parkway. Where the sections abut, the undrained shear strength profile is assumed to be approximately equal to the average of the two adjacent profiles, modified for local conditions based on the local crust thickness.

Areas with comparatively low undrained shear strength were identified near CPT-106, CPT-317 and CPT-9, CPT-328 and CPT-333. Separate profiles are summarized for these locations on Figures 6.4J, 6.4G, 6.4E and 6.4D, respectively. At each of these locations, the interpreted undrained shear strength of the individual CPT test has been assumed to be representative of the 50th percentile values. The 10th and 90th percentile values of undrained shear strength is to be about 80 per cent and 125 per cent of the summarized 50th percentile profile values at these locations where the CPT profile indicated a region of lower strength soil. In these areas, the interpreted undrained shear strength at any point between the identified lower strength CPT location and the next adjacent CPT may be assumed to be linearly varying between the respective 10th, 50th and 90th percentile profiles at the same elevation, based on the horizontal distance between the test locations.

6.1.2 Preconsolidation Pressure

For interpretation purposes, the “preconsolidation pressure” was 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):

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

where: $S_{u(\text{ref})}$ = 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 Lutenege 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 the slope-intercept method (Boone 2010). This interpretation method 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 the estimated preconsolidation pressure using the CPT and oedometer tests is summarized on Figure 6.5.

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 with published correlations and theoretical relationships. Correlations among oedometer test results



developed for this project are illustrated in the summary on Figure 6.6. Figure 6.7 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 was not 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 6.6. The results of data evaluation provided the following approximate correlations:

$$\begin{aligned} C_c &= 0.0092w_n - 0.015 \\ C_r &= 0.11C_c \\ C_s &= 0.26C_c \\ C_{\alpha} &= 0.028C_c \text{ for all stress ranges} \\ w_n &= \text{natural water content expressed as a percent} \end{aligned}$$

Table 6.1: Interpreted Vertical Coefficient of Consolidation, c_v (cm^2/s)

Stress Condition	10 th Percentile	50 th Percentile	90 th Percentile
Recompression	3.1×10^{-4}	9.2×10^{-4}	1.8×10^{-3}
Virgin Compression	4.7×10^{-4}	9.5×10^{-4}	1.6×10^{-3}

Note: c_v based on t_{50} values

The horizontal coefficient of consolidation (c_h) was derived from 20 porewater pressure dissipation tests carried out during four piezocone penetration tests (CPT-339, CPT-340, CPT-342 and CPT-344). The dissipation tests consisted of stopping cone penetration at different depths and monitoring the decay of excess pore water pressures with time. These dissipation tests were conducted in the western portion of the project near areas that will include high-fill embankments and are characterized by flowing artesian pressure condition in the bedrock and upward hydraulic gradient through the overburden. Considering the uncertain nature of the equilibrium pore water pressure (u_0) in such conditions, the method of Burns and Mayne (1998) was adopted to interpret the values of c_h . This method is based on a cavity expansion-critical state model and can provide a theoretical approximation of the entire dissipation response observed in the field. By trial and error, best fit values of c_h and u_0 were obtained for the measured dissipation data.

The dissipation tests were conducted at the same depths as the oedometer test specimen locations in nearby boreholes (BH-341, BH-343 and BH-345). The estimated values of c_h were compared to the c_v values (at in situ effective stress) derived from the oedometer test results, as illustrated on Figure 6.8. The ratio c_h/c_v was found to be between 1 and 3 for the majority of the data points. Laboratory consolidation testing of horizontally-oriented samples also indicated a c_h/c_v ratio of about 2.

These correlations are generally 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 after accounting for 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 6.7 where similarity between the data sets is evident. The data evaluation resulted in the approximate correlations below. These values are generally consistent with, though somewhat lower than, published correlations for similar soil types (e.g., Becker 1981, Kulhawy and Mayne 1990).

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

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

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

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

Where analyses are adversely sensitive to high values of deformation moduli within the "crust" the above correlations should be applied to the interpreted reference undrained shear strength values within the "crust", rather than values reduced considering the effects of fissuring.

The deformation moduli as summarized above are considered to 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 should not 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} . 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 estimated using the following relationship and the interpreted 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 peak angle of internal friction for an assumption of an effective cohesion intercept of zero was estimated to be about 30 degrees. These values are generally consistent with published correlations for similar soil types (e.g., Kulhawy and Mayne 1990). The residual angle of internal friction is estimated to be about 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 considered 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:

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

Where the analyses are sensitive to the porewater pressure parameter at failure, the more adverse of the upper or lower bound values should be used.

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 value for the ratio of in situ horizontal to vertical stresses, K_o , may be assumed to be between 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 on Figure 6.10. Figure 6.10 illustrates that the measured permeabilities, based on the flexible wall permeameter results, are similar to those obtained through interpretation of the oedometer tests. The laboratory test measurements of permeability, however, are only considered appropriate for the small specimens of the Clayey Silt to Silty Clay Deposit. Permeability interpreted from oedometer data within the virgin compression stress range is approximately one half of an order of magnitude lower than the permeability within the in situ vertical stress range. In addition, rising or falling head



tests were completed in selected piezometers or observation wells within the silty clay to clayey silt deposits. The interpreted in situ mass permeability in the vertical direction for the Clayey Silt to Silty Clay Deposit is summarized in the table below. The value in the horizontal direction is assumed to be about twice the values provided below, similar to the ratio of c_h to c_v as discussed in Section 6.1.3.

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

Deposit Type	10 th Percentile	50 th Percentile	90 th Percentile
"Crust" Soils	5×10^{-8}	1×10^{-7}	5×10^{-7}
Unweathered Soils	1×10^{-8}	2×10^{-8}	5×10^{-8}

6.1.7 Dynamic Soil Properties

In situ geophysical testing was carried out at three locations to assist in defining the dynamic soil properties needed to assess seismic design requirements. The geophysical testing consisted of cross-hole seismic testing at the locations of boreholes 301 and 345. Vertical seismic profile (VSP) techniques were used at borehole 326 because the down-hole shear wave source requires a dry borehole to operate and there was water recharge into the PVC casing of borehole 326. Details of the methods of testing are summarized in the Geotechnical Data Report (Golder, 2010b).

The cross-hole seismic technique measures the velocity of elastic wave propagation through the subsurface to infer rock/soil types, stratigraphy and soil conditions. Data is collected using both a compression (P) wave and shear (S) wave source. The recorded data is subsequently analyzed by splitting the three recorded components (vertical, longitudinal and transverse) into depth-wave trains. Compression and shear wave arrivals at the receivers are then picked and velocities are calculated based on the borehole separations. Values for Poisson's ratio (ν), dynamic shear modulus (G_d), dynamic deformation modulus (E_d) and Bulk Modulus (K) may be calculated from compression wave and shear wave velocities as:

$$\nu = \frac{0.5\left(\frac{V_P}{V_S}\right)^2 - 1}{\left(\frac{V_P}{V_S}\right)^2 - 1}$$

$$E_d = \frac{V_P^2 \rho (1 + \nu)(1 - 2\nu)}{(1 - \nu)} = 2G_d(1 + \nu)$$

$$G_d = \frac{E_d}{2(1 + \nu)} = V_S^2 \rho$$

$$K = \frac{E}{3(1 - 2\nu)} = \frac{V_P^2 \rho}{\frac{4}{3}}$$

Where ρ is the bulk density (in kg/m^3), V_P is the compression wave velocity (m/s) and V_S is the shear wave velocity (m/s).



Figures 6.11 and 6.12 illustrate the shear and compression wave velocities and the interpreted dynamic deformation and shear moduli, respectively, at each of the testing locations. In general, the dynamic deformation modulus was found to be dependent upon the estimated in situ void ratio as well as upon the CPT tip resistance, as suggested by Mayne and Rix (1993). For this site, the maximum dynamic shear modulus may be approximated as twice the value that could be estimated using the empirical correlations (Mayne and Rix, 1993) as follows:

$$G_d = \frac{200p_a^{0.305} q_c^{0.695}}{e^{1.13}}$$

6.1.8 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 29 samples for this project using the “Standard Proctor Compaction” test (ASTM D698). These tests were conducted on samples obtained from the boreholes using both thin-wall tube sampling and bulk 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 summarized on Figure 6.13. Interpreted optimum compaction water content and maximum dry density values for the native cohesive soils are summarized in Table 6.3, below. Figure 6.13 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: Interpreted Optimum Compaction Water Content and Maximum Dry Density for Clayey Silt to Silty Clay

Parameter	10 th Percentile	50 th Percentile	90 th Percentile
Optimum Compaction Water Content (%)	11.7	14.8	17.3
Maximum Dry Density (kN/m ³)	17.3	18.1	19.1

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.

Further, 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.

A comparison has been made between the natural water content of the native cohesive 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 6.14 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 6.14 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 6.14 (lower figure) represent approximate 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 illustrates the range of upper and lower bound water contents, wet or dry of optimum, needed to achieve 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 about 3.5% wet of optimum or not less than 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 6.14, the analysis indicates that there is an approximately 35% chance (i.e., 35th percentile) that the crust materials (blue line on Figure 6.14) will have a natural water content equal to or less than the optimum compaction water content (green line on Figure 6.14) and an approximately 60% chance that these soils will have a natural water content equal to or less than the upper bound (3.5% wet of optimum) compaction water content. These analyses suggest:

- the water content at the time of placement (w_p) should be no more than 3.5% 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) should be no less than 4% dry of the laboratory optimum compaction water content to achieve 95% of the maximum dry density;
- there is an approximately 45% probability that excavated crust soils may have an in situ water content within the water content bounds suitable for achieving 95% of the maximum dry density during compaction (adequate compaction);
- there is an approximately 40% probability that excavated crust soils will likely require drying from the in situ water content to achieve adequate compaction and an approximately 14% probability that excavated crust materials will likely 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 there is approximately 30% probability that 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 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 of the crust 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.



6.1.9 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 subgrade resilient modulus of 25 megapascals (MPa) and a modulus of vertical subgrade reaction of 30 MPa per metre (MPa/m) may be assumed for the crust soils of the Clayey Silt to Silty Clay Deposit. These values are generally 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, the interpreted resilient moduli and moduli of subgrade reaction are broadly correlated to the undrained shear strength profiles as described in previous sections of this report.

INTERPRETED 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 should be considered unsuitable for support of structures or for use as engineered fill due to their compositional heterogeneity.

6.3 Upper Granular Deposit

Interpreted geotechnical engineering parameters for the Upper Granular Deposit are summarized in the table below. Where analyses may be sensitive to these parameters, the more adverse condition should be used.

Table 6.5: Interpreted Geotechnical Engineering Parameters for Upper Granular Deposit

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)	1×10^{-6} to 1×10^{-4}



6.4 Lower Granular Deposit

Interpreted geotechnical engineering parameters for the Lower Granular Deposit are summarized in the table below. Where analyses may be sensitive to these parameters, the more adverse condition should be used. Dewatering of the Lower Granular Deposit is not anticipated and, therefore, interpreted permeability or hydraulic conductivity parameters are not provided.

Table 6.6: Interpreted Geotechnical Engineering Parameters for Lower Granular Deposit

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 6.15 presents a summary of data related to recovery of rock cores as well as rock quality designation (RQD) values. 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 6.16 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 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 interpreted values for the strength and deformation modulus parameters.

Table 6.7: Interpreted Intact Rock Properties

Parameter	10 th Percentile	50 th Percentile	90 th 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: Interpreted Rock Engineering Parameters

Parameter	10 th Percentile	50 th Percentile	90 th Percentile
Top 2 m of Bedrock			
TCR (%)	43	92	100
SCR (%)	0	67	91
RQD (%)	0	58	89



Parameter	10 th Percentile	50 th Percentile	90 th Percentile
2 m to 4 m Penetration			
TCR (%)	74	98	100
SCR (%)	61	91	100
RQD (%)	48	89	100
More than 4 m Penetration			
TCR (%)	94	100	100
SCR (%)	72	94	100
RQD (%)	69	89	100

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

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

6.6 Frost Penetration Depth

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



8.0 MANAGEMENT OF SOIL AND GROUNDWATER

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 conditions are summarized:

- In general, native soils not subject to anthropogenic degradation should not exhibit parameter concentrations in excess of risk based standards as provided in Tables 2 or 3 from the MOE (MOE 2004, "Ministry Standards"); and
- All native soils not subject to anthropogenic degradation are expected to routinely exhibit at least one parameter concentration in excess of "background" levels as provided in Table 1 from the MOE (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 soil 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 MOE in all cases in which the expected groundwater extraction rate is 50,000 litres per day or greater.



9.0 CLOSURE

This SCIR 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.

ORIGINAL SIGNED

Storer J. Boone, Ph.D., P.Eng.
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Principal

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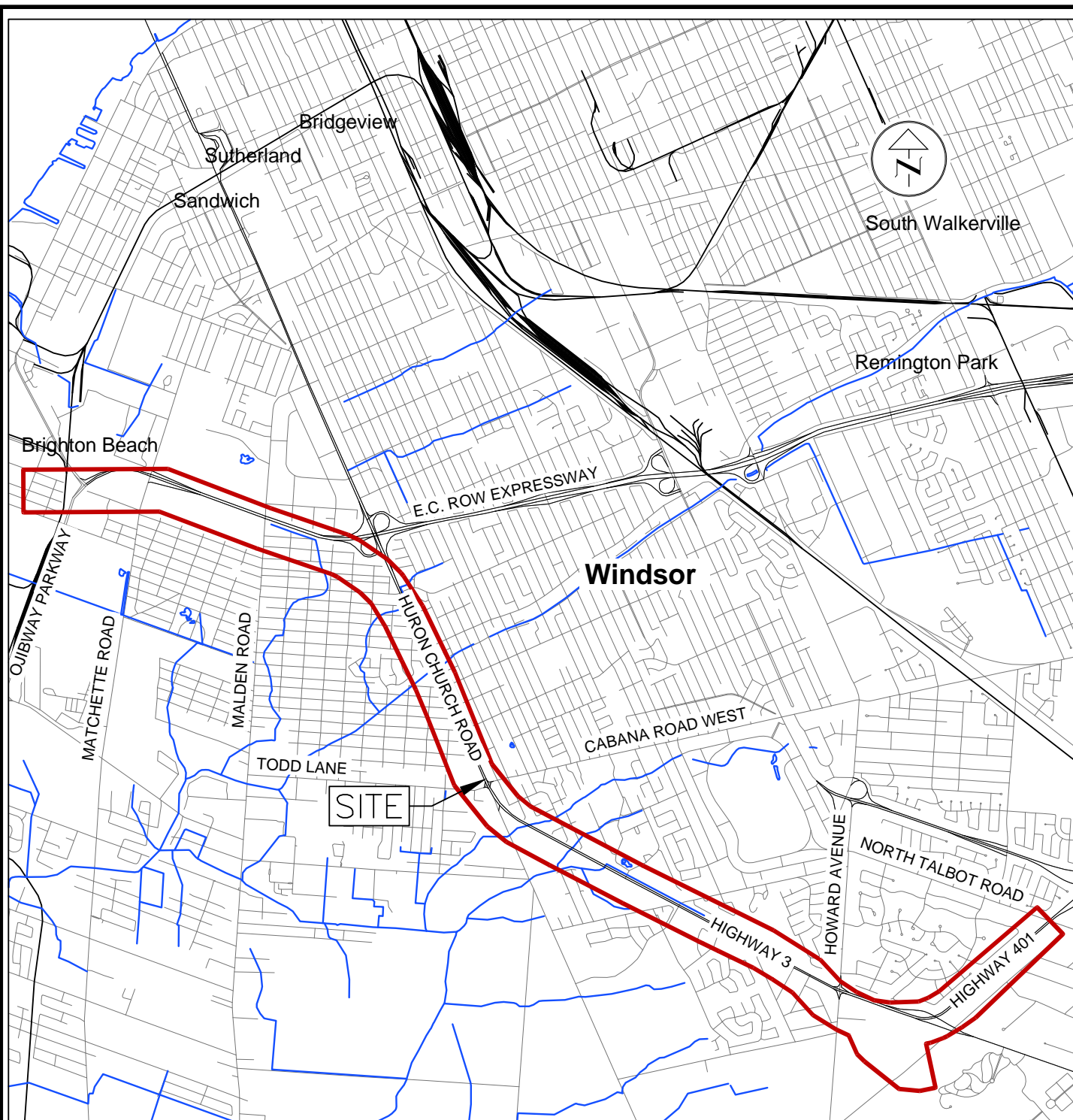
Fintan J. Heffernan, P.Eng.
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Philip R. Bedell, P.Eng.
Senior Consultant

MK/SG/SJB/JW/PRB/FJH/ly

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REFERENCES

- 1) DRAWING BASED ON CANMAP STREETFILES V2005.4.

NOTES

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

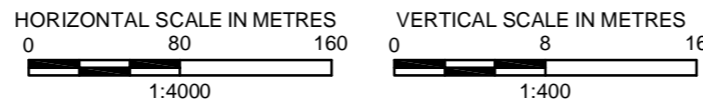
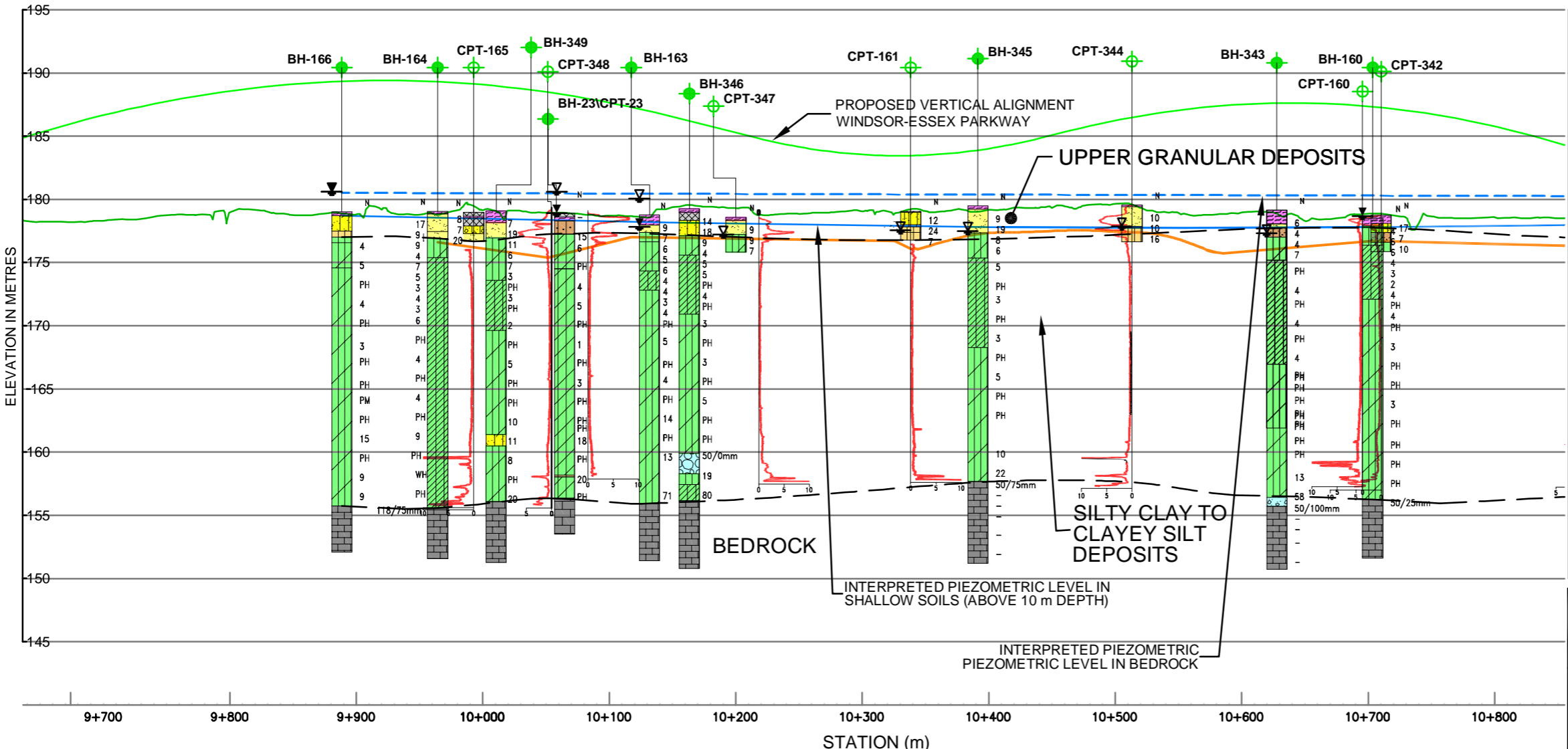
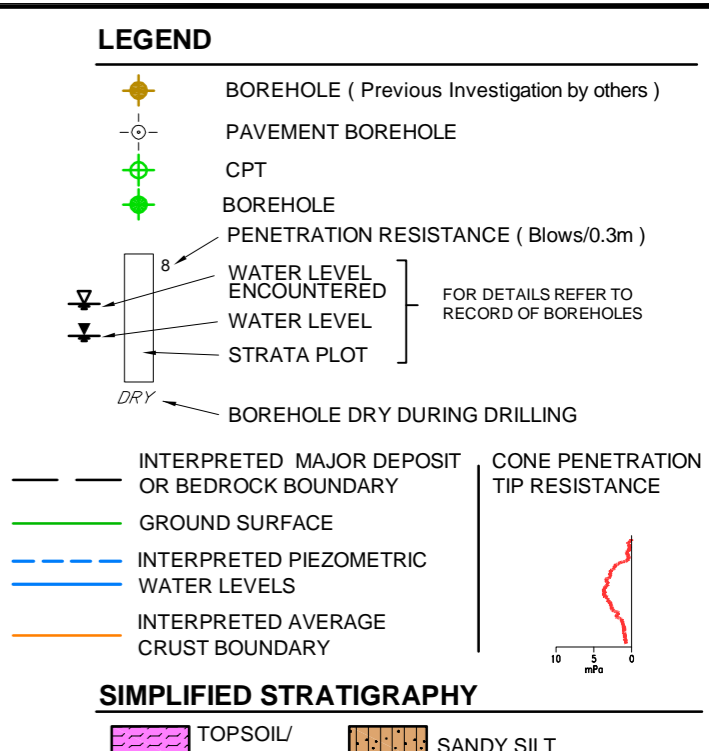
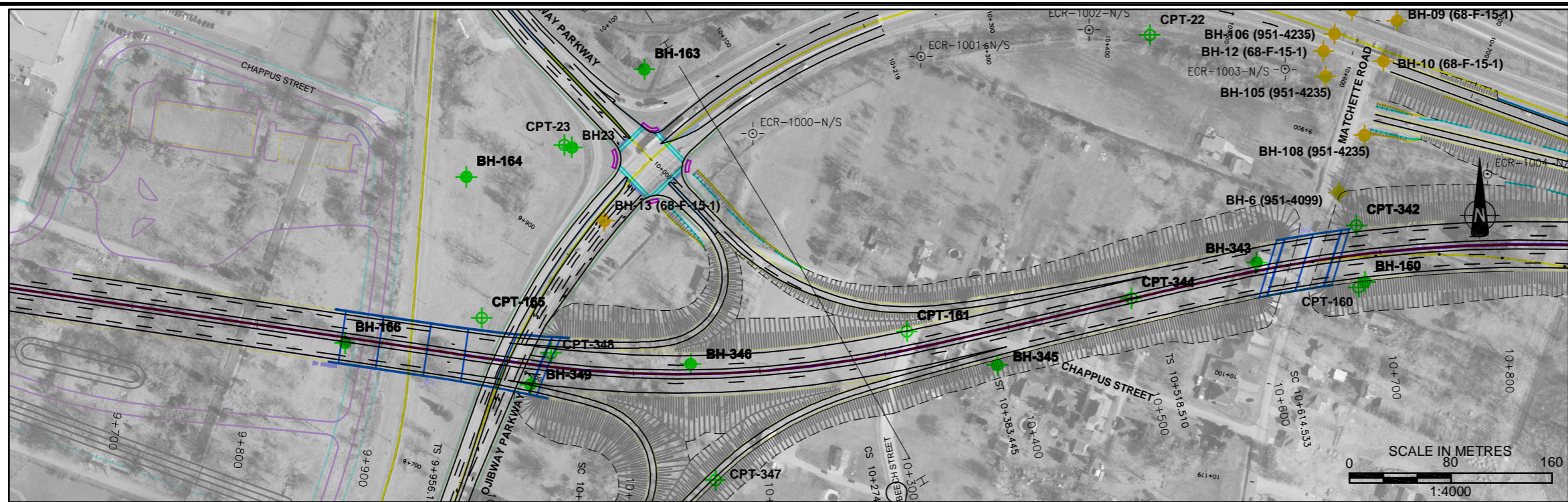
PROJECT
SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

KEY PLAN



PROJECT No. 07-1130-207-0			FILE No. 0711302070-R02001		
CADD	WDF	Jan. 12/11	SCALE	AS SHOWN	REV. 1
CHECK			FIGURE 1.1		



PROJECT

**SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO**

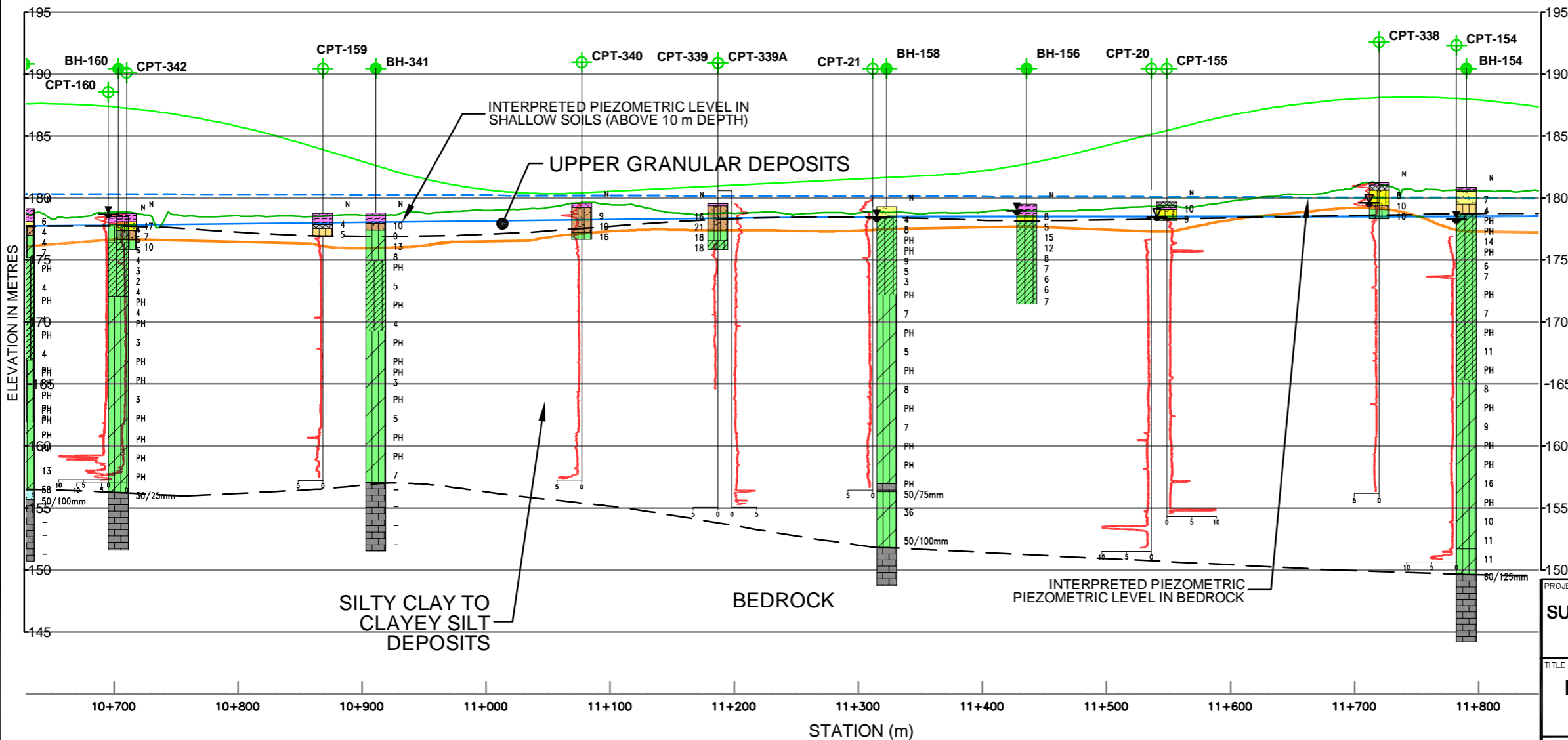
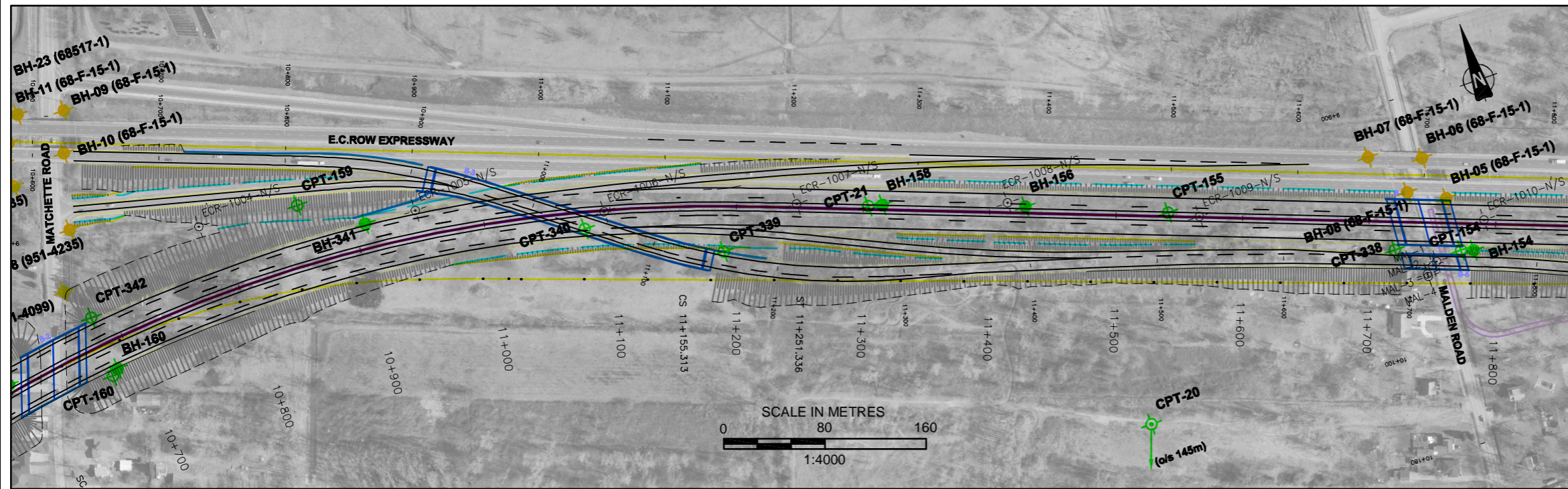
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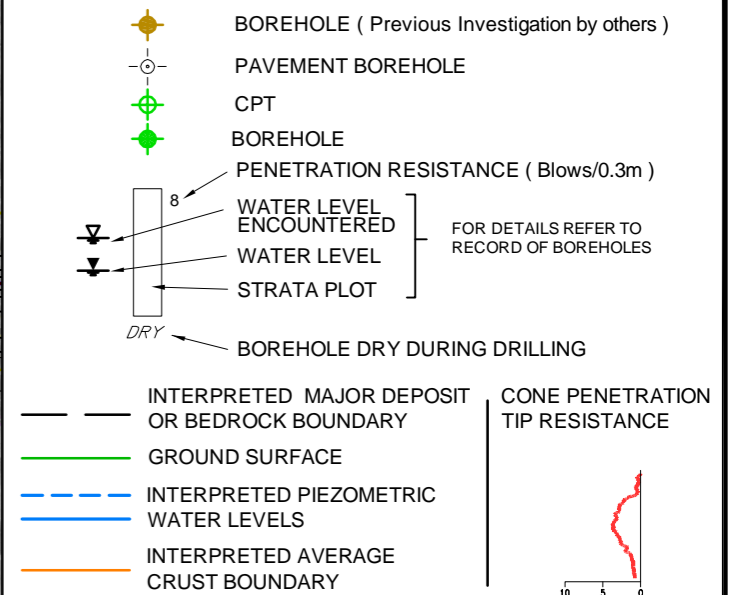
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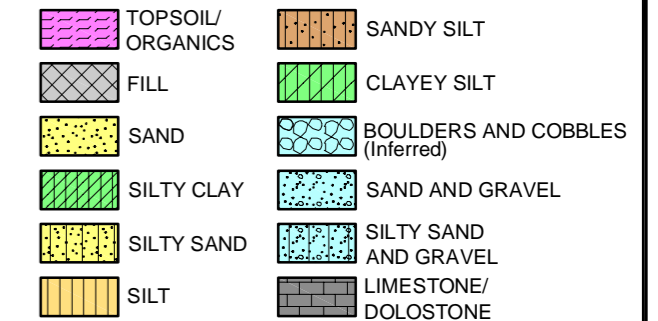
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LEGEND



SIMPLIFIED STRATIGRAPHY




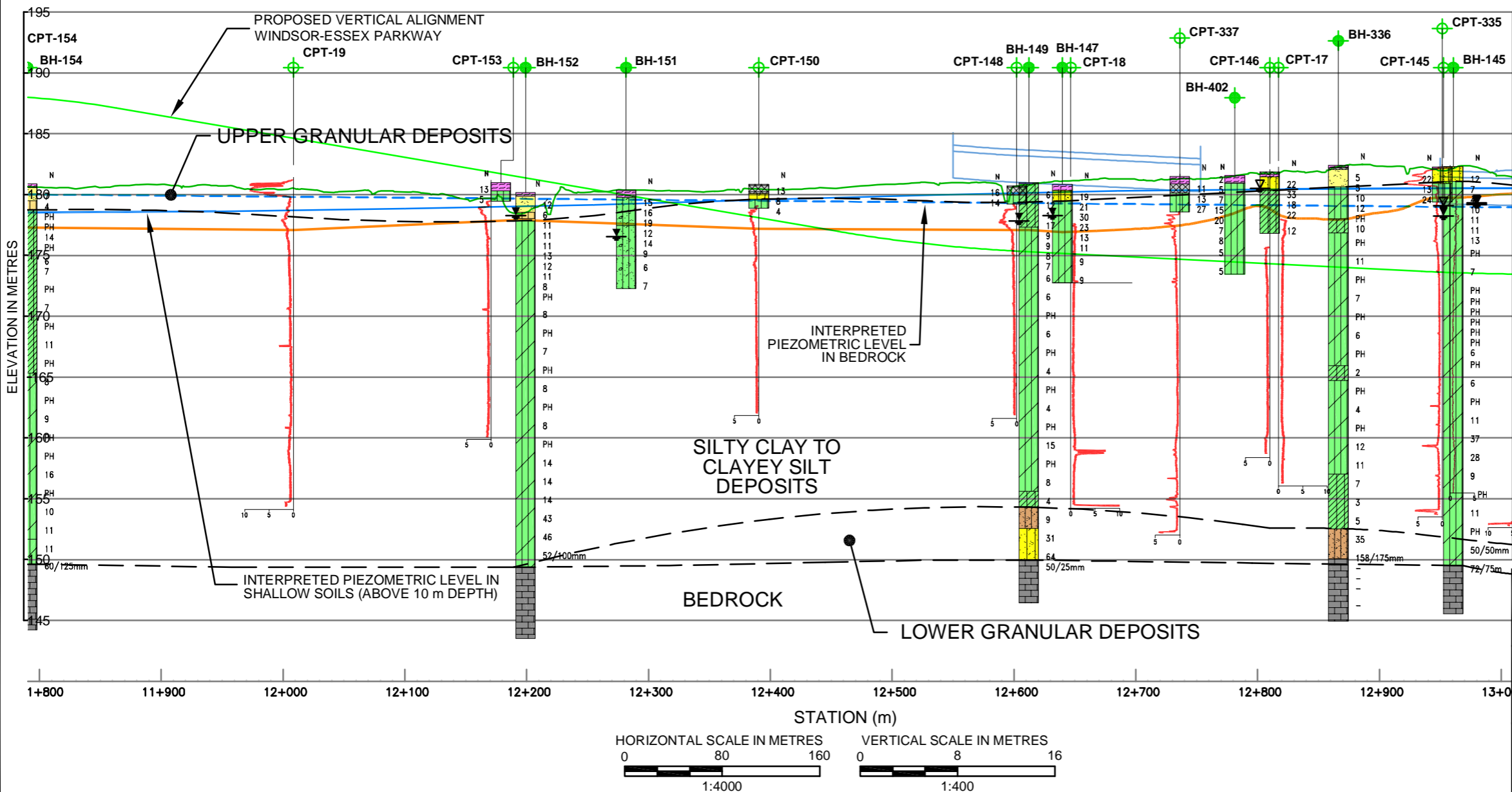
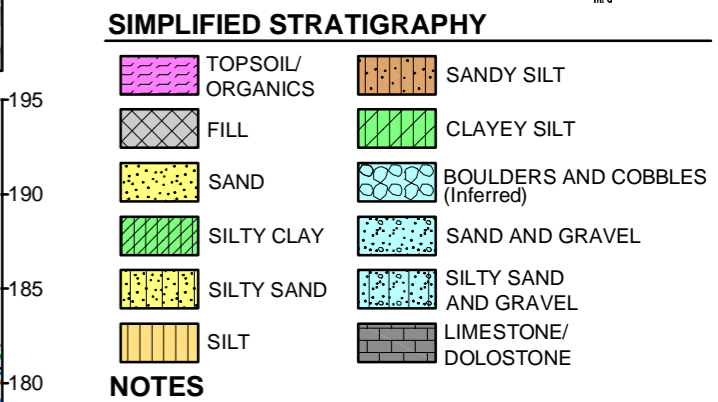
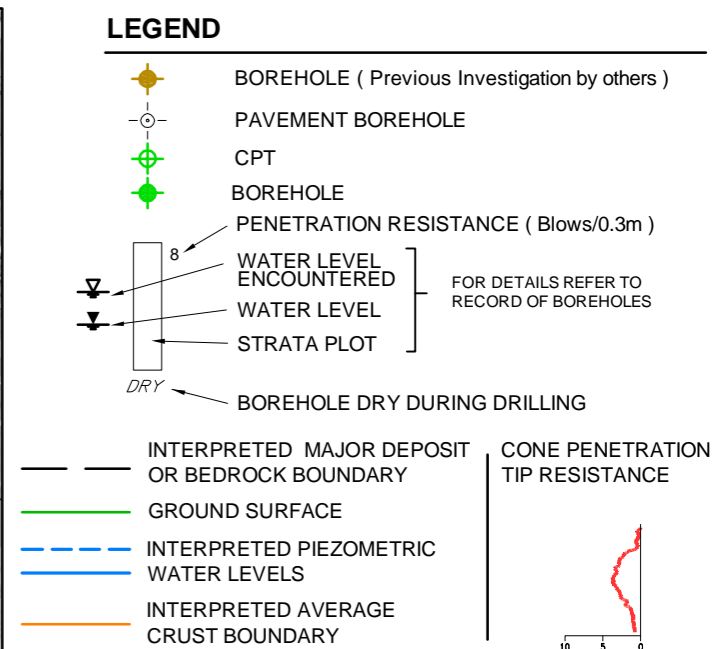
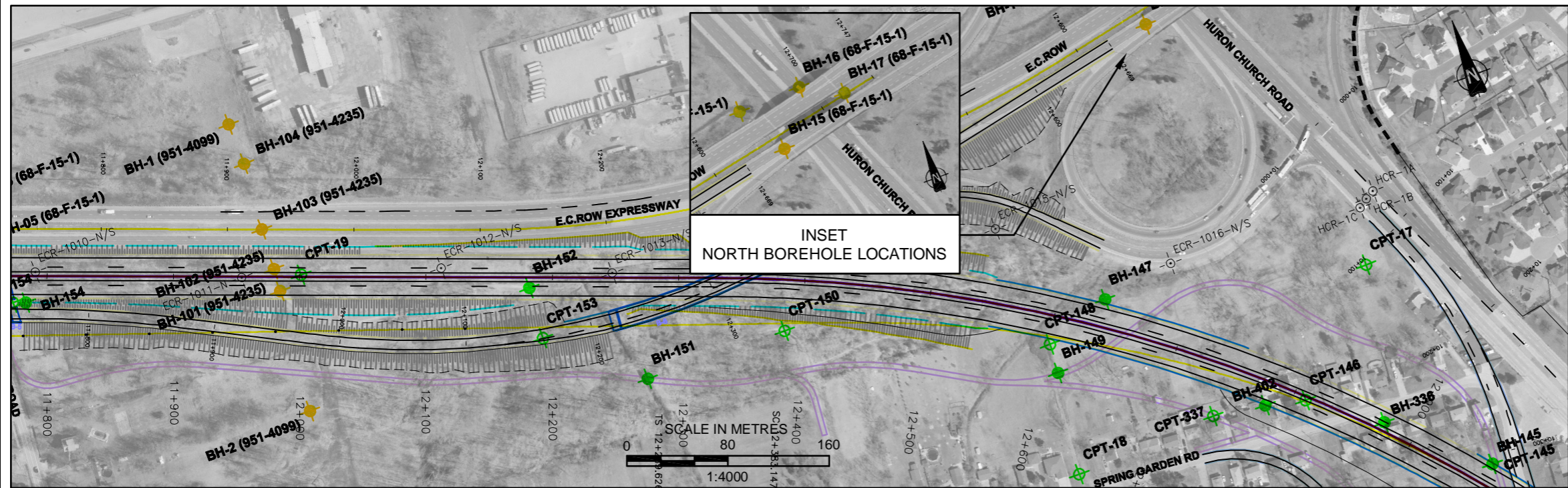
NOTES

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SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR - ESSEX PARKWAY WINDSOR, ONTARIO

INTERPRETED STRATIGRAPHIC PROFILE STN 10+800 TO 11+800

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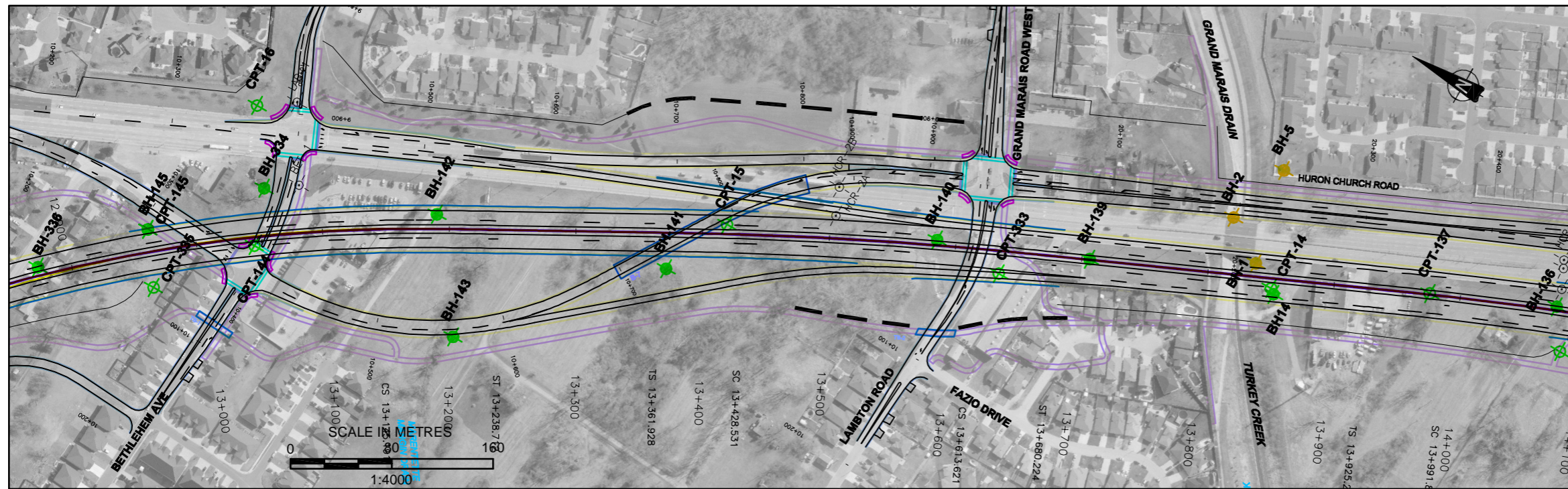
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WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

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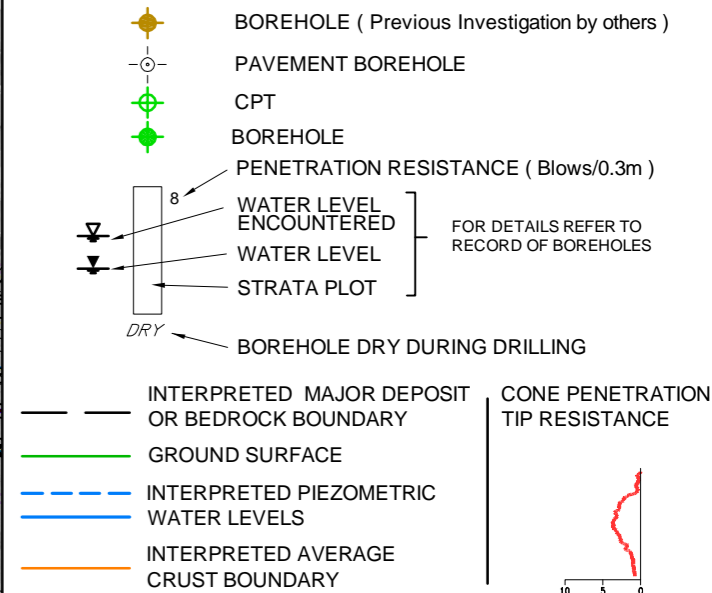
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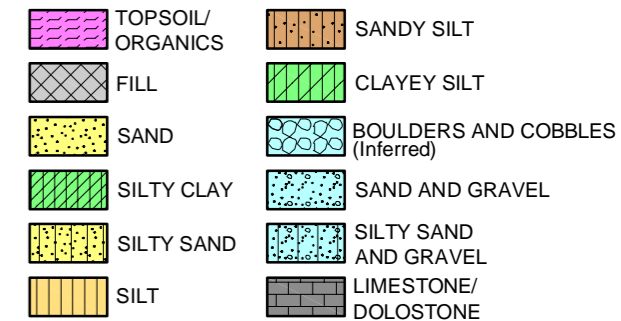
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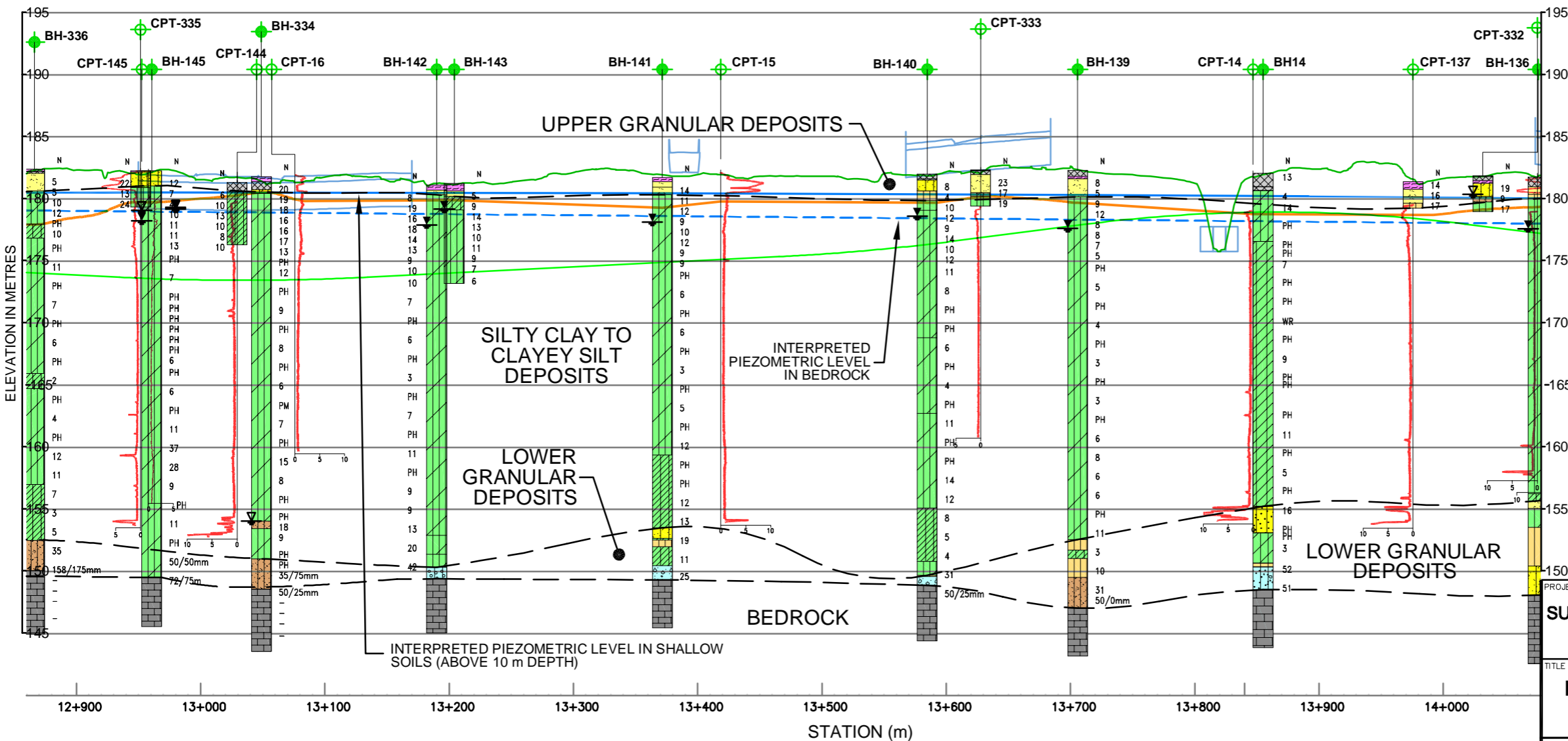


SIMPLIFIED STRATIGRAPHY



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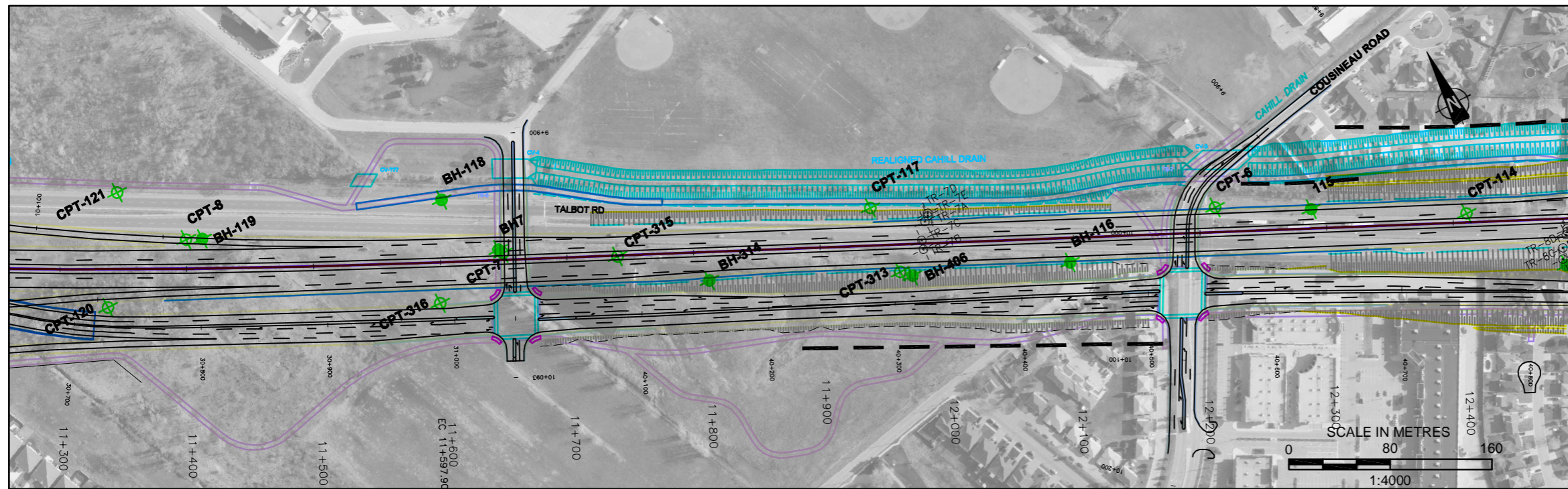
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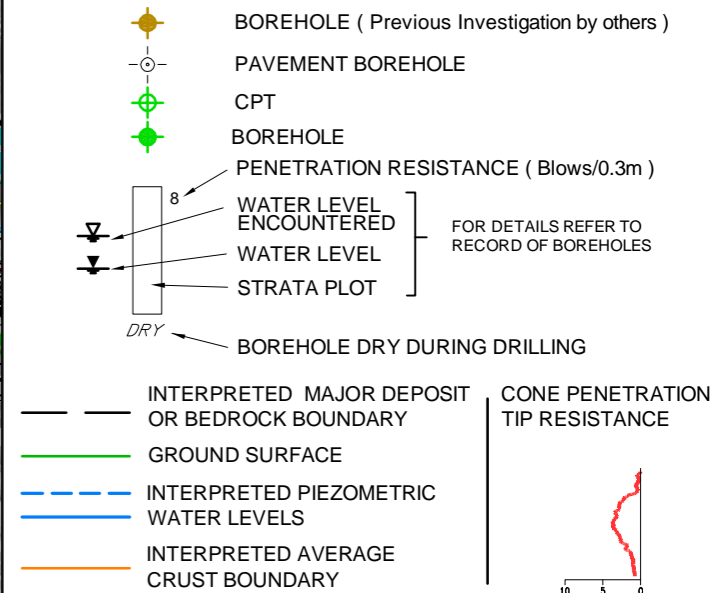
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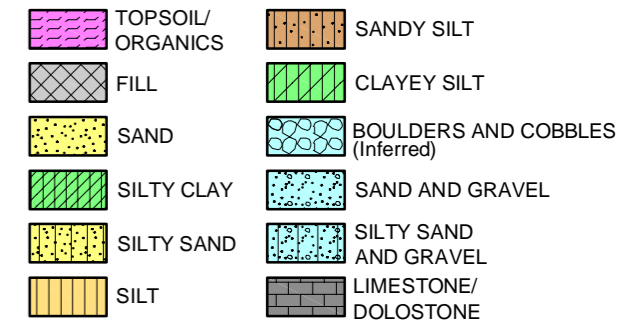
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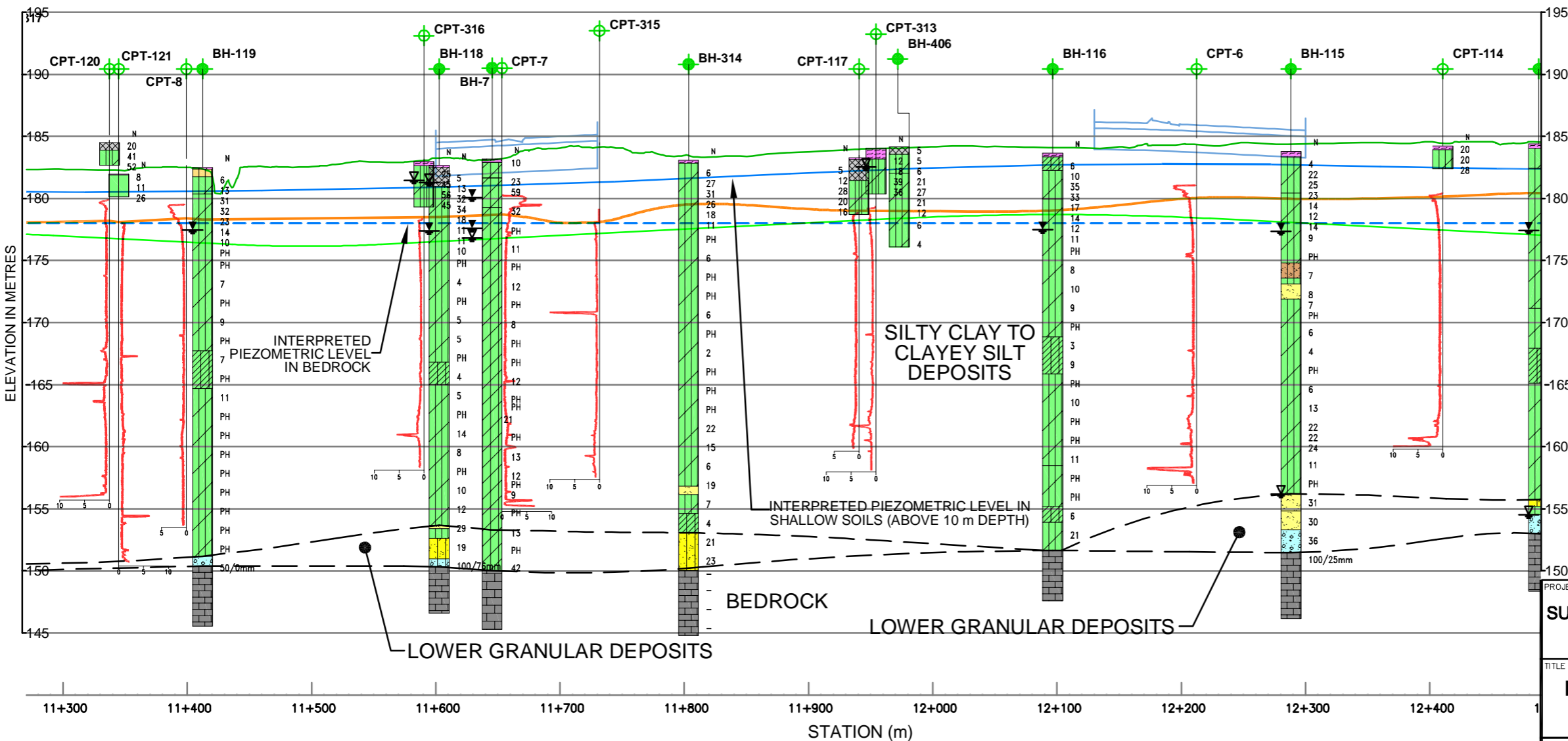


SIMPLIFIED STRATIGRAPHY



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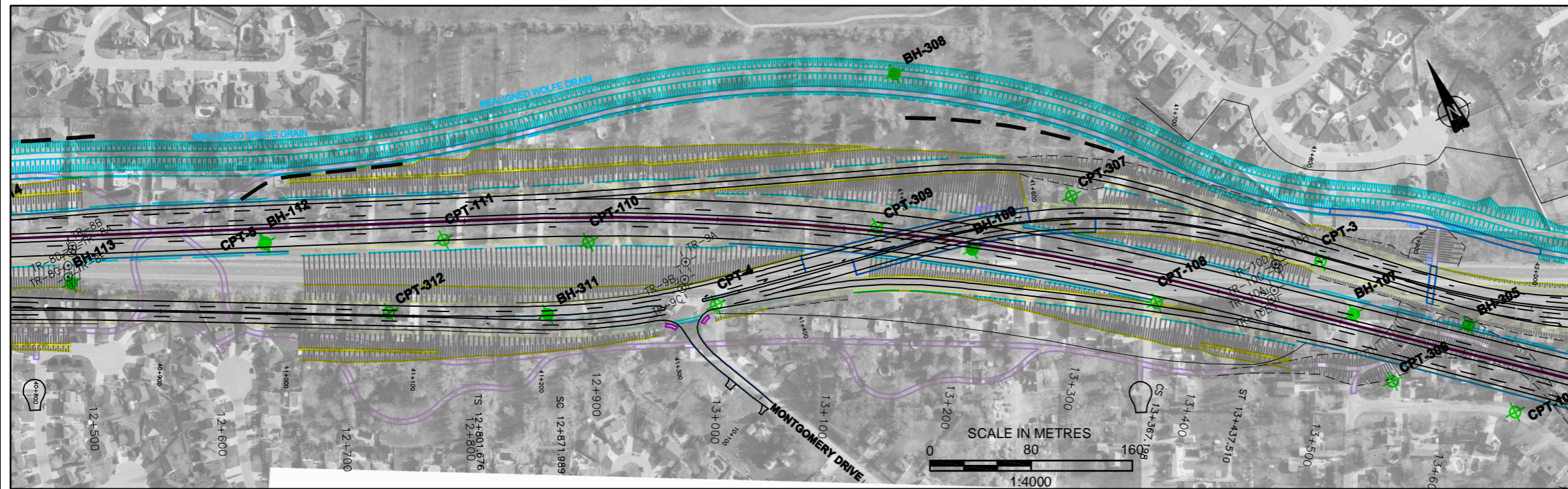


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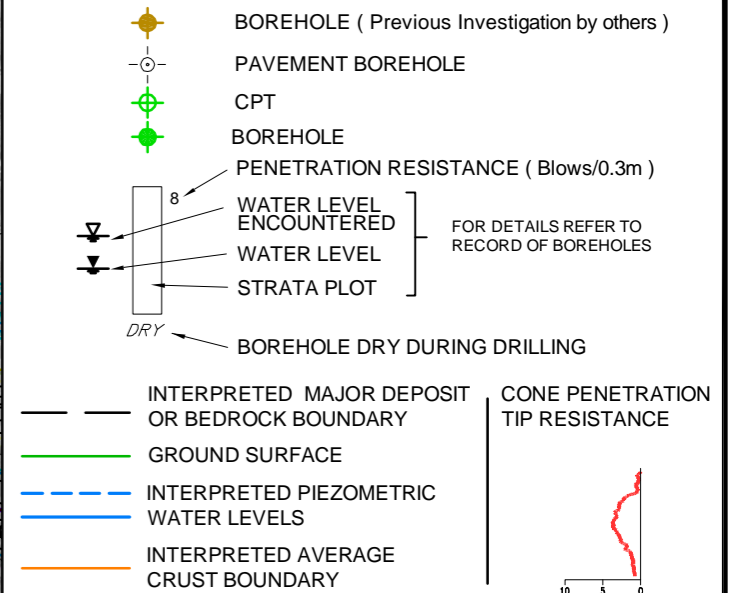
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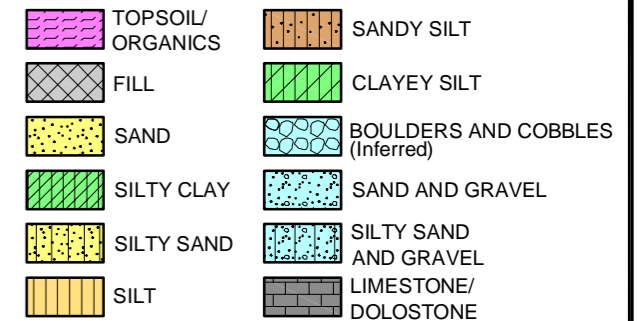
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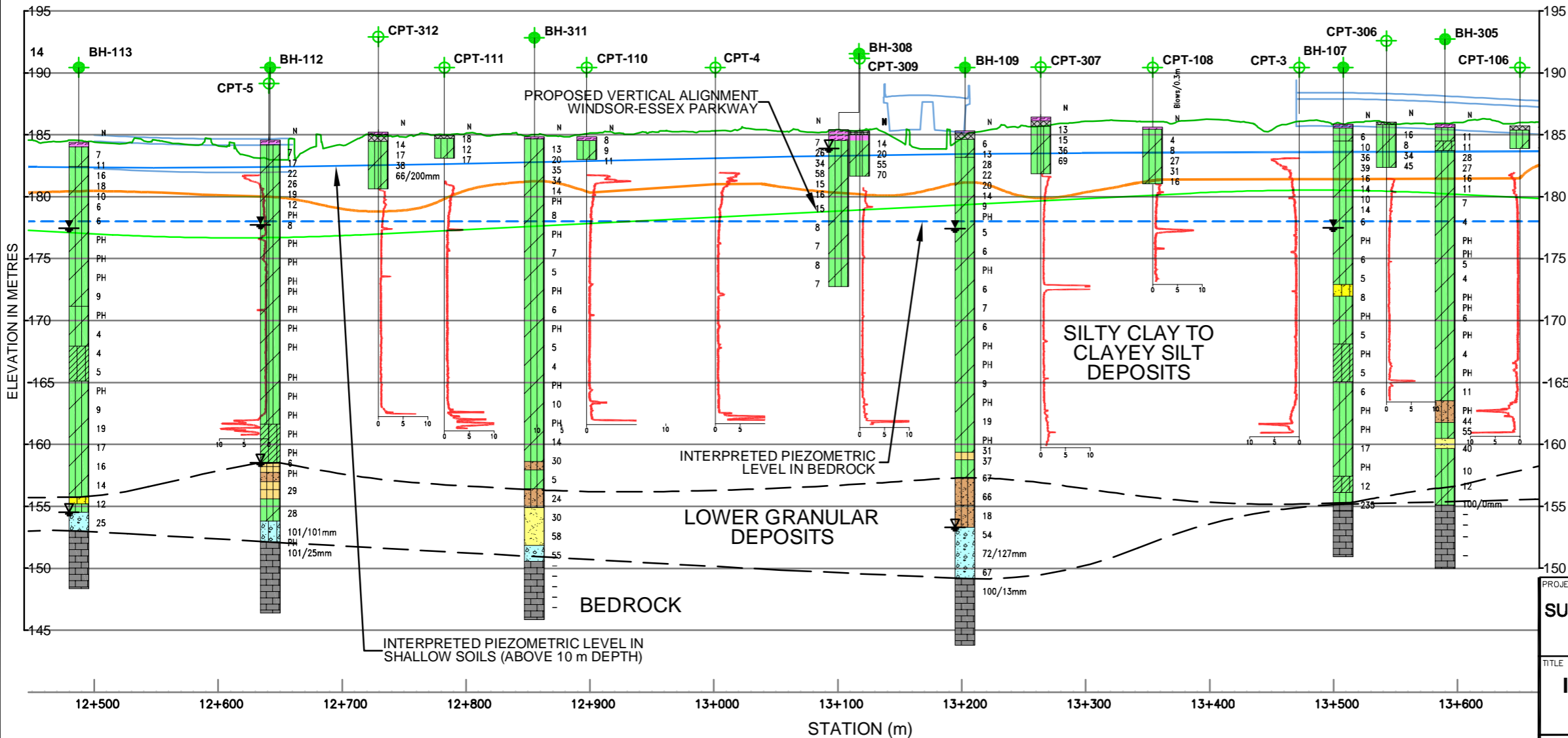


SIMPLIFIED STRATIGRAPHY

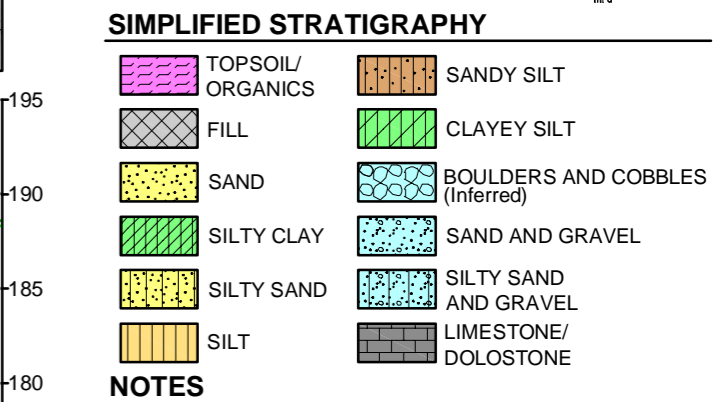
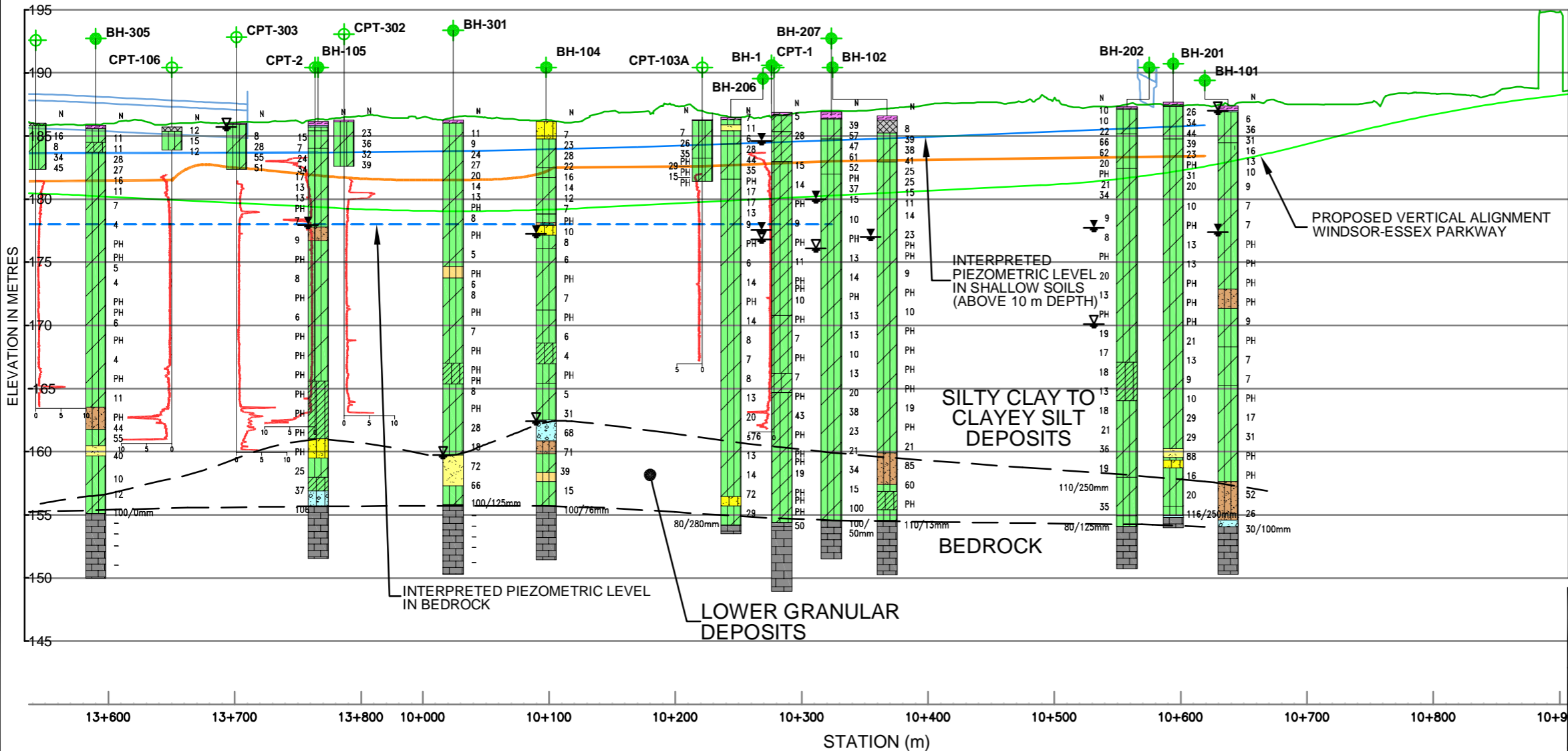
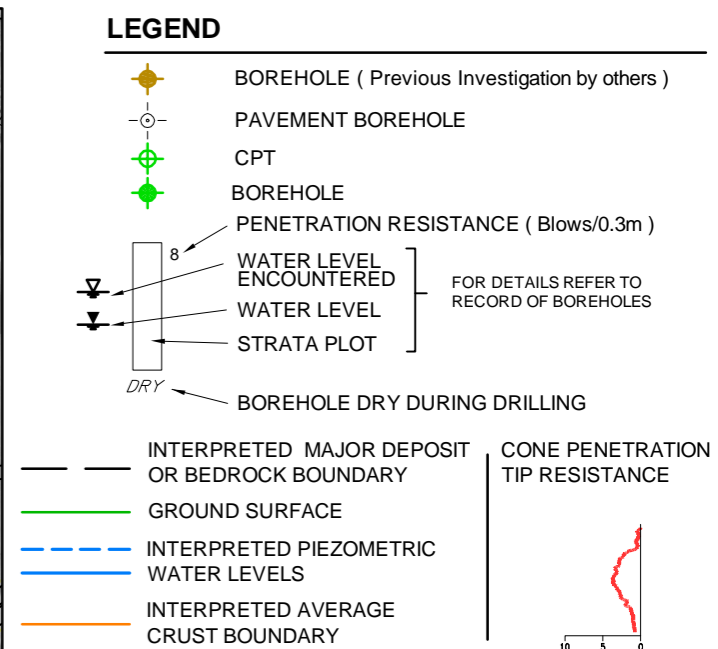
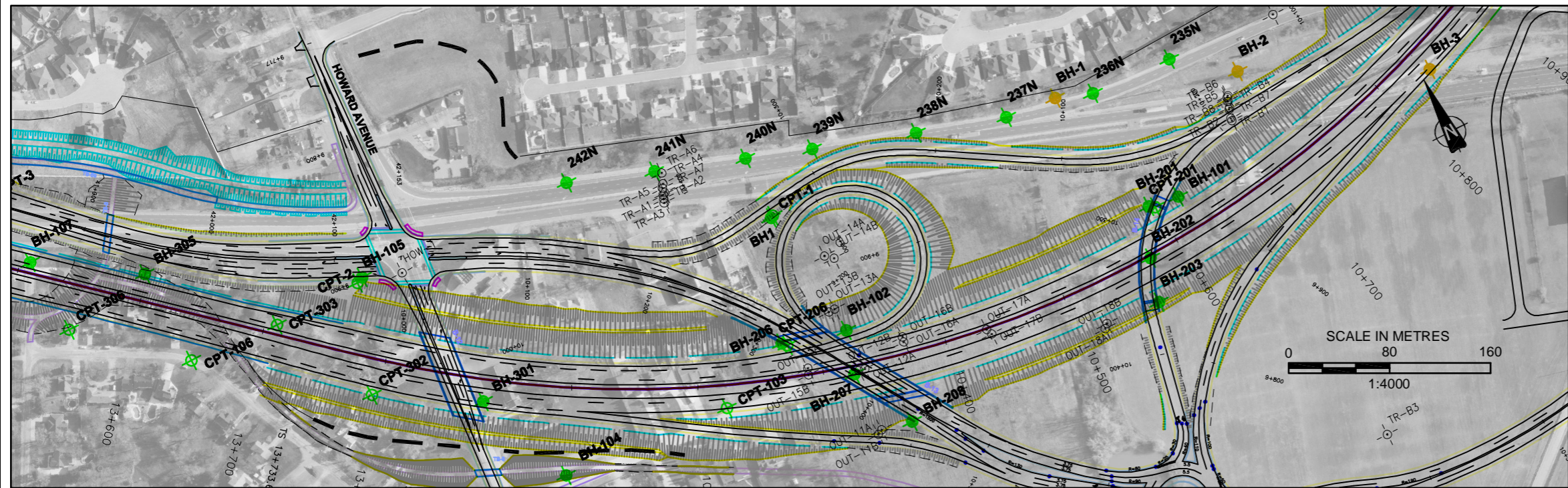


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1. This drawing is to be read in conjunction with the report titled "Subsurface Conditions Interpretation Report, Windsor-Essex Parkway".
2. This interpreted stratigraphy is a simplification of the subsurface conditions. Detailed descriptions of the 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 an interpreted 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 INTERPRETATION REPORT WINDSOR - ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE			
INTERPRETED STRATIGRAPHIC PROFILE STN 12+500 TO 13+600			
PROJECT No. 07-1130-207-0		FILE No. 0711302070-R02051	
CADD	JR/WFH	Dec. 11/10	SCALE AS SHOWN REV. 2
CHECK			5.1H
Golder Associates LONDON, ONTARIO			



- NOTES**
- This drawing is to be read in conjunction with the report titled "Subsurface Conditions Interpretation Report, Windsor-Essex Parkway".
 - 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.
 - Major soil deposit and rock formations are delineated by the boundary line identified above. The boundary established represents an interpreted ground condition; however, variation in the boundaries from those illustrated must be anticipated both parallel and perpendicular to the section line.
 - 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.
 - 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.
 - 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.
 - Borehole width in profile is not to scale.

PROJECT

SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR - ESSEX PARKWAY
WINDSOR, ONTARIO

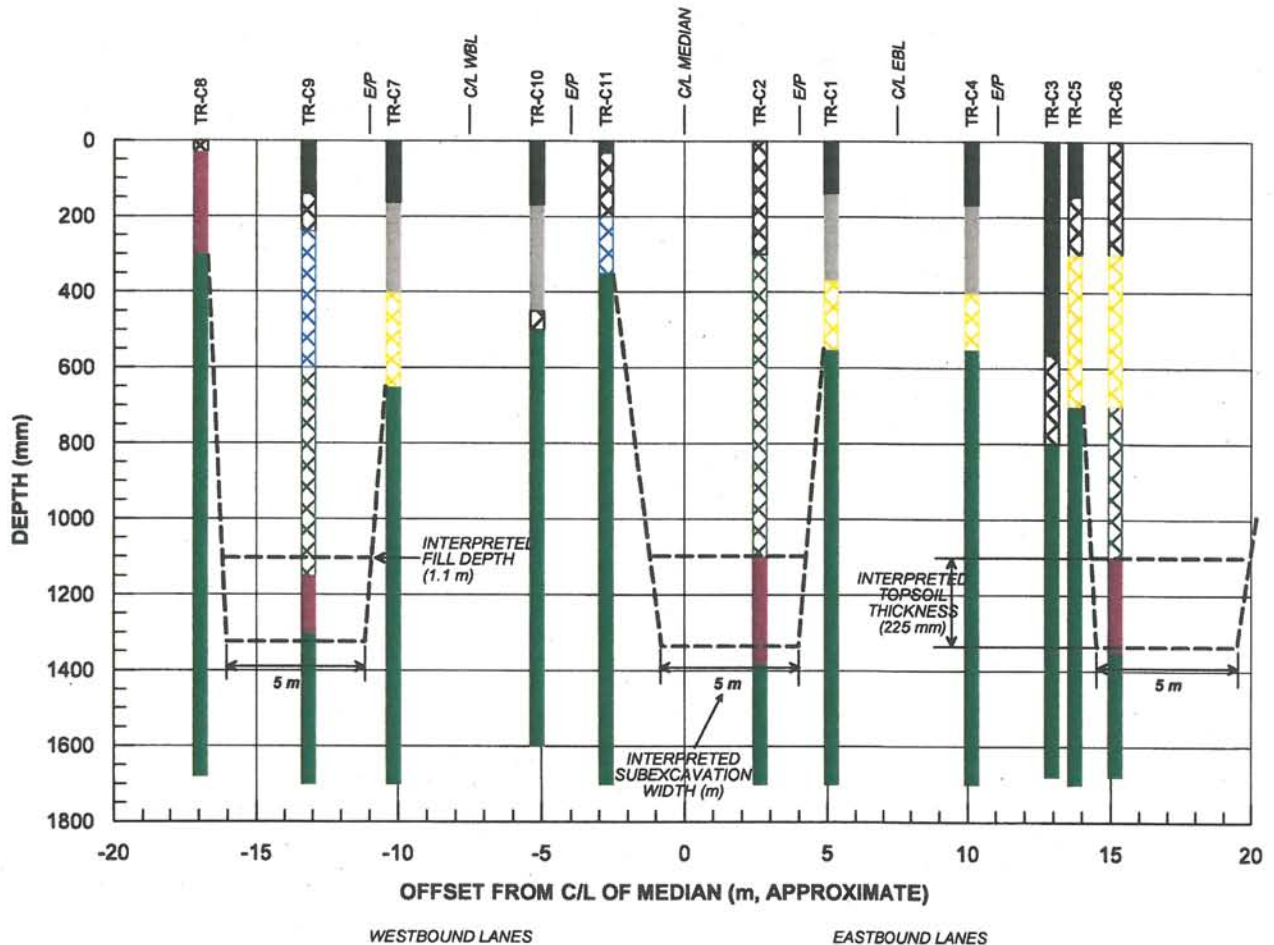
TITLE

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

PROJECT No. 07-1130-207-0	FILE No. 0711302070-R02051
CADD JRV/VJH Dec. 11/10	SCALE AS SHOWN REV. 2
CHECK	5.11

Golder Associates
LONDON, ONTARIO

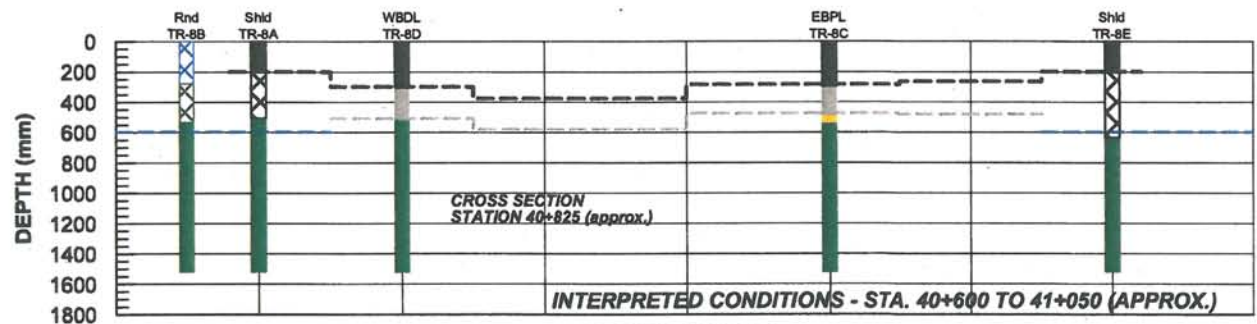
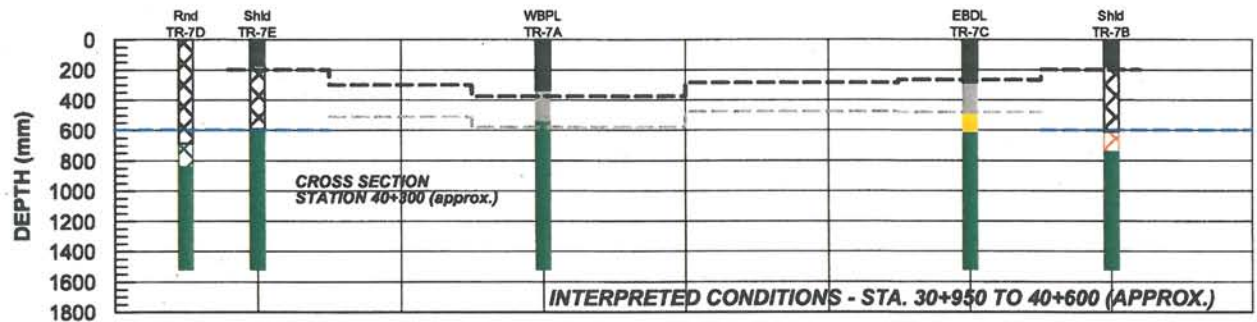
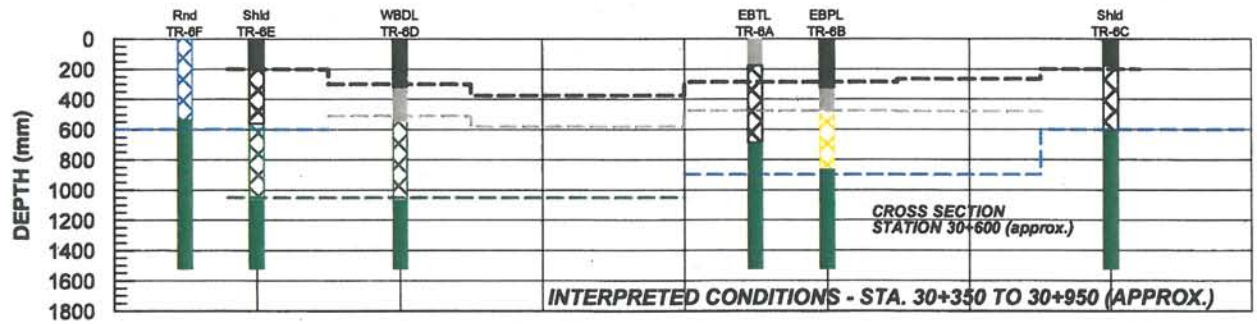
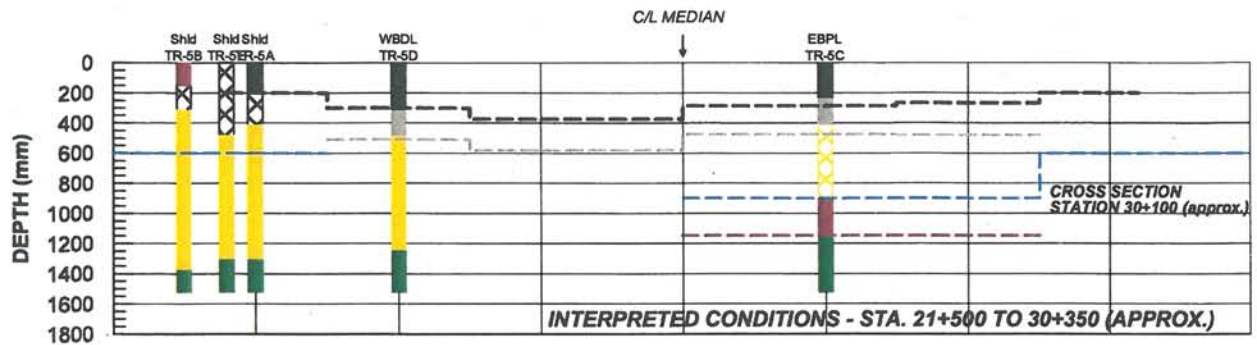
**INTERPRETED CROSS SECTION
EXISTING HIGHWAY 401
STATION 10+900 TO STATION 11+300 (APPROXIMATE)**



- 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 INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
INTERPRETED PAVEMENT AND			
SHALLOW SUBSURFACE CONDITIONS			
EXISTING HIGHWAY 401			
PROJECT No. 05-1140-003		FILE No. DRIC PAVT 5-2-A.grf	
DRAWN MEB		Dec 7-09	
CHECK SJB		REV. 1	
Golder Associates LONDON, ONTARIO		FIGURE 5.2A	



CROSS SECTIONS

WESTBOUND

INTERPRETED BASELINE DEPTHS

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

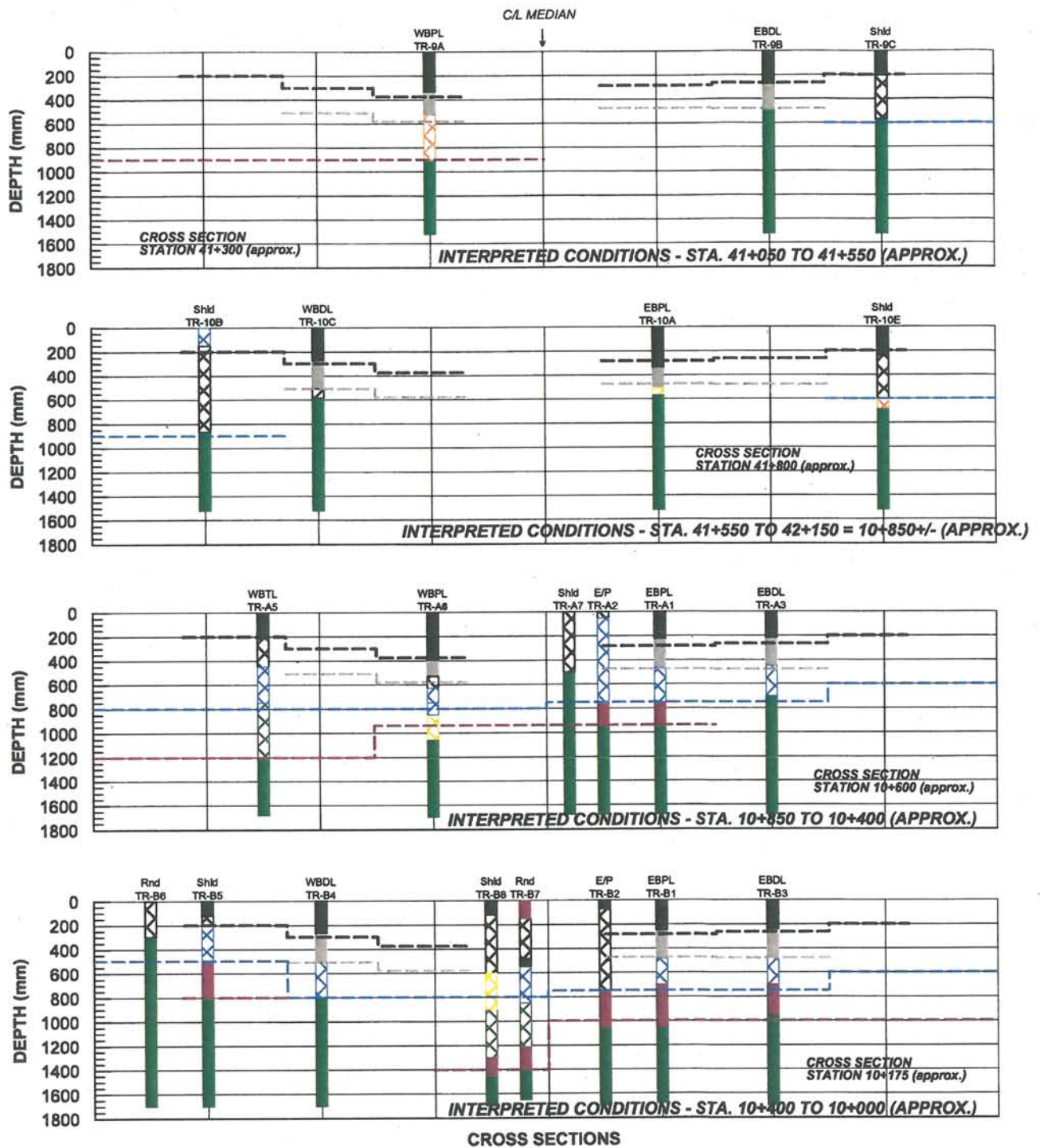
- ASPHALT
- CONCRETE
- GRANULAR FILL
- CLAYEY FILL
- BURIED TOPSOIL

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

EASTBOUND

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

PROJECT			
SUBSURFACE CONDITIONS INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
INTERPRETED PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS EXISTING HIGHWAY 3/TALBOT ROAD			
PROJECT No. 05-1140-003		FILE No. DRIC PAVT 5-2-B.grf	
DRAWN MEB		SCALE AS SHOWN	
CHECK SJS		REV. 1	
Golder Associates		FIGURE 5.2B	



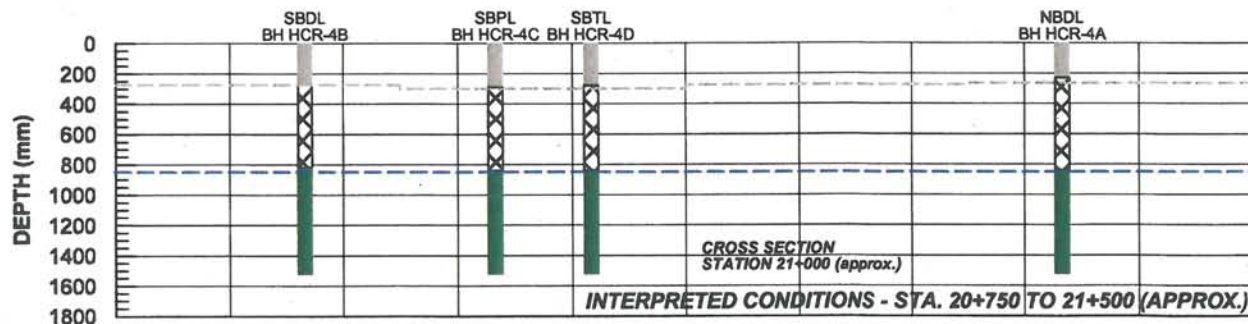
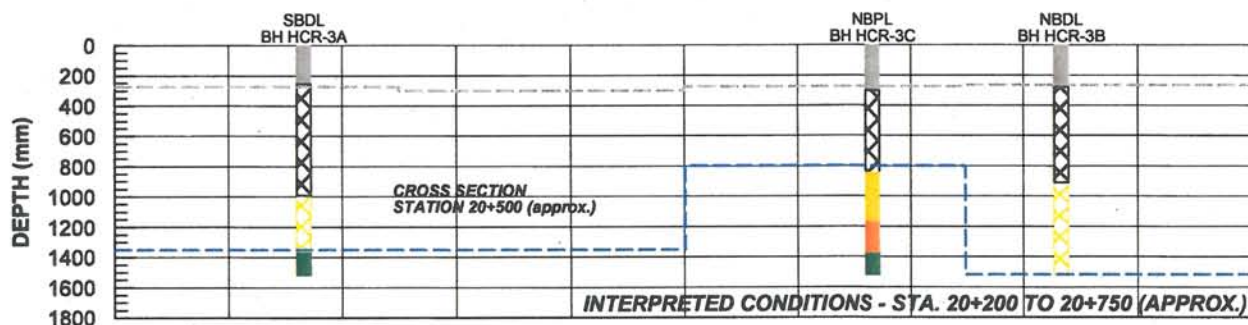
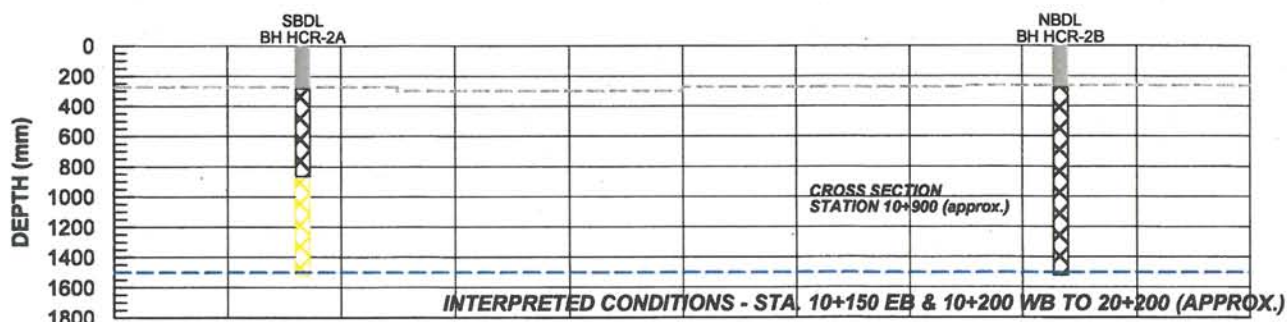
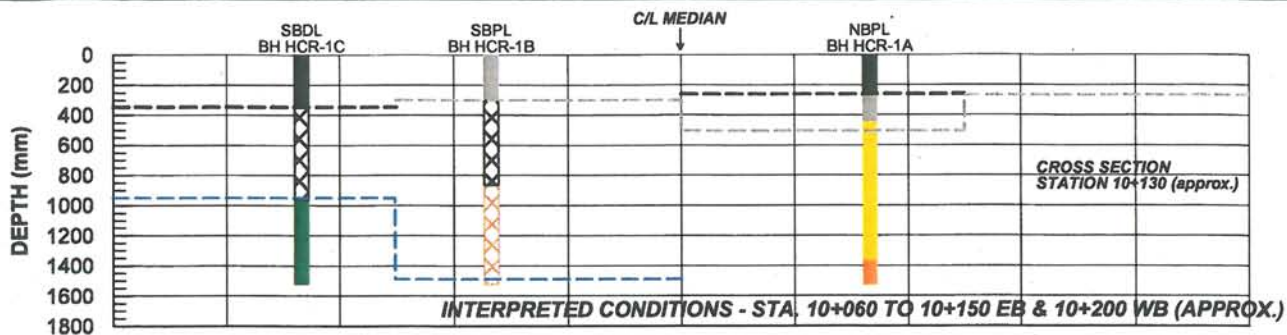
WESTBOUND

EASTBOUND

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

INTERPRETED DEPTHS	
	ASPHALT
	CONCRETE
	GRANULAR BASE
	SILTY SAND FILL
	SAND & GRAVEL FILL
	SAND FILL
	CLAYEY FILL
	CLAYEY SILT/SILTY CLAY
	TOPSOIL
---	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 INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
INTERPRETED PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS EXISTING HIGHWAY 3/TALBOT ROAD			
PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-C.grt
DRAWN	MEB	Dec 7-08	SCALE AS SHOWN/REV. 1
CHECK			FIGURE 5.2C
		Golder Associates LONDON, ONTARIO	



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

- INTERPRETED DEPTHS
--- ASPHALT
--- CONCRETE
--- GRANULAR FILL

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

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

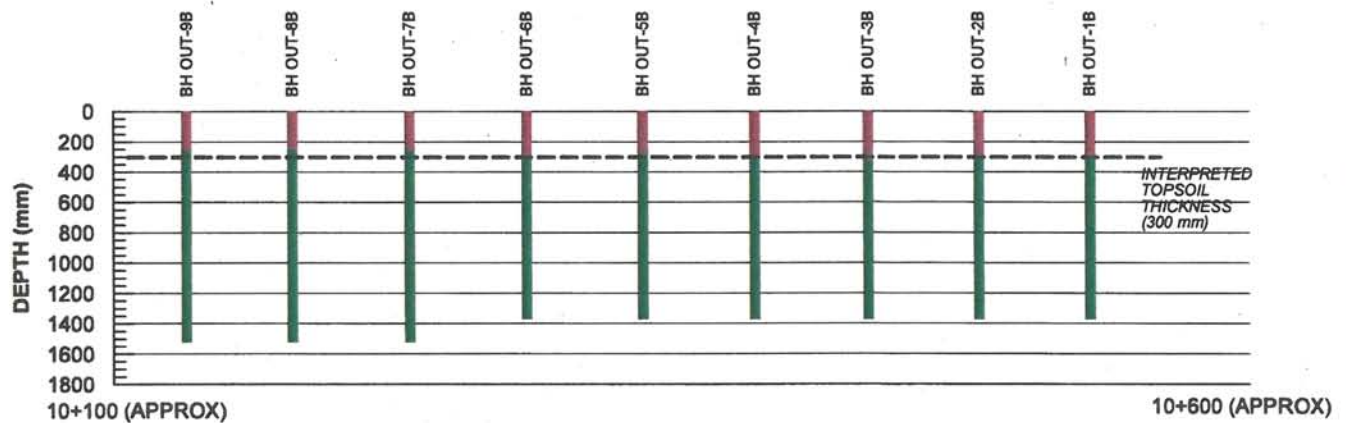
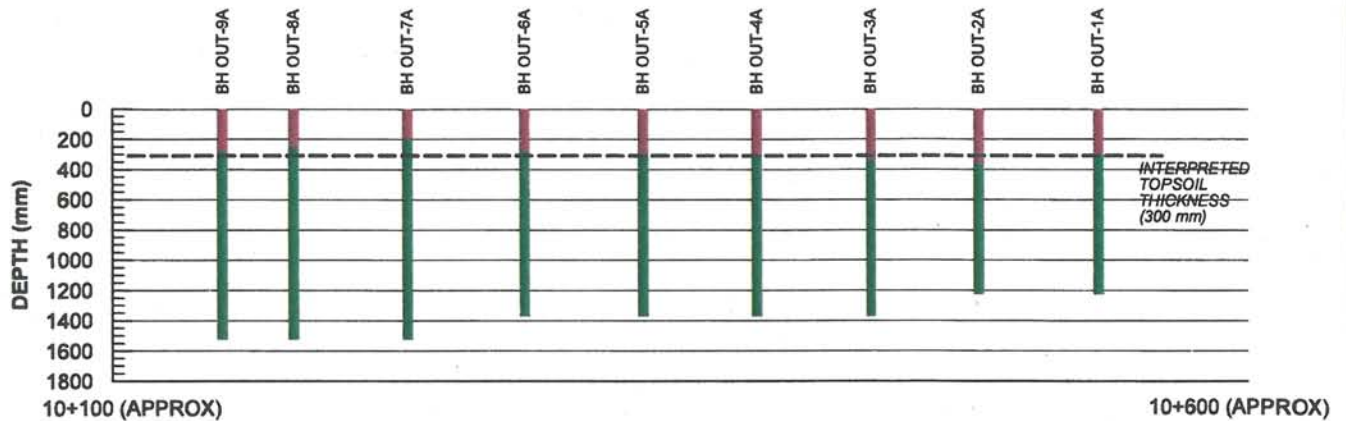
TITLE INTERPRETED PAVEMENT AND
SHALLOW SUBSURFACE CONDITIONS
EXISTING HURON CHURCH ROAD



PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-D.grf
DRAWN	MEB	Dec 7-09	SCALE AS SHOWN
CHECK	3/8/10	12/10/09	REV. 1

FIGURE 5.2D

**INTERPRETED 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**

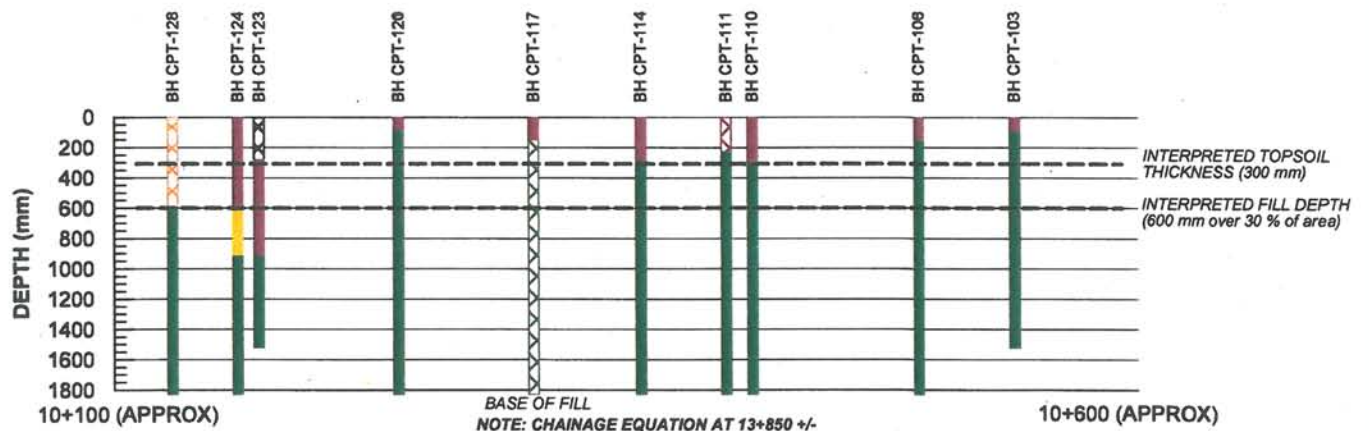
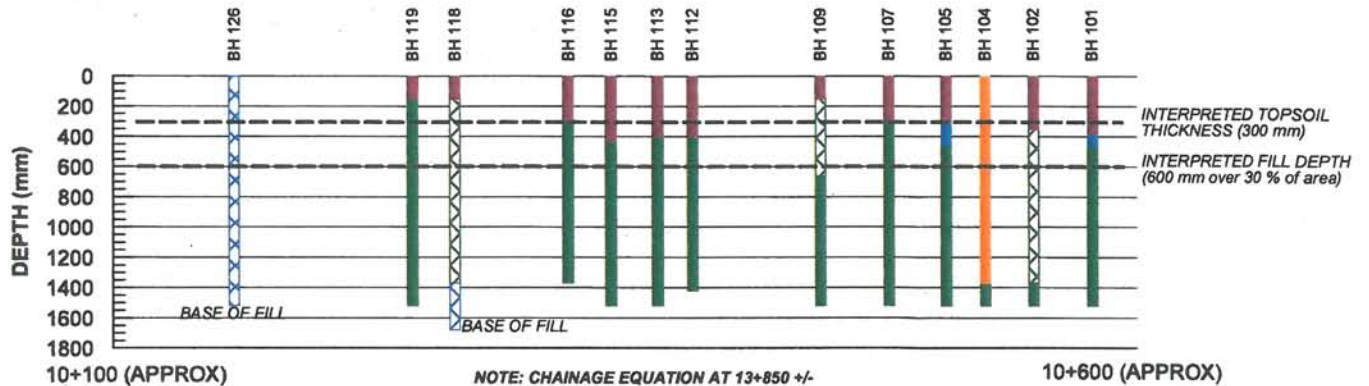
- NOTES:**
1. TWO PLOTS SHOWN FOR CLARITY.
 2. ALL DIMENSIONS ARE APPROXIMATE AND FOR ILLUSTRATION PURPOSES ONLY.
 3. FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

TOPSOIL
CLAYEY SILT/SILTY CLAY

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE INTERPRETED PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED HIGHWAY 3 REALIGNMENT			
PROJECT No. 05-1140-003		FILE No. DRIC PAVT 5-2-E.grf	
DRAWN MEB		Dec 7-09	
CHECK SJB		SCALE AS SHOWN	
FIGURE 5.2E			



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



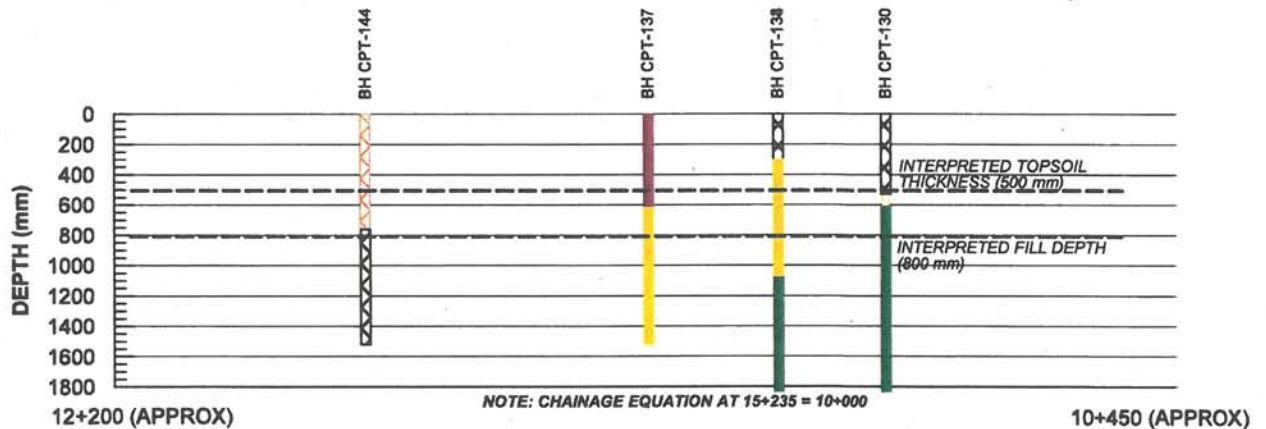
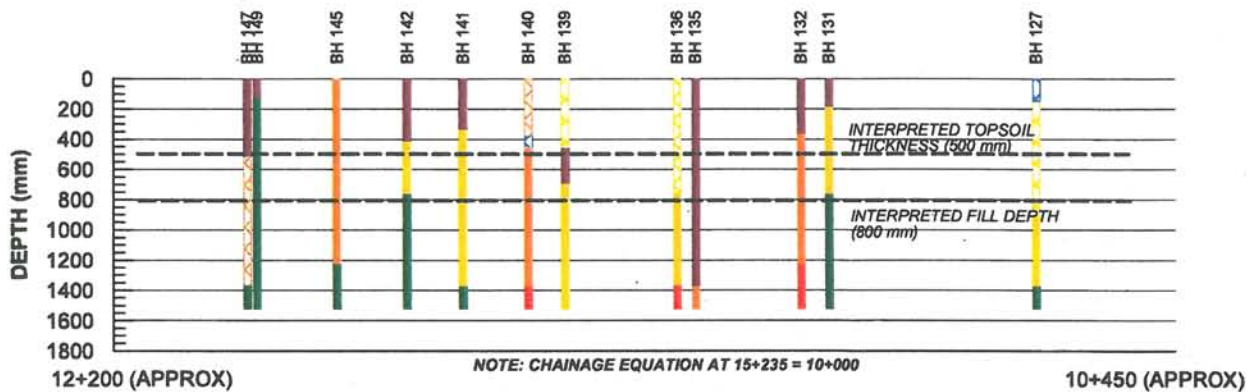
**FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
TWO PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION**

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

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE INTERPRETED PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS HIGHWAY 3/TALBOT ROAD CORRIDOR			
PROJECT No. 05-1140-003		FILE No. DRIC PAVT 5-2-F.grf	
DRAWN MEB		Dec 7-09	
CHECK SJS		SCALE AS SHOWN	
		FIGURE 5.2F	

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

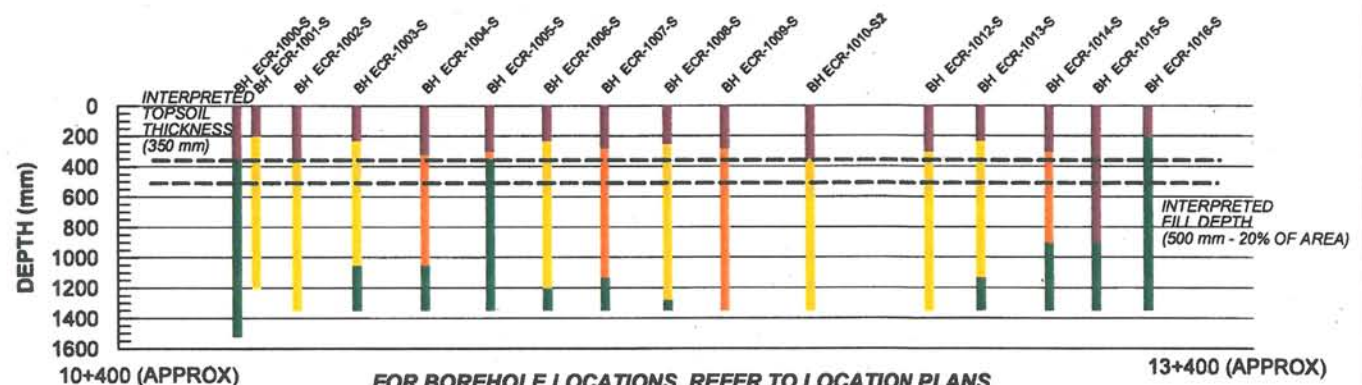
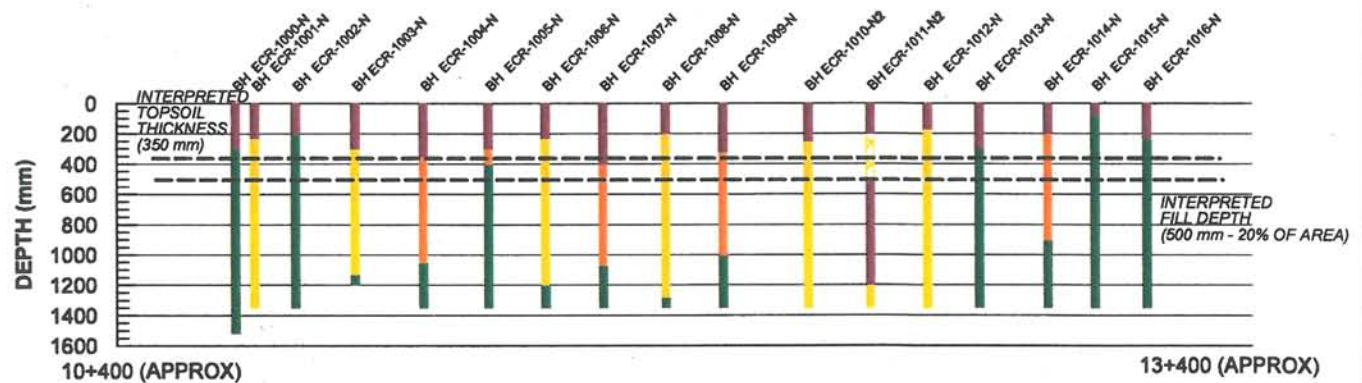
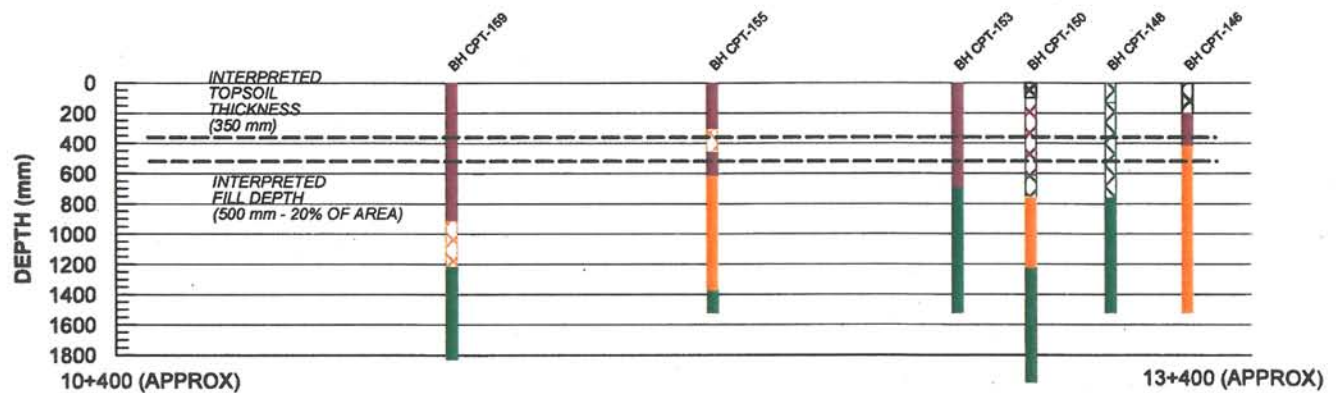
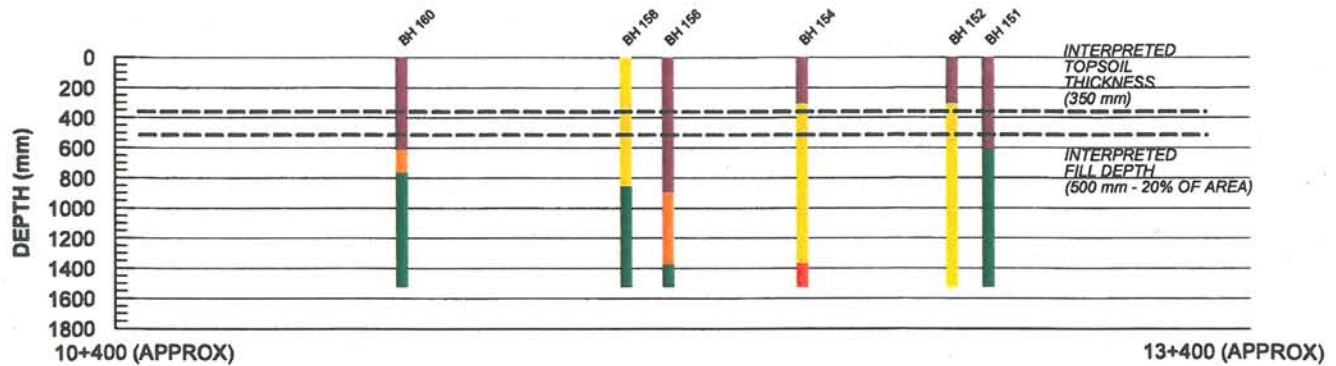


**FOR BOREHOLE LOCATIONS, REFER TO LOCATION PLANS
TWO PLOTS SHOWN FOR CLARITY OF DATA PRESENTATION**

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

PROJECT			
SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE			
INTERPRETED PAVEMENT AND SHALLOW SUBSURFACE CONDITIONS PROPOSED WINDSOR-ESSEX PARKWAY AND SERVICE ROADS HURON CHURCH ROAD CORRIDOR			
PROJECT No. 05-1140-003		FILE No. DRIC PAVT 5-2-G.grf	
DRAWN MEB		SCALE AS SHOWN	
CHECK SSB		REV. 1	
		FIGURE 5.2G	



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

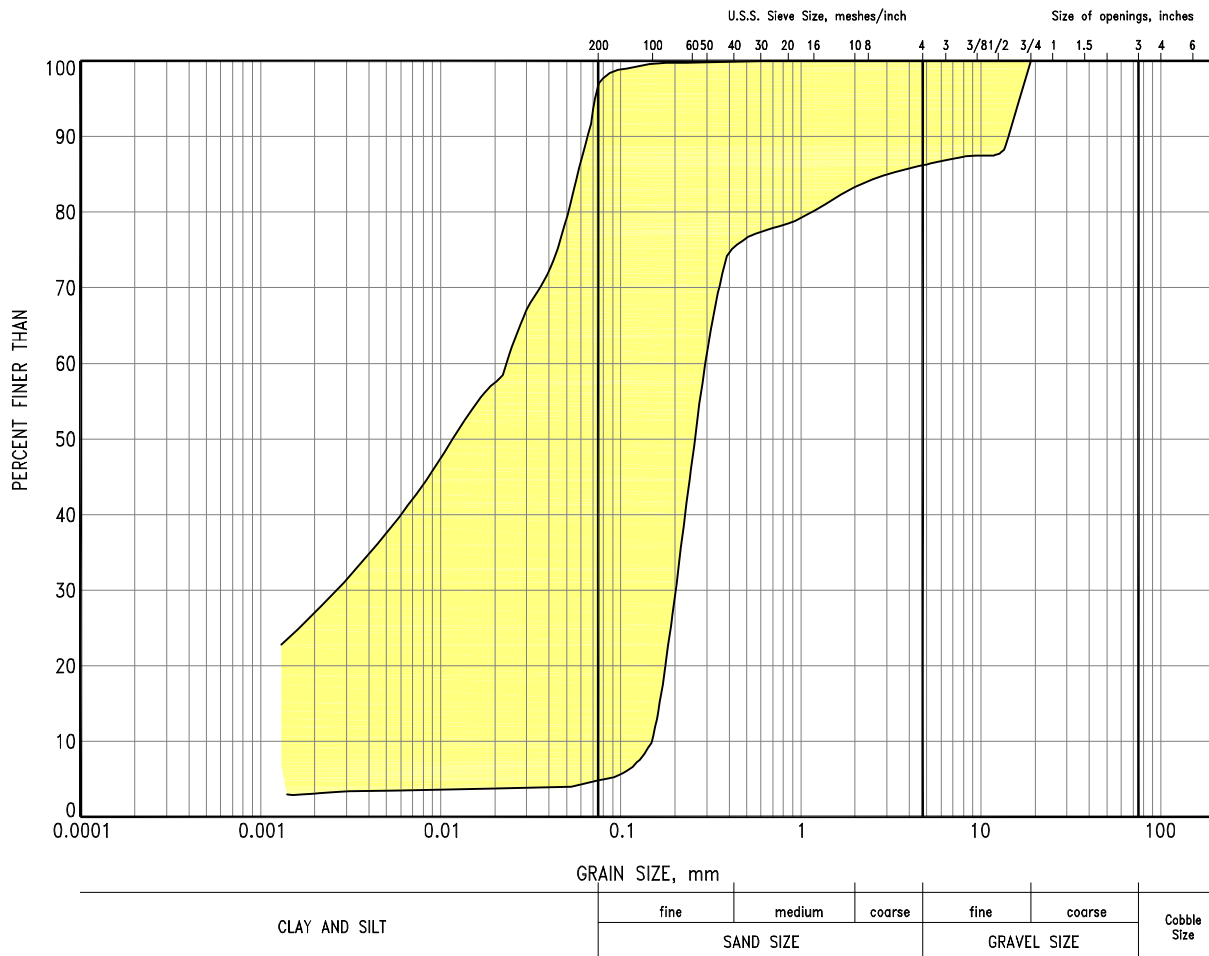
PROJECT
SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

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



PROJECT No.	05-1140-003	FILE No.	DRIC PAVT 5-2-H.grf
DRAWN	MEB	Dec 7-09	SCALE AS SHOWN
CHECK	SJS	REV. 1	

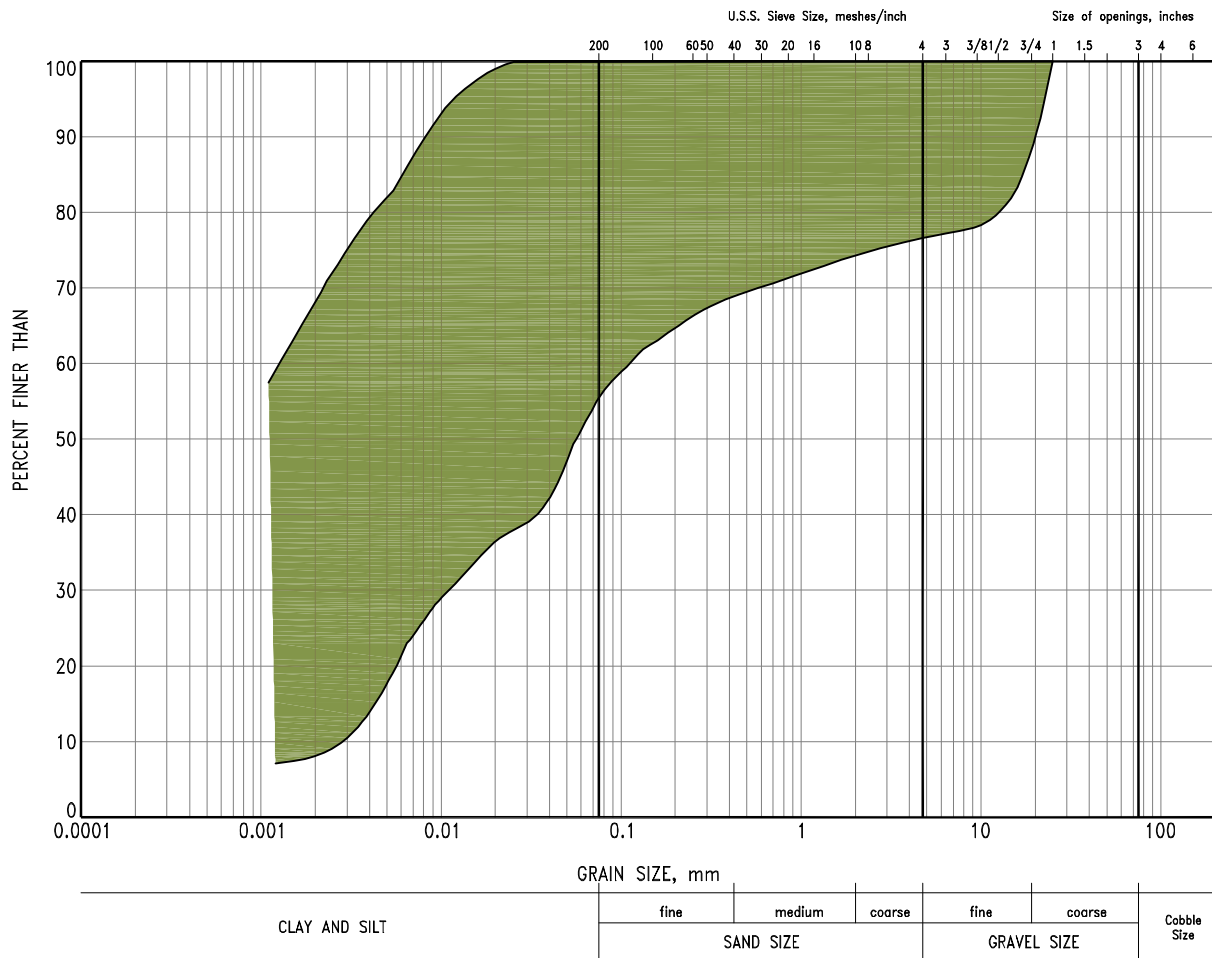
FIGURE 5.2H




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 INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE GRAIN SIZE DISTRIBUTION ENVELOPE UPPER GRANULAR DEPOSITS			
PROJECT No. 07-1130-207-0		FILE No. 0711302070-R02053	
CADD LMK/DCH Dec. 11/10		SCALE AS SHOWN REV. 2	
CHECK		FIGURE 5.3	





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 INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
GRAIN SIZE DISTRIBUTION ENVELOPE			
CLAYEY SILT TO SILTY CLAY DEPOSITS			
PROJECT No.		07-1130-207-0	
FILE No.		0711302070-R02054	
CADD	LMK/DCH	Dec. 11/10	SCALE AS SHOWN
CHECK			REV. 2
 Golder Associates LONDON, ONTARIO			FIGURE 5.4

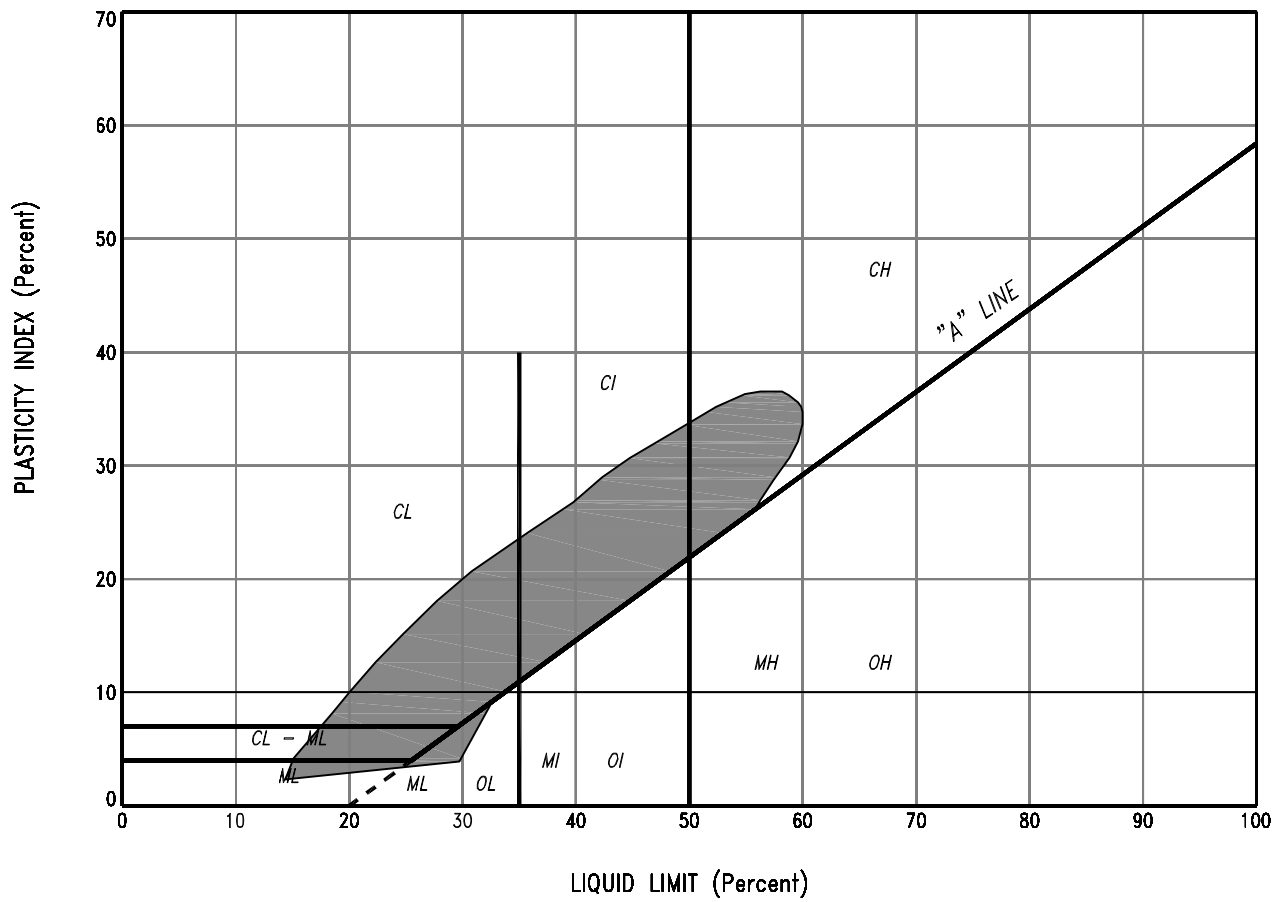


Natural fissuring of silty clay to clayey silt deposit exposed during excavation in Sarnia, Ontario.

NOTES

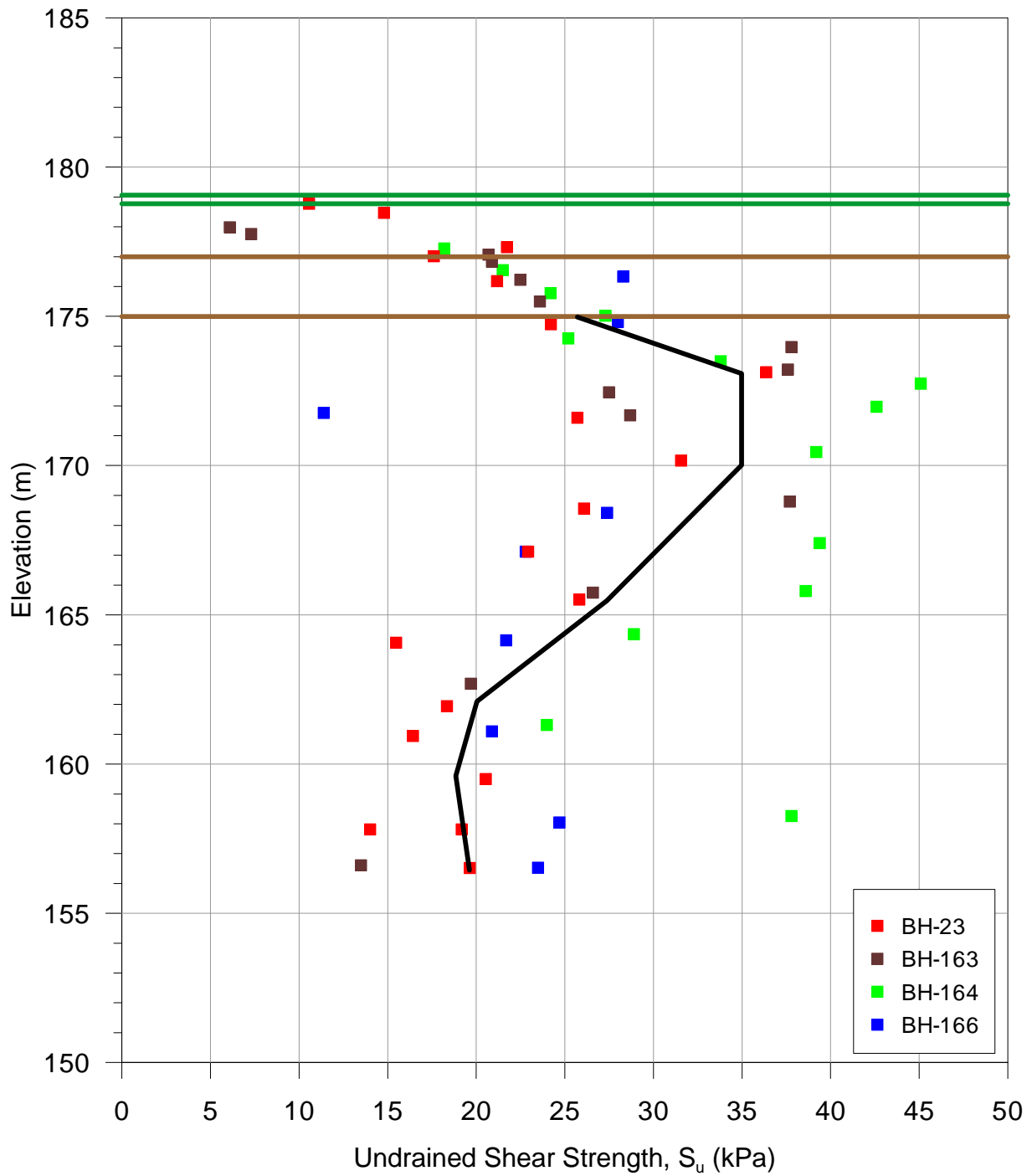
1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO					
TITLE FISSURING OF CLAYEY SILT TO SILTY CLAY DEPOSITS					
 Golder Associates LONDON, ONTARIO		PROJECT No. 07-1130-207-0		FILE No. 0711302070R02055	
				SCALE AS SHOWN	
		DRAWN MK Nov. 18/10		REV. 2	
		CHECK		FIGURE 5.5	



SOIL TYPE PLASTICITY
 C = Clay L = Low
 M = Silt I = Intermediate
 O = Organic H = High

PROJECT			
SUBSURFACE CONDITIONS INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
ATTERBERG LIMITS ENVELOPE			
PROJECT No.		FILE No.	
07-1130-207-0		0711302070-R02056	
DESIGN		SCALE	AS SHOWN
CADD	LMK/DCH	REV.	2
CHECK		FIGURE 5.6	
REVIEW			



— RANGE OF INTERPRETED 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

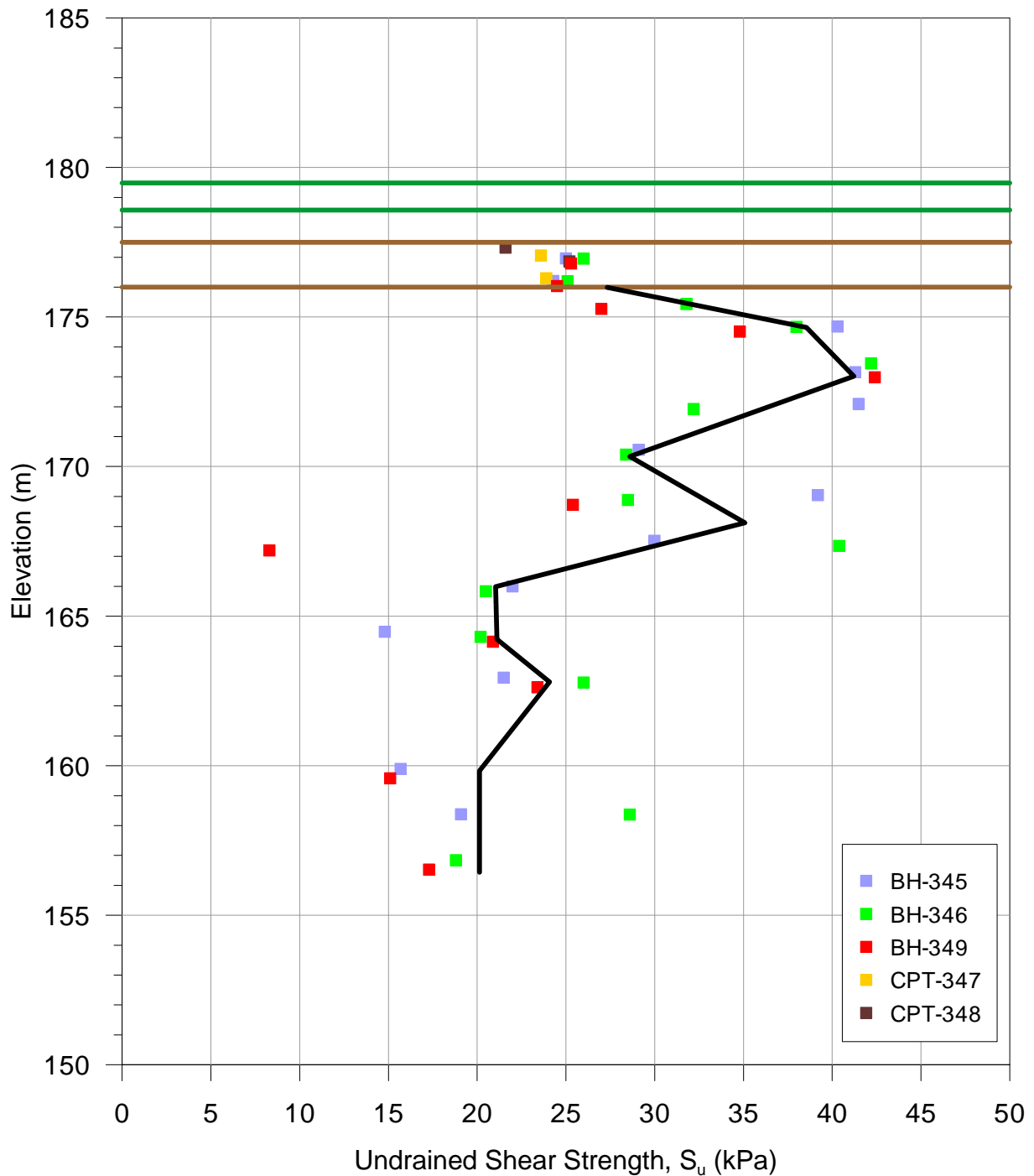
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 8%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 9+900 TO 10+000
(WEST OF OJIBWAY PARKWAY)**



PROJECT No.	0711302070	FILE No.	0711302070R0257A
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7A			



NOTES

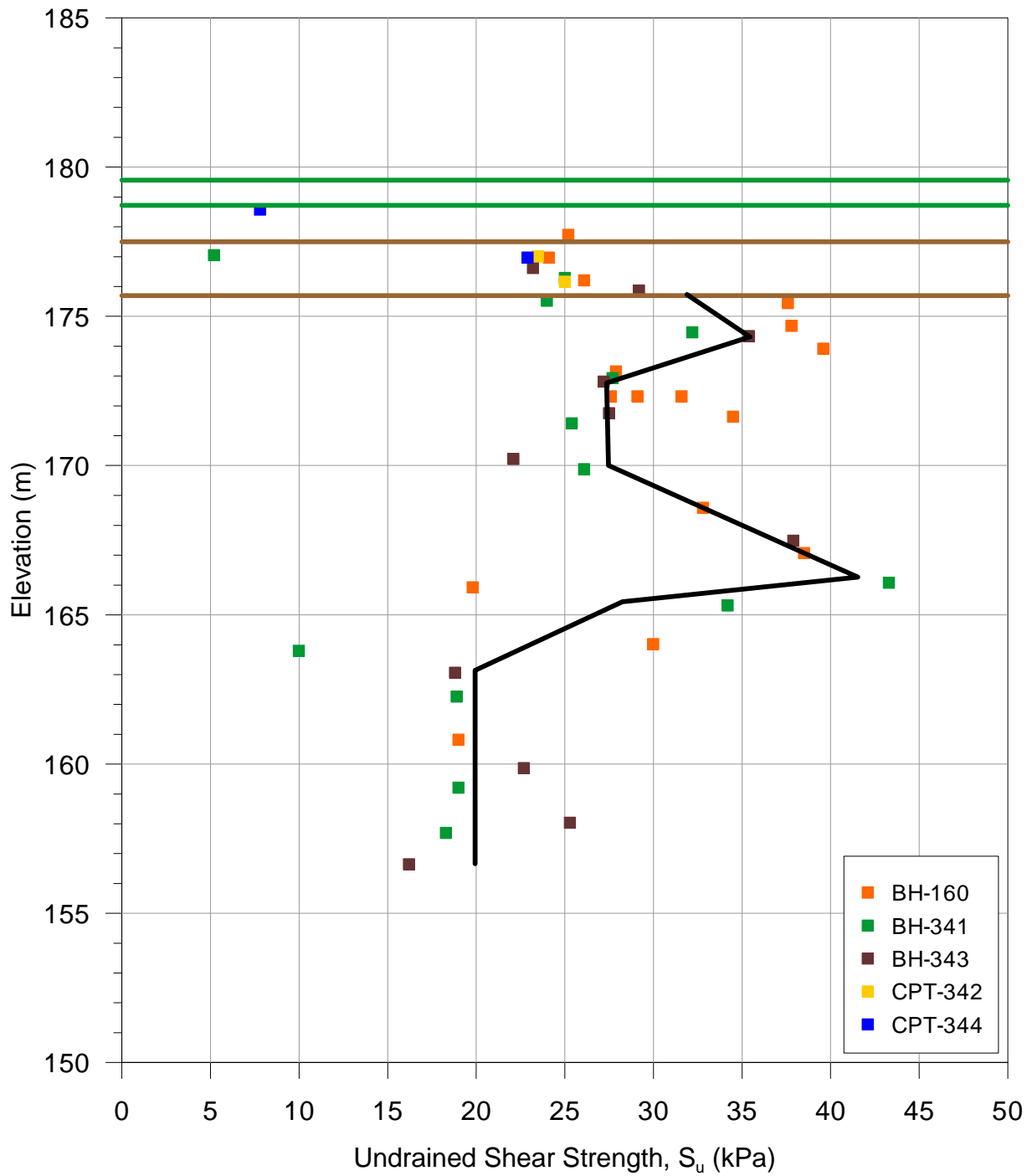
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 8%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 10+000 TO 10+500**



PROJECT No.	0711302070	FILE No.	0711302070R0257B
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
			FIGURE 5.7B



RANGE OF INTERPRETED 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

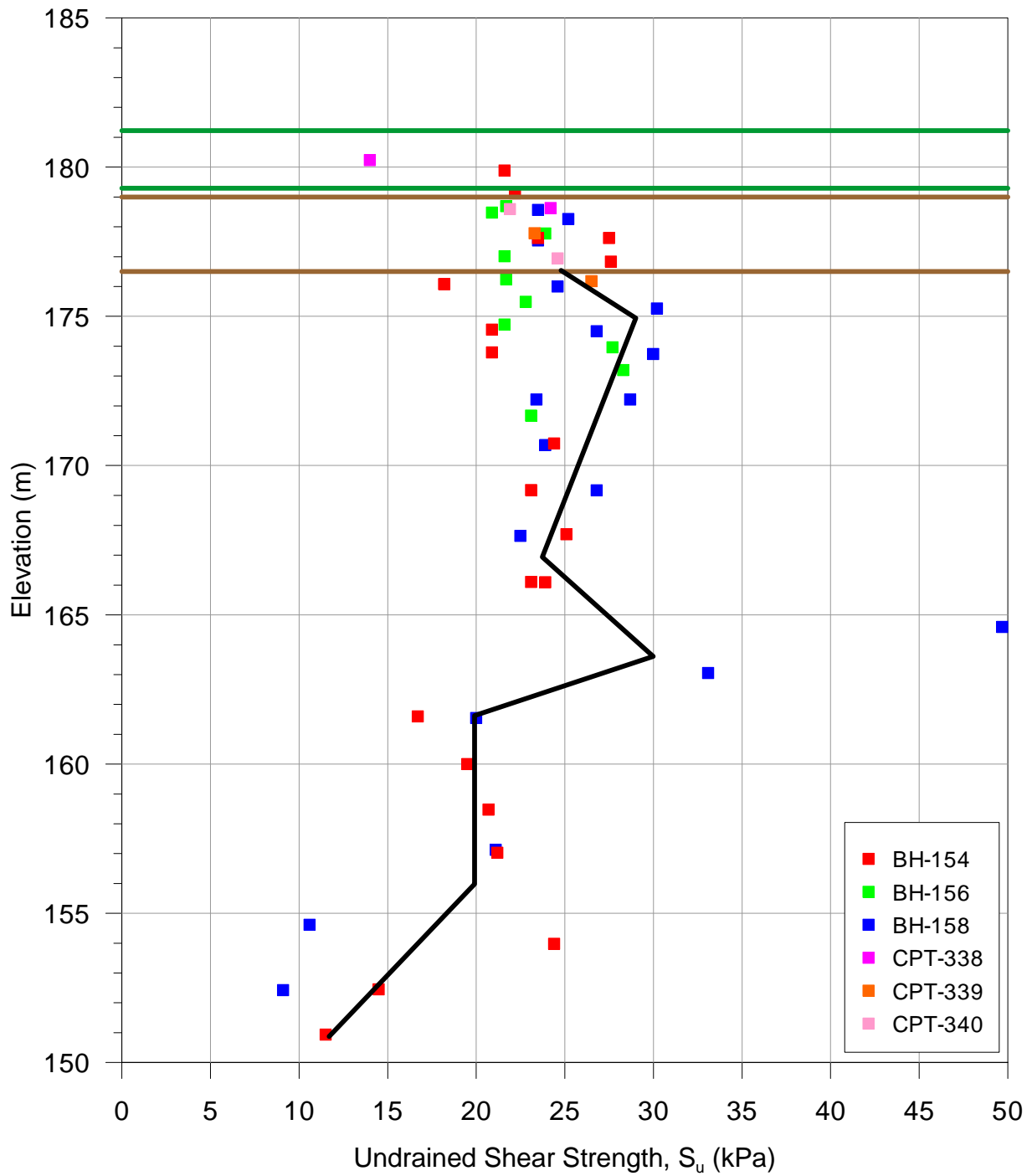
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
 INTERPRETED WATER CONTENT PROFILE
 STATION 10+500 TO 11+000



PROJECT No.	0711302070	FILE No.	0711302070R0257C
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7C			



NOTES

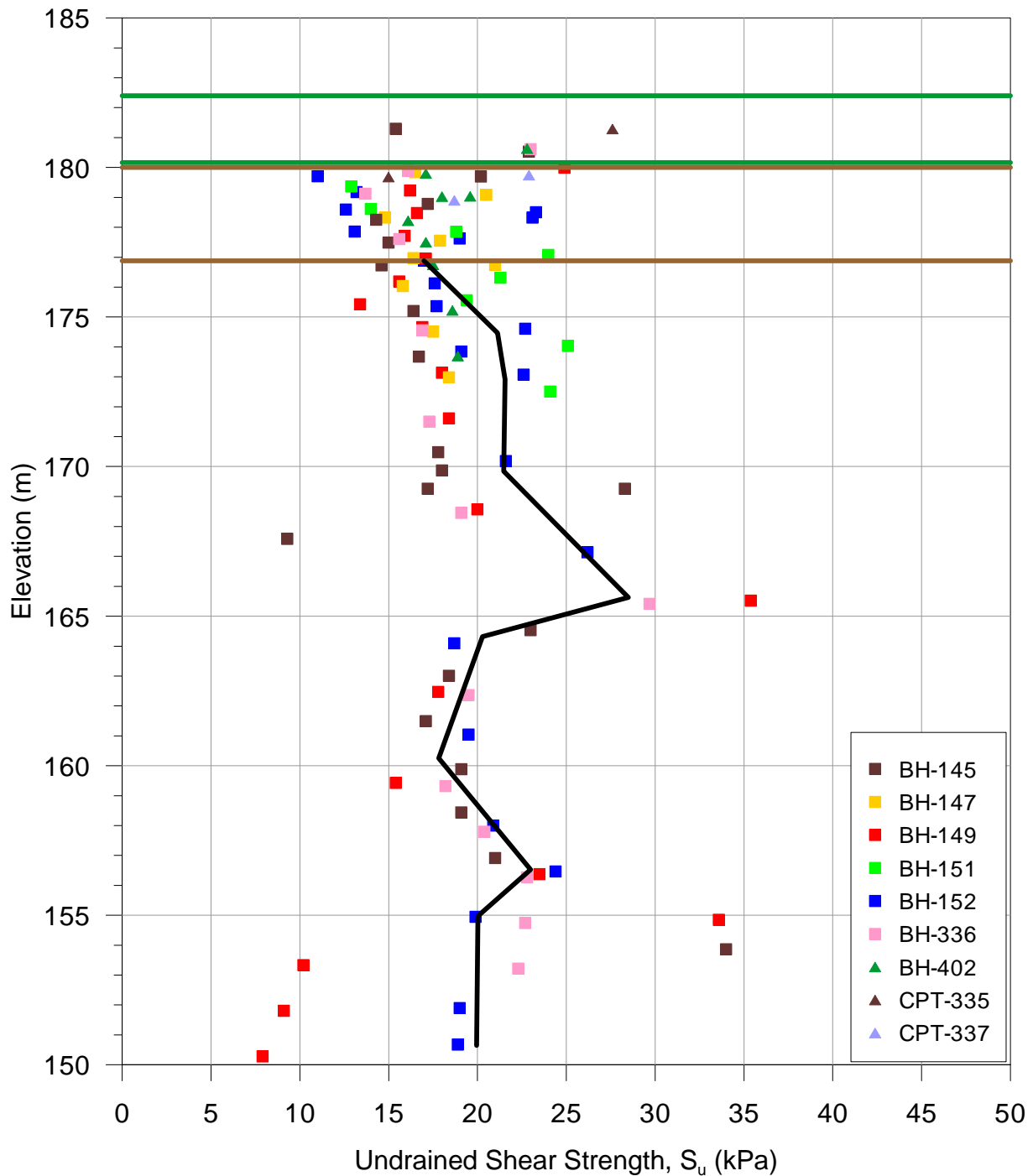
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 11+000 to 12+100**



PROJECT No.	0711302070	FILE No.	0711302070R0257D
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7D			



NOTES

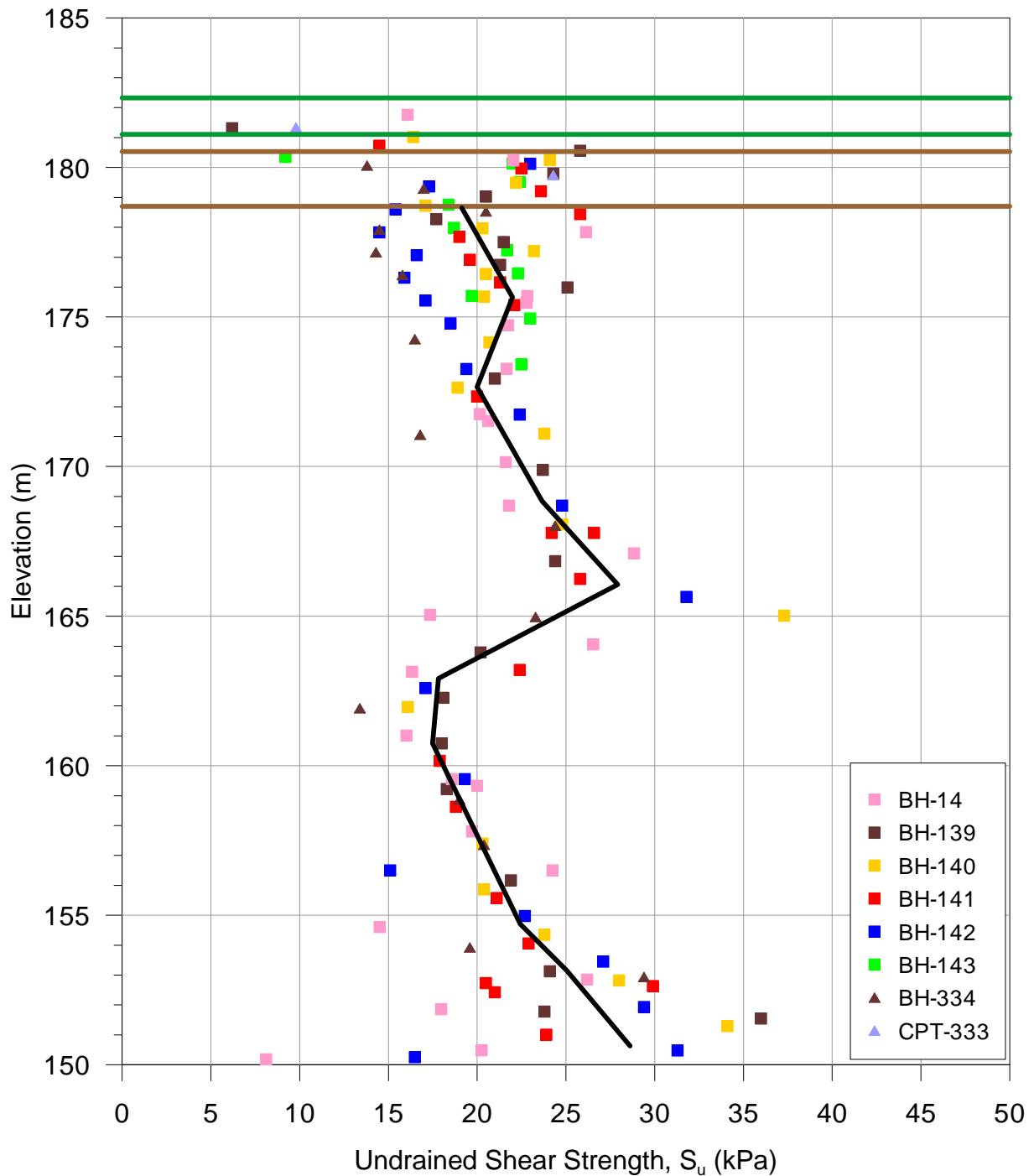
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 12+100 TO 13+000**



PROJECT No.	0711302070	FILE No.	0711302070R0257E
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7E			



RANGE OF INTERPRETED 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

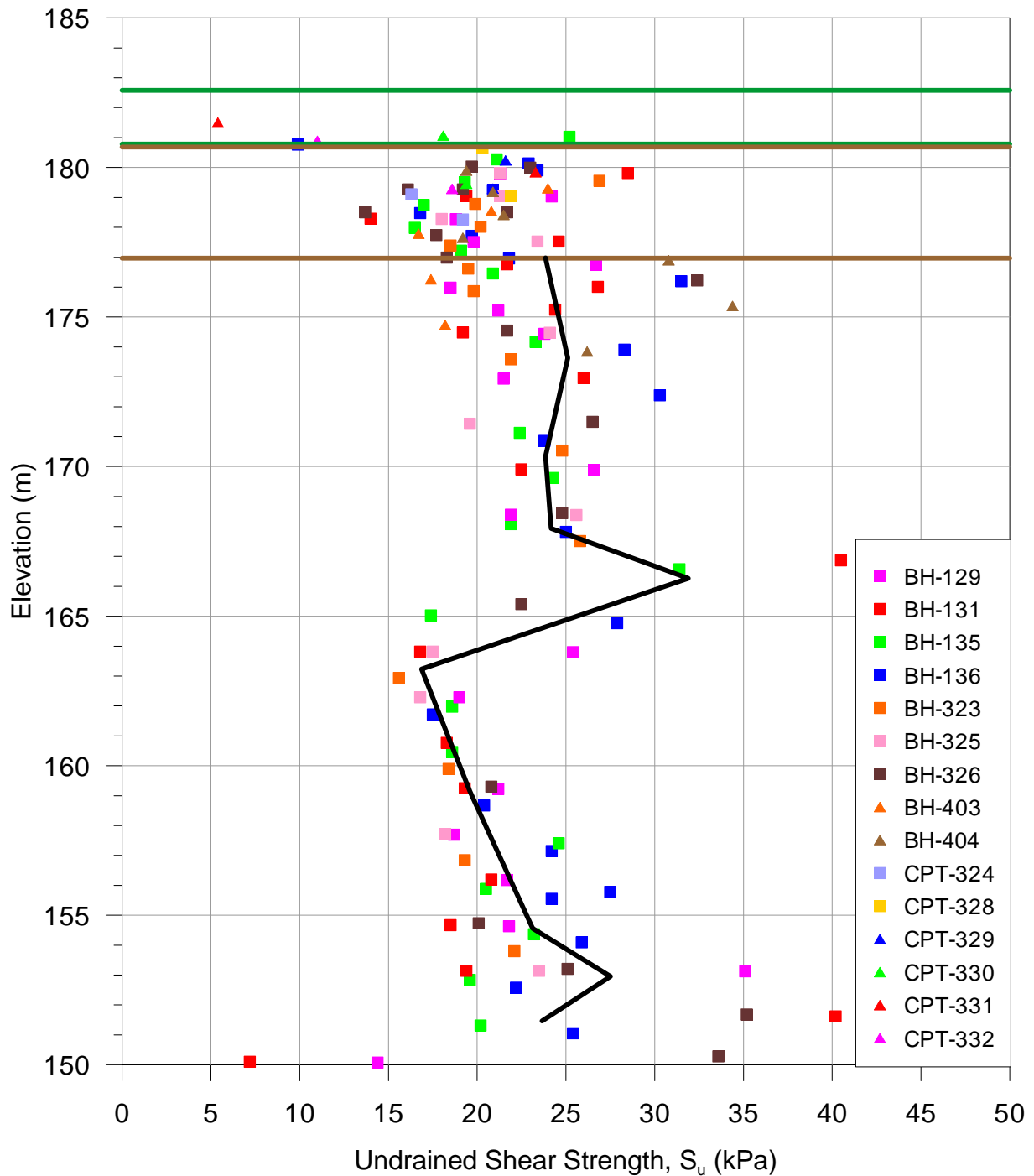
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 13+000 TO 13+900**



PROJECT No.	0711302070	FILE No.	0711302070R0257F
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7F			



NOTES

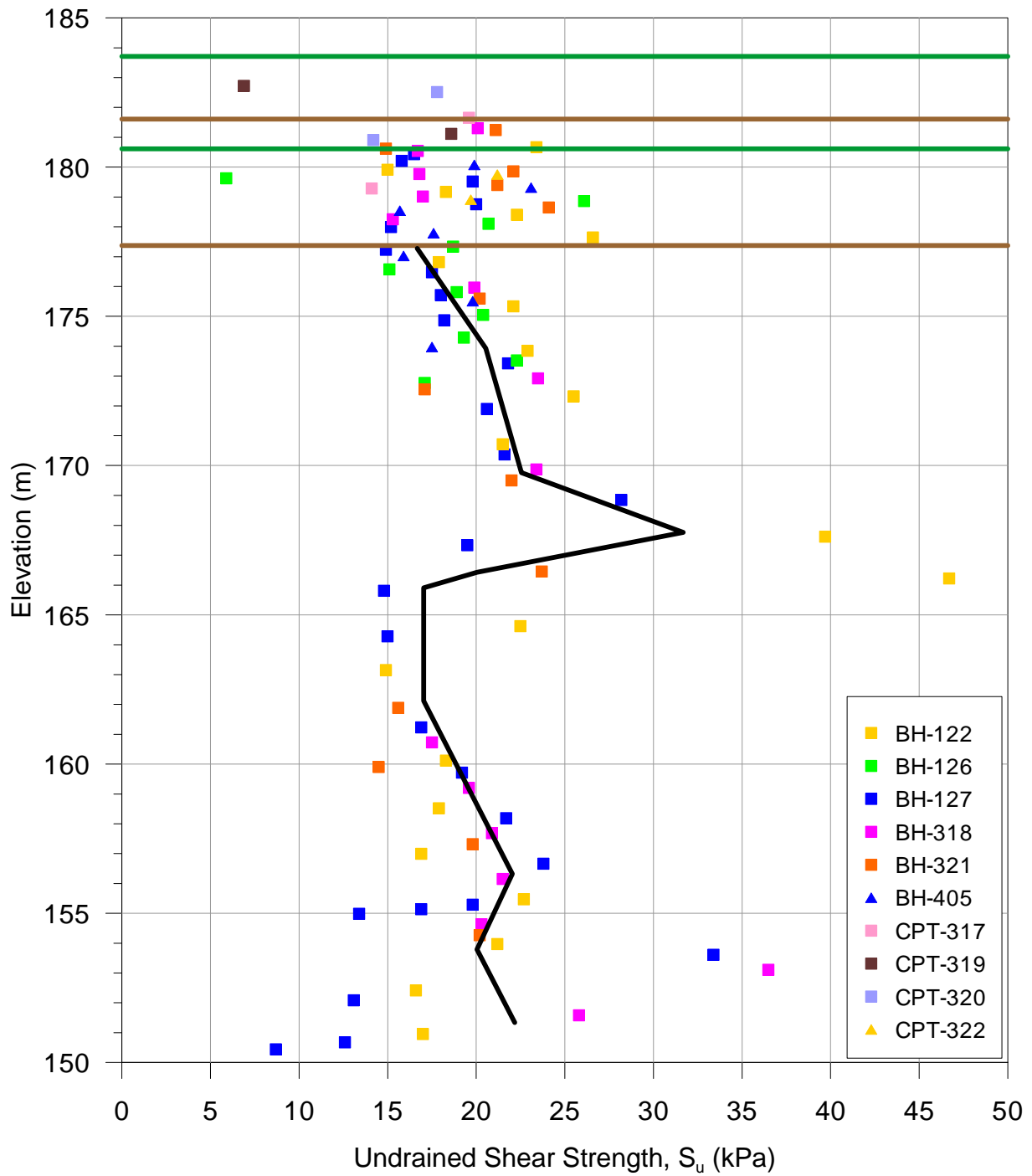
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 13+900 TO 10+300**



PROJECT No.	0711302070	FILE No.	0711302070R0257G
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7G			



NOTES

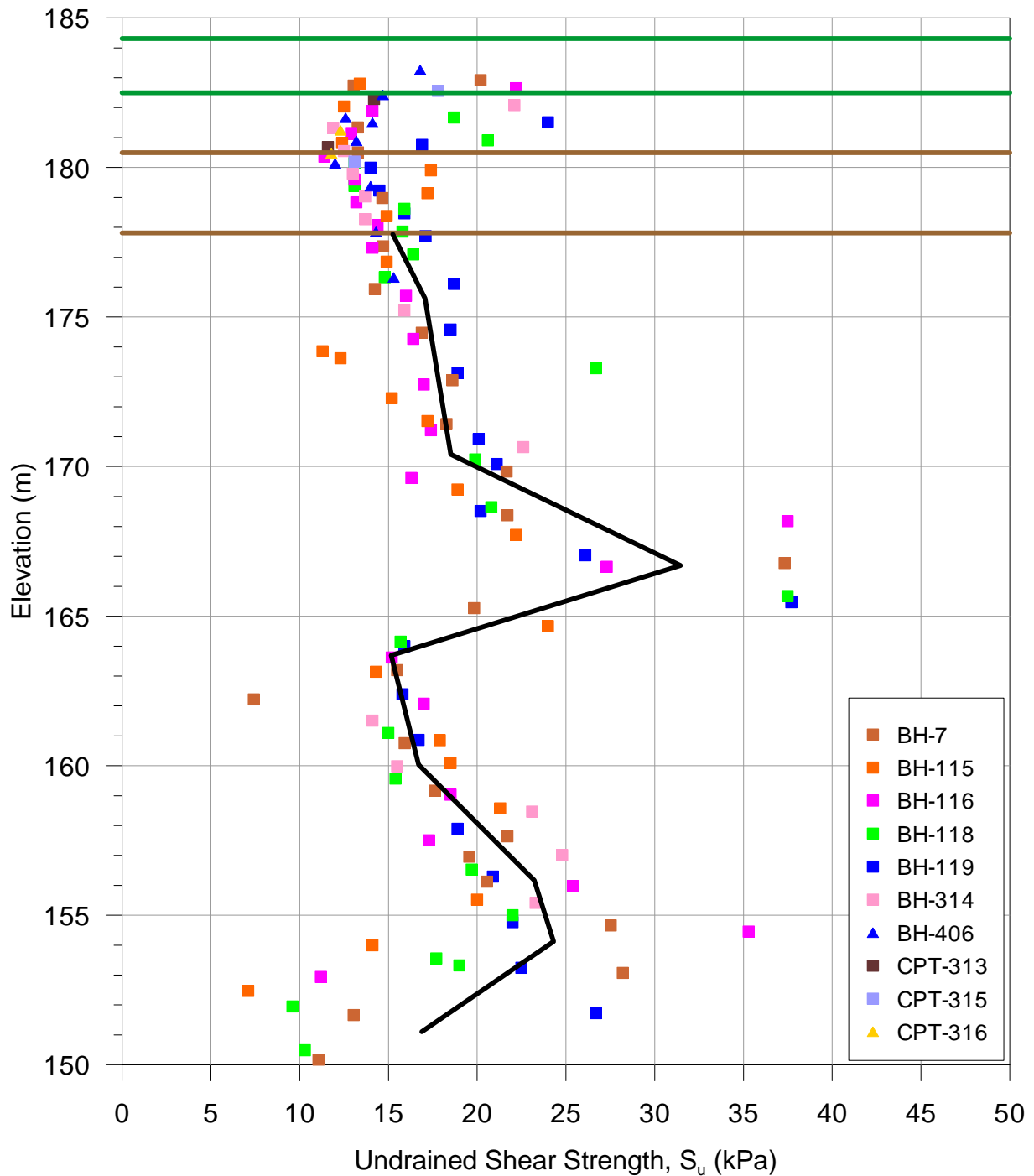
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 10+300 TO 11+300**



PROJECT No.	0711302070	FILE No.	0711302070R0257H
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
			FIGURE 5.7H



— RANGE OF INTERPRETED 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

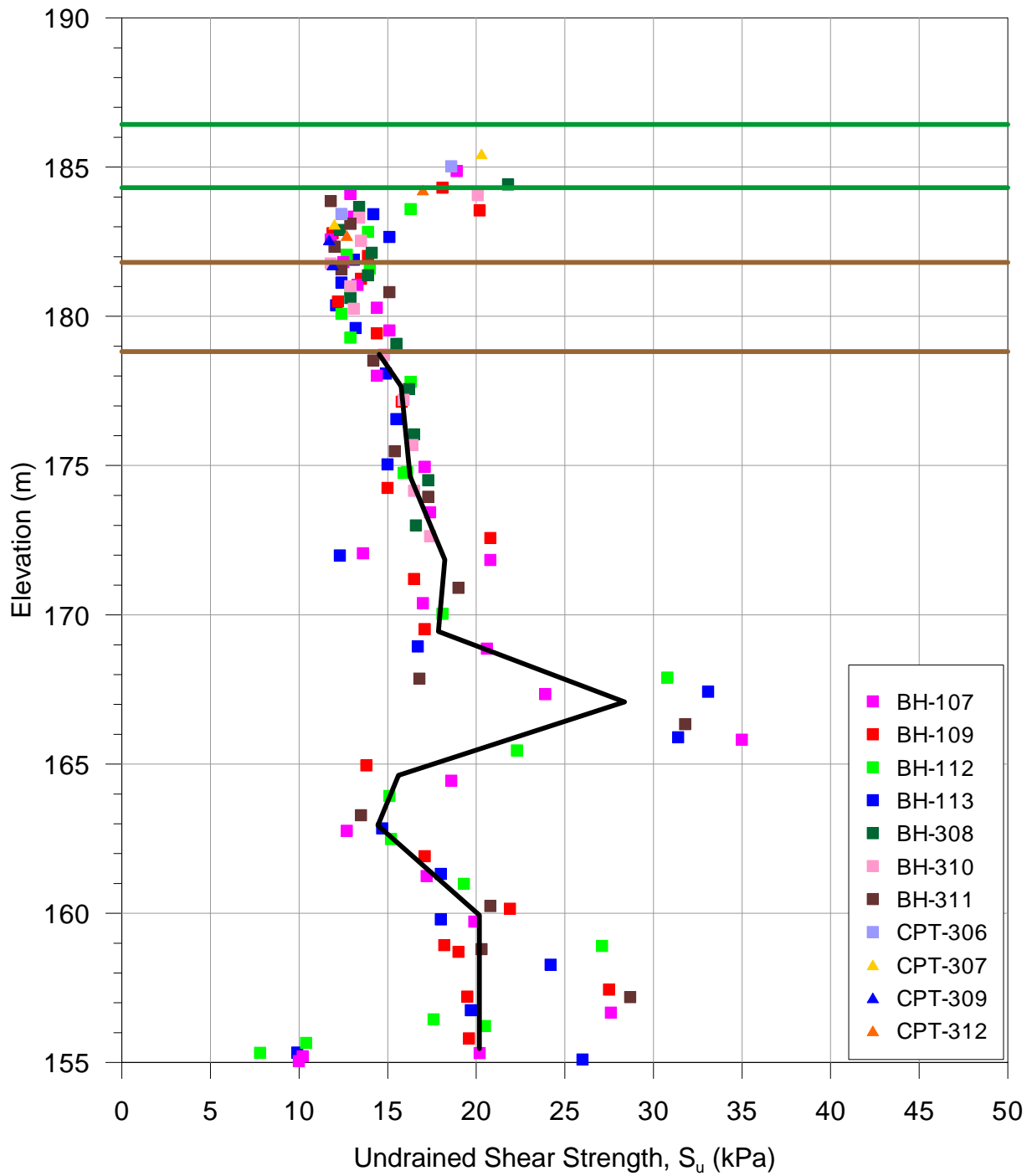
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 11+300 TO 12+600**



PROJECT No.	0711302070	FILE No.	0711302070R02571
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.71			



— RANGE OF INTERPRETED 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

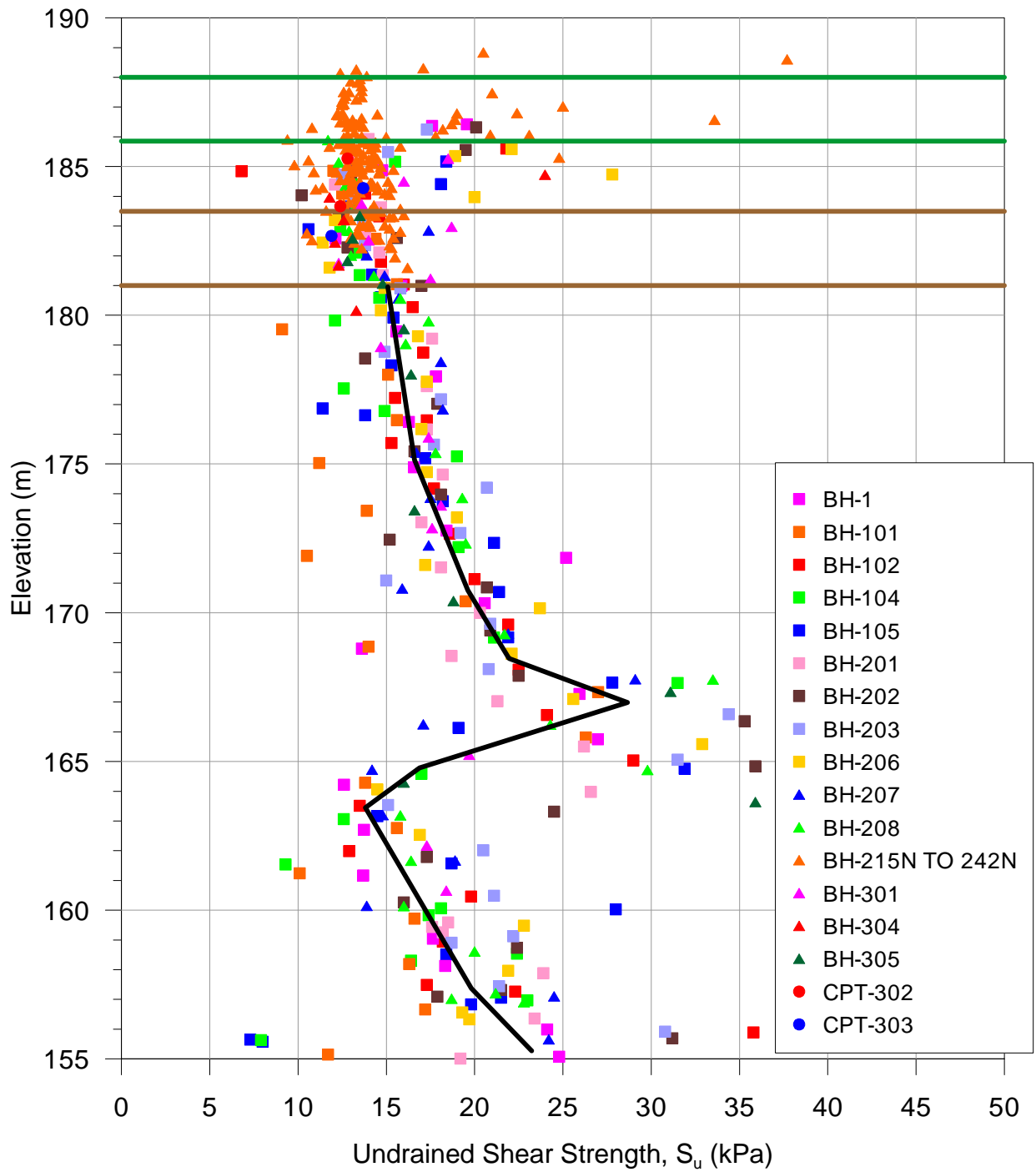
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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

TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 12+600 TO 13+600**



PROJECT No.	0711302070	FILE No.	0711302070R0257J
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7J			



NOTES

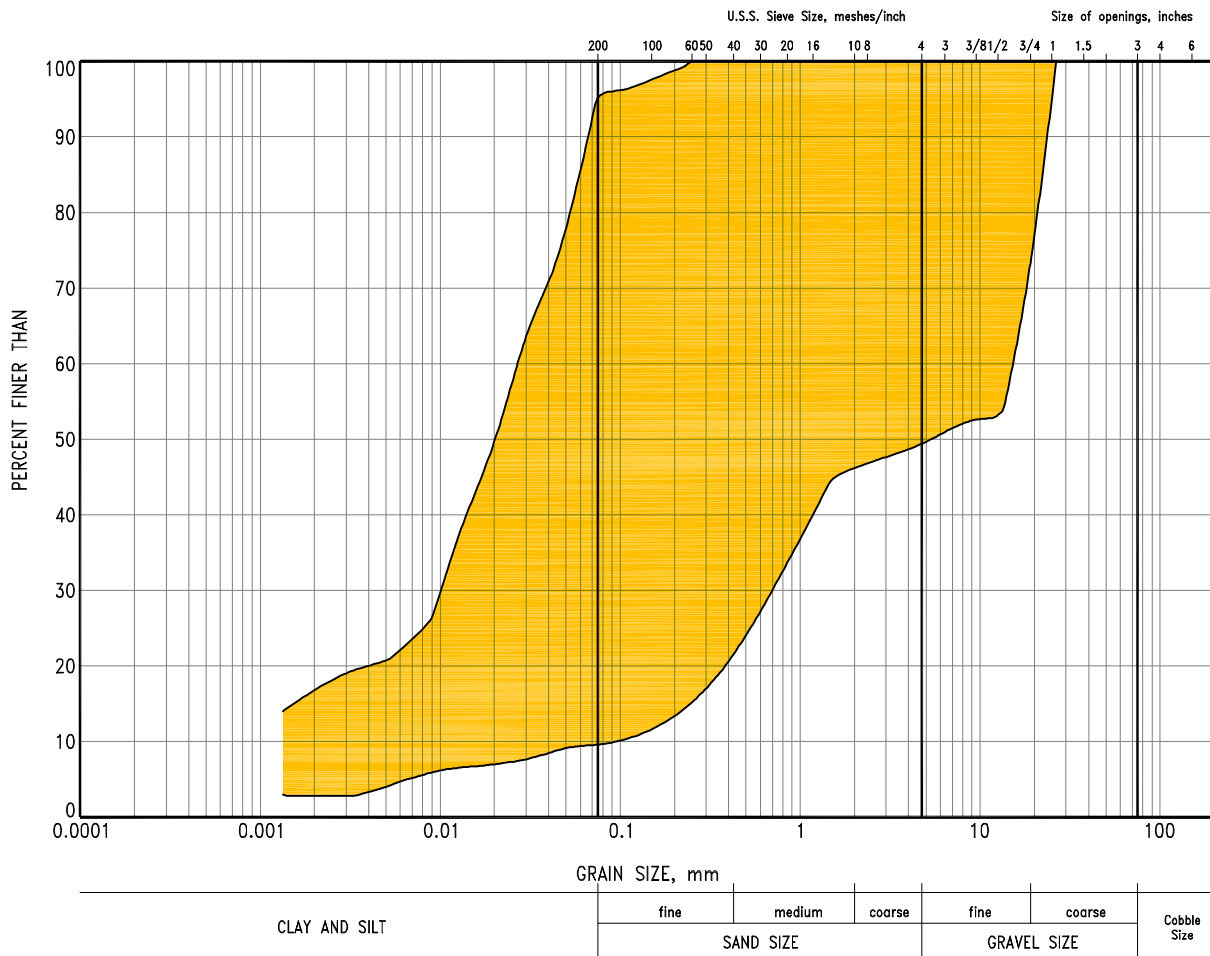
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY WATER CONTENT DETERMINATION AND THE INTERPRETED WATER CONTENT PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. 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 AND MINUS A WATER CONTENT OF 5%, RESPECTIVELY.

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


TITLE
**INTERPRETED WATER CONTENT PROFILE
STATION 13+600 TO 10+900**



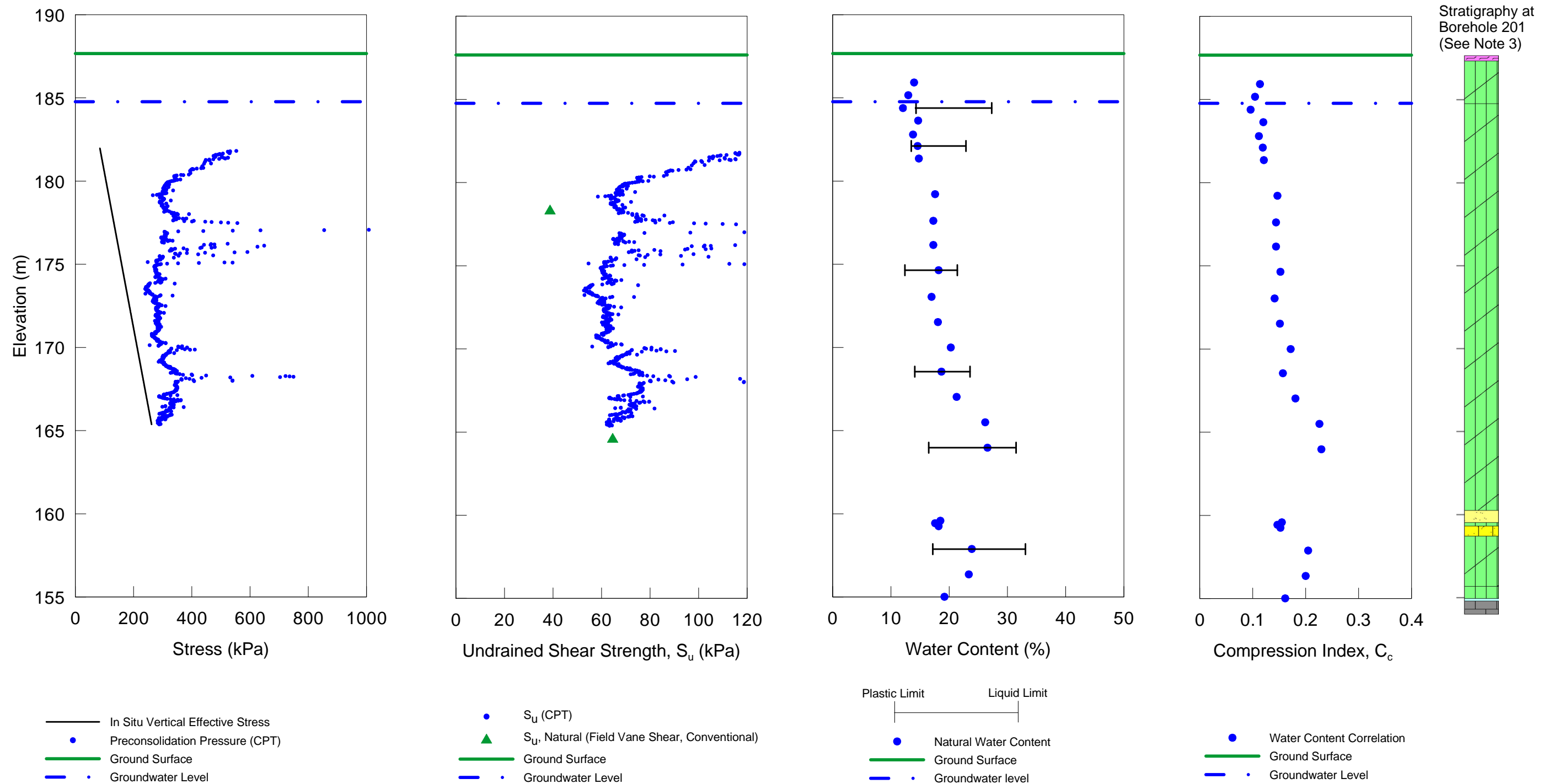
PROJECT No.	0711302070	FILE No.	0711302070R0257K
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 5.7K			



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 INTERPRETATION REPORT			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
GRAIN SIZE DISTRIBUTION ENVELOPE			
LOWER GRANULAR DEPOSITS			
PROJECT No. 07-1130-207-0		FILE No. 0711302070-R02058	
CADD	LMK/DCH	Dec. 11/10	SCALE AS SHOWN
CHECK			REV. 2
 Golder Associates LONDON, ONTARIO			FIGURE 5.8

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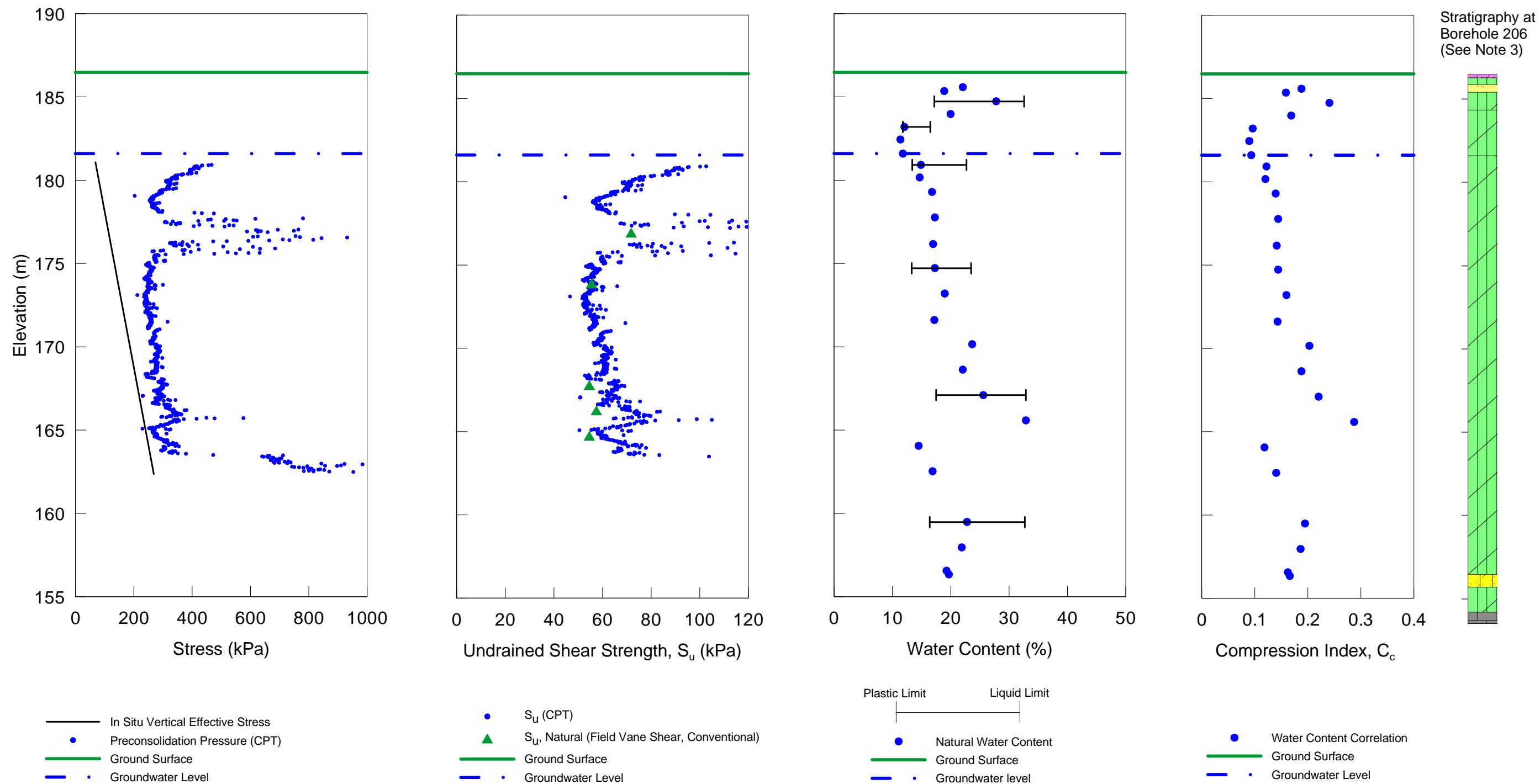


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-201/CPT-201			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0251A
			SCALE AS SHOWN
	DRAWN MK	Nov. 17/10	REV. 2
	CHECK		
			Figure 6.1A

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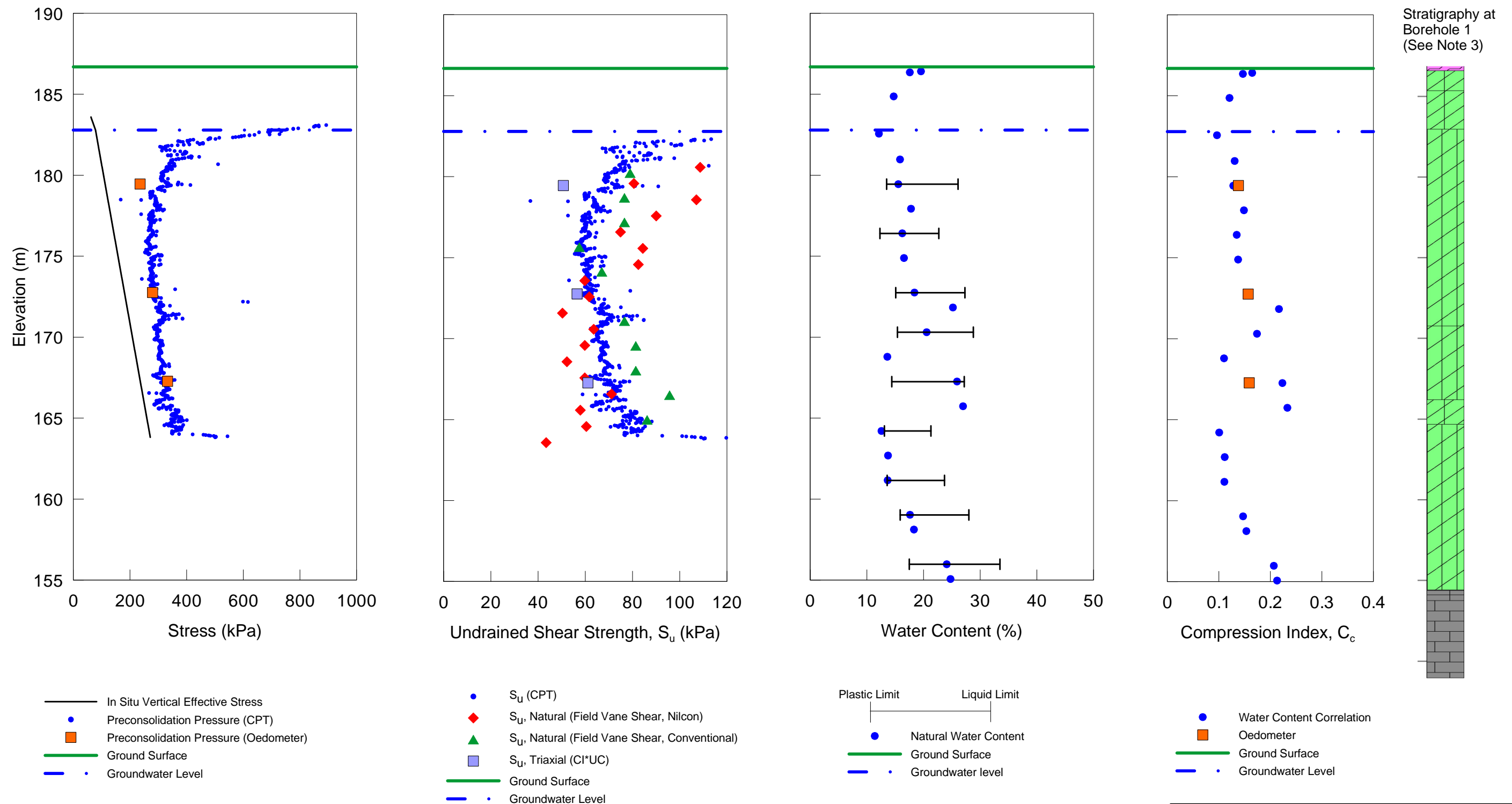


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-206/CPT-206			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261B
	DRAWN MK Nov. 17/10		SCALE AS SHOWN
	CHECK		REV. 2
	Figure 6.1B		

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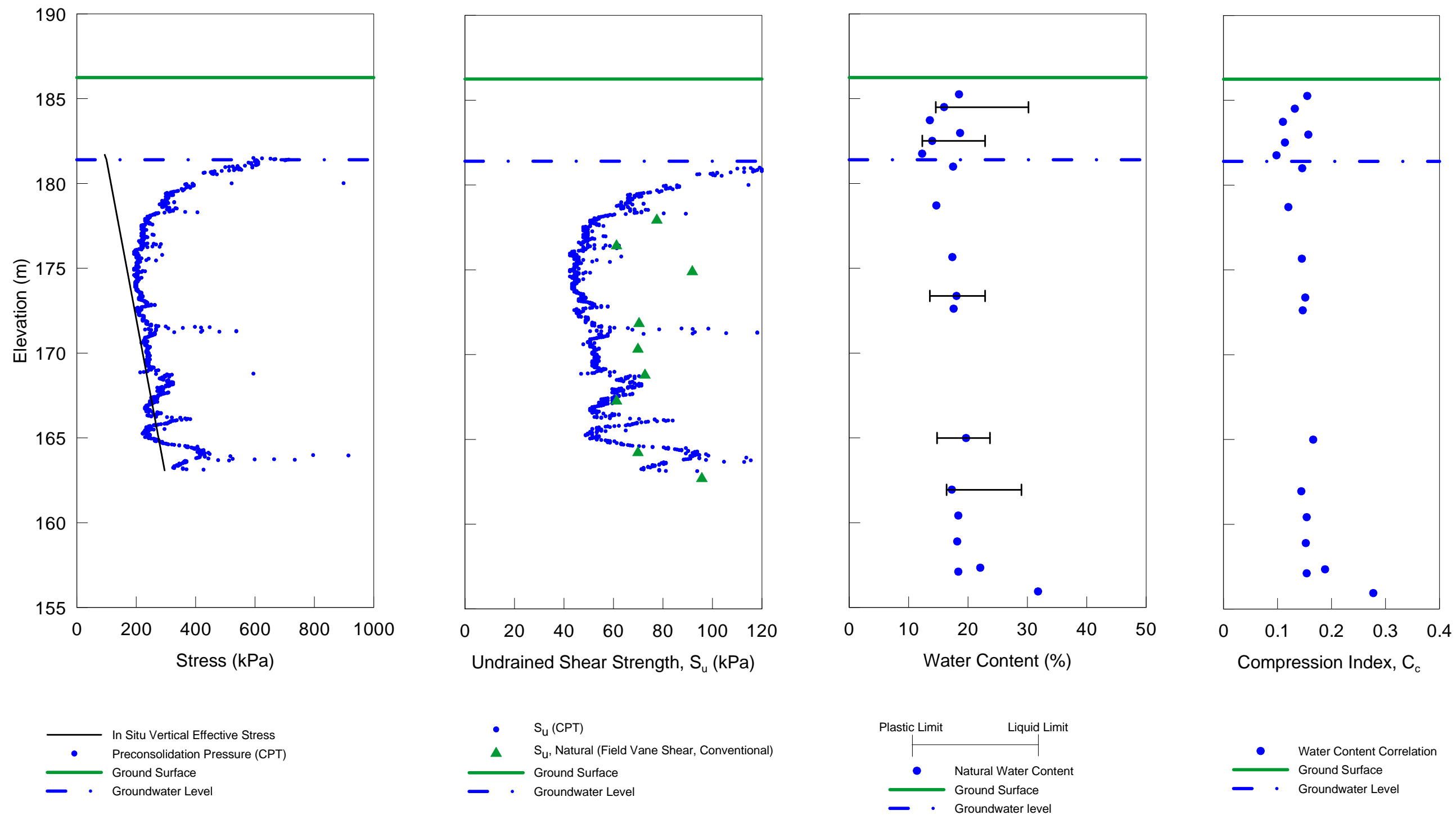


NOTES

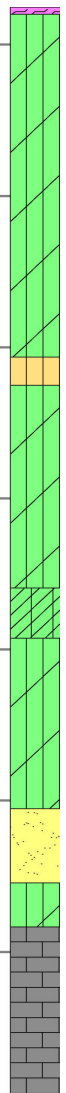
1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-1/CPT-1			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261C
			SCALE AS SHOWN
	DRAWN MK	Nov. 17/10	REV. 2
	CHECK		
			Figure 6.1C

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Stratigraphy at
Borehole 301
(See Note 3)

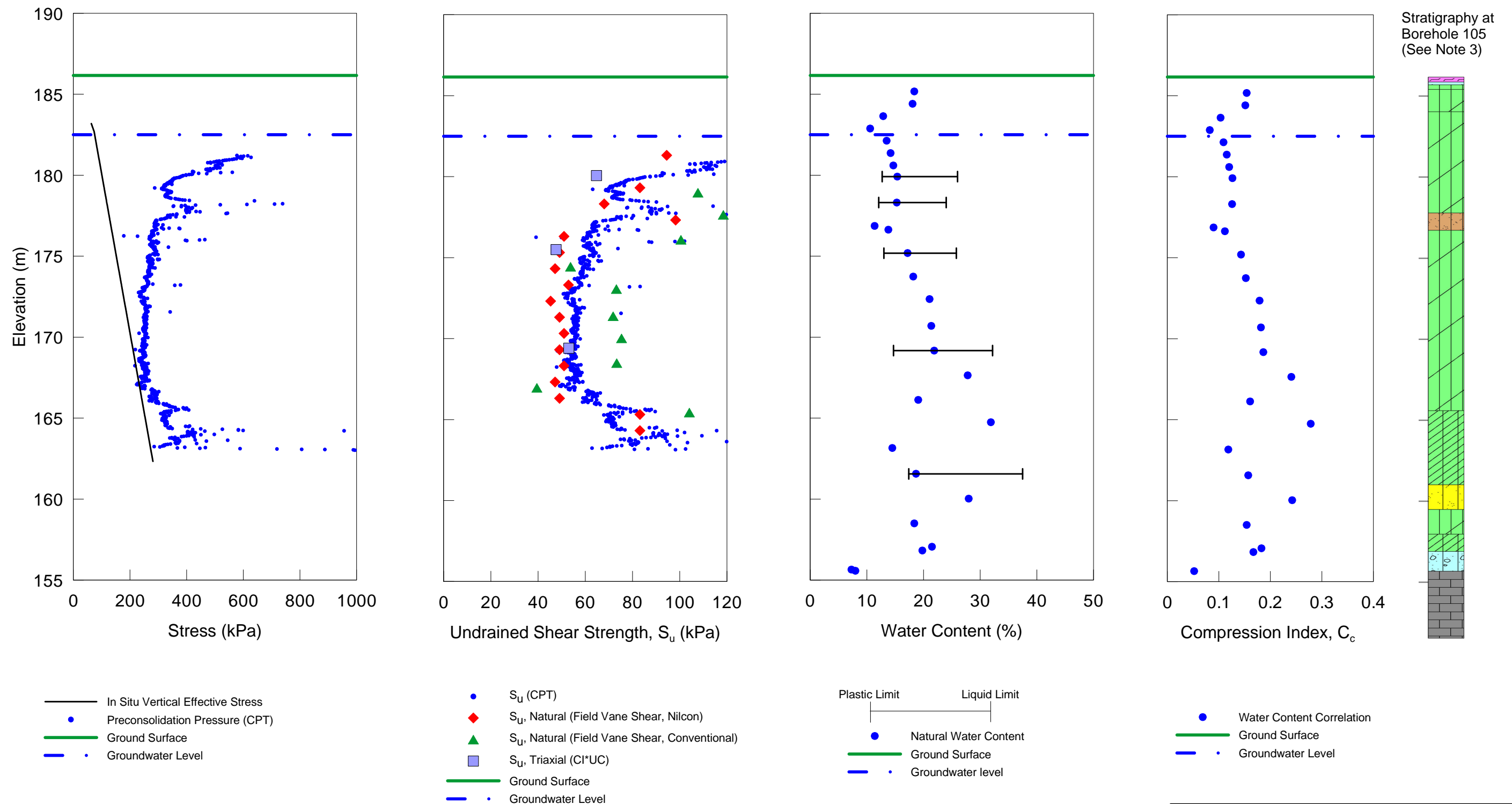


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-301/CPT-302			
	PROJECT No.	07-1130-207-0	FILE No. 0711302070R0261D
	DRAWN	MK	Nov. 17/10
	CHECK		
			SCALE AS SHOWN REV. 2
Figure 6.1D			

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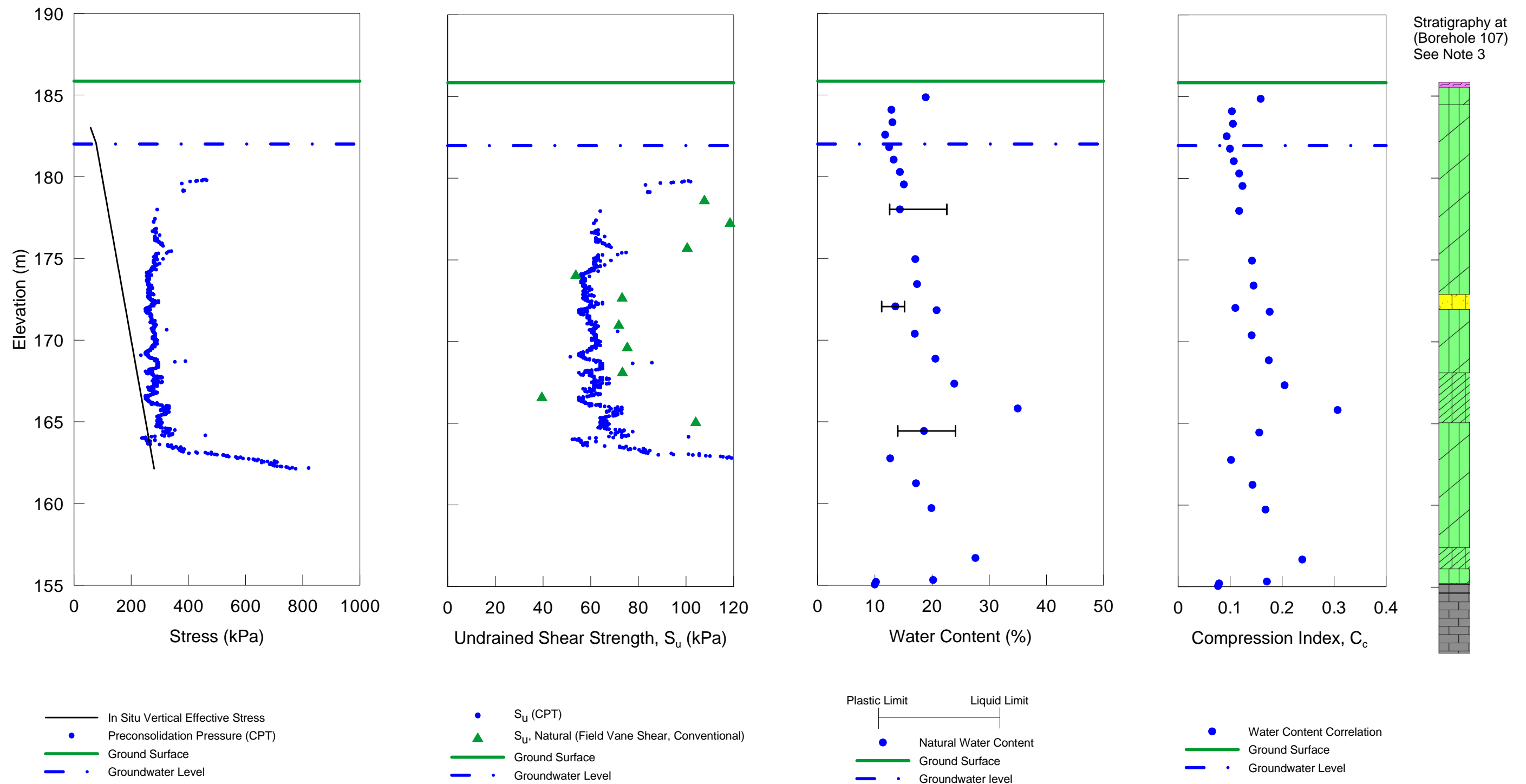


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-105/CPT-2			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0281E
	DRAWN MK		SCALE AS SHOWN
CHECK		Nov. 17/10	REV. 2
Figure 6.1E			

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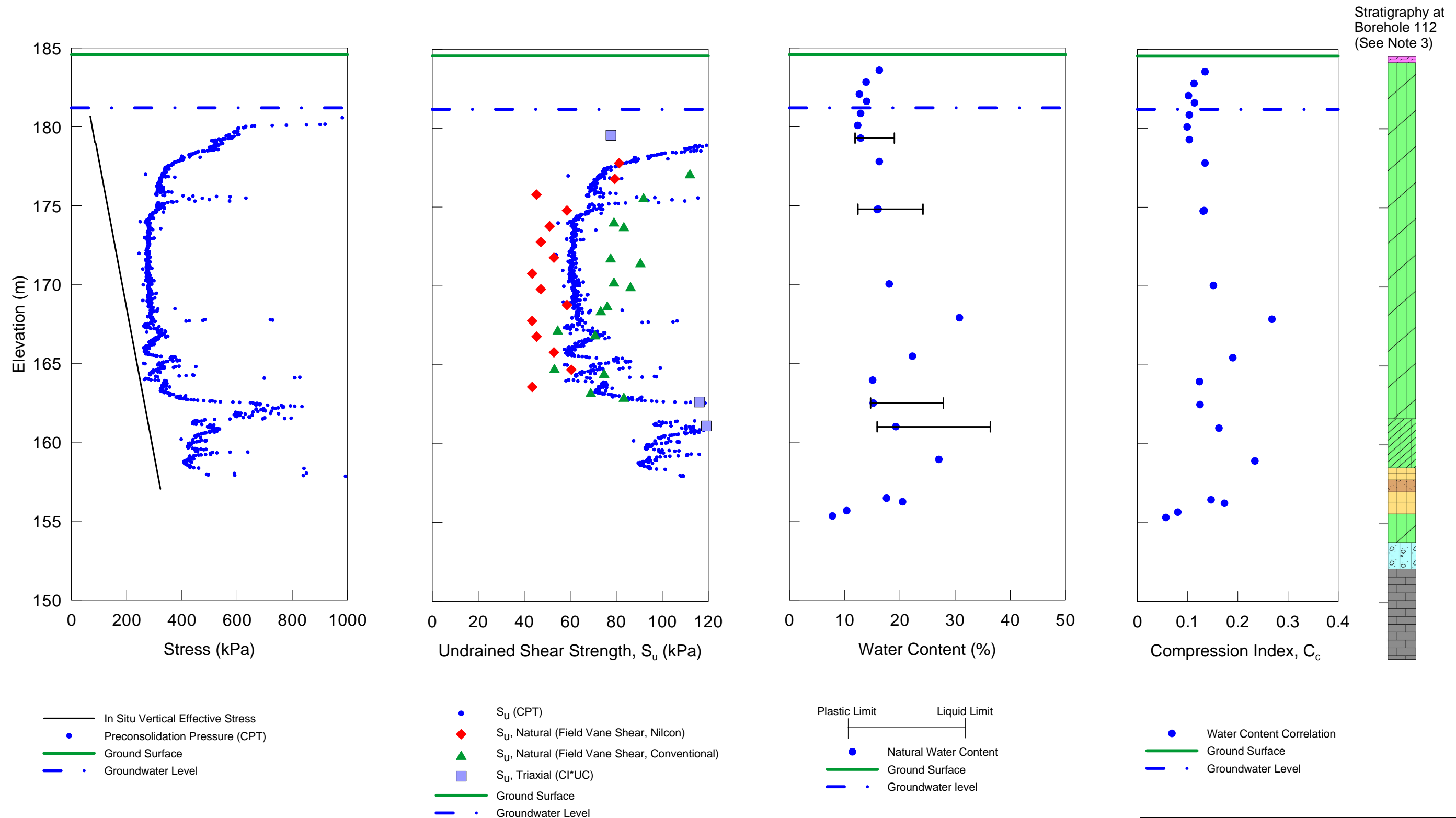


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-107/CPT-3			
	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261F
	DRAWN MK		Nov. 17/10
CHECK			
SCALE AS SHOWN			REV. 2
Figure 6.1F			

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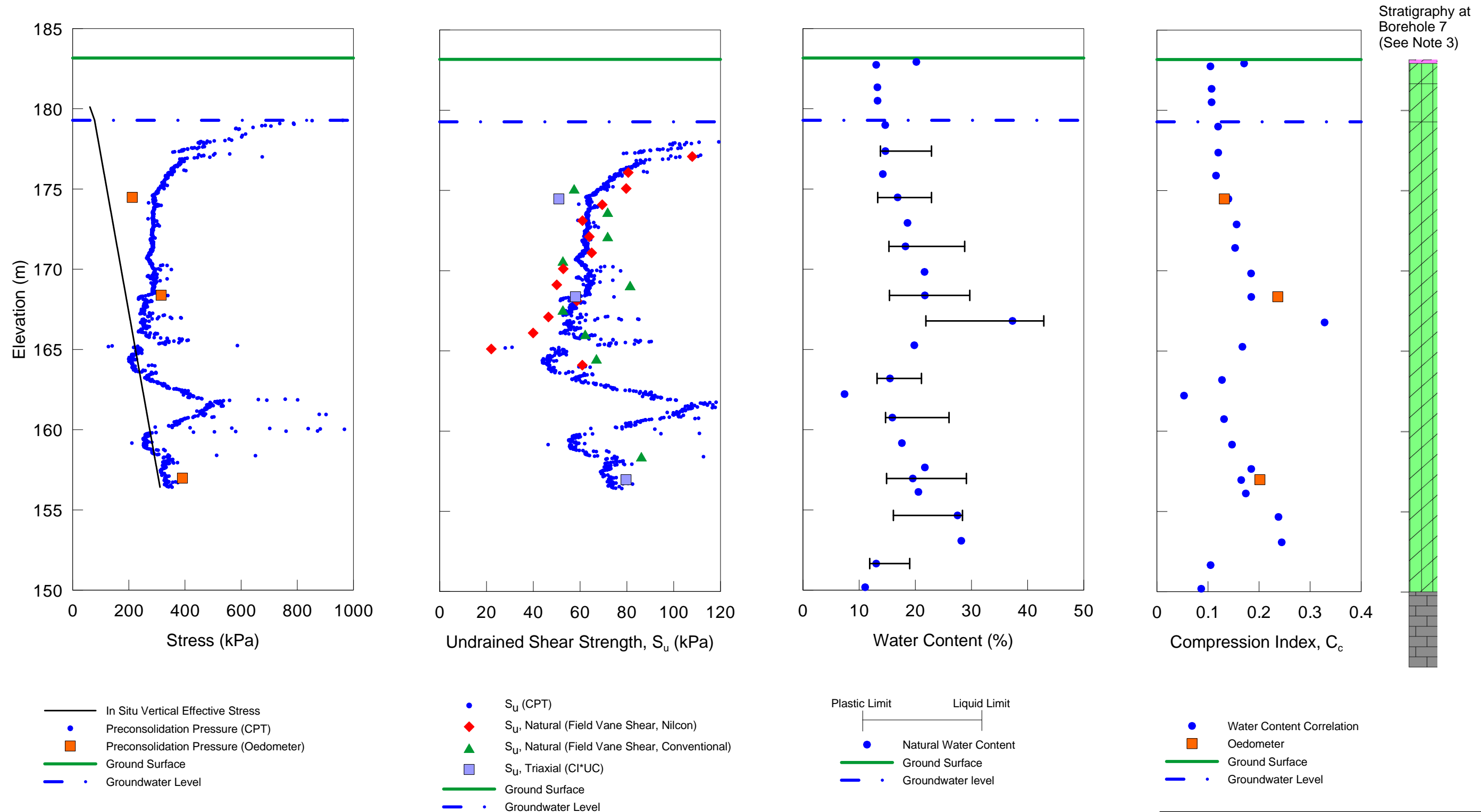


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO				
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-112/CPT-6				
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261G	
			SCALE AS SHOWN	
	DRAWN MK Nov. 17/10		REV. 2	
	CHECK			
Figure 6.1G				

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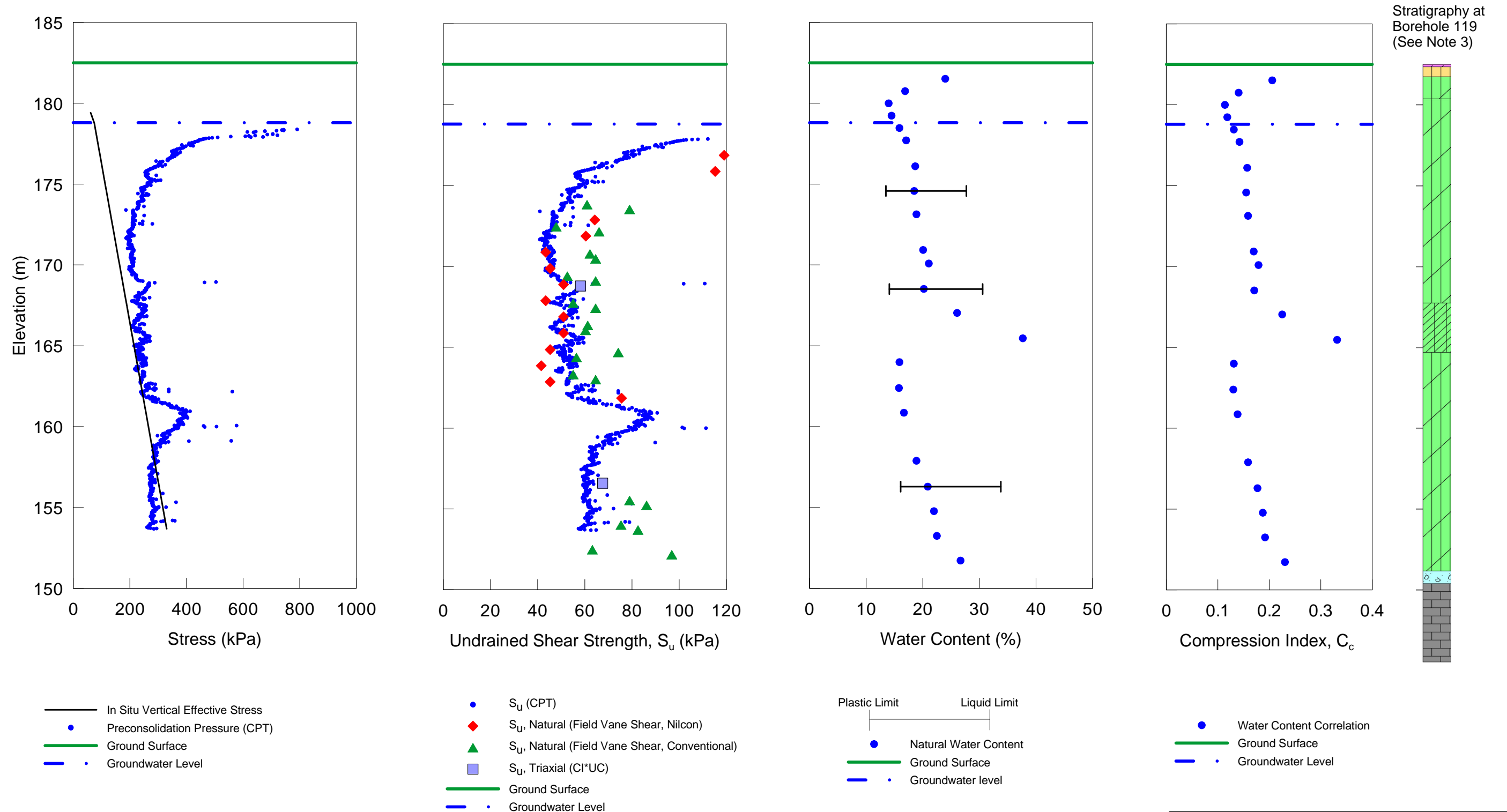


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-7/CPT-7			
	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261H
	DRAWN MK		Nov. 17/10
CHECK			SCALE AS SHOWN
			REV. 2
Figure 6.1H			

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NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

PROJECT

SUBSURFACE CONDITIONS INTERPRETATION REPORT


WINDSOR-ESSEX PARKWAY

WINDSOR, ONTARIO

TITLE

SUMMARY OF SUBSURFACE TEST DATA

BOREHOLE BH-119/CPT-8

Golder Associates
LONDON, ONTARIO

PROJECT No. 07-1130-207-0

FILE No. 0711302070R02611

DRAWN MK Nov. 17/10

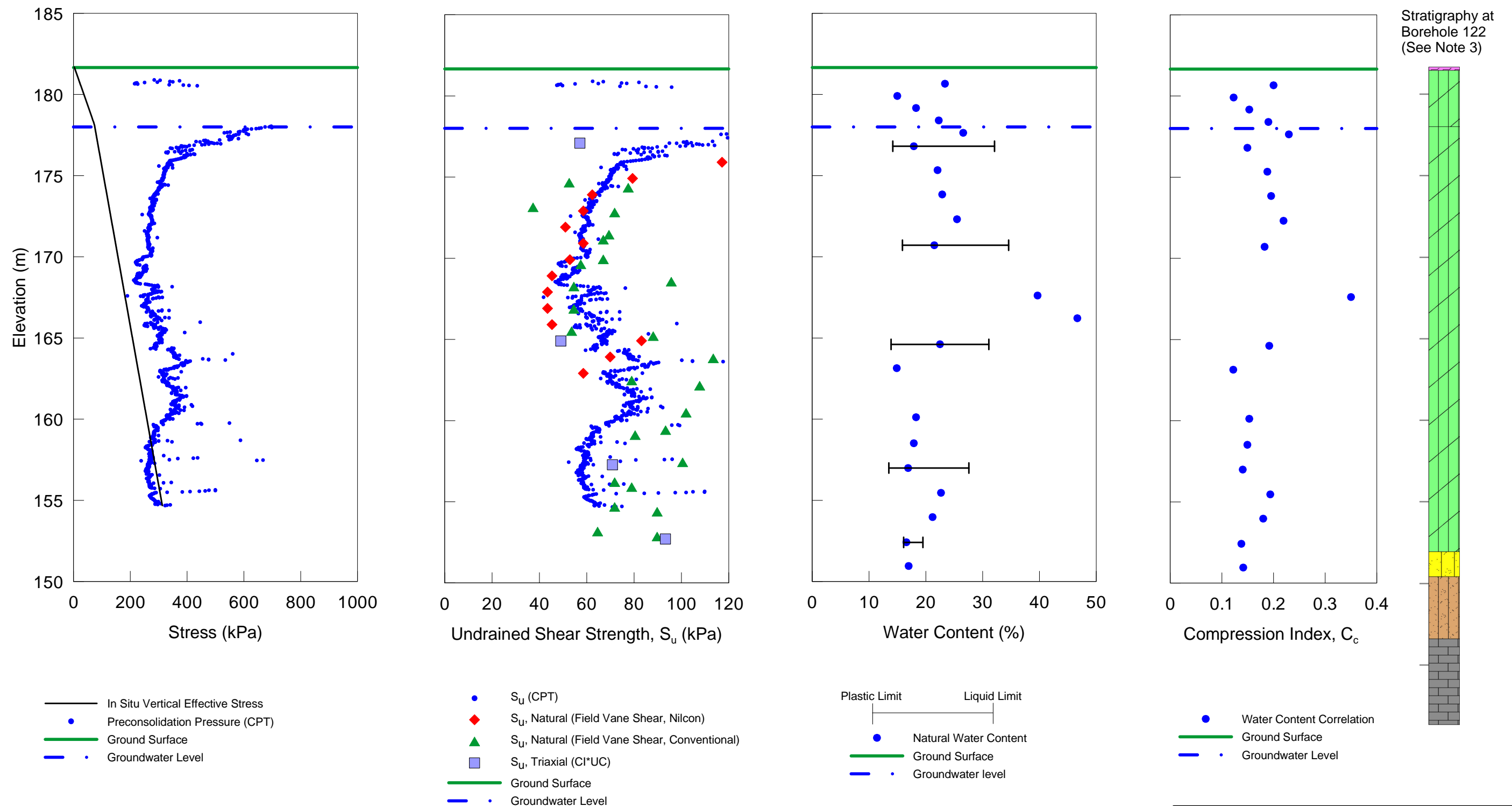
CHECK

SCALE AS SHOWN

REV. 2


Figure 6.11

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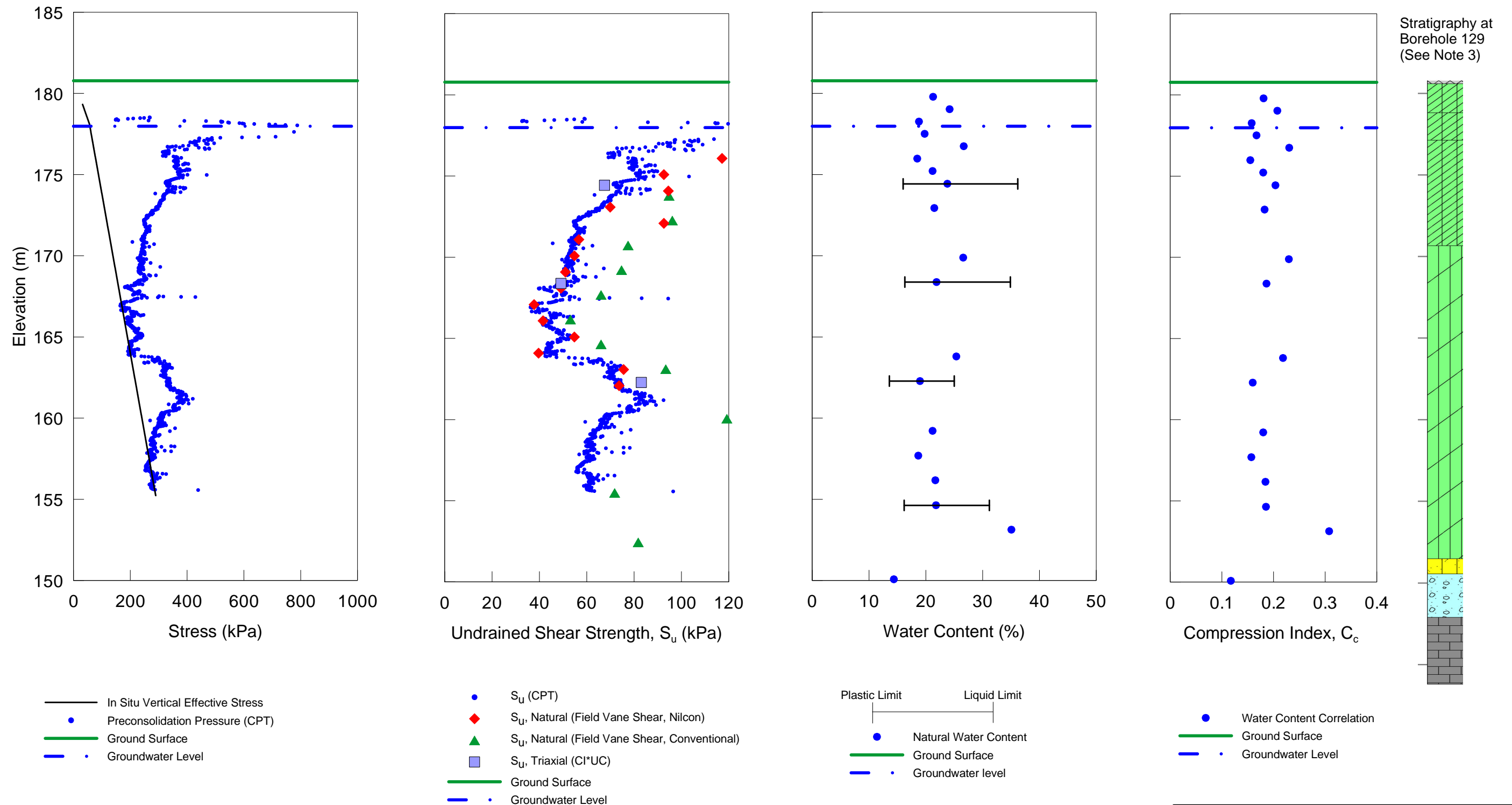


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-122/CPT-10			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 071130207GR0261J
			SCALE AS SHOWN
	DRAWN MK Nov. 17/10		REV. 2
	CHECK		
			Figure 6.1J

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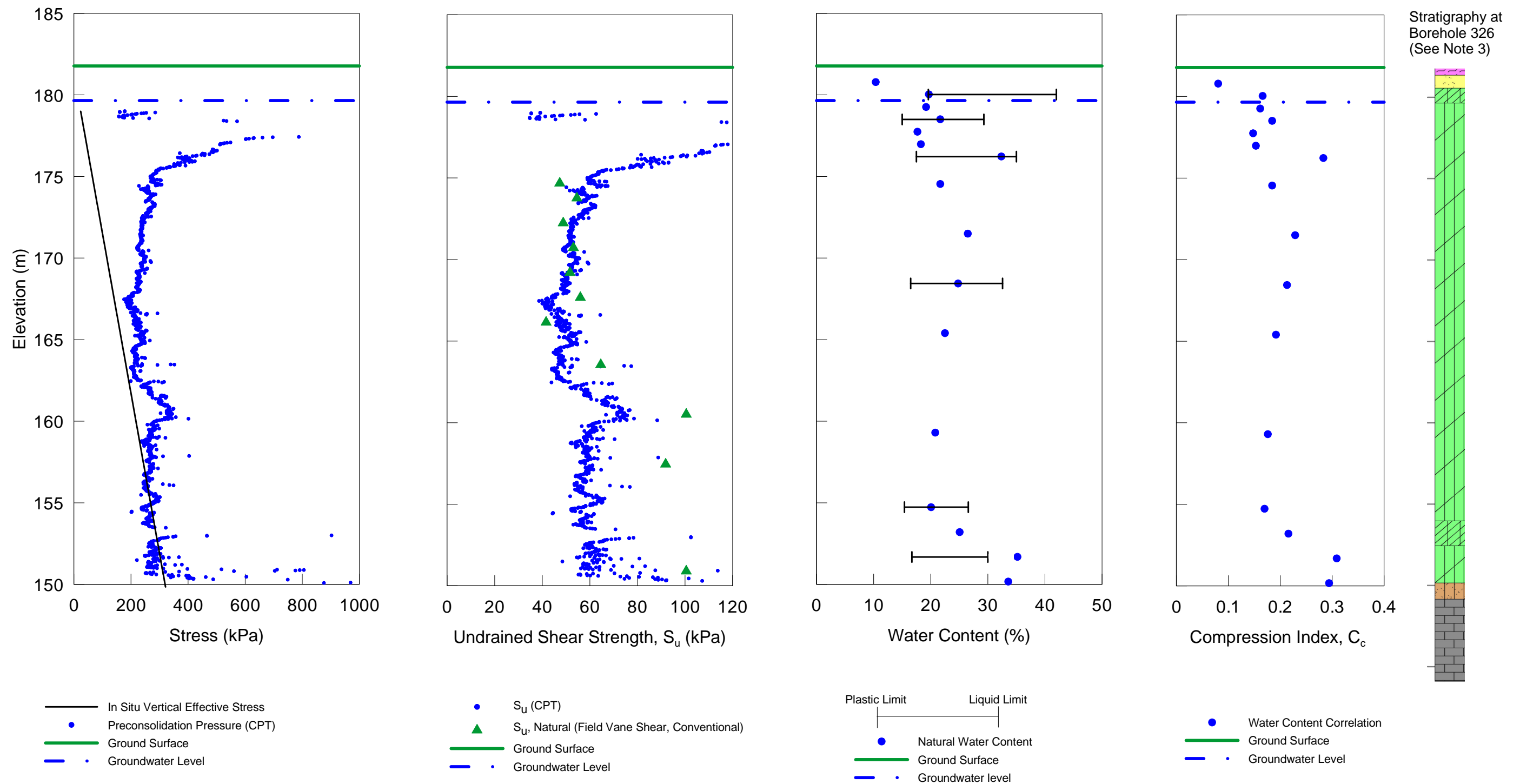


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO				
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-129/CPT-11				
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261K	
			SCALE AS SHOWN	
	DRAWN MK Nov. 17/10		REV. 2	
	CHECK			
Figure 6.1K				

N:\active\2007\1130 - Geotechnical\1130-2007-1130-207-0 - URS - DRIC APPROACH GSR - WINDSOR\Drawing\Grapher files\071130207060261L.grf

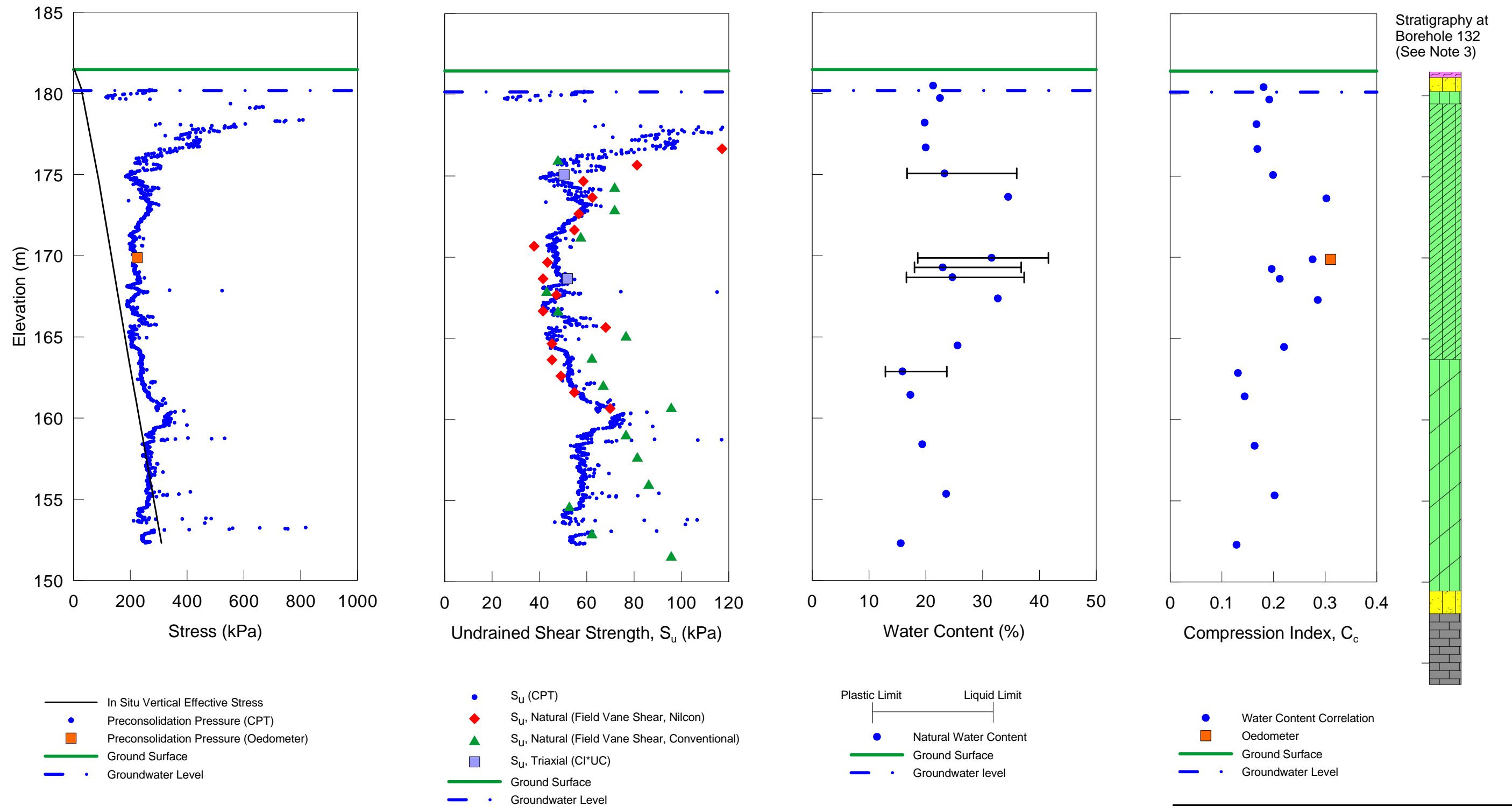


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-326/CPT-130			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261L
			SCALE AS SHOWN
	DRAWN MK Nov. 17/10		REV. 2
	CHECK		
			Figure 6.1L

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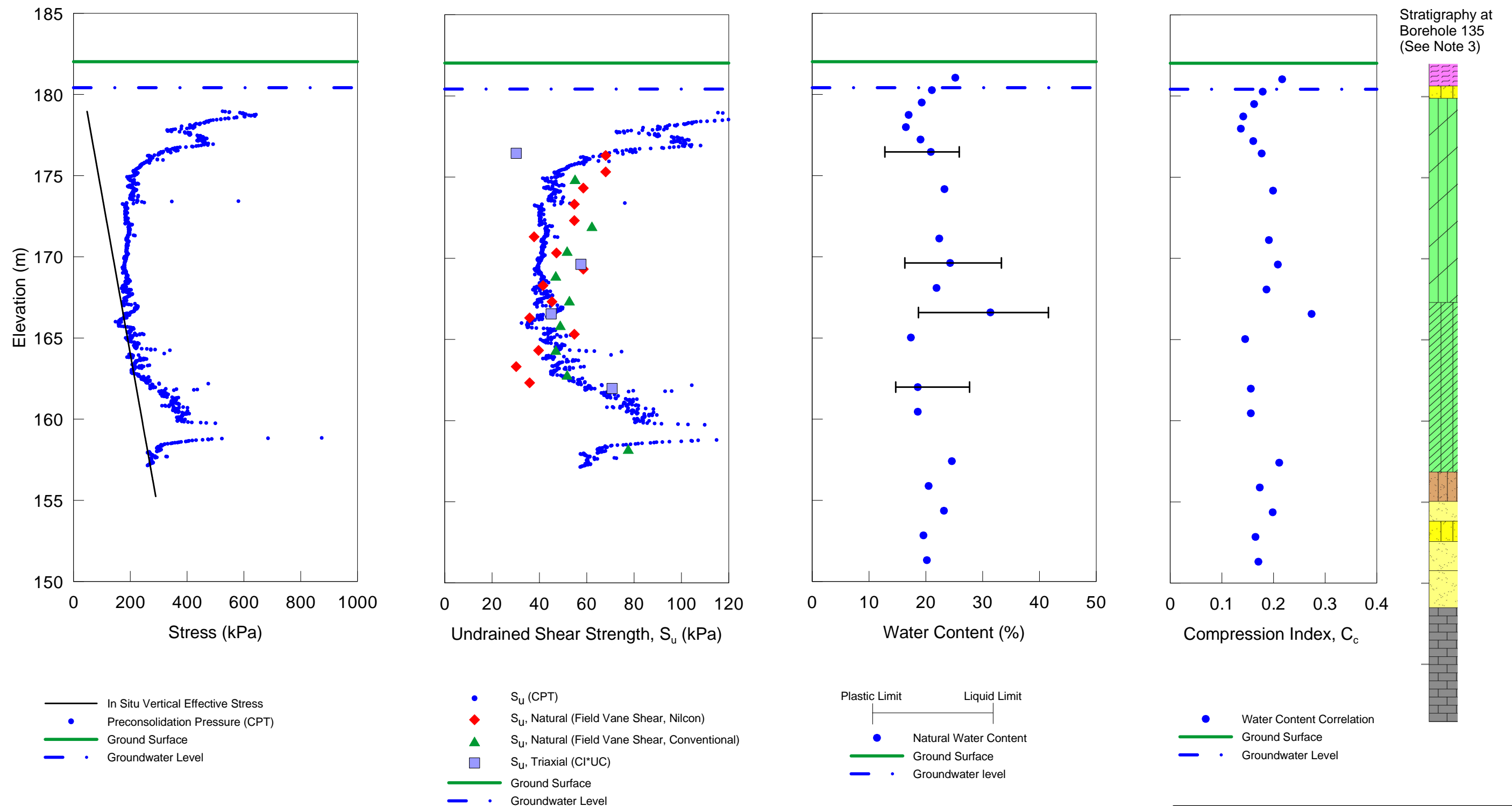


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO				
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-132/CPT-12				
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0251M	
			SCALE AS SHOWN	
	DRAWN MK Nov. 17/10		REV. 2	
	CHECK			
Figure 6.1M				

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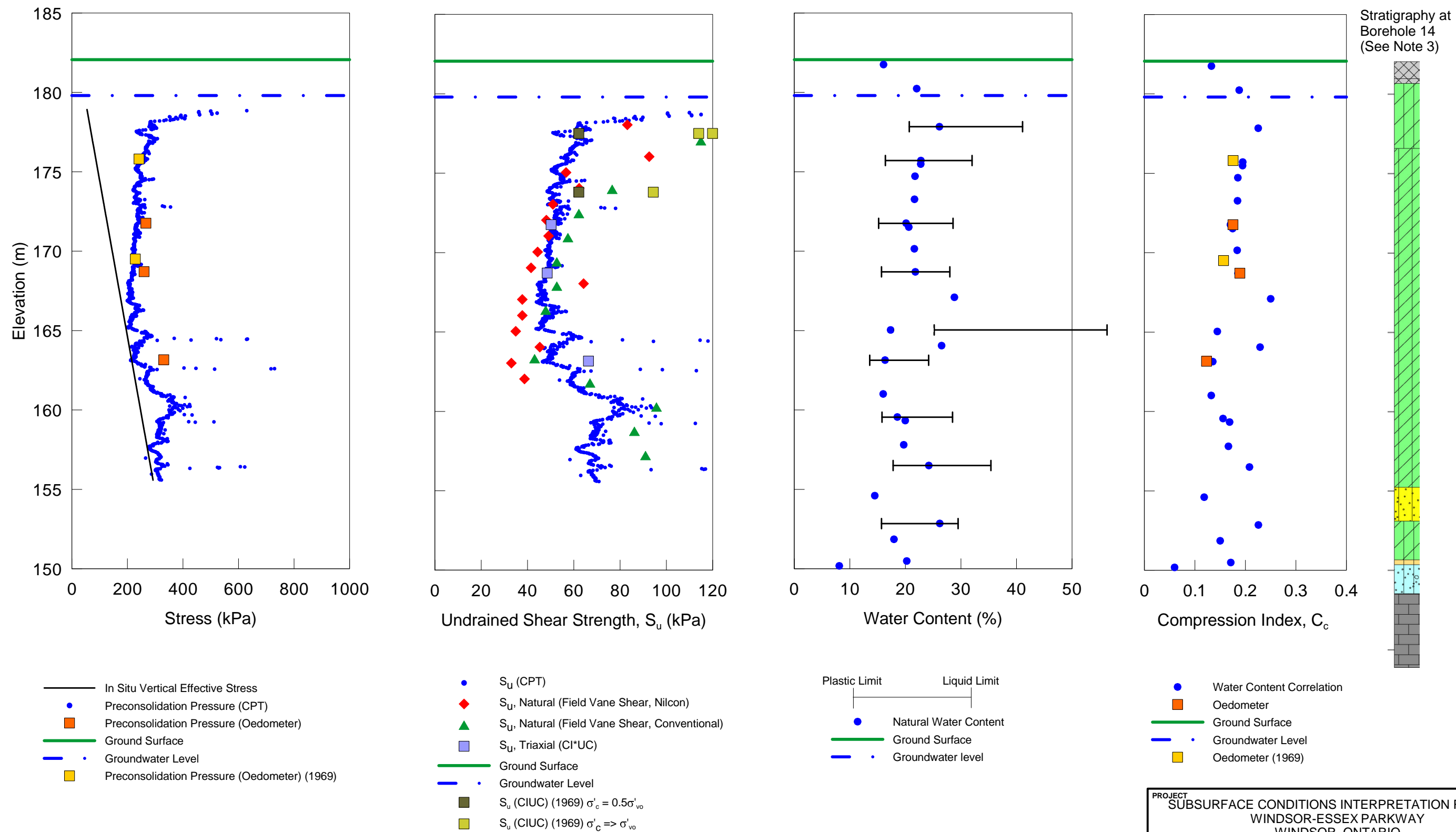


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-135/CPT-13			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0251N
			SCALE AS SHOWN
	DRAWN MK	Nov. 17/10	REV. 2
	CHECK		
			Figure 6.1N

N:\drive\2007\1130 - Geotechnical\1130-2007-1130-207-0 - URS - DRIC APPROACH GSR - WINDSOR\Drafting\Grapher files\0711302070602610.grf



NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


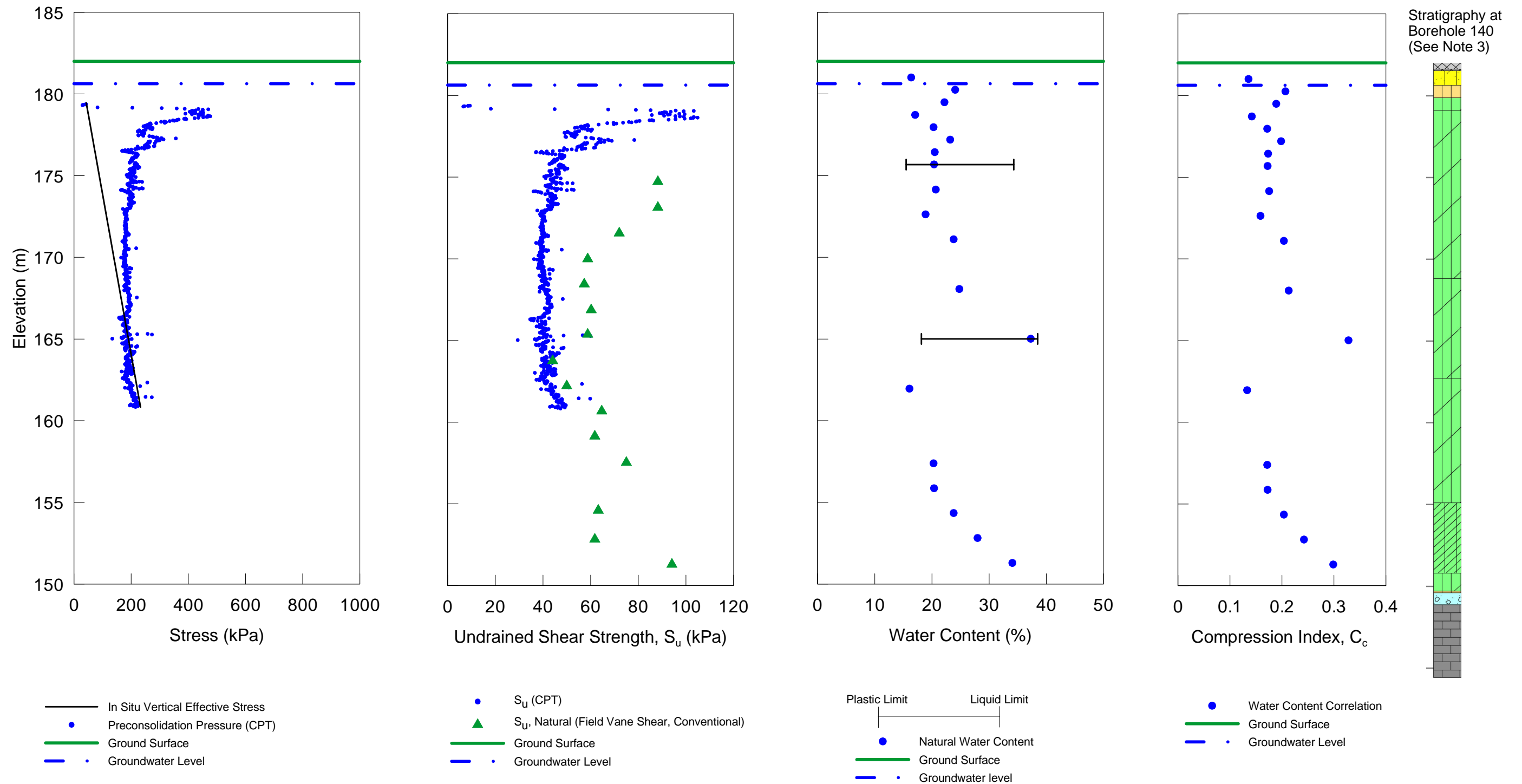
PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-14/CPT-14			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R02610
	DRAWN MK		SCALE AS SHOWN
CHECK		Nov. 17/10	REV. 2


Figure 6.10

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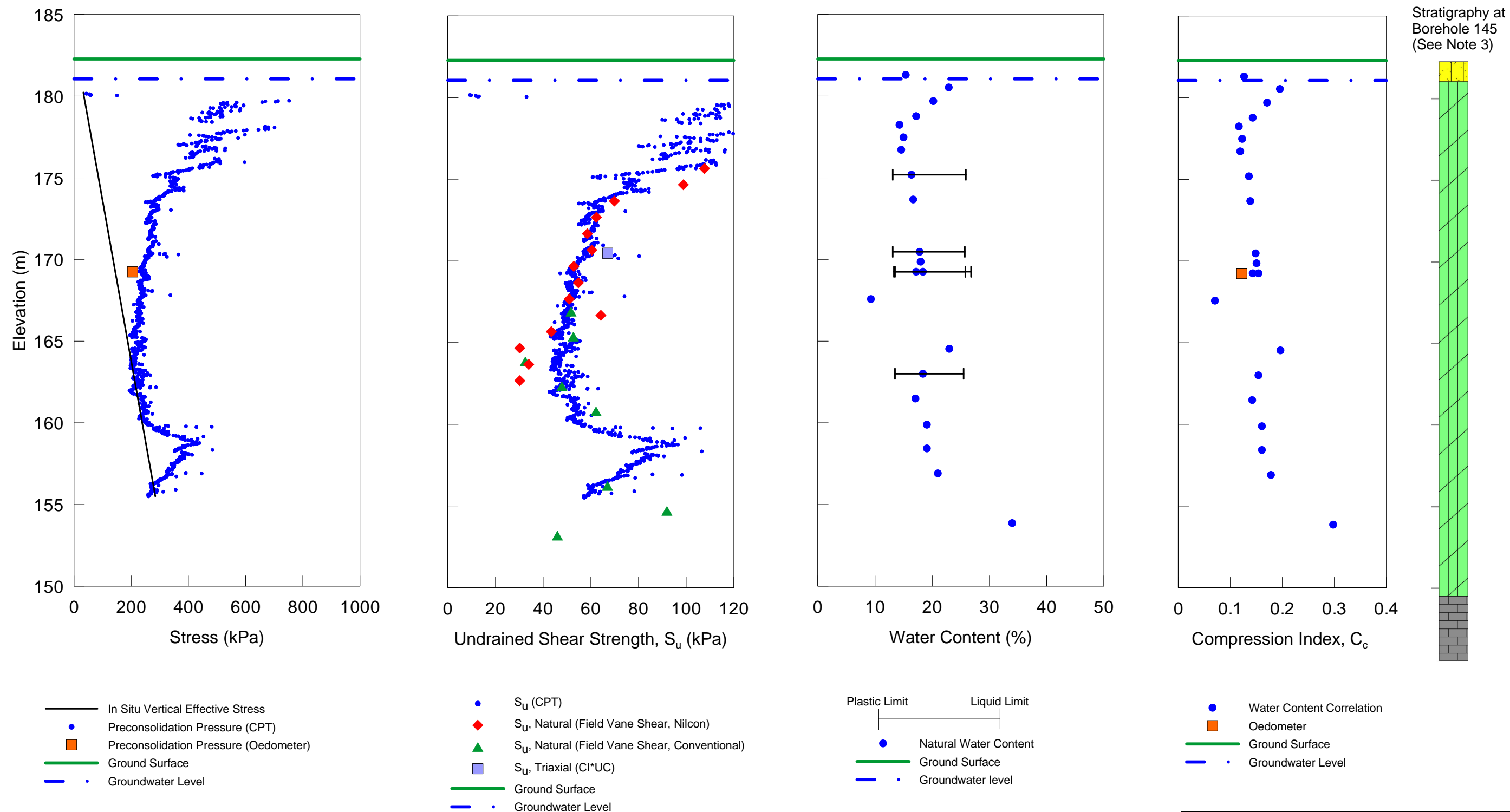


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-140/CPT-333			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0281P
			SCALE AS SHOWN
	DRAWN	MK	Nov. 17/10
	CHECK		
			Figure 6.1P

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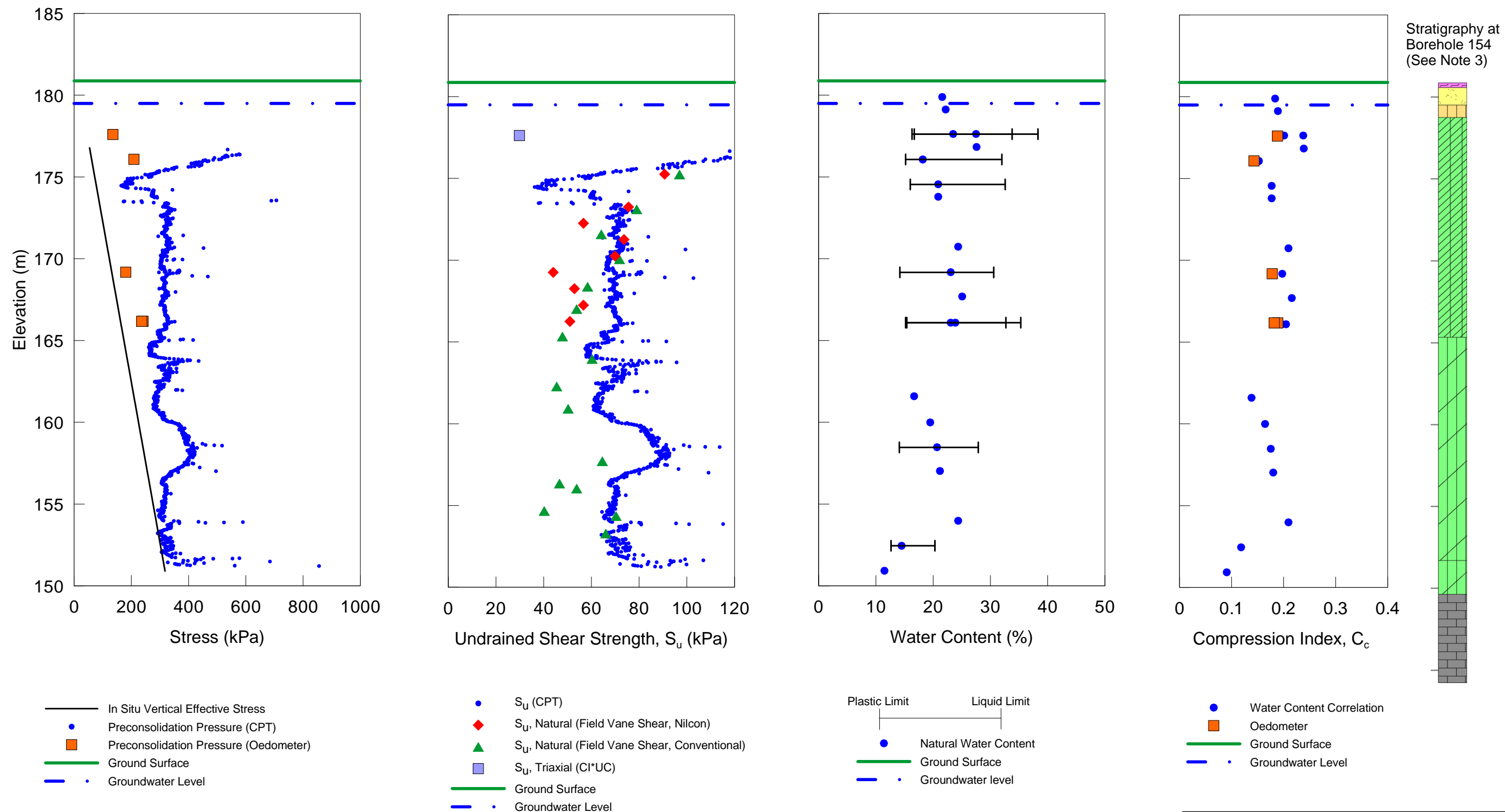


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-145/CPT-145			
	PROJECT No.	07-1130-207-0	FILE No. 0711302070R0261Q
	DRAWN	MK	Nov. 17/10
	CHECK		
			SCALE AS SHOWN REV. 2
Figure 6.1Q			

N:\drive\2007\1130 - Geotechnical\1130-2007-1130-207-0 - URS - DRIC APPROACH GSR - WINDSOR\Drawing\Grapher files\071130207060261R.grf

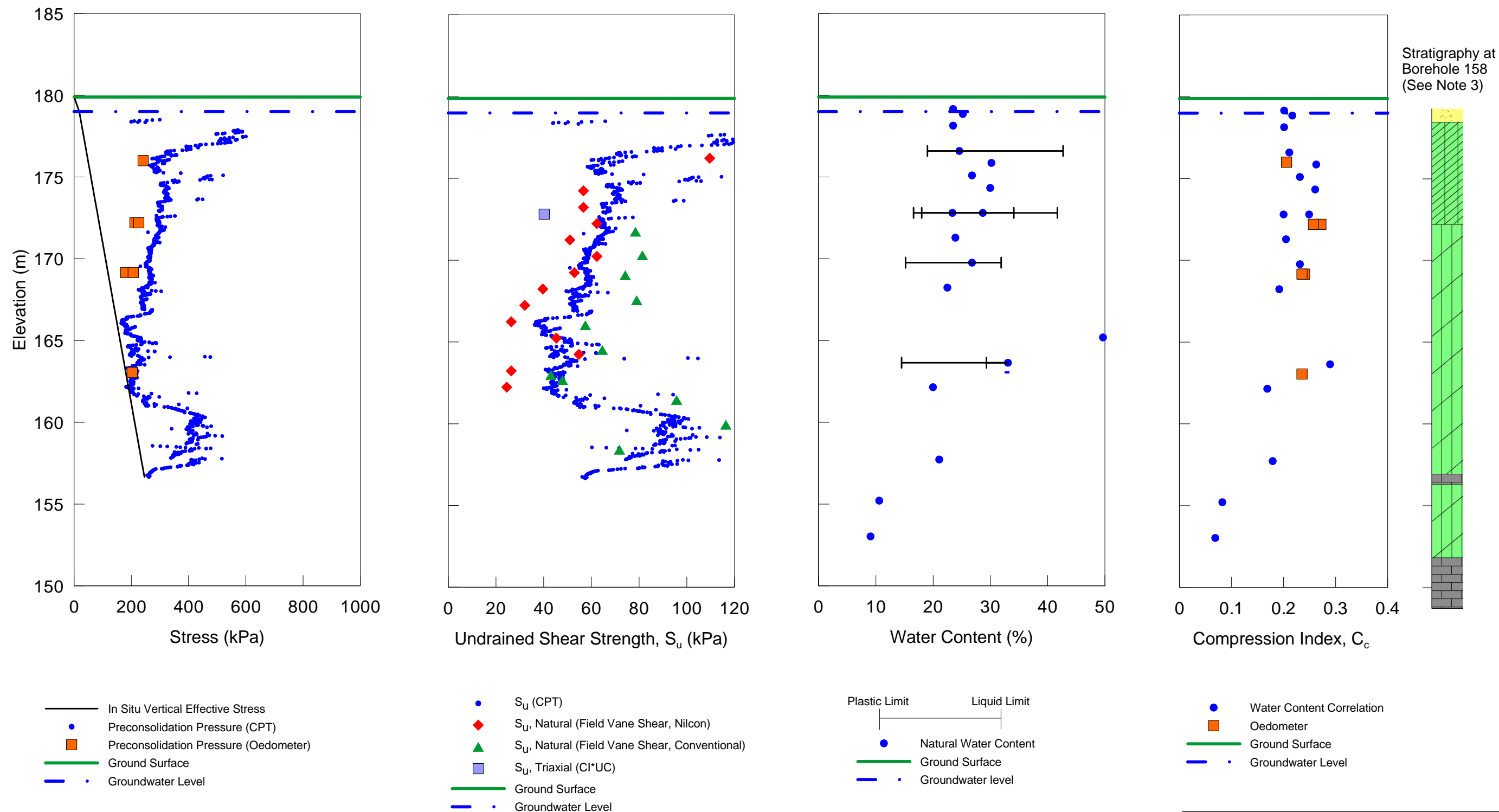


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-154/CPT-154			
	PROJECT No.	07-1130-207-0	FILE No. 0711302070R0261R
	DRAWN	MK	Nov. 17/10
CHECK			SCALE AS SHOWN
REV. 2			
Figure 6.1R			

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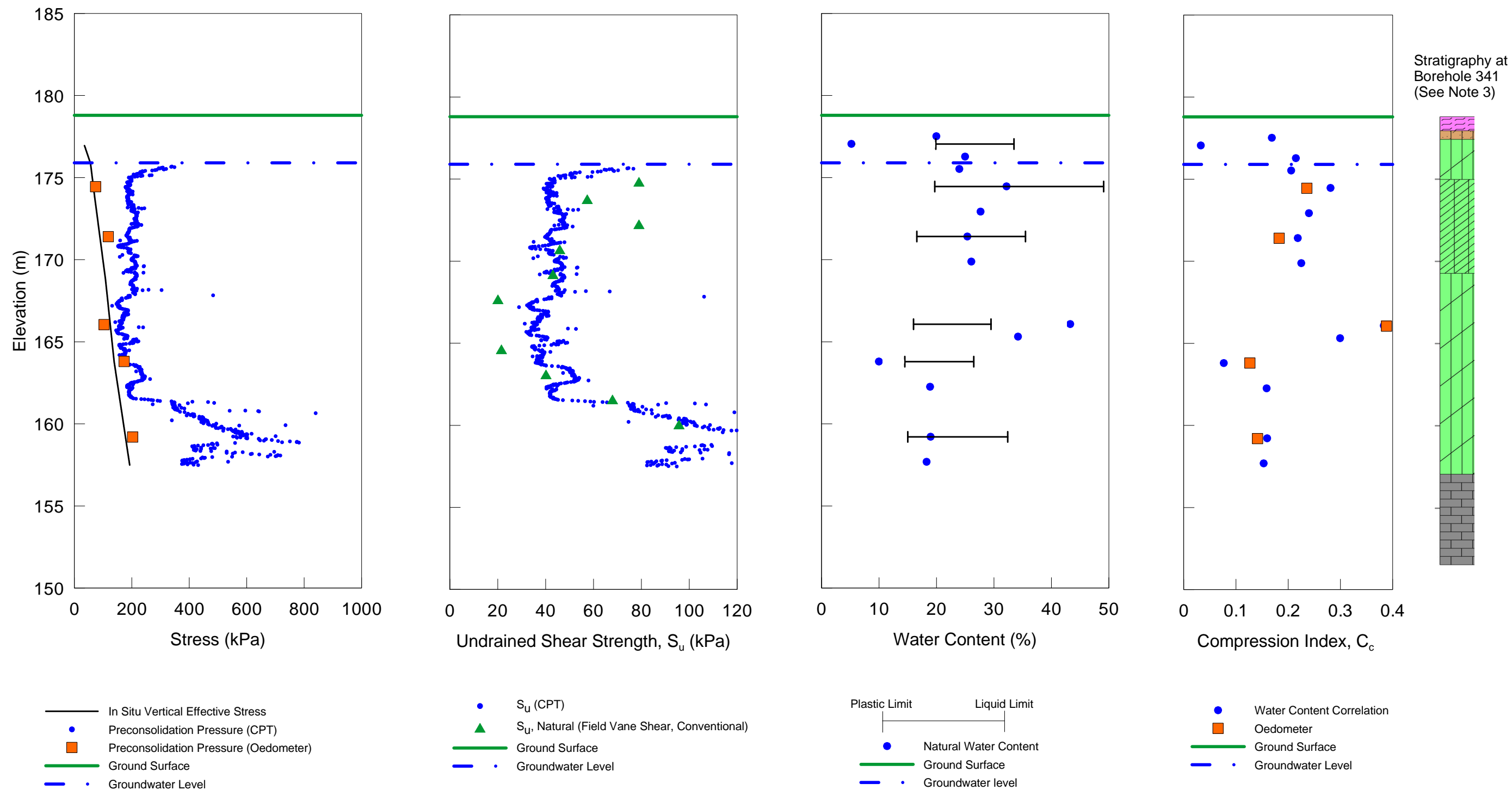


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-158/CPT-21			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261S
	DRAWN MK		Nov. 17/10
CHECK			
SCALE AS SHOWN			REV. 2
Figure 6.1S			

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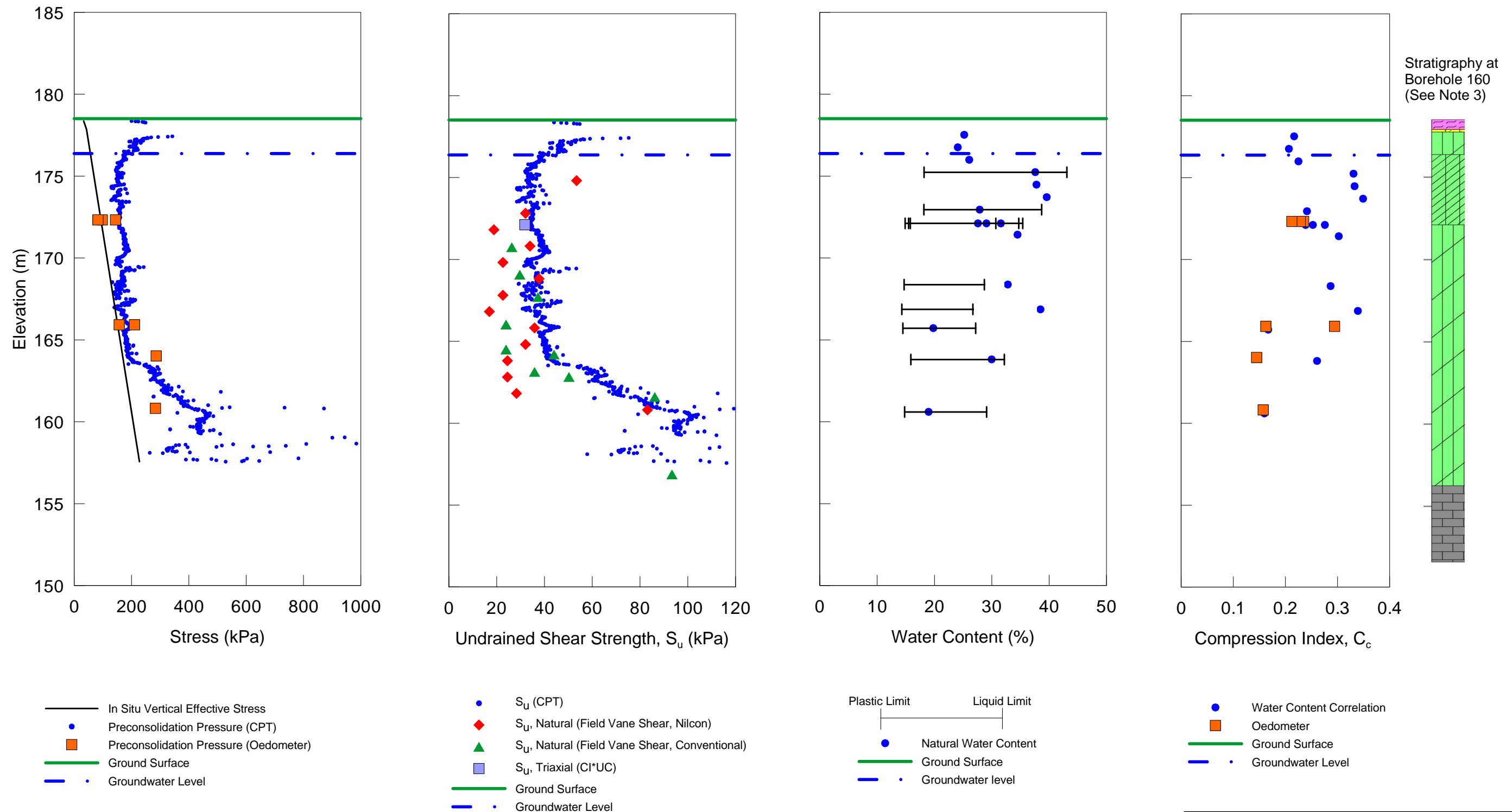


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-341/CPT-159			
	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261T
	DRAWN MK Nov. 17/10		SCALE AS SHOWN REV. 2
CHECK		Figure 6.1T	

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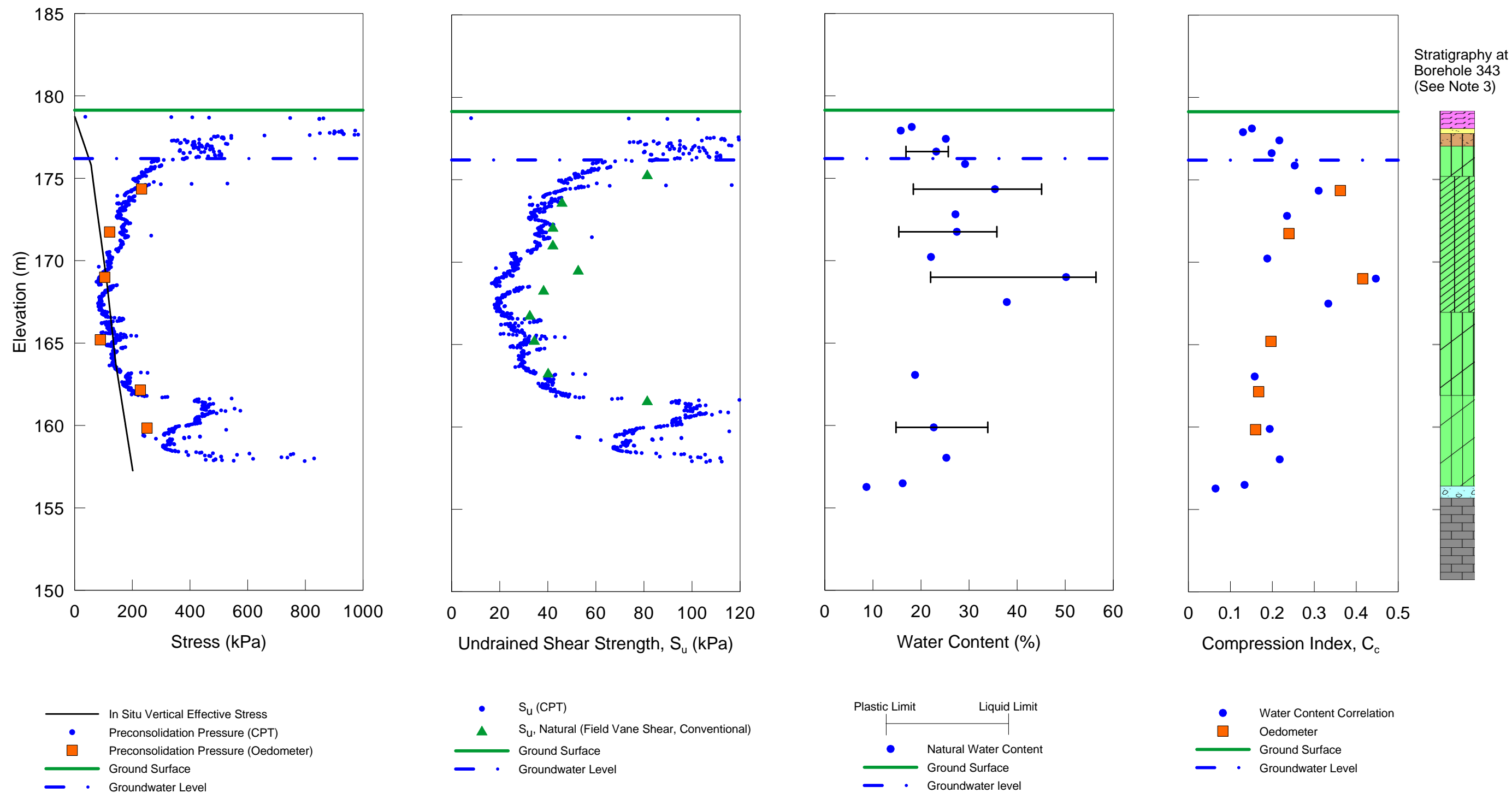


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-160/CPT-160			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0281U
			SCALE AS SHOWN
	DRAWN MK Nov. 17/10		REV. 2
	CHECK		
			Figure 6.1U

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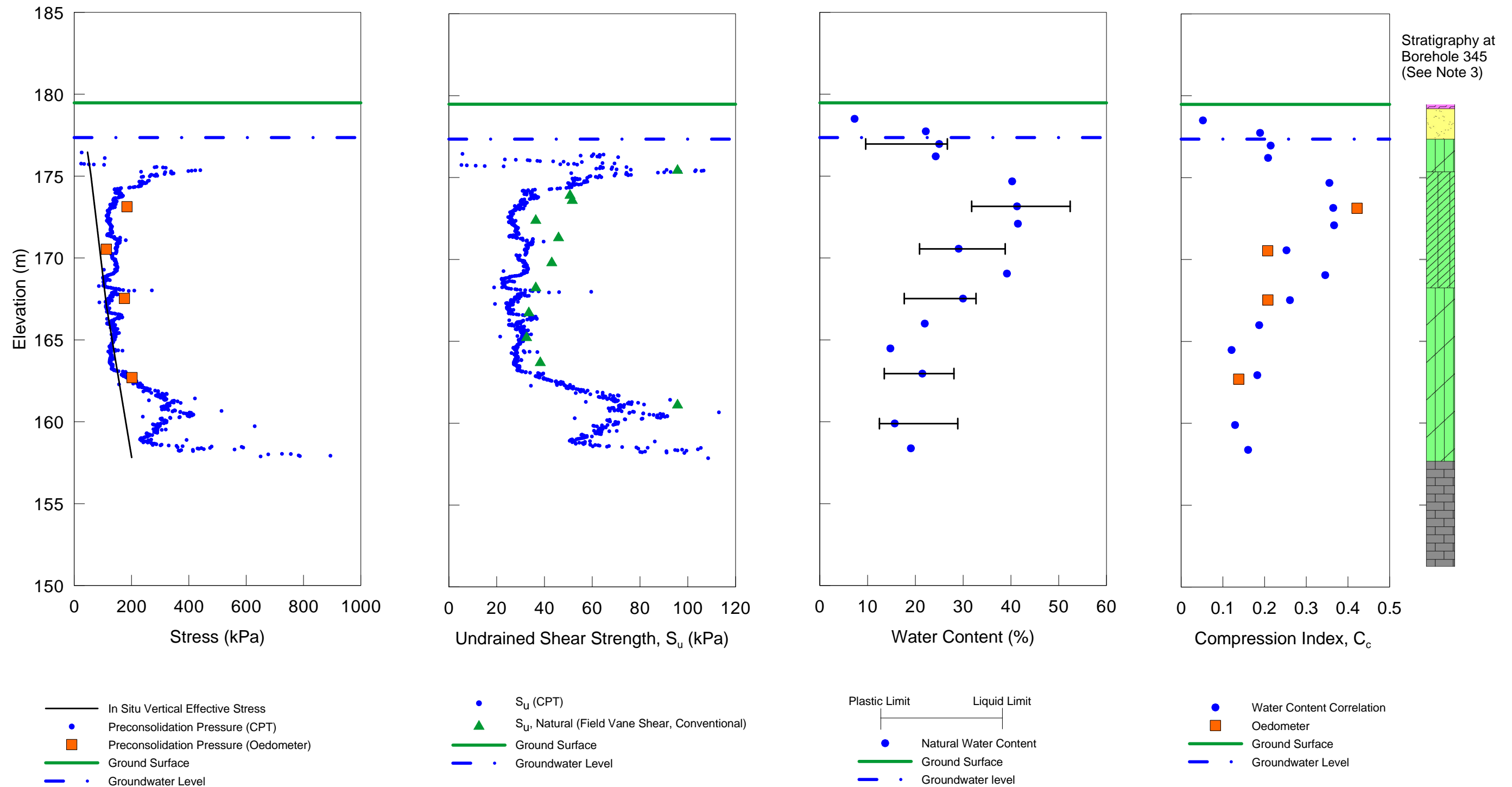


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-343/CPT-342			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261V
	DRAWN MK Nov. 17/10		SCALE AS SHOWN
	CHECK		REV. 2
	Figure 6.1V		

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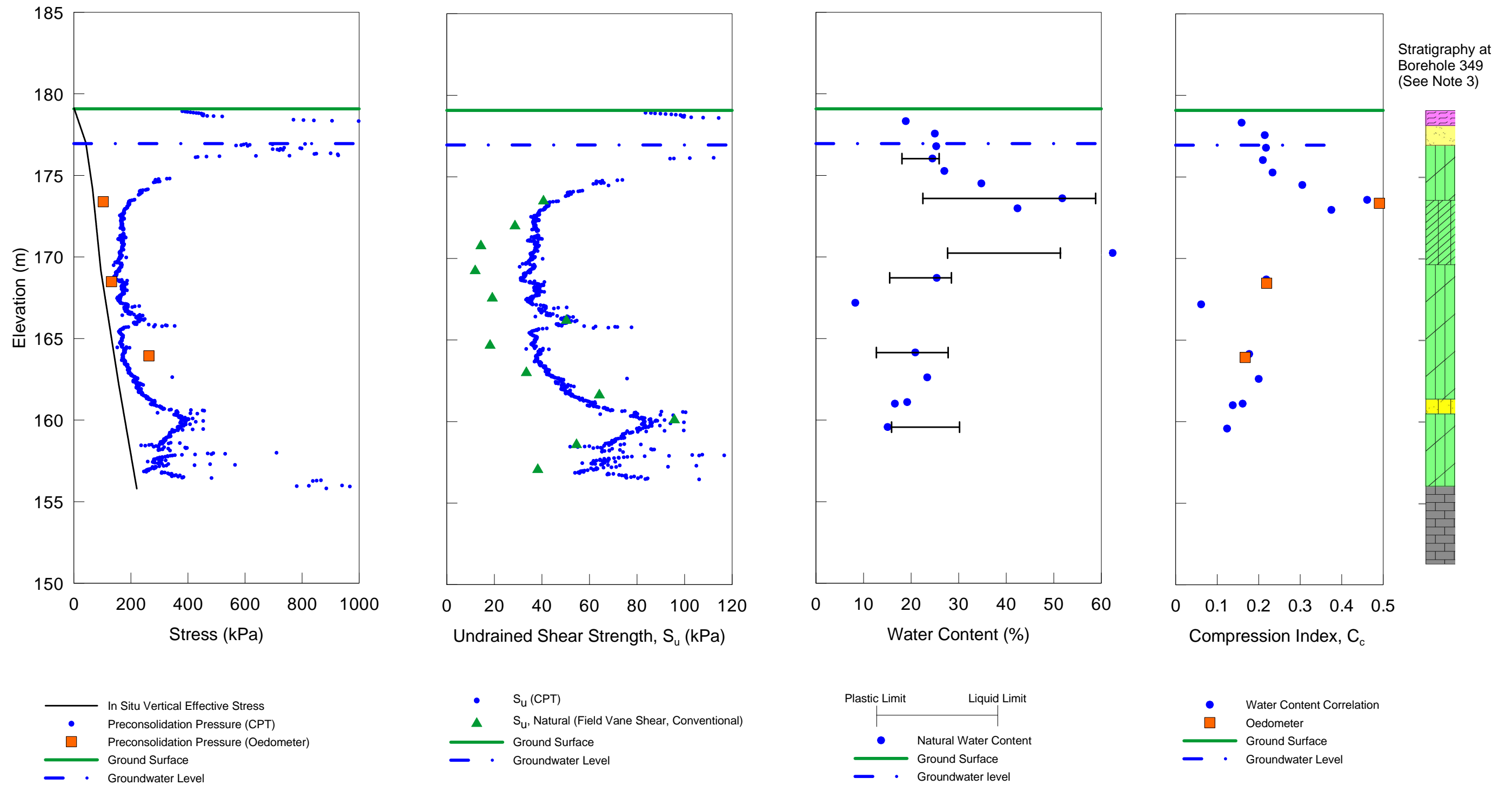


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO				
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-345/CPT-161				
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261W	
			SCALE AS SHOWN	
	DRAWN MK Nov. 17/10		REV. 2	
	CHECK			
Figure 6.1W				

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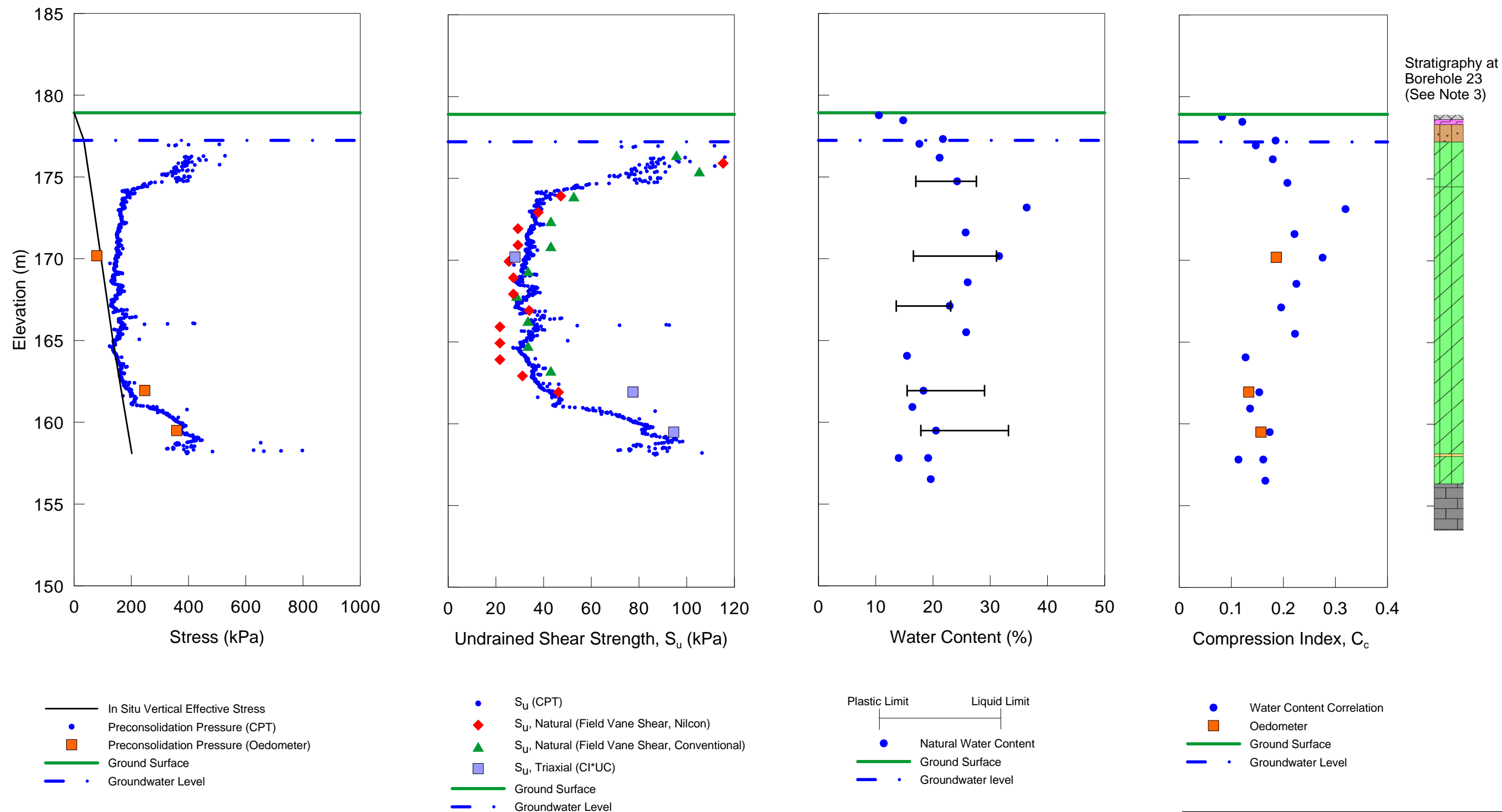


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-349/CPT-348			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0261X
	DRAWN MK Nov. 17/10		SCALE AS SHOWN REV. 2
CHECK		Figure 6.1X	

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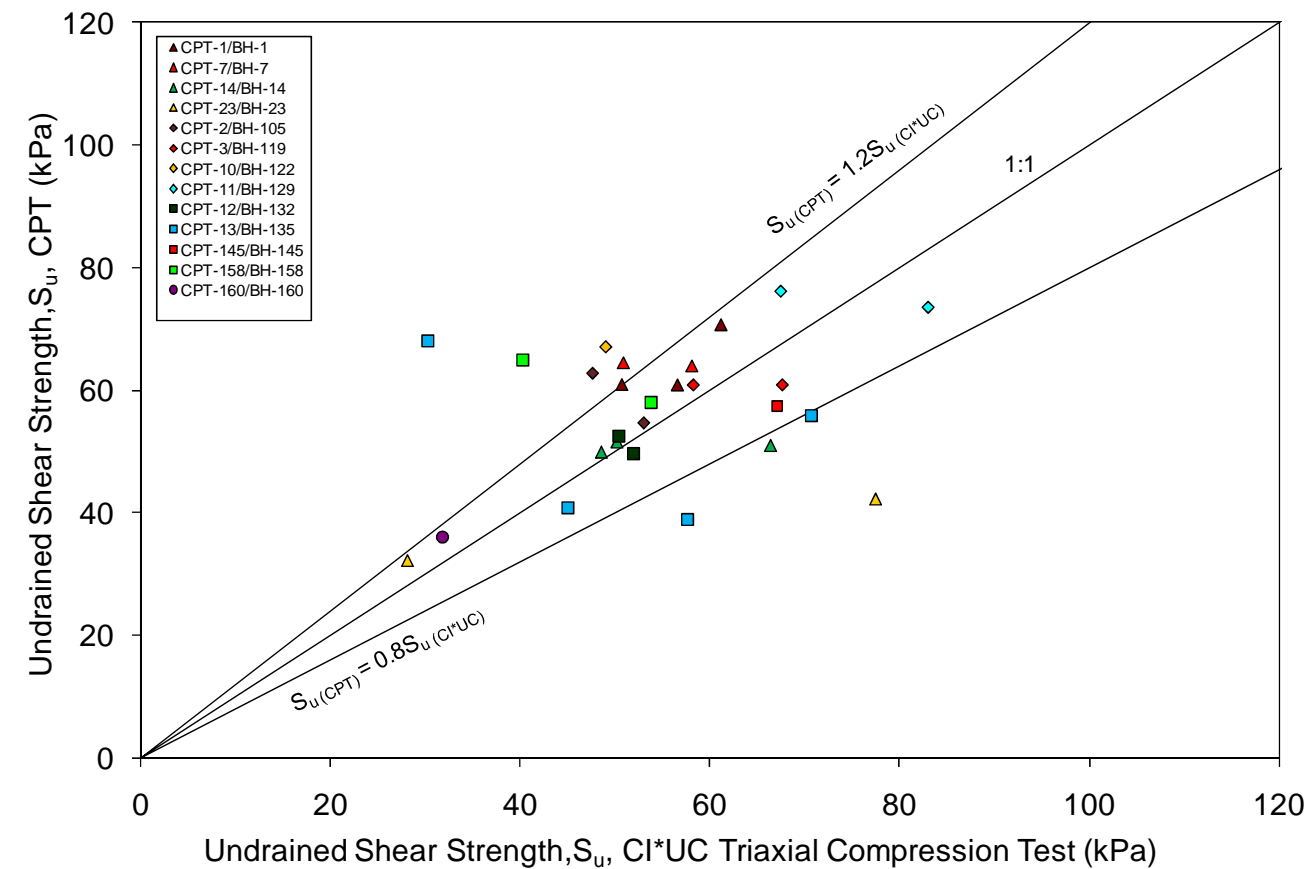
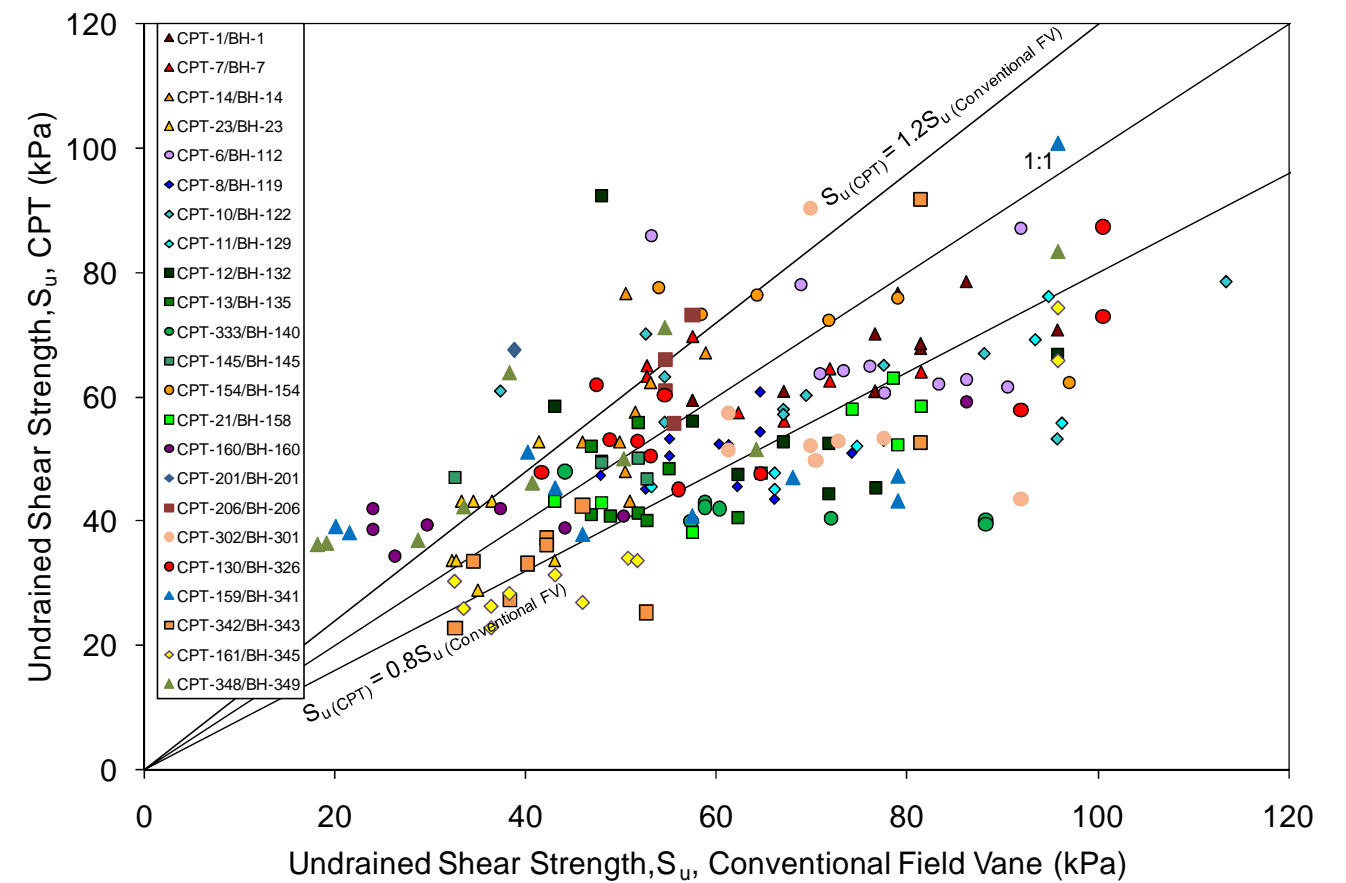
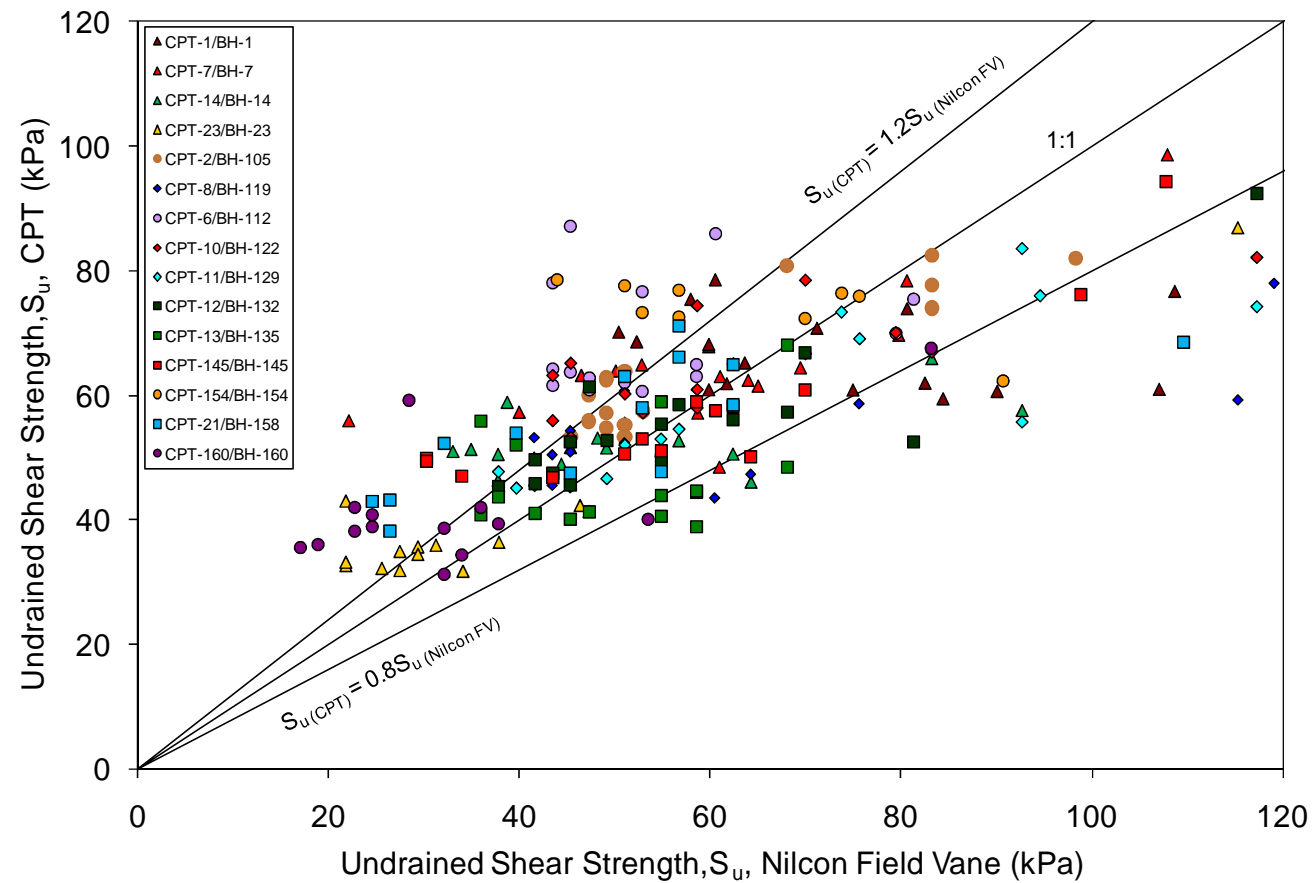


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.


PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SUBSURFACE TEST DATA BOREHOLE BH-23/CPT-23			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R0251Y
	DRAWN MK		Nov. 17/10
CHECK			
SCALE AS SHOWN			REV. 2
Figure 6.1Y			

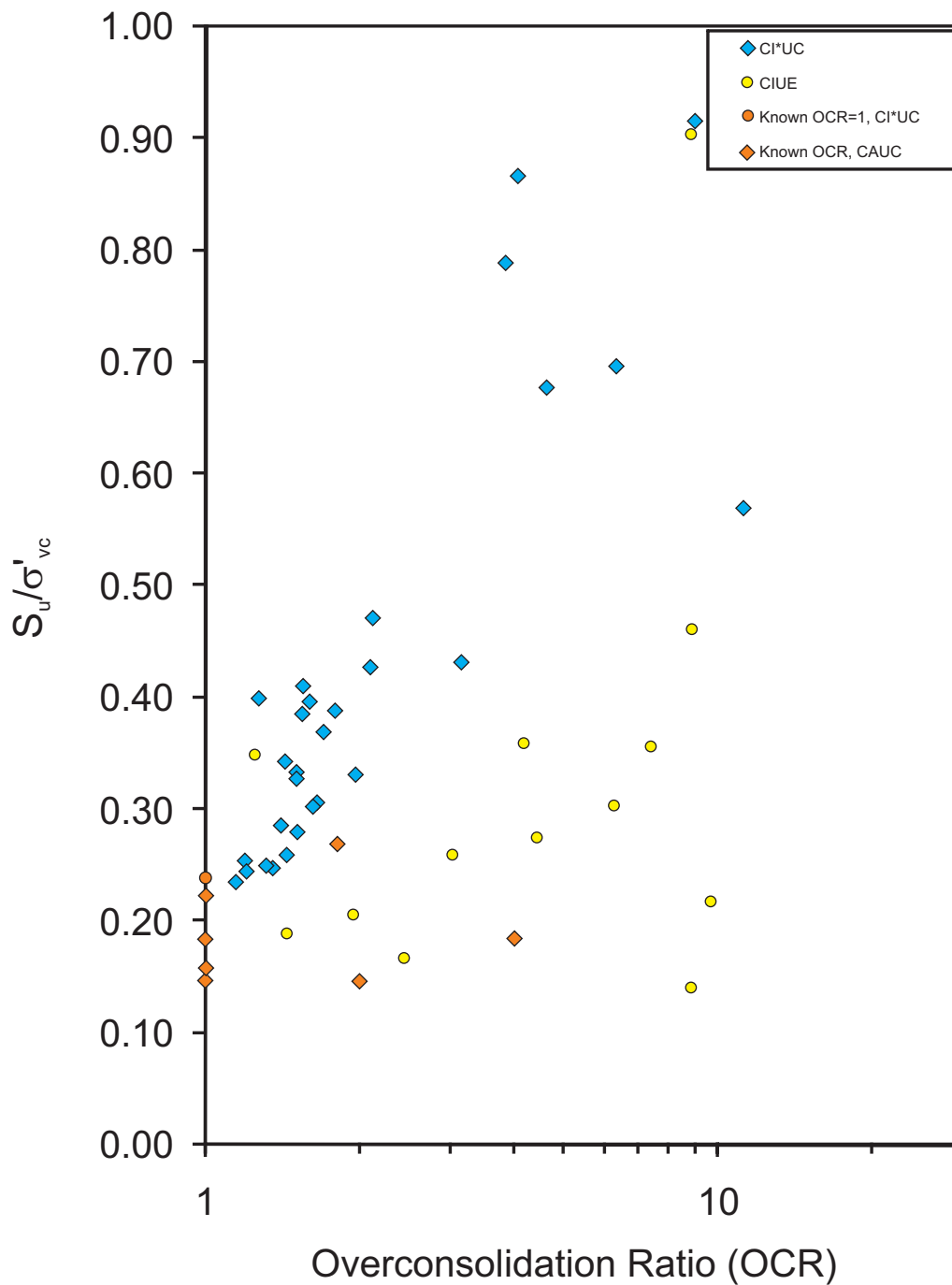
N:\active\2007\1130 - Geotechnical\1130-2000\07-1130-2070-Q URS - DRIC APPROACH GSR - WINDSOR\Drafting\Grapher files\0711302070R02062.grf



NOTES


1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

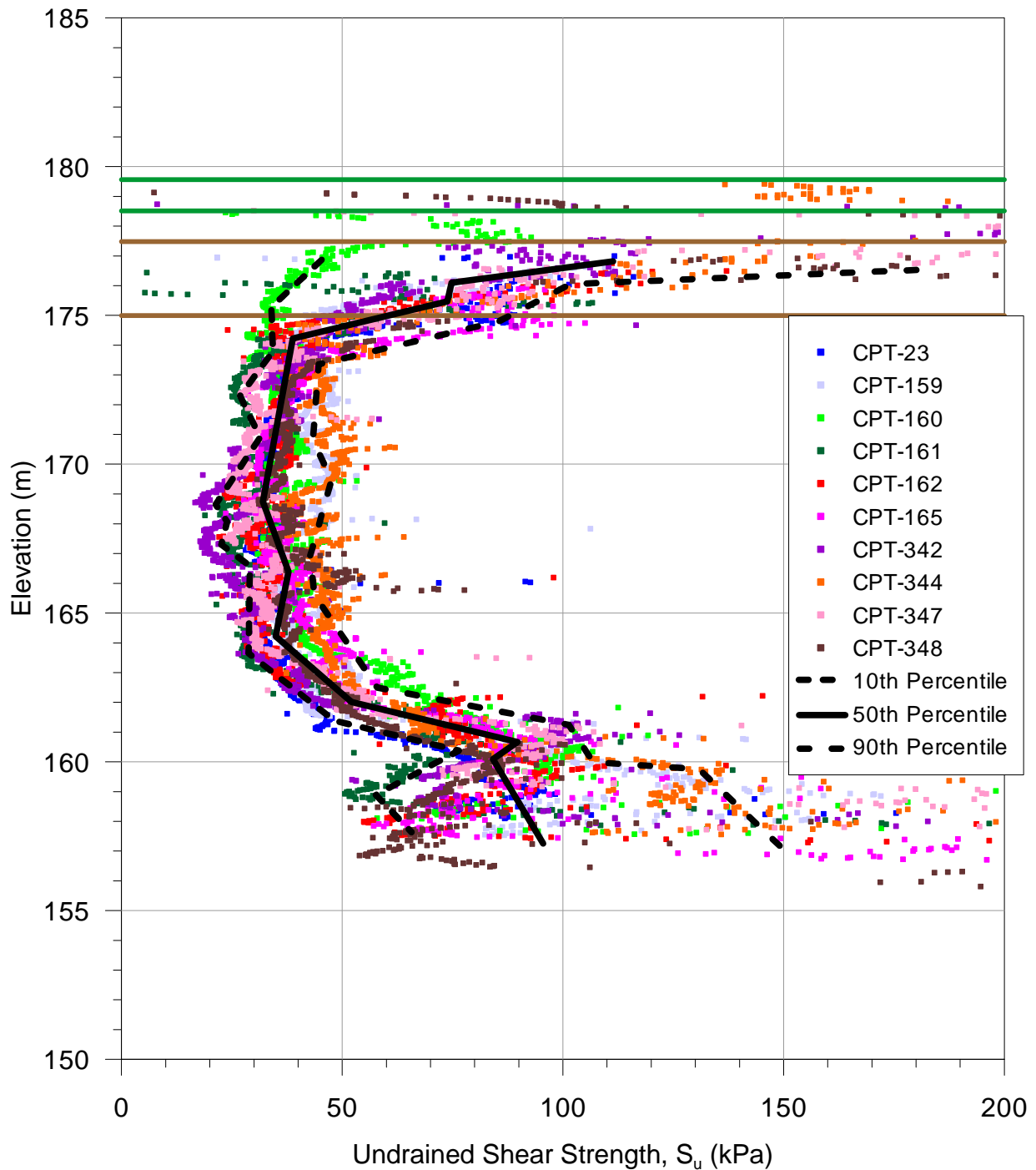
PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY UNDRAINED SHEAR STRENGTH CPT, LABORATORY AND FIELD VANE TESTS			
	PROJECT No.	0711302070	FILE No. 0711302070R02062
	DRAWN	MK	NOV. 16/10
CHECK			
			SCALE AS SHOWN REV. 2
Figure 6.2			



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY LABORATORY UNDRAINED SHEAR STRENGTH DATA COMPARISON			
PROJECT No. 07-1130-207-0		FILE No. 0711302070-R02063	
CADD	MK	Nov. 18/10	SCALE AS SHOWN REV. 2
CHECK			
			FIGURE 6.3



NOTES

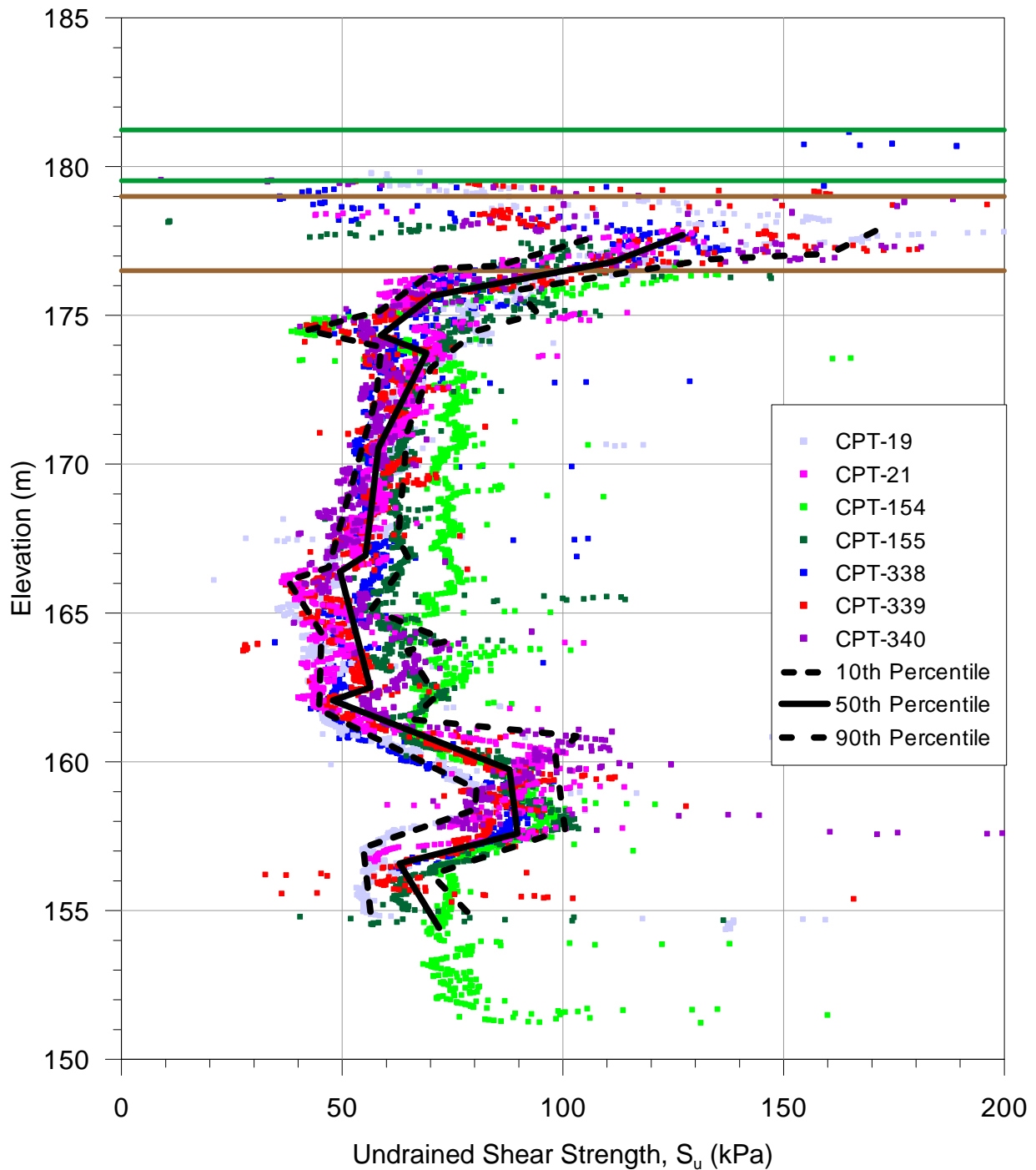
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 9+900 TO 11+000**



PROJECT No.	0711302070	FILE No.	0711302070R0284A
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4A			



NOTES

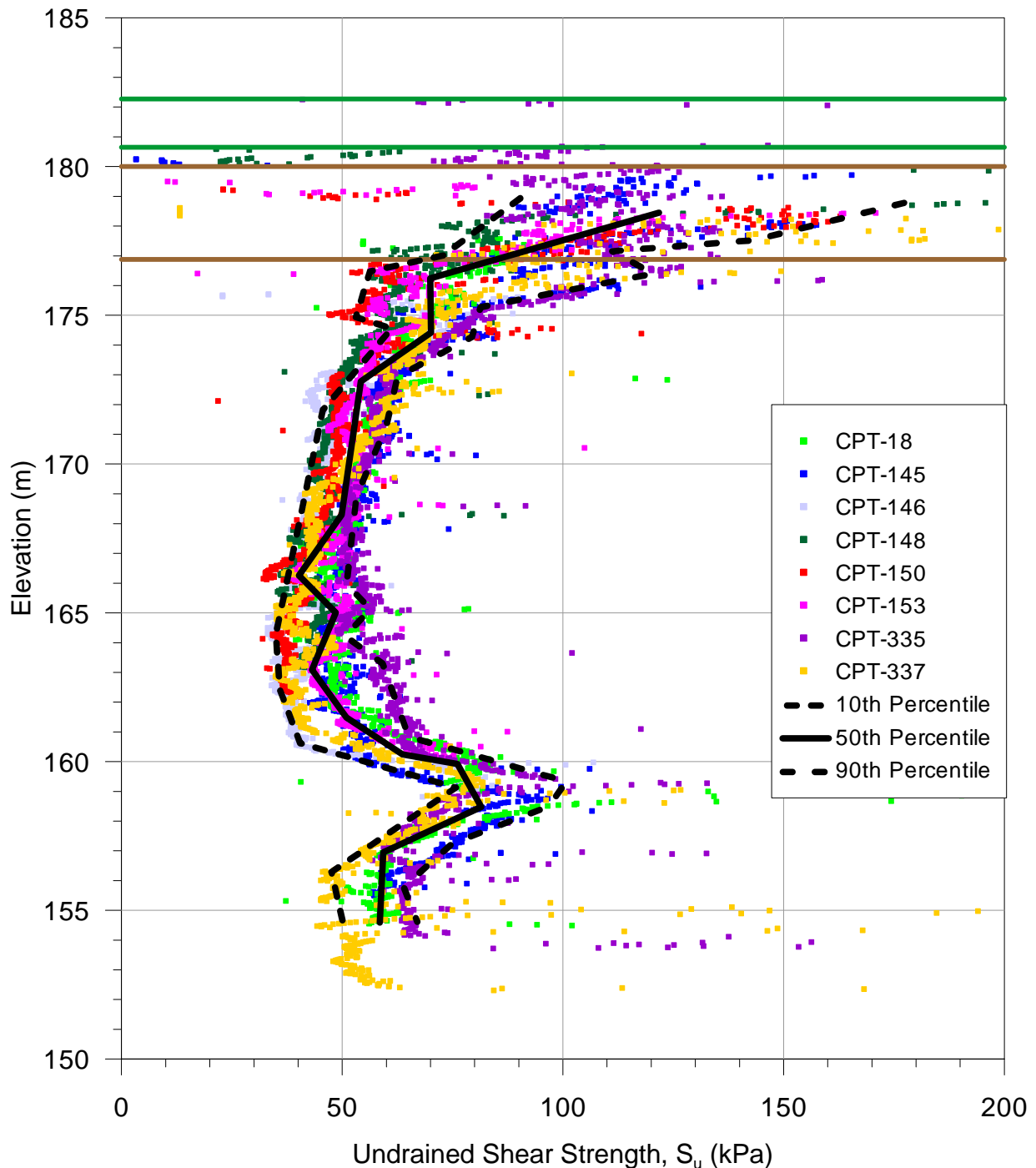
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 11+000 to 12+100**



PROJECT No.	0711302070	FILE No.	0711302070R0264B
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
			FIGURE 6.4B



— RANGE OF INTERPRETED 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

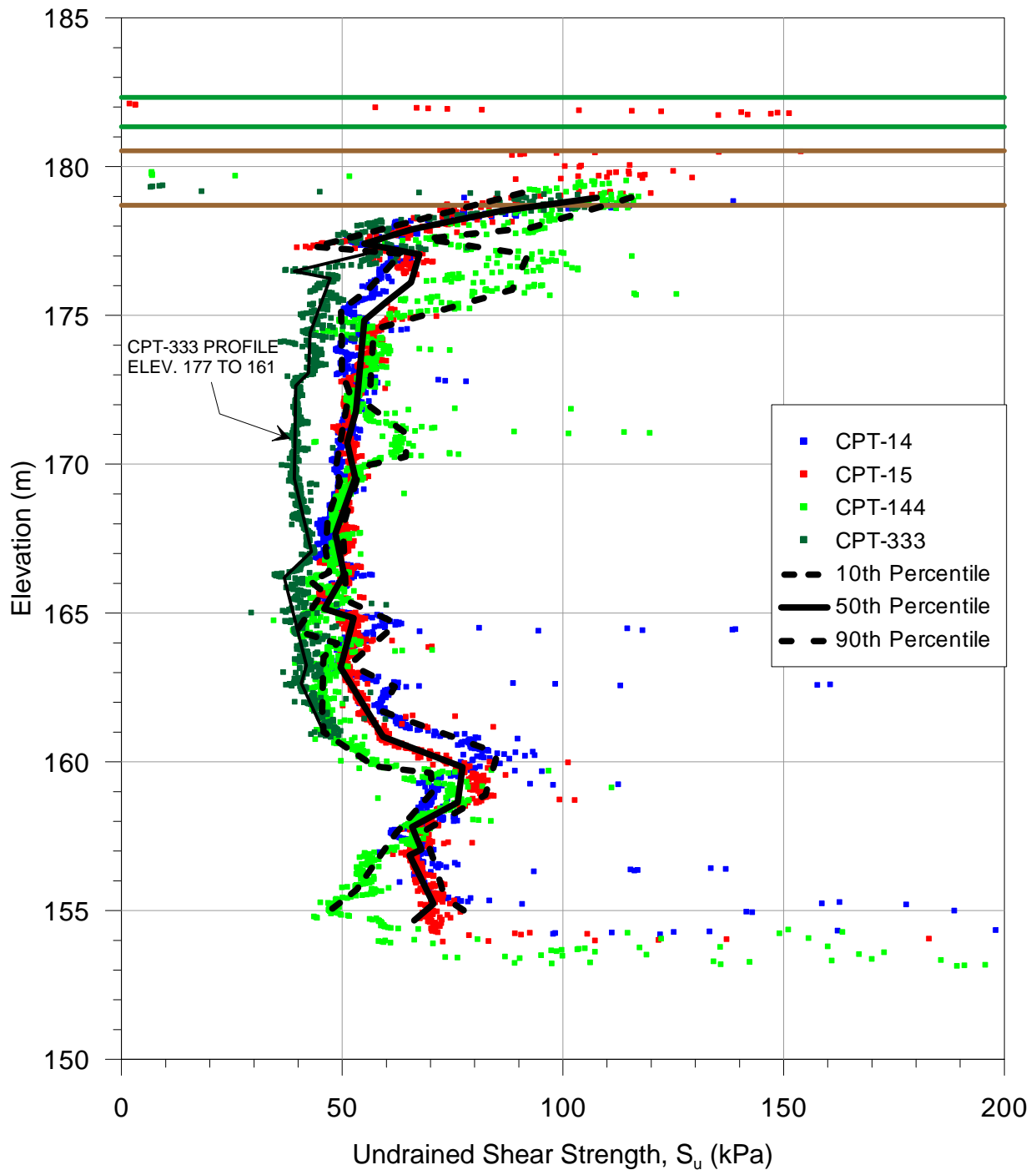
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 12+100 TO 13+000**



PROJECT No.	0711302070	FILE No.	0711302070R0284C
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4C			



NOTES

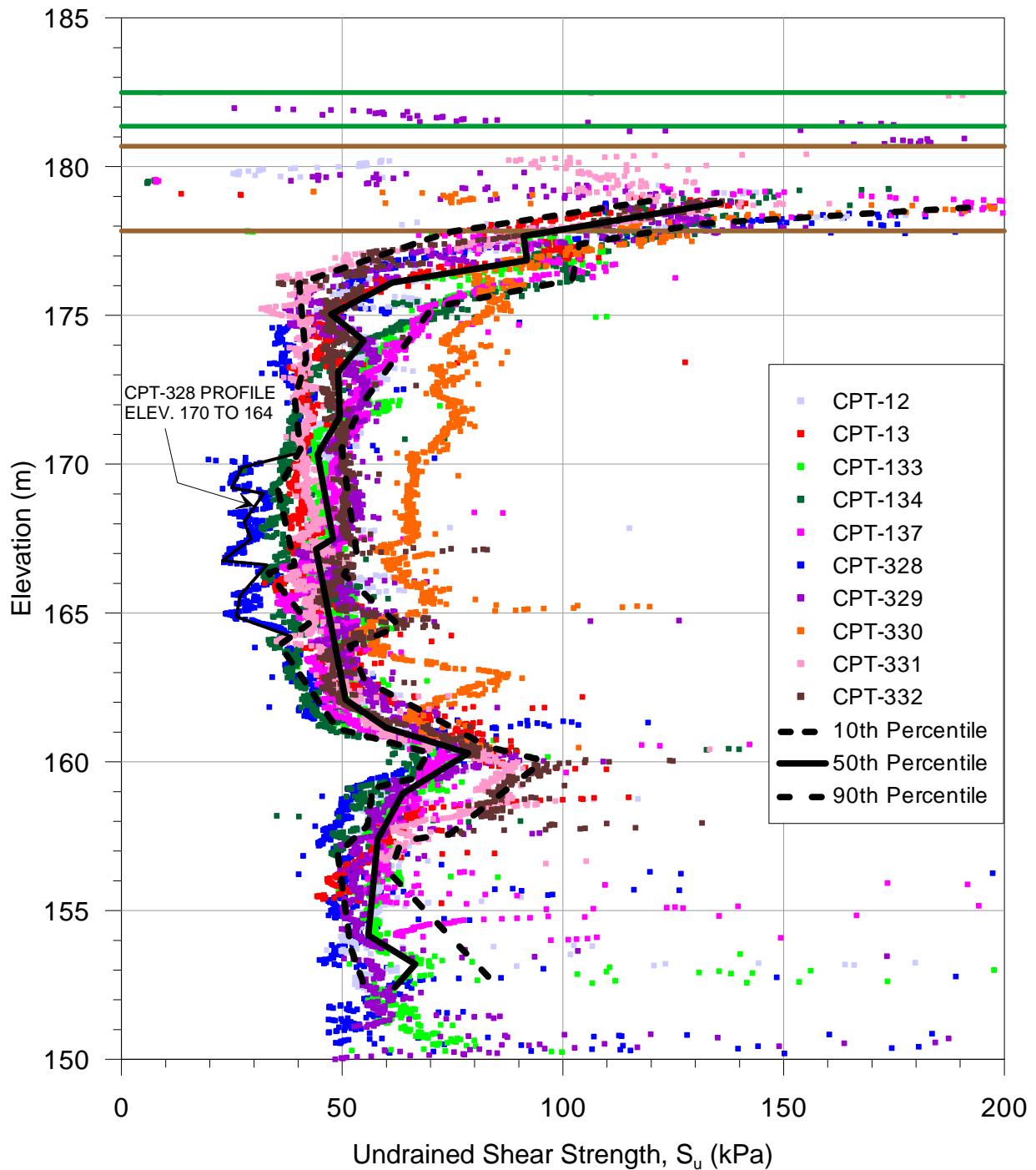
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 13+000 to 13+900**



PROJECT No.	0711302070	FILE No.	0711302070R0284D
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4D			



NOTES

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

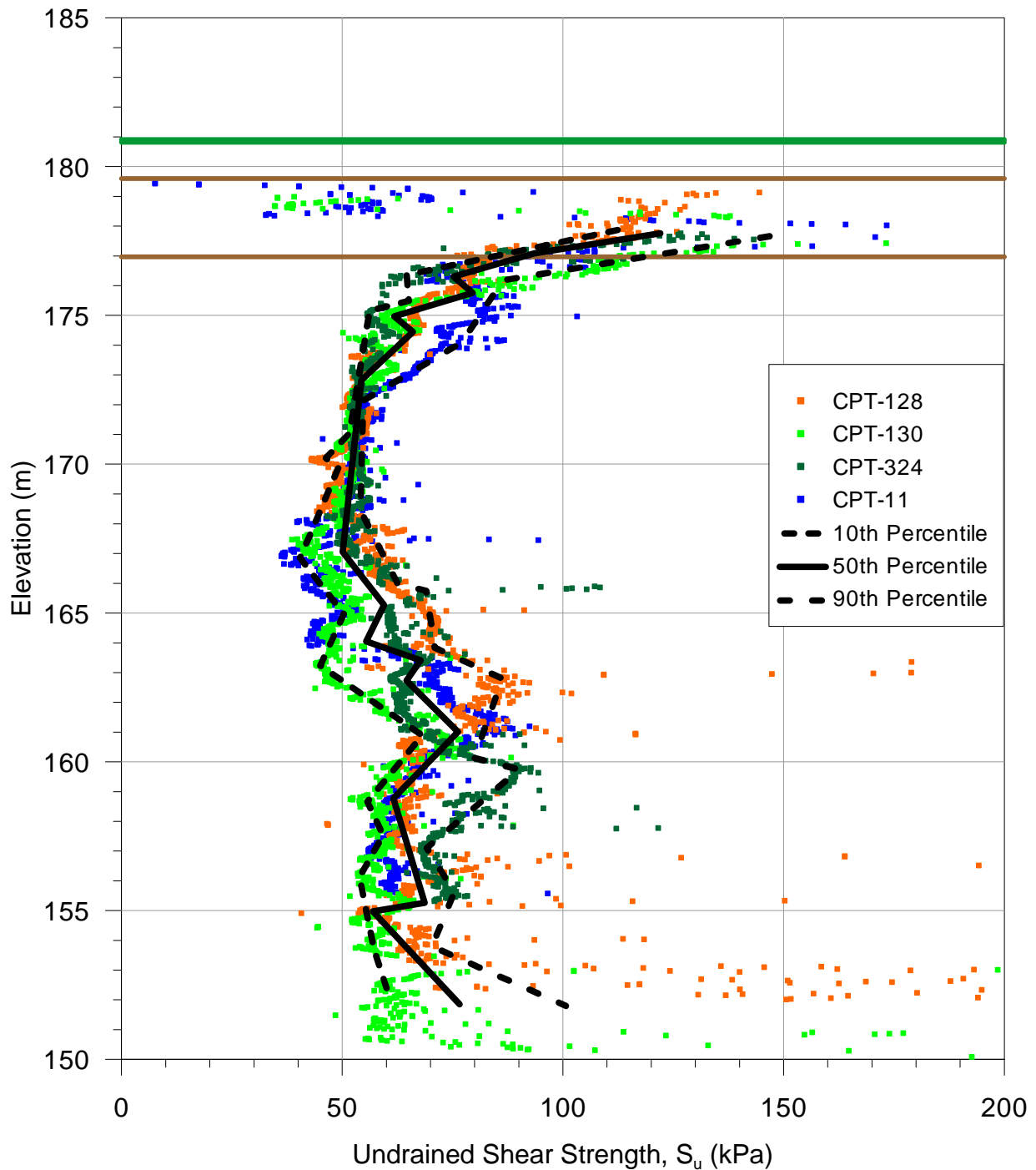
PROJECT
SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 13+900 to 14+550**



PROJECT No.	0711302070	FILE No.	0711302070R0284E
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2

FIGURE 6.4E



— RANGE OF INTERPRETED 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

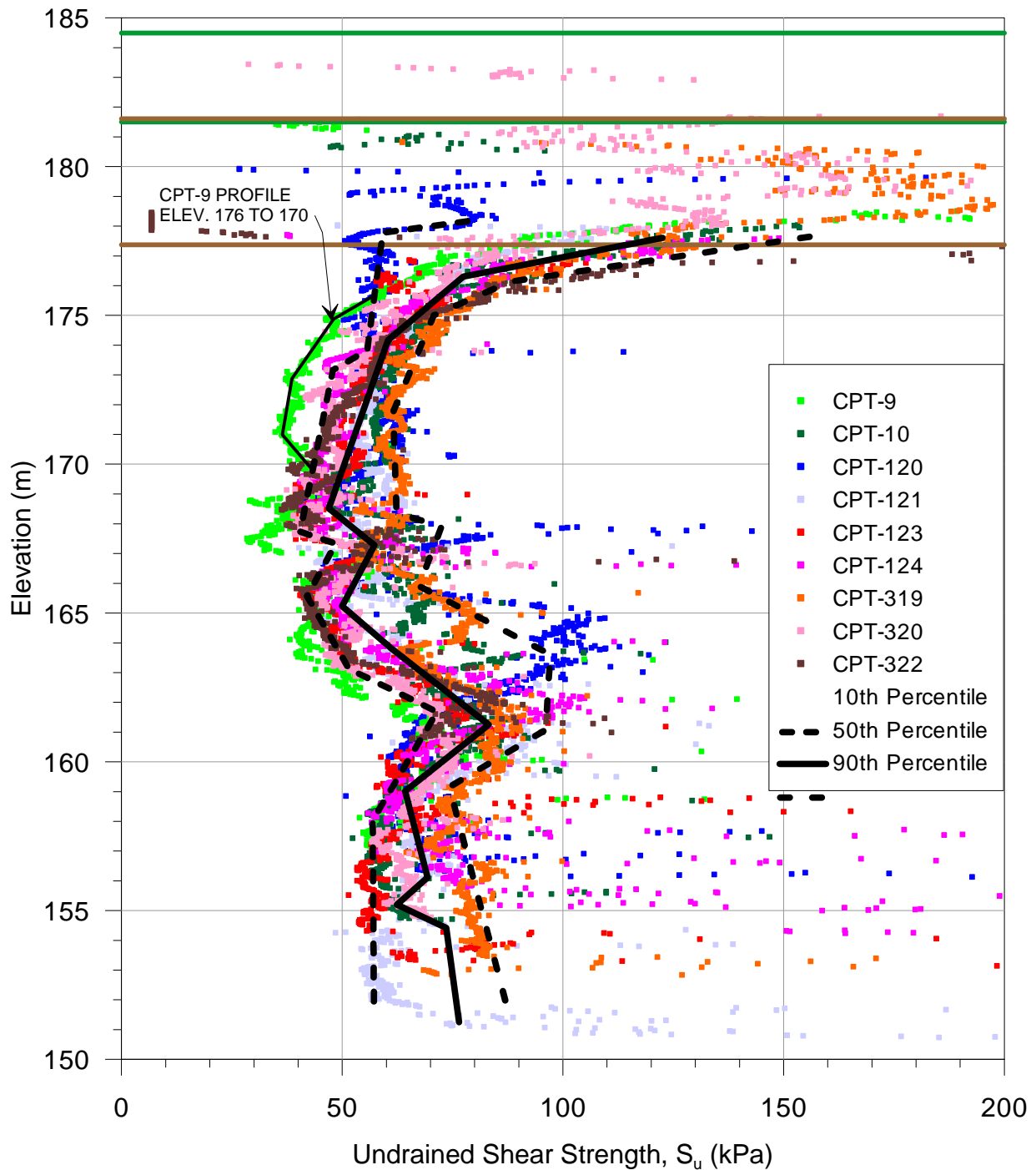
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 14+550 TO 10+300**



PROJECT No.	0711302070	FILE No.	0711302070R0264F
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4F			



NOTES

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

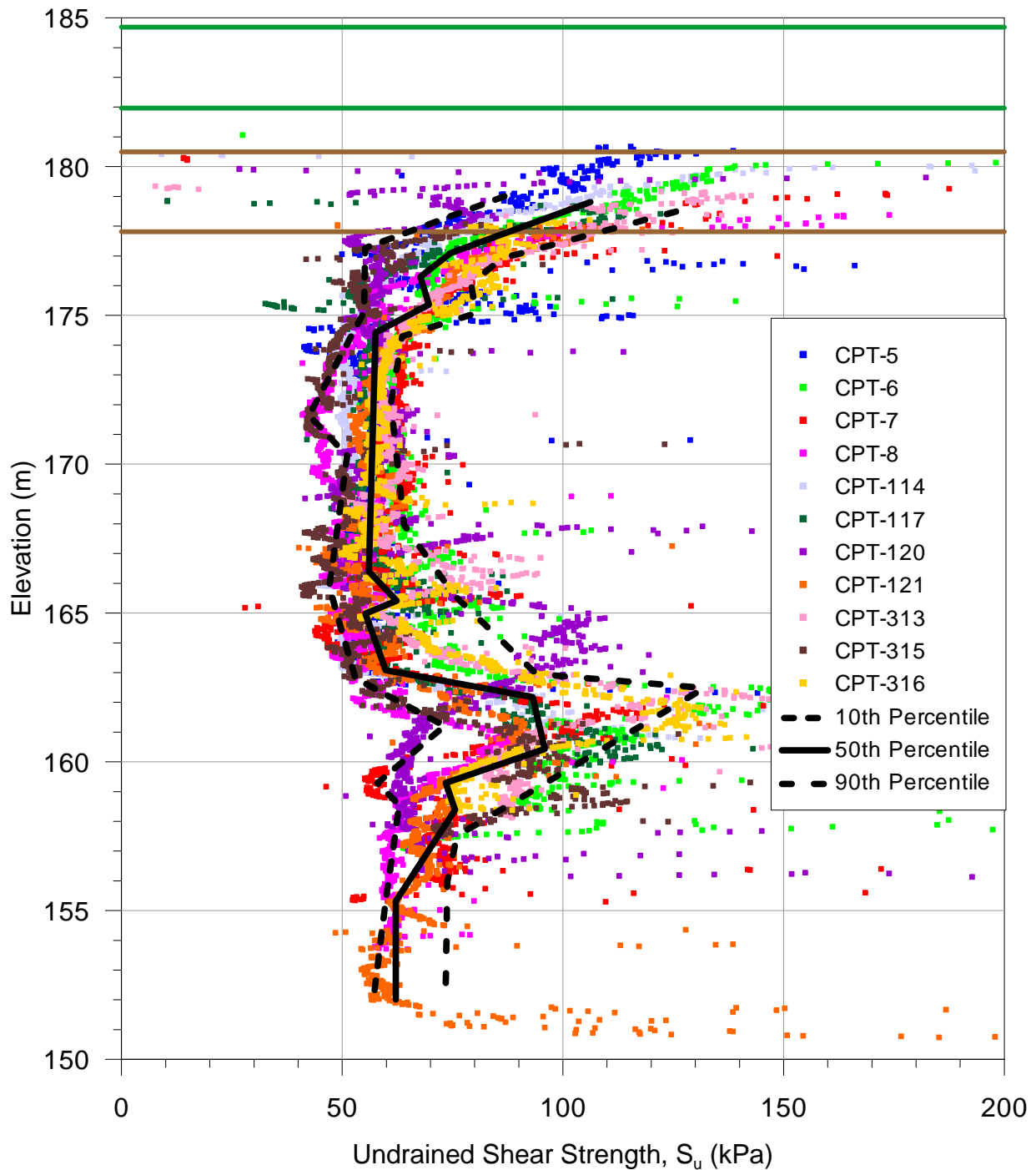
PROJECT
SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 10+300 to 11+300**



PROJECT No.	0711302070	FILE No.	0711302070R0264G
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2

FIGURE 6.4G



NOTES

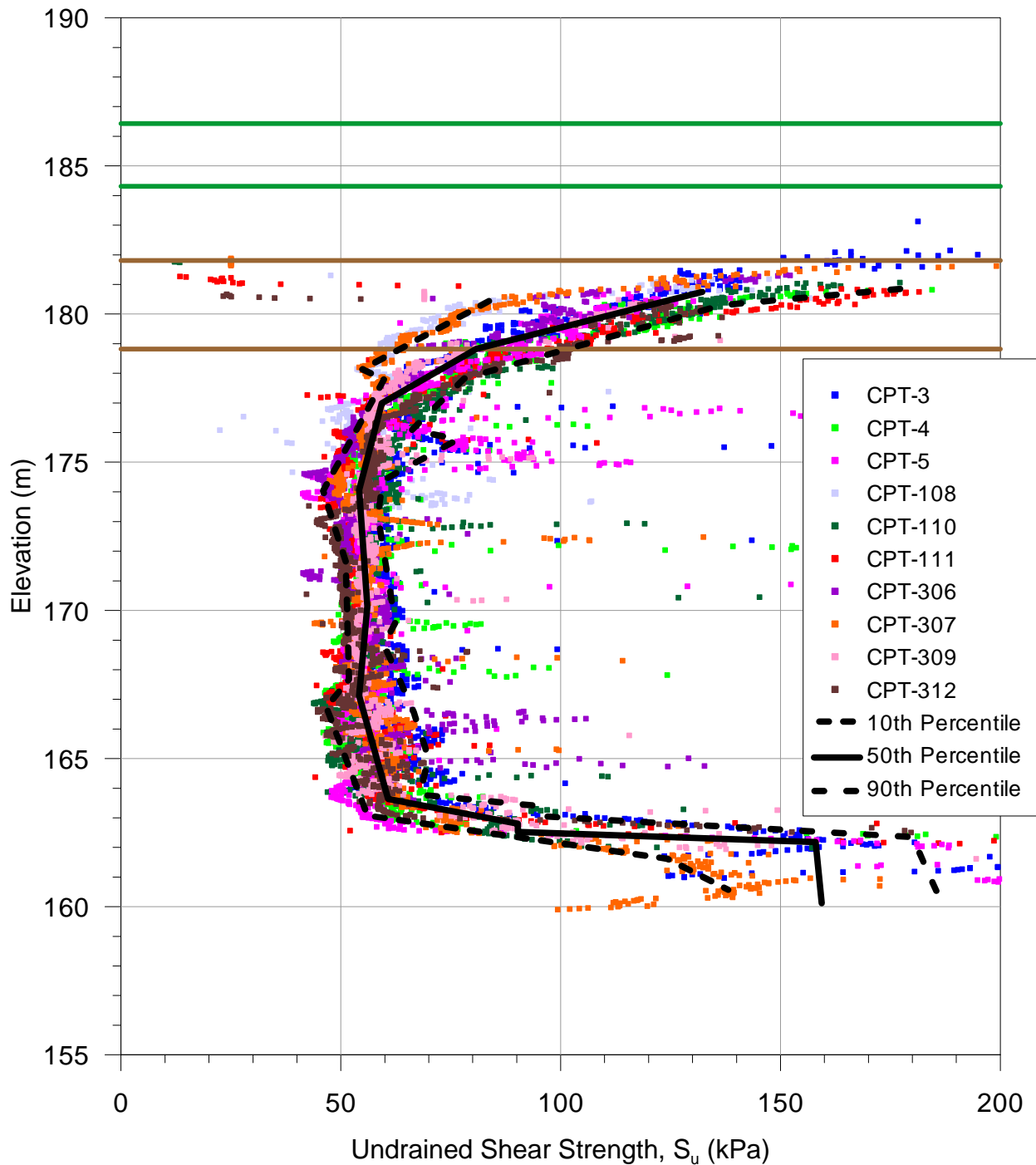
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 11+300 TO 12+600**



PROJECT No.	0711302070	FILE No.	0711302070R0284H
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4H			



— RANGE OF INTERPRETED 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 INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

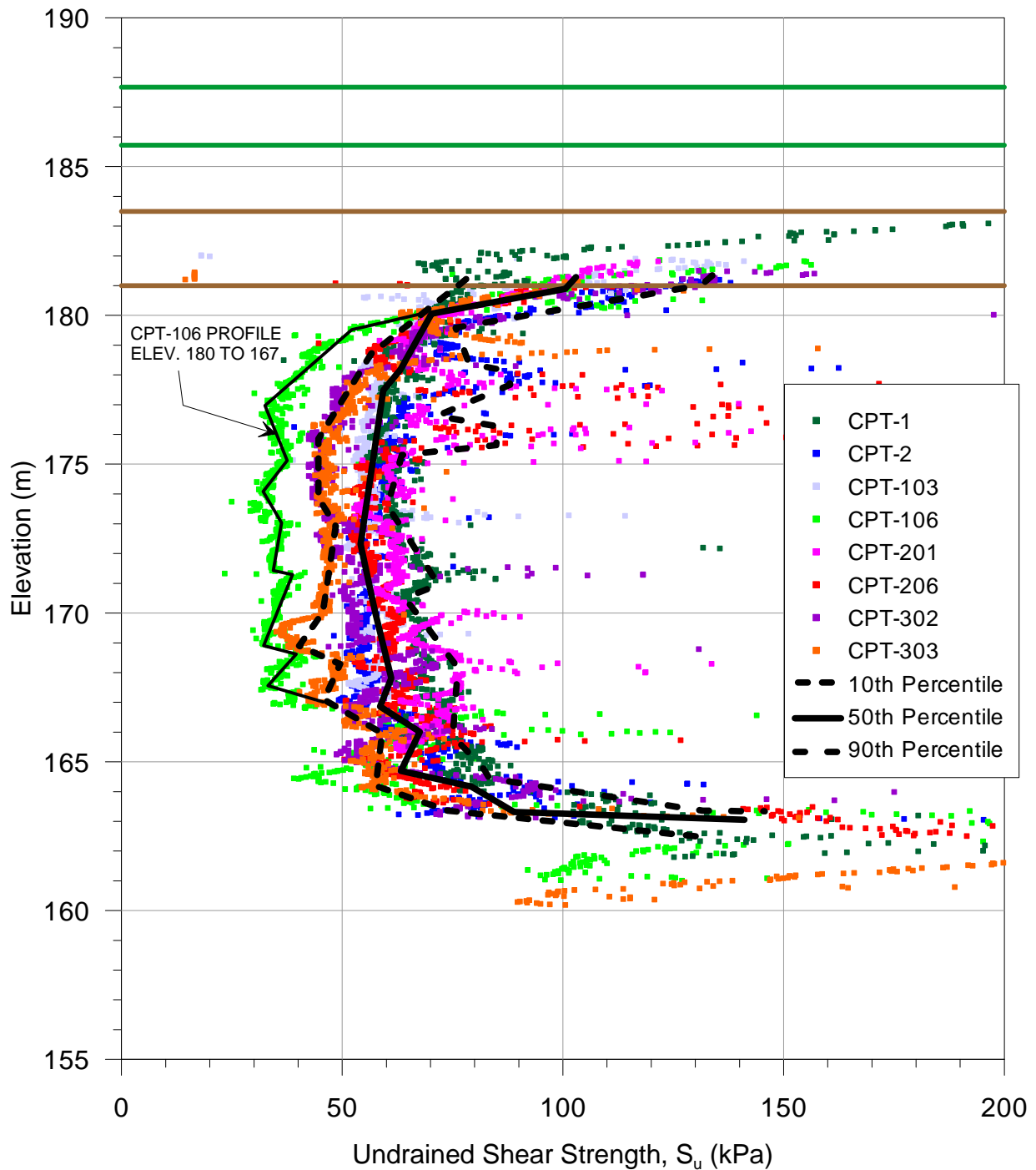
PROJECT
SUBSURFACE CONDITIONS INTERPRETATION REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 12+600 TO 13+600**



PROJECT No.	0711302070	FILE No.	0711302070R0264I
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2

FIGURE 6.4I



NOTES

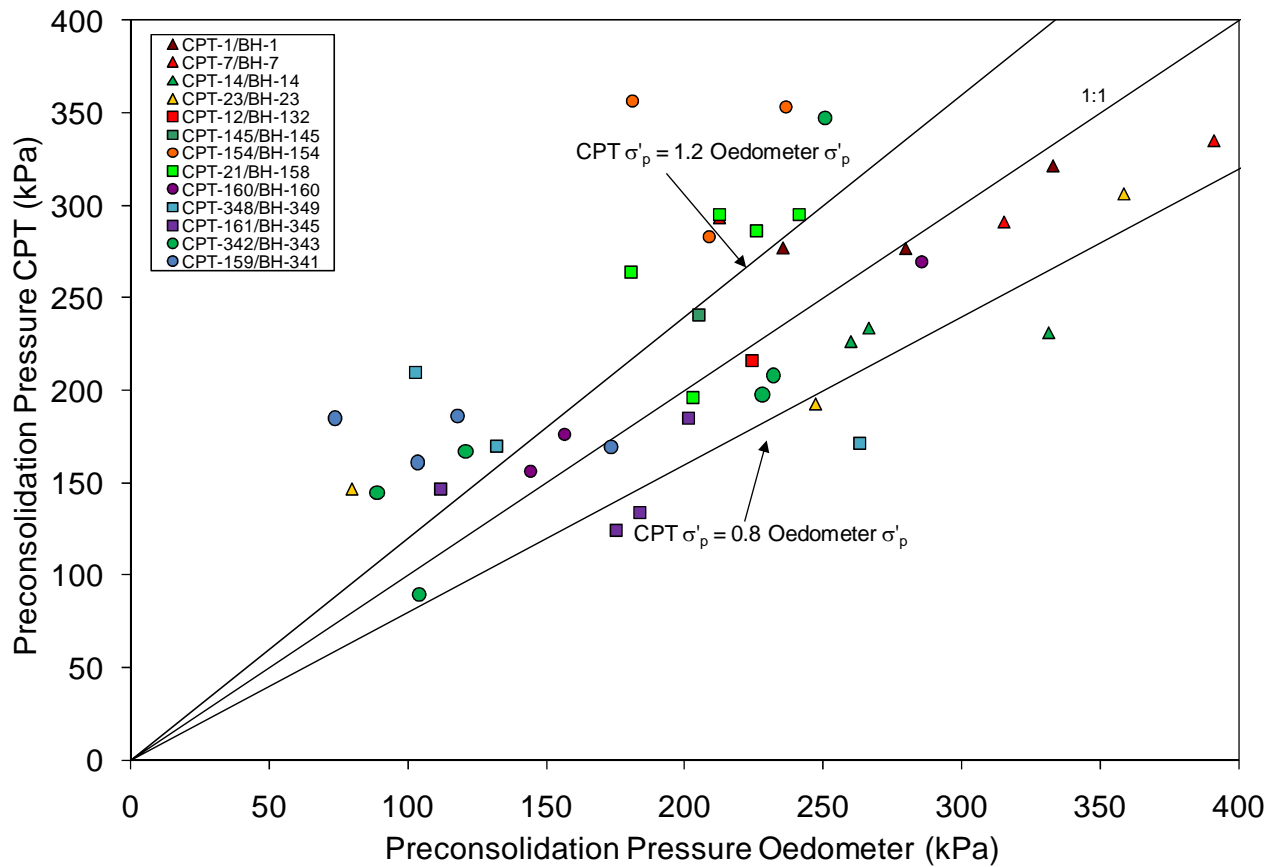
1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD CPT DATA AND THE INTERPRETED UNDRAINED SHEAR STRENGTH PROFILE AS DESCRIBED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

TITLE
**INTERPRETED UNDRAINED SHEAR STRENGTH
PROFILE STATION 13+600 TO 10+900**



PROJECT No.	0711302070	FILE No.	0711302070R0264J
DRAWN	MK	Nov. 11/10	SCALE AS SHOWN
CHECK			REV. 2
FIGURE 6.4J			



NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

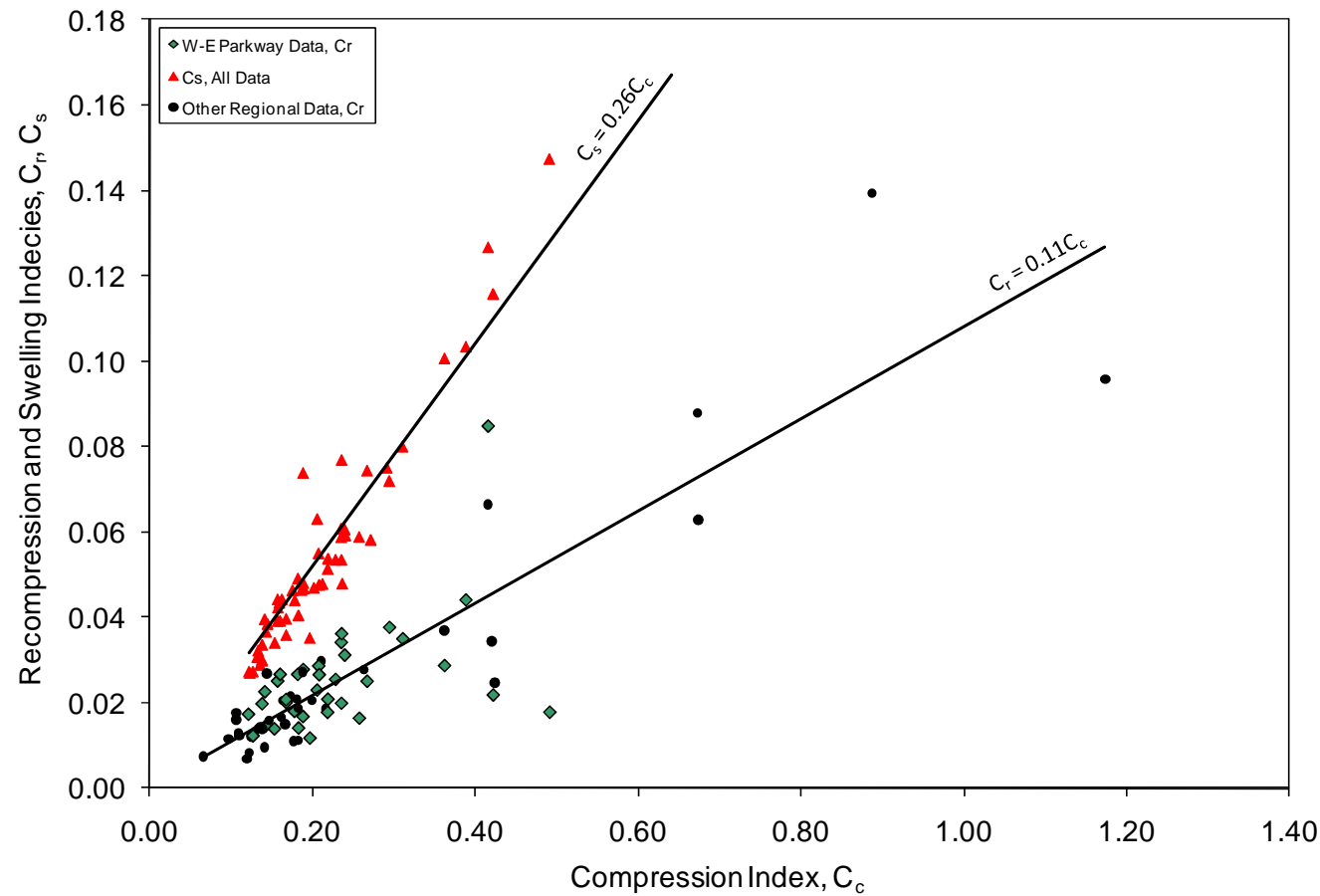
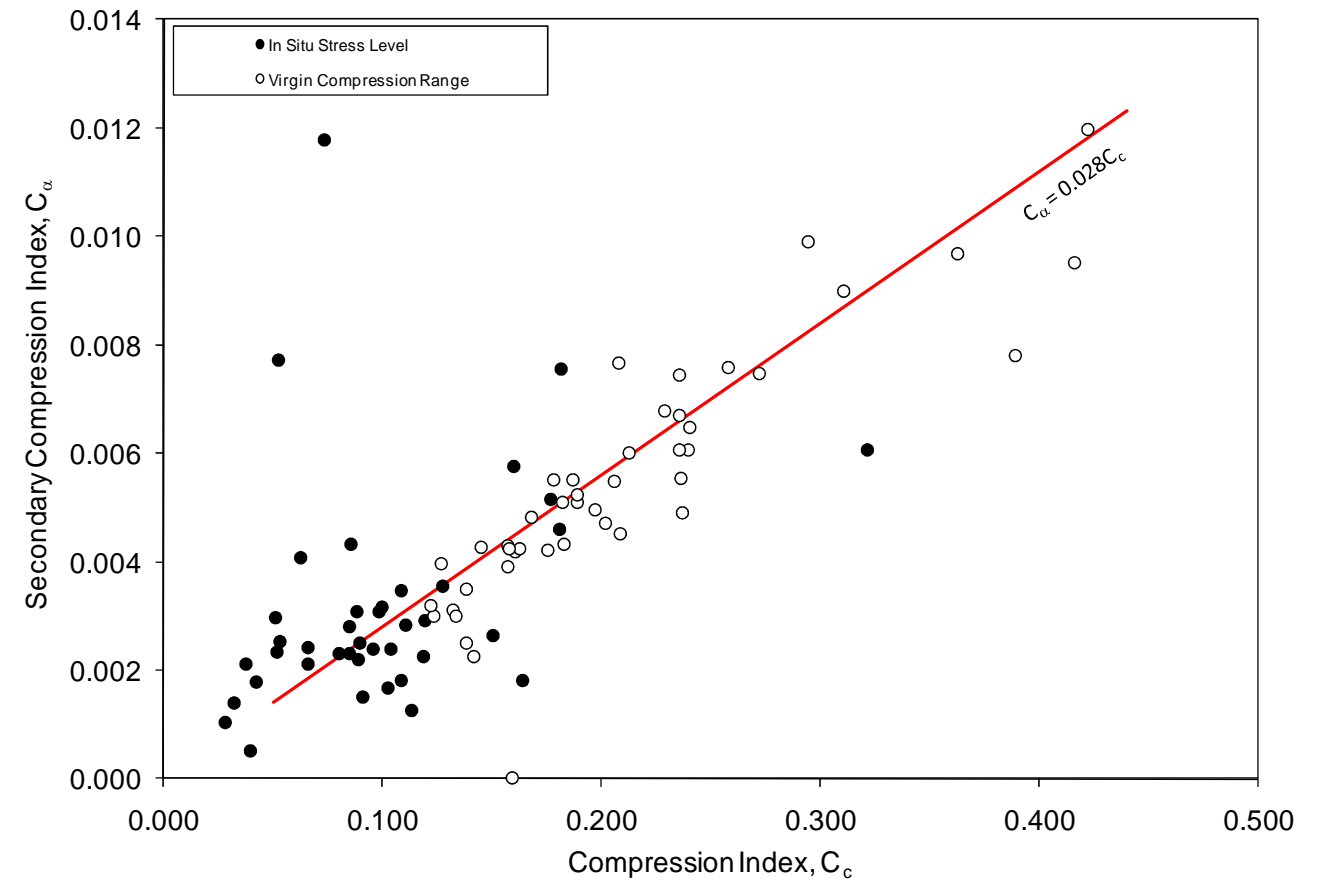
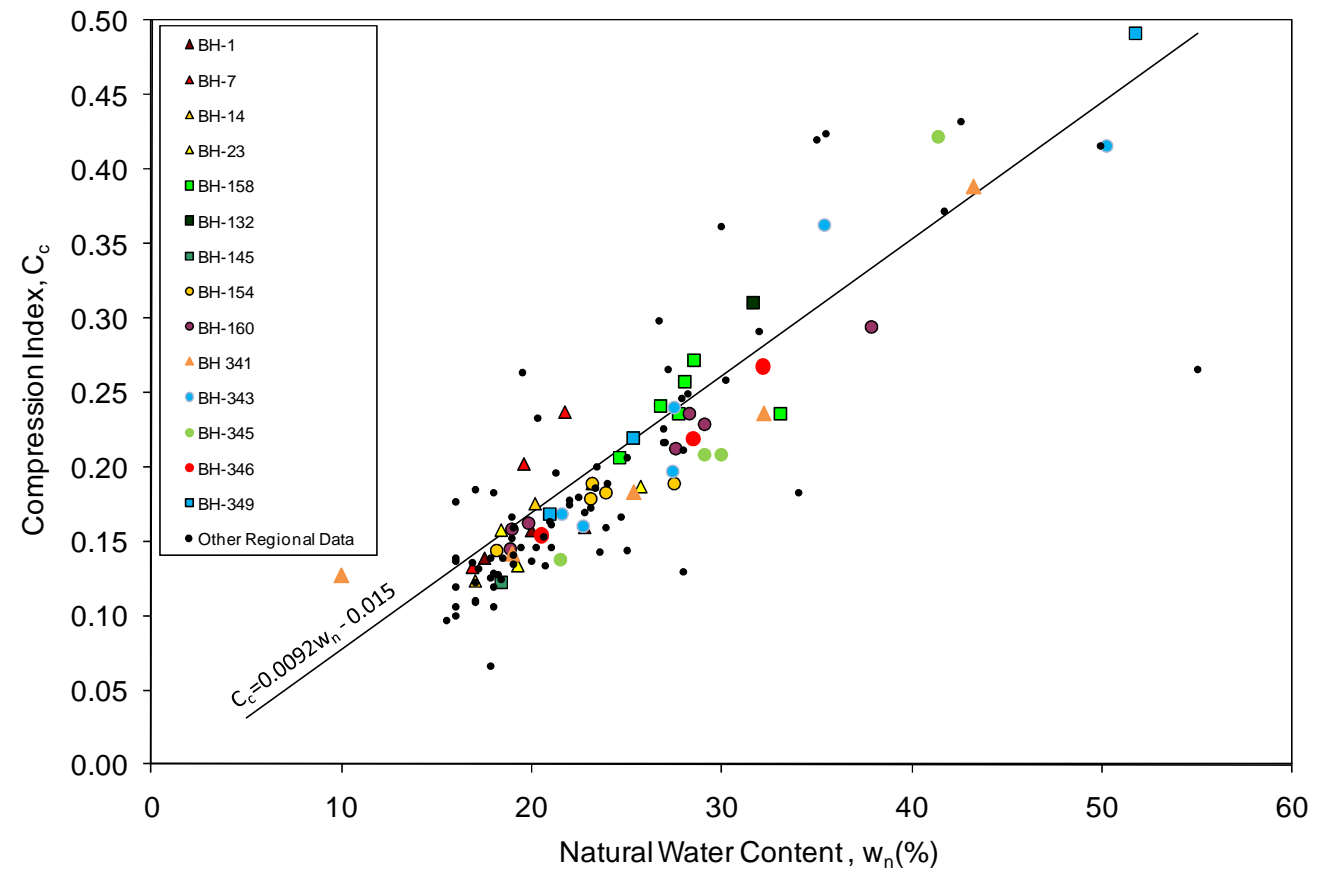
TITLE
DATA SUMMARY
PRECONSOLIDATION PRESSURES DERIVED
FROM CPT AND OEDOMETER TESTS



PROJECT No.	07-1130-207-0	FILE No.	0711302070R02065
DRAWN	MK	Nov. 18/10	SCALE AS SHOWN
CHECK			REV. 2


FIGURE 6.5

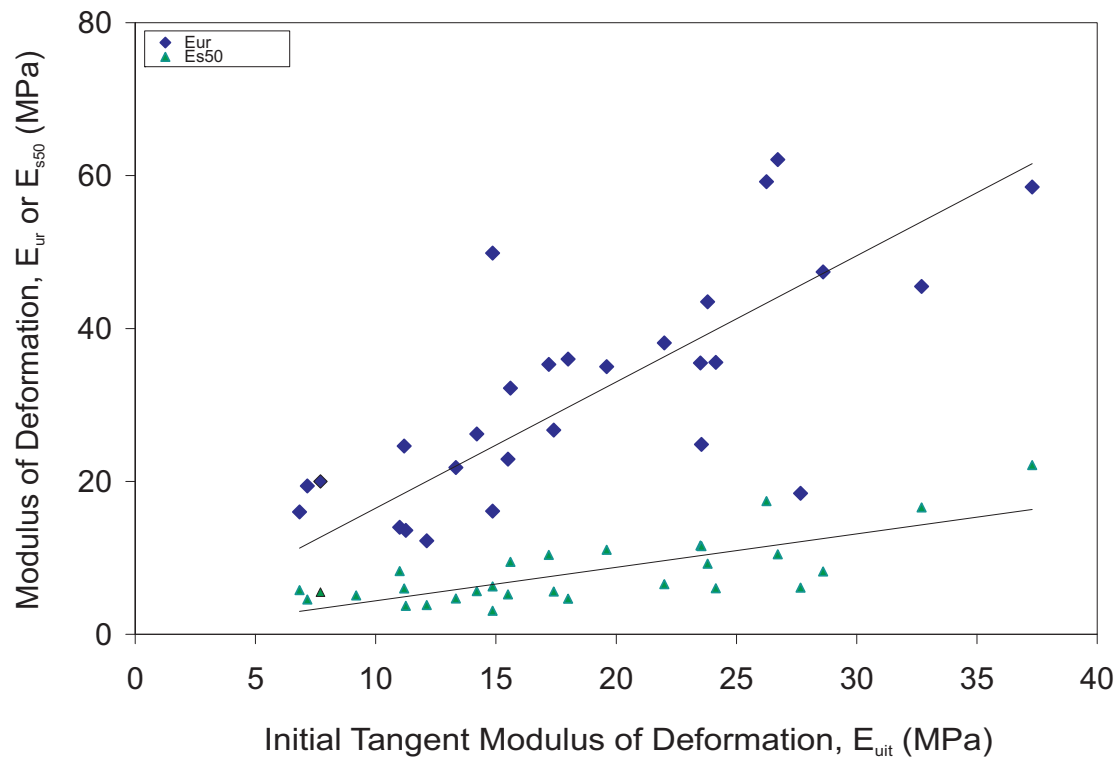
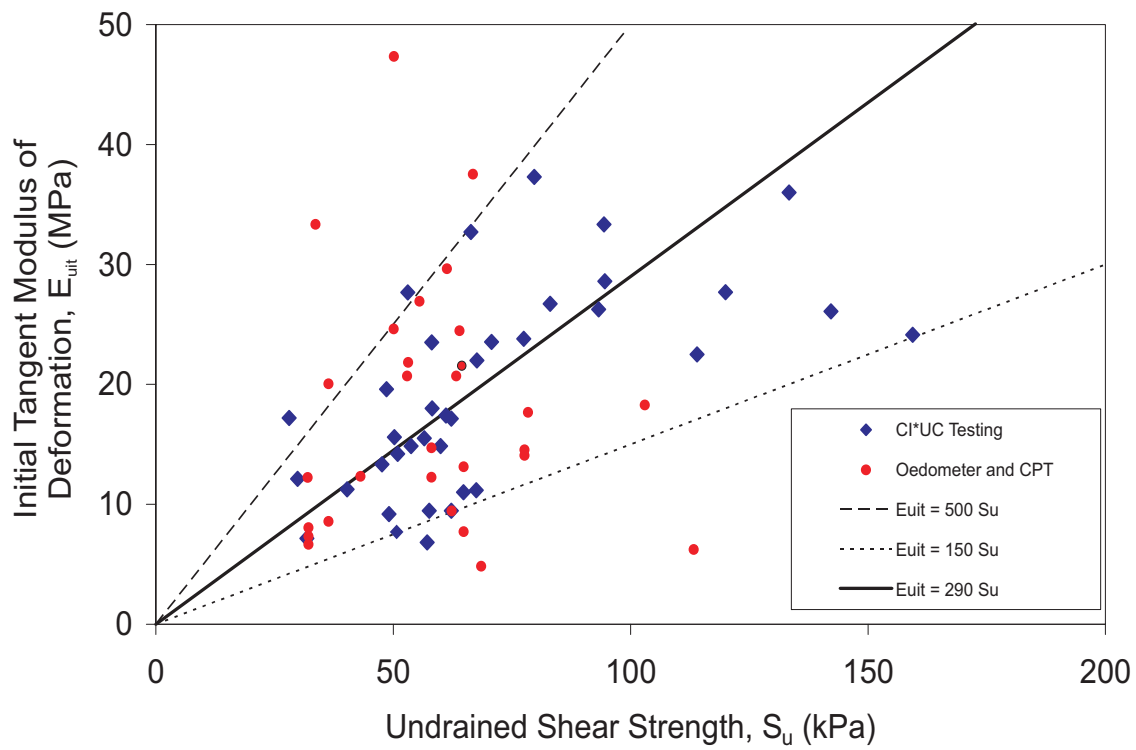
I:\active\2007\1130 - Geotechnical\1130-2000\07-1130-2070 - URS - DRIC APPROACH GSR - WINDSOR\Drafting\Grapher files\0711302070R02066.grf



NOTES


1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY COMPRESSION INDICES			
	PROJECT No. 0711302070		FILE No. 0711302070R02066
	DRAWN	MK	NOV. 16/10
	CHECK		
SCALE AS SHOWN			REV. 2
Figure 6.6			

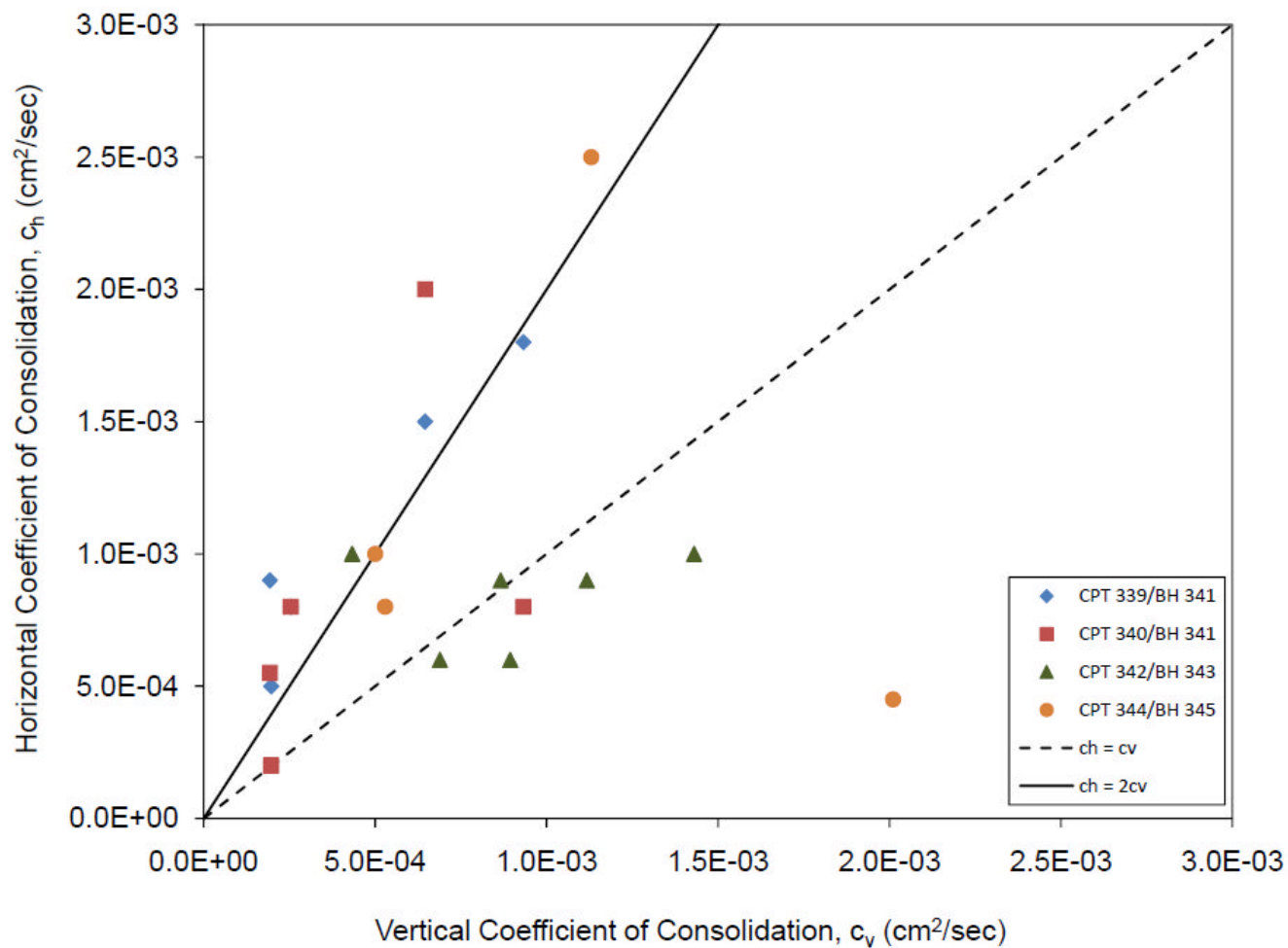


NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT		SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO	
TITLE			
DATA SUMMARY STRESS-STRAIN PARAMETERS			
	PROJECT No.		07-1130-207-0
	FILE No.		0711302070-R02068
	SCALE	AS SHOWN	REV. 2
	CADD	MK	Nov. 18/10
	CHECK		
			FIGURE 6.7





NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

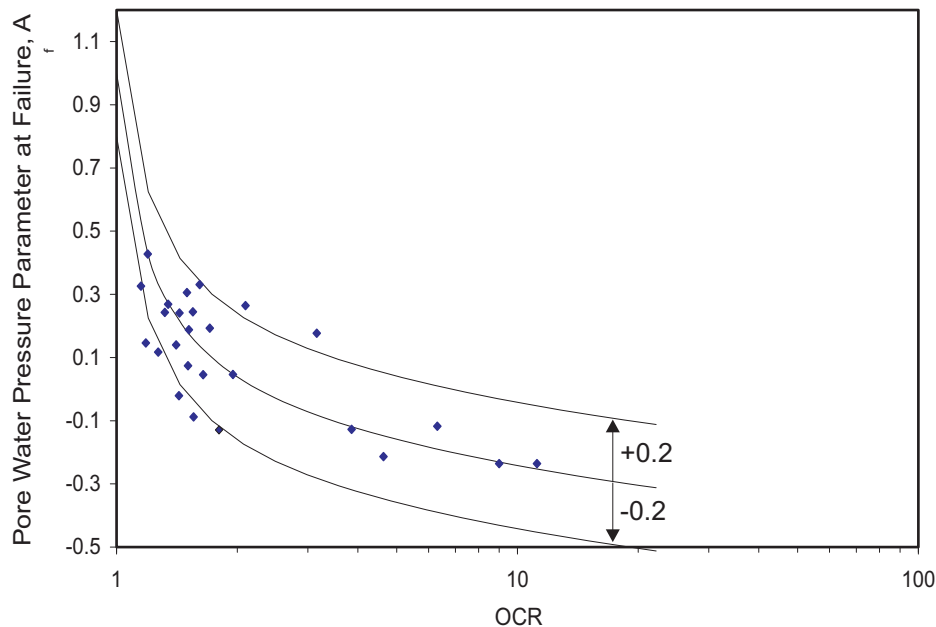
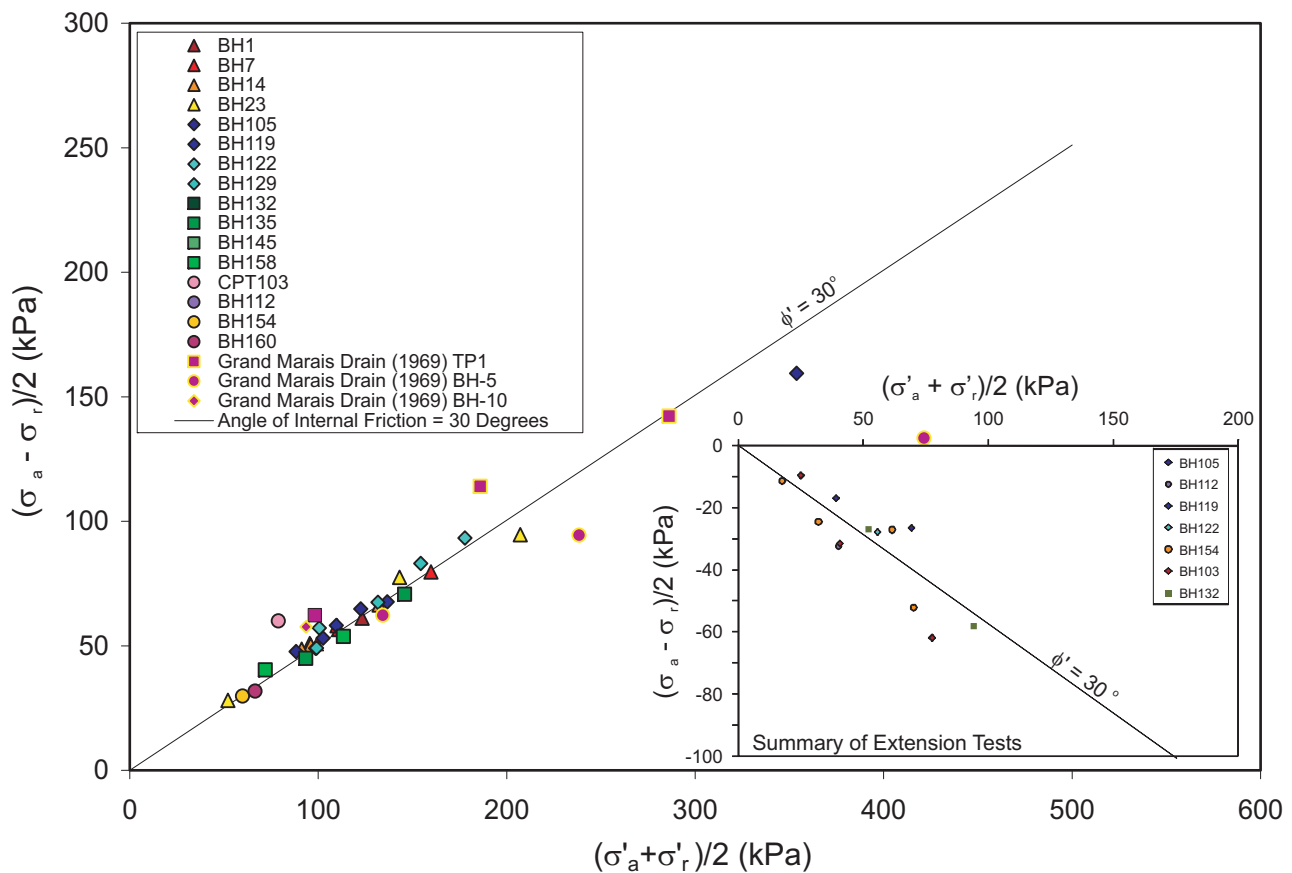
TITLE

DATA SUMMARY COEFFICIENTS OF CONSOLIDATION



PROJECT No.	07-1130-207-0	FILE No.	0711302070R02068
DRAWN	MK	Nov. 18/10	SCALE AS SHOWN
CHECK			REV. 0


FIGURE 6.8

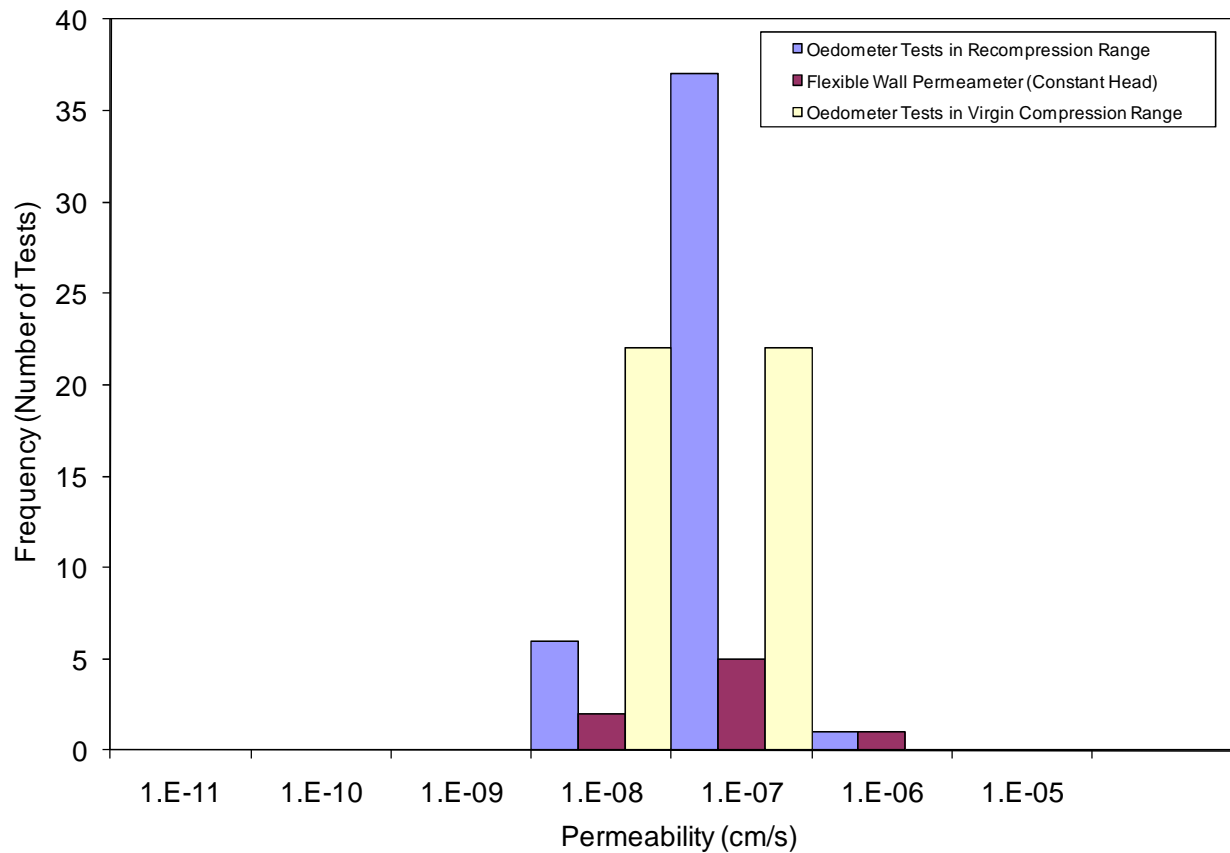


σ'_a = axial effective stress at failure
 σ'_r = radial effective confining stress at failure
 OCR = overconsolidation ratio
 A_f = pore water parameter at failure

NOTES


1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY EFFECTIVE STRESS STRENGTH PARAMETERS			
PROJECT No.	07-1130-207-0	FILE No.	0711302070-R02069
CADD	MK	Nov. 18/10	SCALE AS SHOWN REV. 2
CHECK			
			FIGURE 6.9

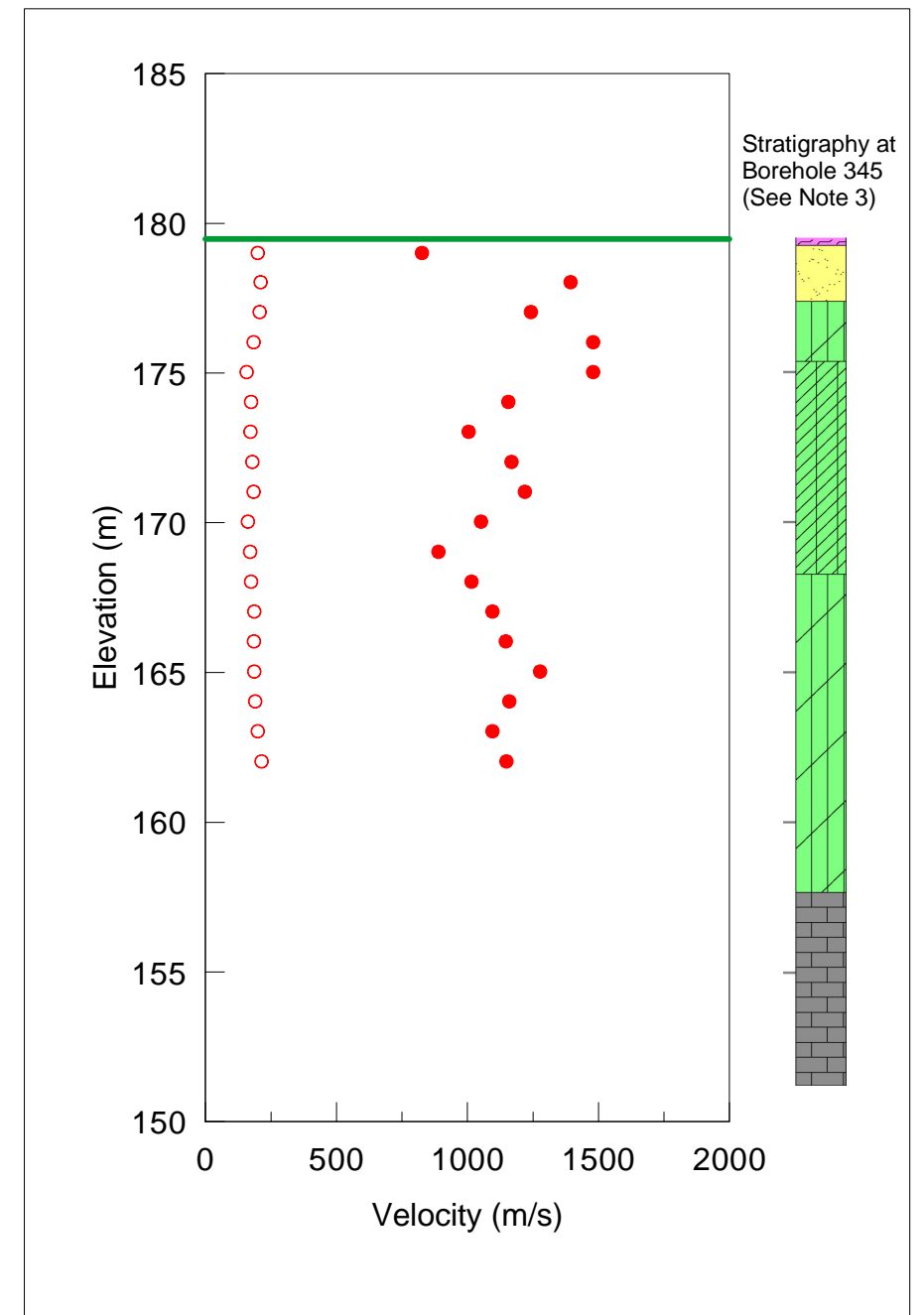
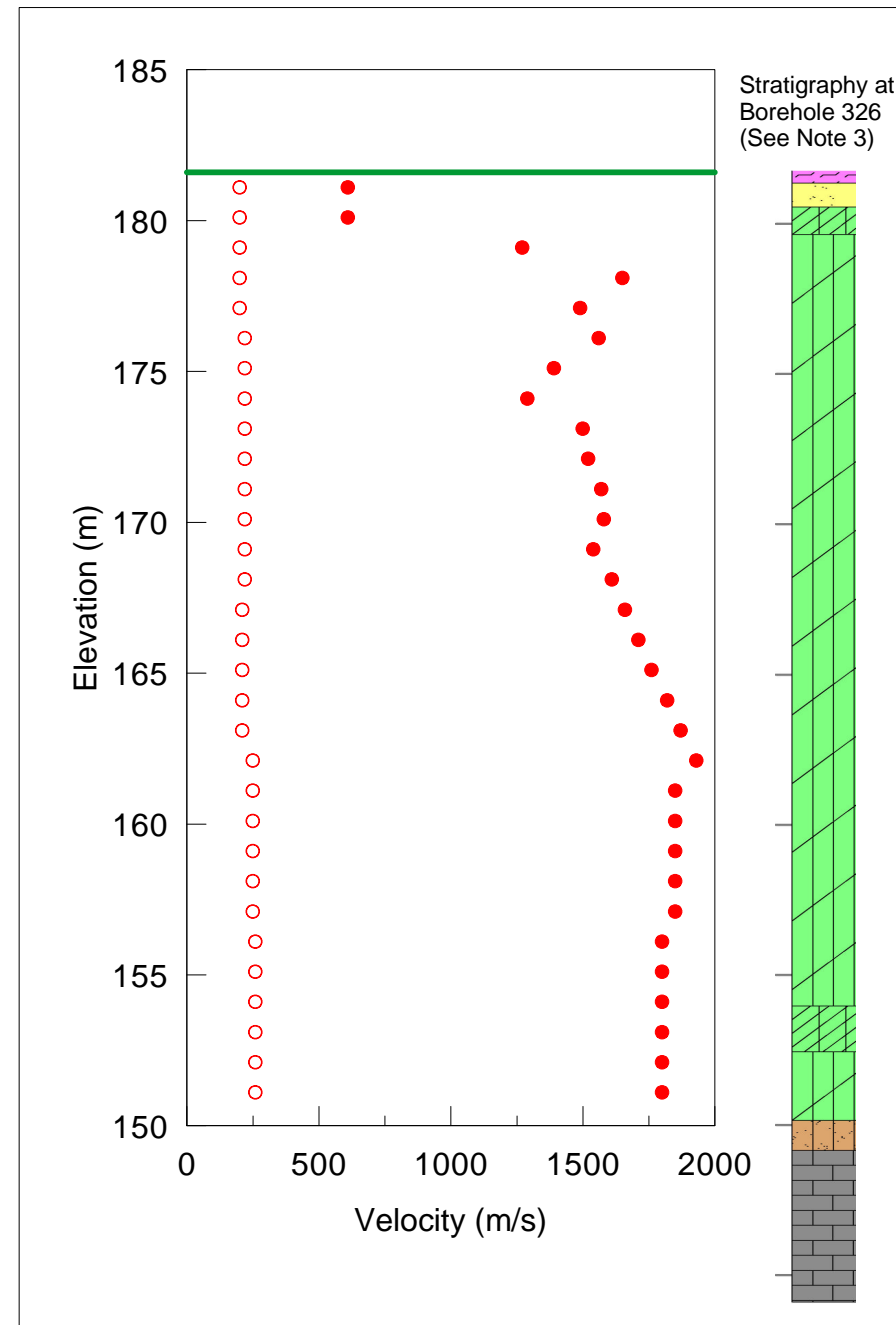
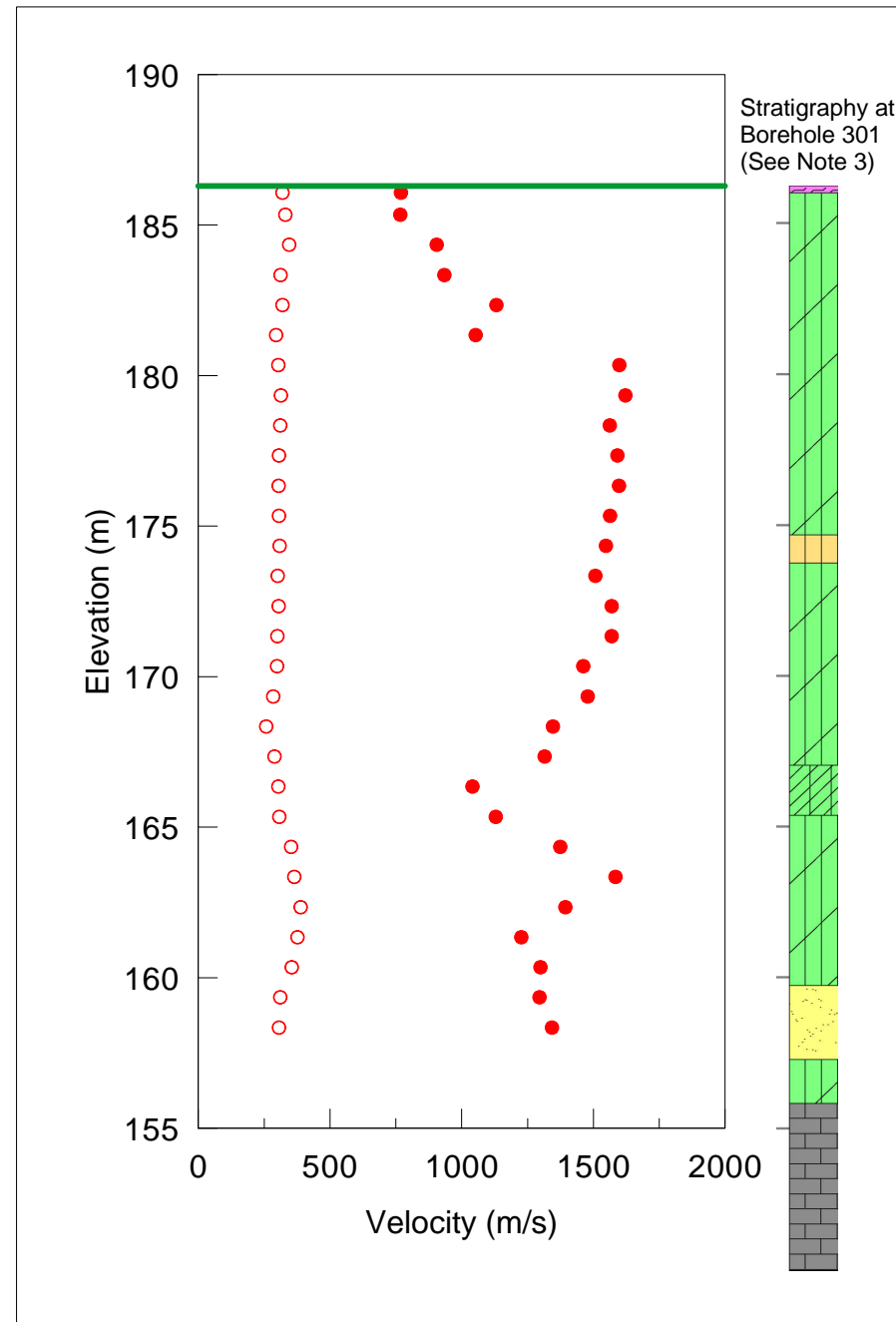


NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT			
SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE			
DATA SUMMARY MEASURED AND INFERRED PERMEABILITY FROM LABORATORY TESTS			
PROJECT No. 07-1130-207-0		FILE No. 0711302070R02610	
DRAWN MK Nov. 18/10		SCALE AS SHOWN REV. 2	
CHECK		FIGURE 6.10	
 Golder Associates LONDON, ONTARIO			

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LEGEND

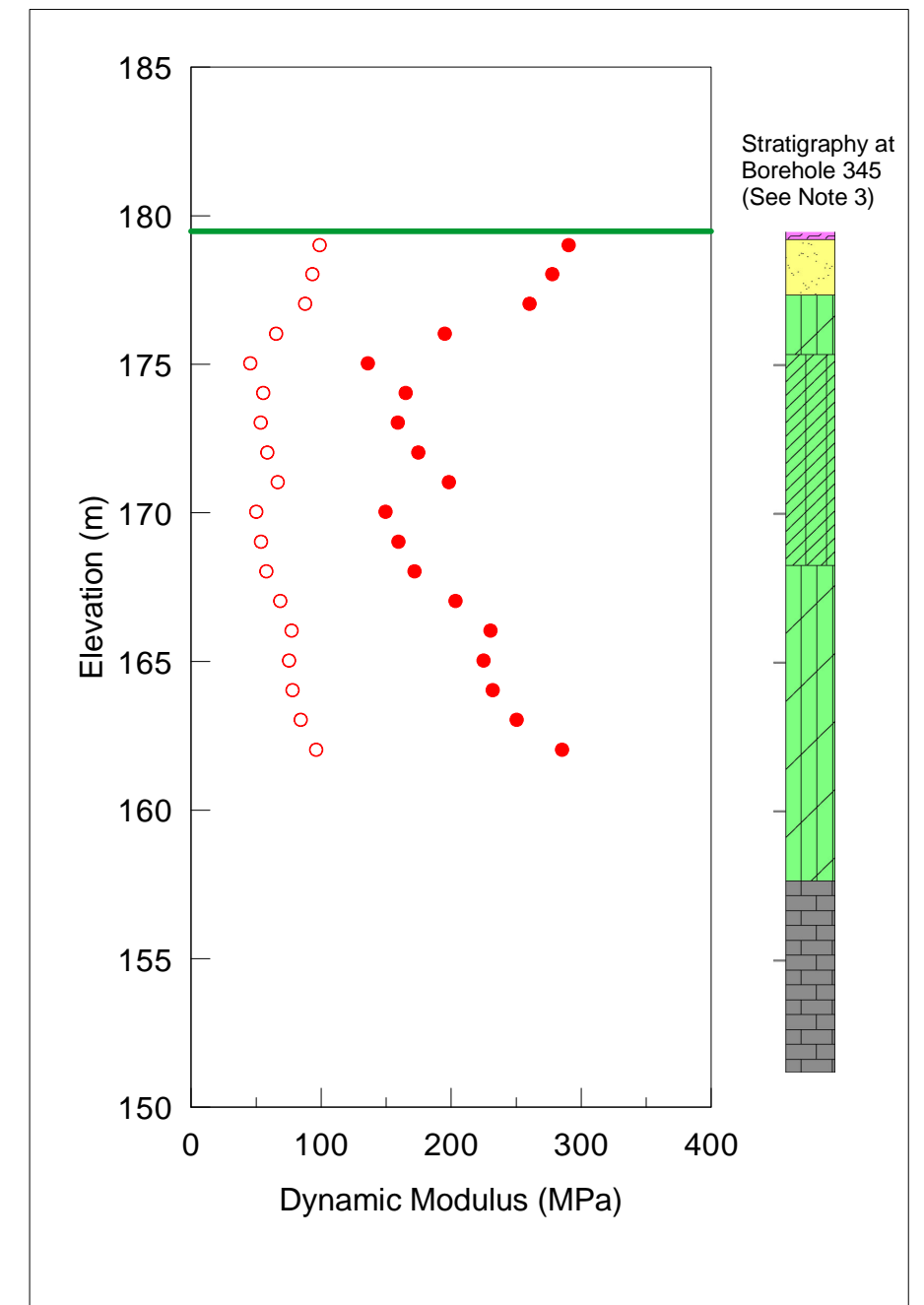
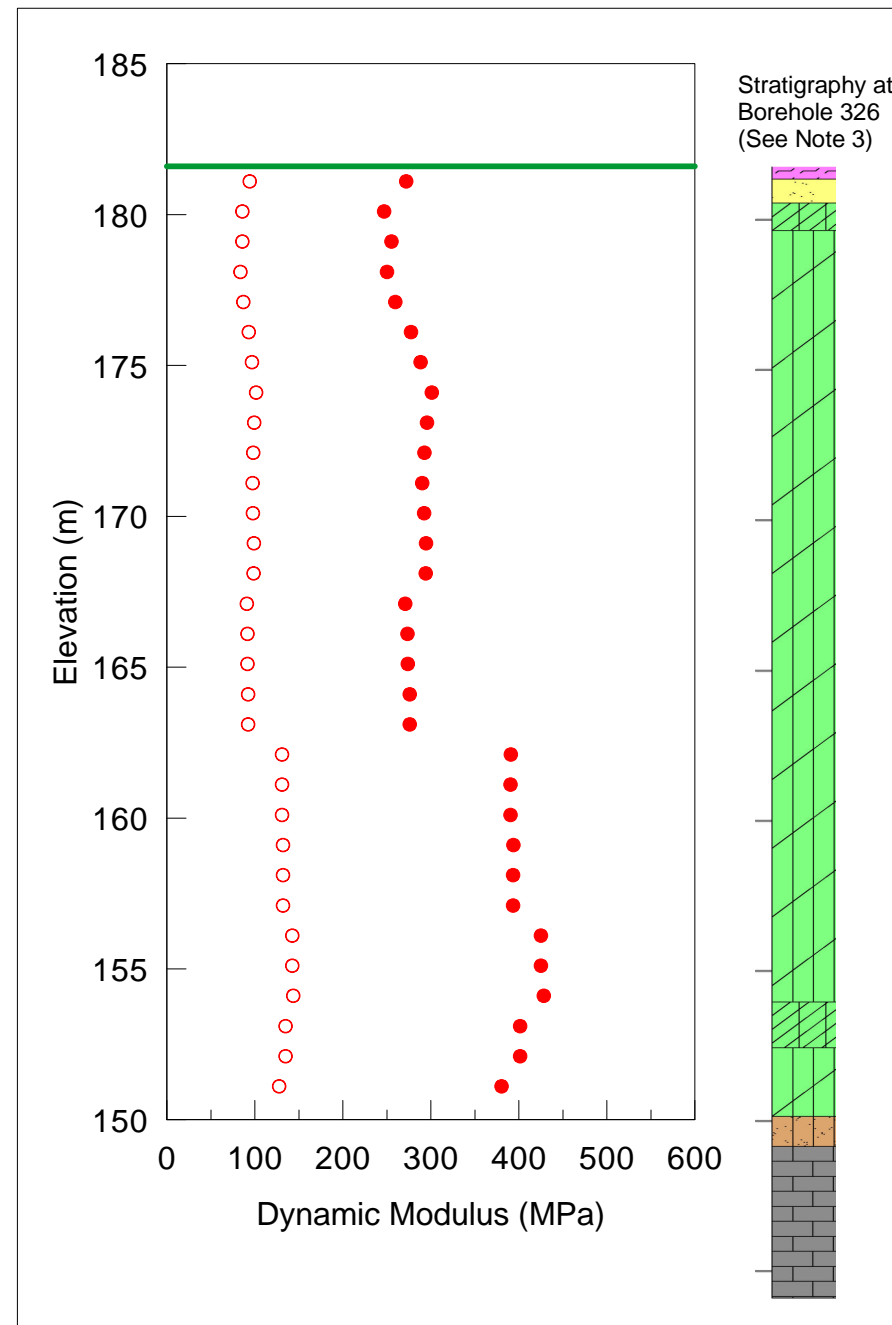
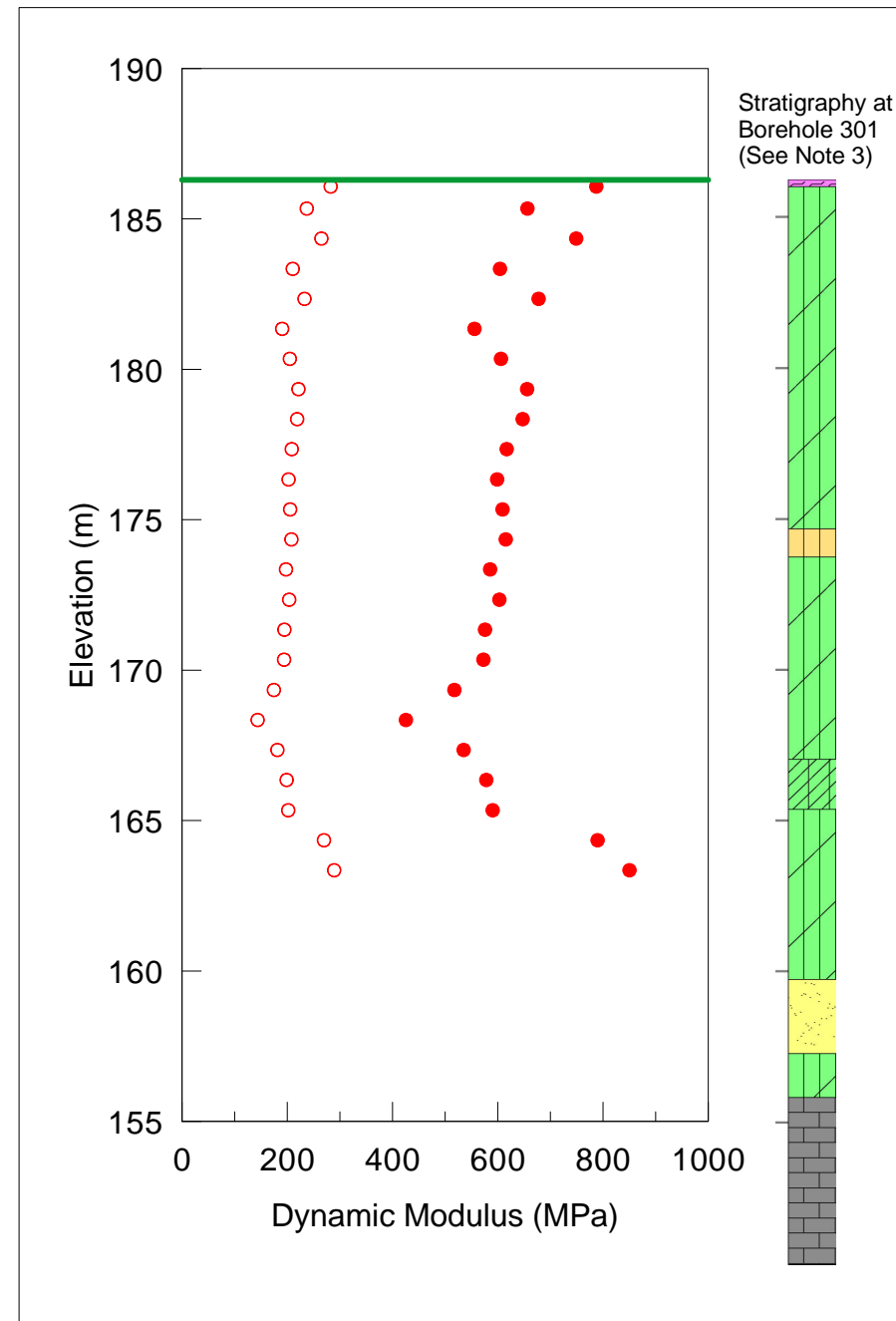
- Shear Wave Velocity
- Compression Wave Velocity
- Ground Surface

NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF SHEAR AND COMPRESSION WAVE VELOCITIES			
	PROJECT No. 0711302070		FILE No. 0711302070R02611
			SCALE AS SHOWN
	DRAWN MK NOV. 16/10		REV. 0
	CHECK		
Figure 6.11			

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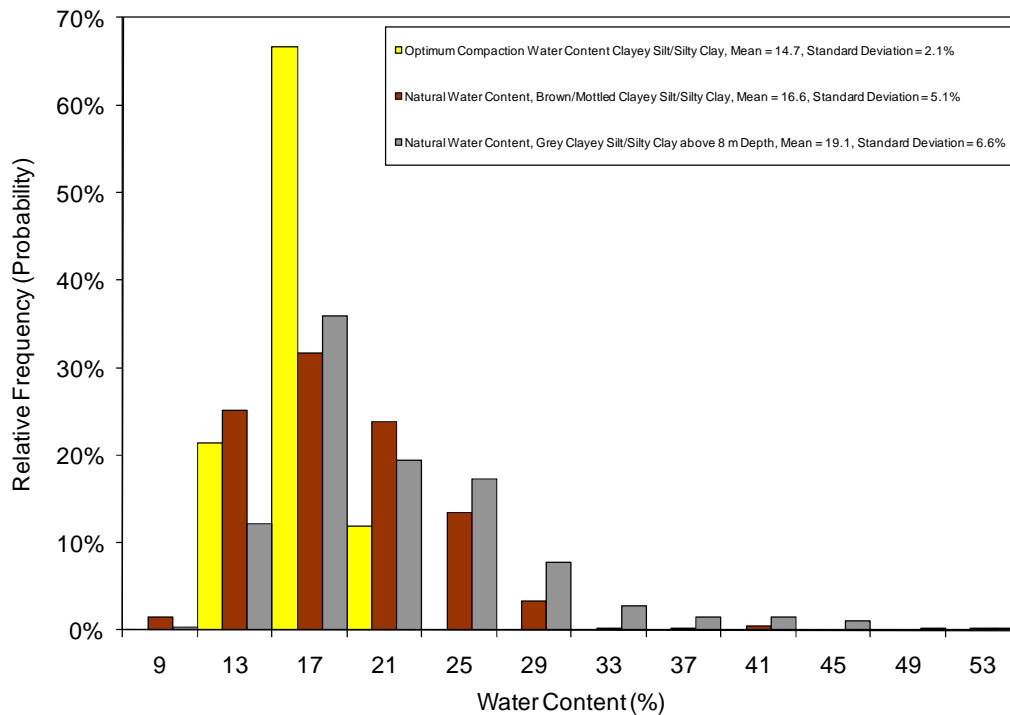
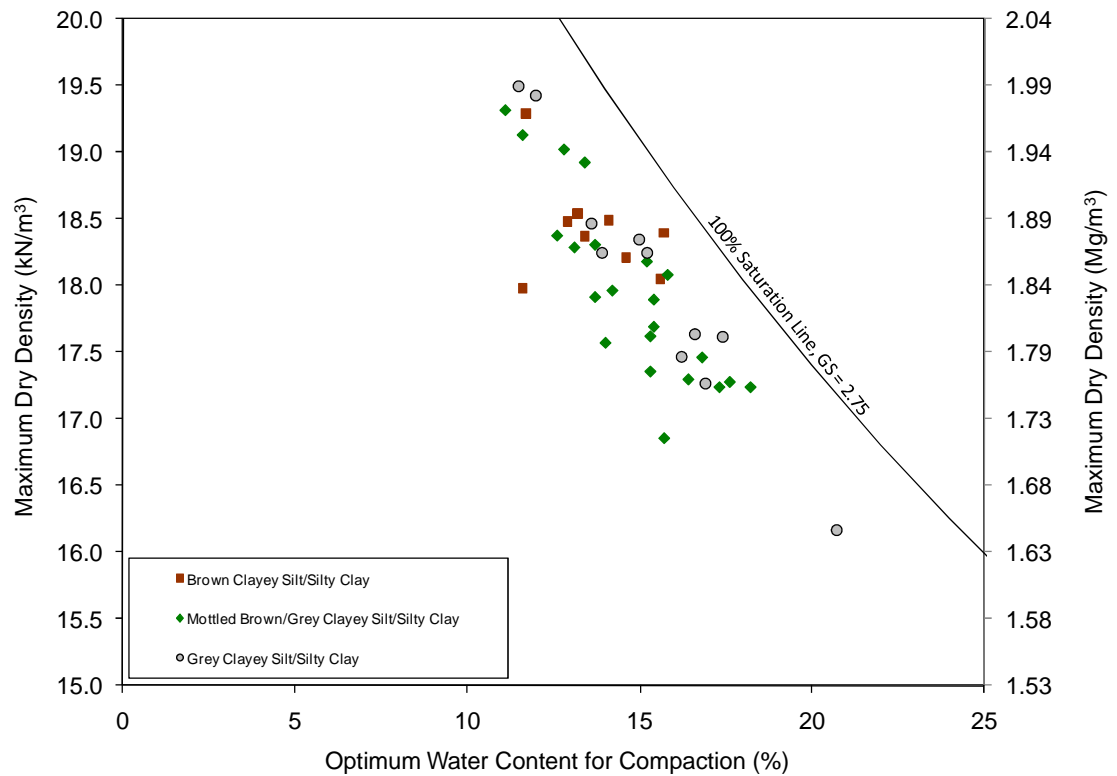
LEGEND

- Dynamic Shear Modulus, G_d
- Dynamic Deformation Modulus, E_d
- Ground Surface

NOTES


1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.
3. FOR LEGEND OF STRATIGRAPHY, REFER TO FIGURES 5.1A TO 5.1I.

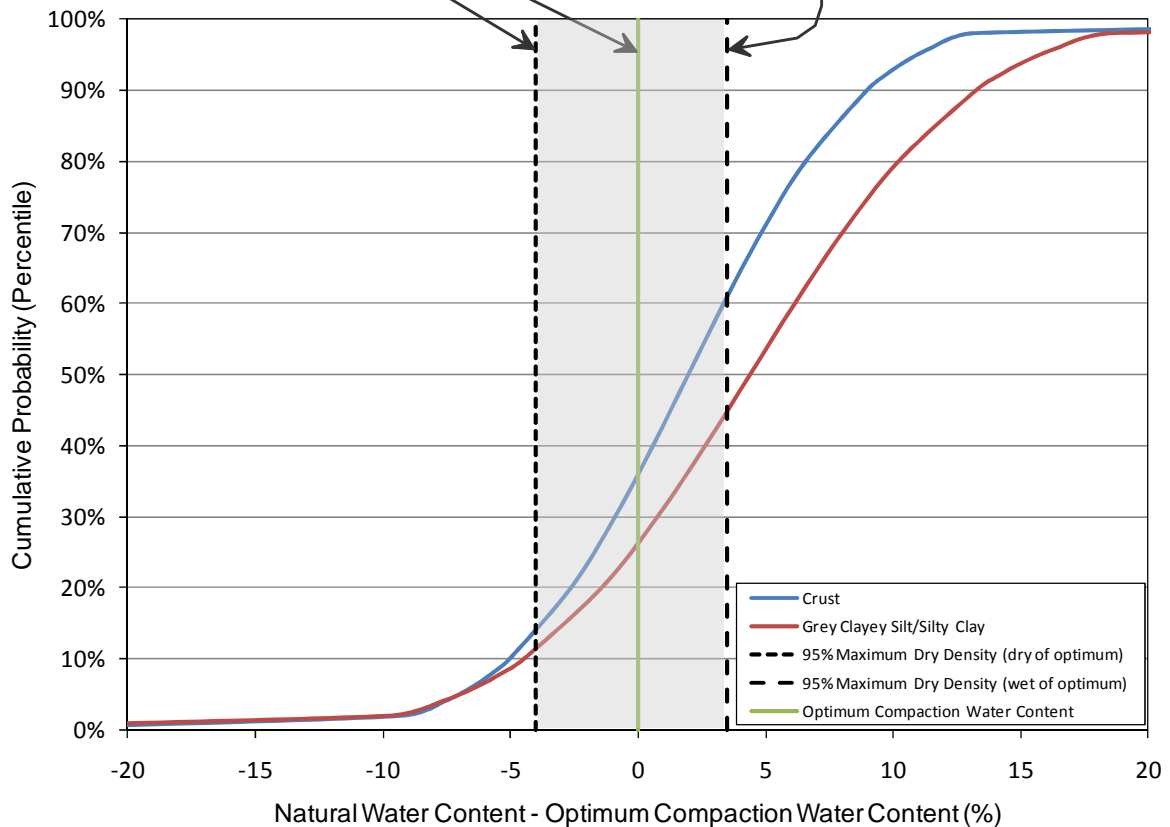
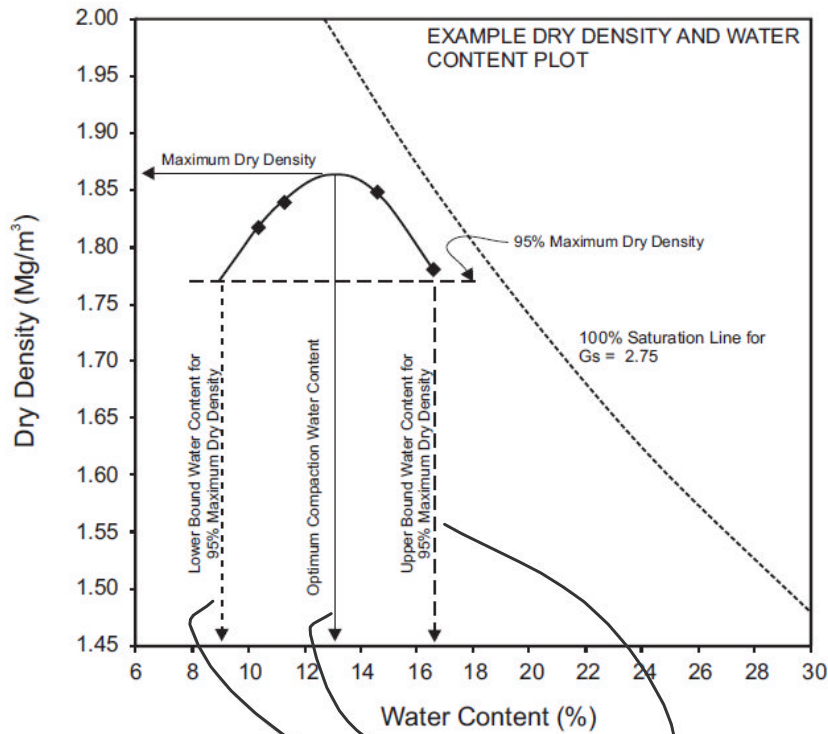
PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE SUMMARY OF DYNAMIC SOIL MODULI			
	PROJECT No. 0711302070		FILE No. 0711302070R02612
			SCALE AS SHOWN
	DRAWN MK NOV. 16/10		REV. 0
	CHECK		
Figure 6.12			



NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY MAXIMUM DRY DENSITY AND OPTIMUM COMPACTION WATER CONTENT			
 Golder Associates LONDON, ONTARIO	PROJECT No. 07-1130-207-0		FILE No. 0711302070R02613
			SCALE AS SHOWN REV. 2
	DRAWN MK Nov. 18/10	FIGURE 6.13	
	CHECK		



NOTES

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

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

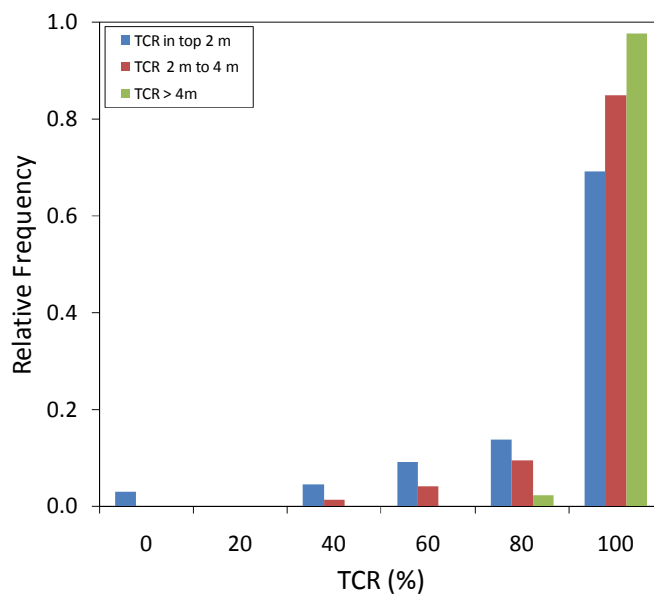
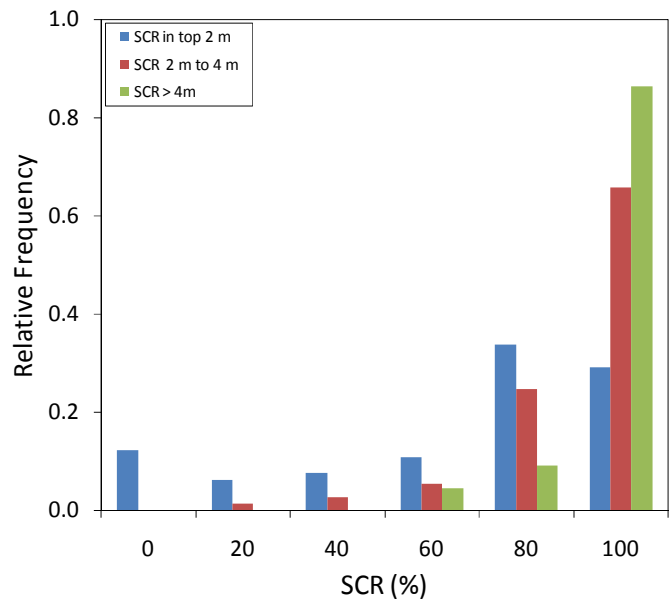
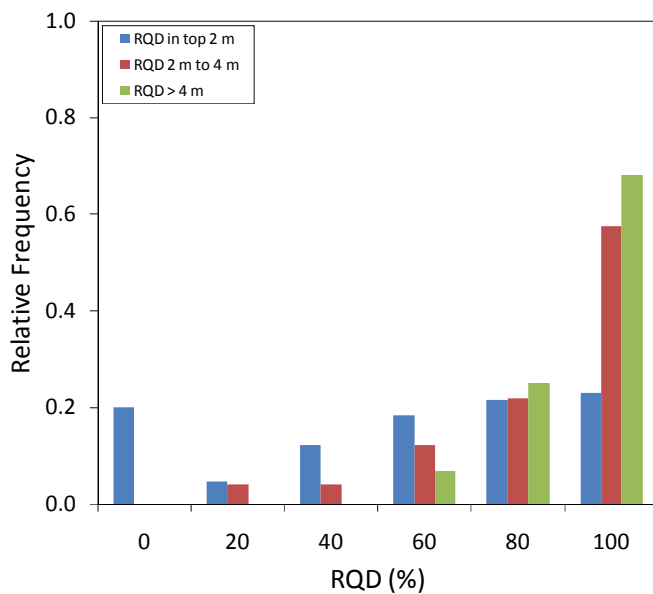
TITLE

DATA SUMMARY COMPARISON OF WATER CONTENT AND MAXIMUM DRY DENSITY DATA




PROJECT No.	07-1130-207-0	FILE No.	0711302070R02614
DRAWN	MK	Nov. 18/10	SCALE AS SHOWN
CHECK			REV. 2

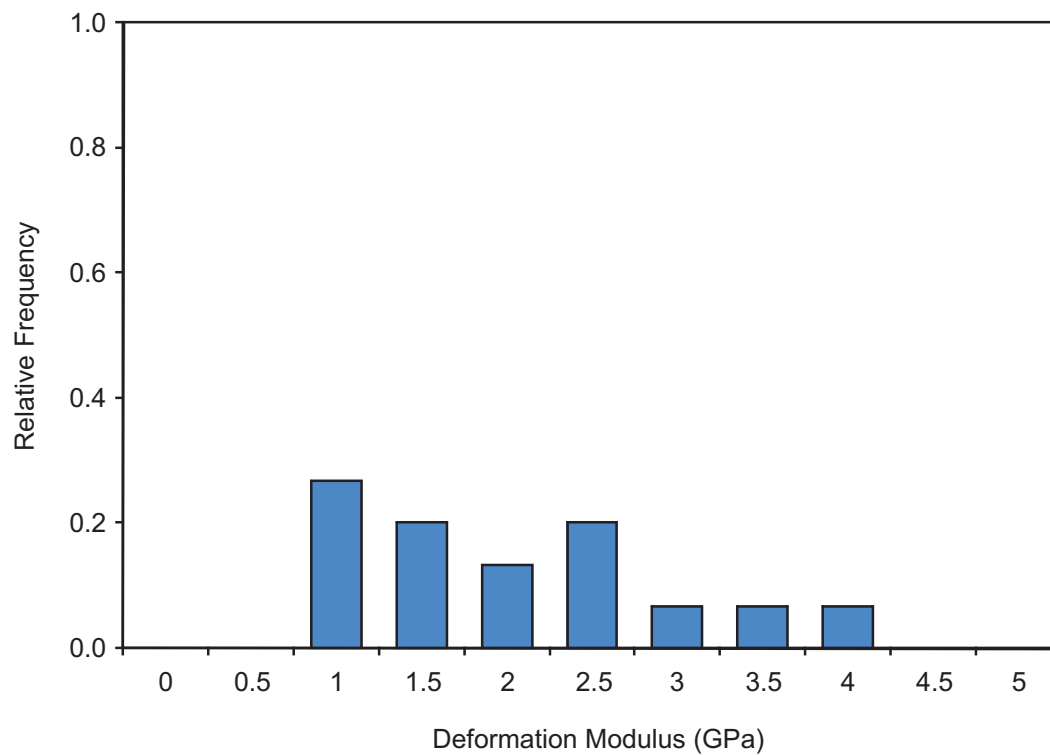
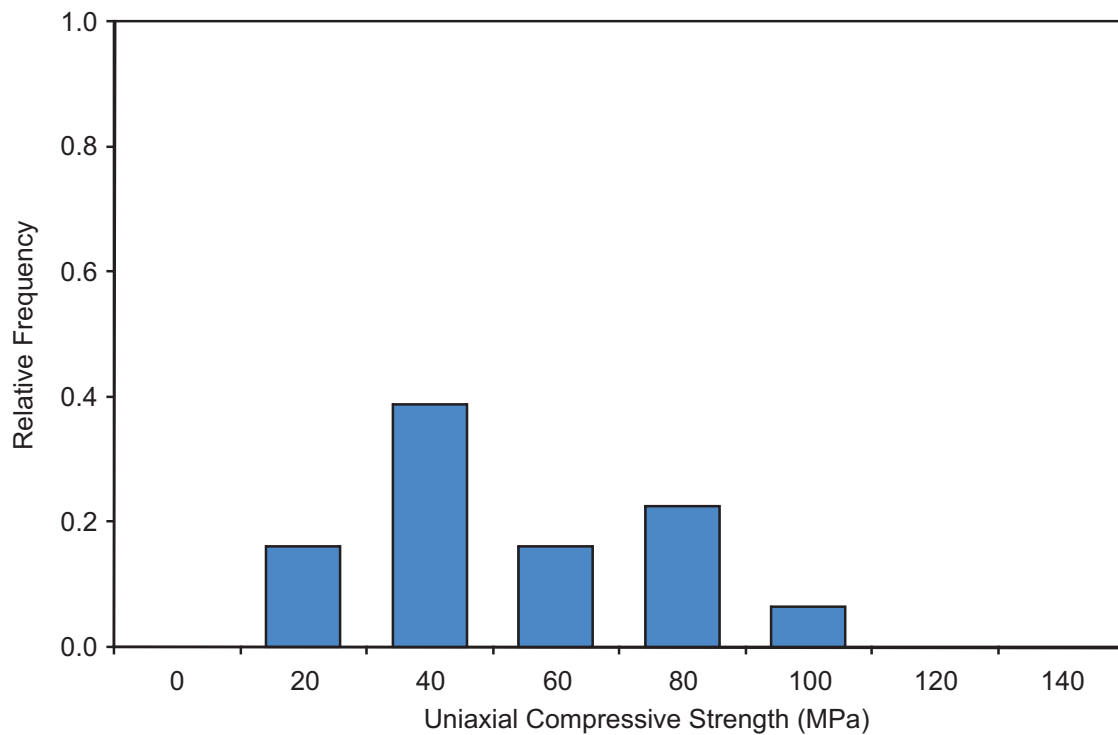
FIGURE 6.14



NOTES


1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY".
2. THIS FIGURE REPRESENTS A SUMMARY OF FIELD AND LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE.

PROJECT SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE DATA SUMMARY ROCK CORING			
	PROJECT No.	07-1130-207-0	FILE No. 0711302070R02615
	DRAWN	MK	Nov. 18/10
	CHECK		
			SCALE AS SHOWN REV. 2
FIGURE 6.15			



NOTES

1. THIS FIGURE IS TO BE READ WITH THE REPORT TITLED "SUBSURFACE CONDITIONS INTERPRETATION REPORT, WINDSOR-ESSEX PARKWAY"
2. THIS FIGURE REPRESENTS A SUMMARY OF LABORATORY TESTING DATA AS DISCUSSED IN THE TEXT OF THE REPORT REFERENCED ABOVE

PROJECT		SUBSURFACE CONDITIONS INTERPRETATION REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE					
DATA SUMMARY ROCK COMPRESSION TESTS					
		PROJECT No.		07-1130-207-0	
		CADD	MK	Nov. 18/10	
		CHECK			
		FILE No.		0711302070-R020613	
		SCALE		AS SHOWN	REV. 2
		FIGURE 6.16			



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