CITY OF BRAMPTON

Condition Assessment of the Churchville Floodwalls and Earth Dykes

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EXECUTIVE SUMMARY

KGS Group was retained by the City of Brampton to undertake a condition assessment of the Churchville flood protection barrier, including earth dykes and floodwalls (City of Brampton Project P2023-097). The purpose of the project is to determine the condition of the flood barrier and to develop an asset management strategy for the City's maintenance and management of the flood protection barrier.

The key findings of the study are summarized below.

Condition Assessment

Earth Dykes

- The overall condition of the earth dykes was satisfactory. There were no observed slope movements, significant depressions or erosion that would suggest significant concerns related to the slope stability and performance of the dykes.
- There were two (2) areas along the earth dykes where the crest elevation was found to be lower than its surrounding dyke component:
  - East of Churchville Road bridge – this area was partially reconstructed while installing a sanitary sewer in 2006.
  - Near the stormwater box culvert outlet – location of overtopping during the ice jam flood in 2022.
- The culverts and backflow prevention check valves through the earth dyke sections were in satisfactory condition without any significant deficiencies.
- Vegetation growth was dense at select earth dyke sections.

Concrete Floodwalls

- The concrete walls were in satisfactory condition with no movement or significant structural deficiencies.
- Localized exposed rebar was visible near the base of the floodwall at one area (near Ch. 0+060).
- Several larger/mature trees were observed growing near the concrete floodwall at localized areas.
- A wooden staircase located on private property is partially anchored into the concrete floodwall near 7772 Churchville Road. It is not clear that the concrete floodwall was originally designed to support the stair structure.
- A flap gate (OF_3984) at the southeast corner of the wall segment (Ch. 0+110) was partially obstructed with debris on the wet-side, which may prevent it from opening.

Geotechnical Investigation Results

A geotechnical investigation was completed to assess the earth dyke fill, floodwall backfill and foundation soils to support the condition assessment. The investigation program consisted of CPTu soundings, exploratory test holes and index laboratory testing to characterize the subsurface soils.

A summary of the materials observed during the investigation program is provided below.

- **Low Permeability Dyke Fill** – Firm to Stiff, Low to Intermediate Plasticity Silty Clay Fill. The dyke fill is suitable to retain water and has a low risk for piping/internal erosion.
- **Floodwall Backfill** – Compact Sand and Gravel Fill.
- **Foundation Soils –** Compact, Poorly Graded Sand and Gravel with Cobbles (immediately below the earth dykes). The sand and gravel was 0.8 to 1.6 m thick and was underlain by stiff to very stiff, low to intermediate plasticity Silty Clay Till. The sand and gravel foundation soil was found to be pervious.

Seepage and Stability Analyses

Earth Dykes

All analyzed dyke sections met the required Factor of Safety (FS) for piping/internal erosion for the sand and gravel foundation soil for the assumed flood conditions (0.3 m freeboard) and indicated a low risk of piping failure through the foundation soils during the flood event. For slope stability, the estimated FS for the analyzed loading cases meet the LRIA/CDA/USACE criteria under both normal and flood conditions.

Floodwall

The analyzed floodwall foundation is estimated to have a maximum horizontal seepage gradient of 0.42 m/m below the base of the wall, which resulted in a FS of 1.7 which is below the recommended FS of 3, assuming no low pervious blanket present at the wet-side of the floodwall. A relatively higher seepage quantity (0.63 gal/min, per metre length) was estimated at the floodwall section due to the pervious sand and gravel foundation soil and shorter seepage path.

The analyzed floodwall sections met the minimum required sliding factors of safety for all loading cases for the assumed flood conditions (0.3 m freeboard). The overturning criteria was also found to be acceptable. The stem wall, heel slab and toe slab of the floodwall have the required strength capacity to withstand the expected loads. Based on sensitivity analysis, the stem wall will still have sufficient strength after the failure of one rebar; however, it does not have adequate strength for the failure of two consecutive rebars.

Asset Management Strategy

A list of deficiencies for the earth dykes and floodwalls was developed based on the background document review, condition assessment results and analyses completed as part of this study. Recommendations were developed to address the deficiencies and establish the City's asset management strategy for the maintenance of the earth dykes and floodwalls. The deficiencies and the associated recommendations are listed in Table ES-1.

Expected Remaining Lifespan

The remaining lifespan of the Churchville flood barriers is dependent on the continued maintenance and care of the structures. Re-evaluation of the remaining lifespan of the structures should be carried out during future engineering studies.

- **Earth Dykes** – Generally, earth dykes can be relied on indefinitely provided they continue to meet the current stability criteria and that their overall conditions are kept satisfactory (i.e., vegetation growth is controlled, prompt repair of any damage caused by erosion or external factors such as human activities or extreme weather events, etc.). As the Churchville earth dykes were found to meet the stability criteria and were found to have a low risk of piping, the earth dykes are expected to continue to perform well in the foreseeable future provided they are properly maintained as recommended in ES-1.

- **Concrete Floodwalls** – Typically, concrete structures have an expected life between 70-90 years if there have been so significant changes to their design assumptions, however this is dependent on their overall condition and shorter/longer lifespans may be expected. Based on the as-found condition of the floodwalls during this study, the floodwalls are expected to continue to perform satisfactorily for the next 35-55 years (considering a construction date of 1989) provided they are properly maintained and repaired as necessary.

Recommendations are provided with the following Priority Ranking system:

High: Work that needs to be done to meet current regulations and safety requirements. Generally, it is the result of an identified deficiency and needs to be attended to within the next 2 years.

Medium: These deficiencies may include additional work that could improve safety or issues that may become deficiencies. These items should be addressed before the next formal condition assessment/study.

Low: These are opportunities for improvement. These issues are not currently considered to be urgent and can be scheduled at the City’s convenience.

TABLE ES-1: DEFICIENCIES AND RECOMMENDATIONS FOR THE CHURCHVILLE EARTH DYKES AND FLOODWALLS

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
1	The river levels for the flood events (100-year return period and regional flood event) considered as part of this study correspond to the original 1985 flood protection study. Segment 1 (earth dyke) does not have adequate freeboard (0.3m) for 100-year flood as recommended in the 1985 report (existing freeboard is 0.1 m). However, this location did not overtop during the ice jam in February 2022.	The flooding risk (water levels associated with annual probability) in Churchville should be further assessed both for open water (100-year up to 350-year return period) and for ice jam conditions. Evaluate the freeboard along the length of the earth dykes and floodwalls. If the freeboard deficiencies are found, consider raising the earth dyke and/or wall using suitable dyke fill material to accommodate 0.3m freeboard. Re-assess the seepage and stability analyses based on the revised hydraulic study.	Study (High)	$ 60,000	$ -	Stormwater Programs
2	There are two (2) locations where the top (crest) elevation of the earth dyke is lower than its surrounding parts. Ch. 0+282 (east of Churchville Road bridge) Ch. 0+420 (location of overtopping during 2022 flood)	Raise the earth dyke section using suitable dyke fill material. The repair should involve technical specification/design by an engineer, removal of the surficial topsoil/organic rich material, placement and compaction of new fill approved by a geotechnical engineer. Topographical surveys should be carried out before and after placement of new dyke fill to confirm crest elevation data.	Repair (Medium)	$ 15,000	$ 30,000	
3	A section of the earth dyke east of the Churchville Road bridge was removed to facilitate the construction of a sanitary sewer crossing the Credit River in 2006. There was no information available detailing the foundation preparation, dyke reconstruction materials, construction methodology, etc. A depression was observed at the crest of this dyke section which may be associated with settlement following reconstruction.	Complete a confirmatory site survey to locate the reconstructed section and carry out a drilling investigation including soil sampling to assess the dyke fill. SPT drilling and sampling is preferred to assess the soil consistency and fill quality.	Background Review / Investigation (High)	$ 10,000	$ 15,000	
4	A higher horizontal seepage gradient and relativity higher seepage quantities were estimated at the base of the floodwall under flood conditions, with the assumption that no-low pervious blanket is present at the wet-side toe of the wall.	Complete a geotechnical investigation at the wet-side of the floodwall to determine the characteristics and thickness of the pervious soil (sand and gravel — alluvium deposit). Shallow test holes, frequent sampling at several locations and lab testing should be carried out to assess the subsurface soils. Re-assess the seepage analysis with the updated geotechnical information.	Investigation / Study (High)	$ 25,000	$ 20,000	
		If no low-permeable soils are present, consider installation of a clay blanket to reduce hydraulic gradient and increase the seepage path below the floodwalls during flood conditions.	Repair (Medium)	$ 30,000	$ [1]120,000	

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
5	There are no agreements with private landowners or easements to carry out required maintenance activities of the floodproofing infrastructure on private properties.	Consider options for establishing access and maintenance responsibilities between the City and property owners, such as acquiring easements and/or establishing a maintenance agreement.	Study (Medium)	$ -	$ 180,000	
		Subtotal:		**$ 140,000**	**$ 365,000**	
Short Term Maintenance & Repairs						
6	Several larger/mature trees were observed growing in close proximity to the concrete floodwall, and vegetation growth was dense at select earth dyke sections.	Carry out tree clearing and brush vegetation overgrowth throughout the floodwall and earth dykes as necessary. Carry out regular mowing. Apply herbicide where required to prevent future overgrowth. Recommend clearing overgrowth 3m from toe of dykes and floodwall. Prior to any tree removal, arborist report must be obtained and a Tree Removal Application filed with the City. Obtain permission from private property owner if tree is not on public lands.	Maintenance	$ -	$ 8,000	Road Operations/Contract Services
		For substantially large tree removals, carry out the removal and restoration of the earthfill materials under the supervision of a geotechnical engineer.	Maintenance	$ -	$ 5,000	
7	At the floodwall near 7772 Churchville Road, some of the supports for the wooden stairs are anchored into the concrete floodwall. It is not clear that the concrete floodwall was originally designed to support the stairwell.	Remove or alter the stairs so as to be independent from the concrete wall.	Repair (Low)	$ -	$ 10,000	
8	There is exposed rebar near the base of the floodwall (wet-side of the wall near ground level) between properties 7780 and 7772 Churchville Rd. The rebar should have at least 2 inches (50mm) of concrete cover. Sensitivity analysis indicated that the wall does not have adequate strength for failure of two consecutive rebars.	Complete technical specification by engineer and carry out localized repair of the wall/rebar.	Repair (Medium)	$ -	$ 15,000	
		Subtotal:		**$ -**	**$ 38,000**	
Annual Inspections & Maintenance						

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
9	Flap gates and inline check valves require periodic inspections to ensure functionality is maintained.	Flap gates and inline check valves should be inspected at least annually and during/after ice jam and flood events.	Maintenance	$ -	$ -	Stormwater Programs
10	No standalone document exists for documenting the operations and maintenance requirements of the earth dykes and floodwalls.	Develop site-specific OMS procedures for the Churchville flood barrier. Ensure that all personnel responsible for dyke and floodwall surveillance/maintenance are trained in dyke safety and are able to recognize basic deficiencies that may lead to more serious safety issues.	Study (Medium)	$ 20,000	$ -	Capital Works Retaining Wall OSIM Inspections
			Subtotal:	**$ 20,000**	**$ -**	

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STATEMENT OF LIMITATIONS AND CONDITIONS

Limitations

This report has been prepared for The Corporation of the City of Brampton ("City of Brampton") in accordance with the agreement between KGS Group and City of Brampton (the "Agreement"). This report represents KGS Group's professional judgment and exercising due care consistent with the preparation of similar reports. The information, data, recommendations and conclusions in this report are subject to the constraints and limitations in the Agreement and the qualifications in this report. This report must be read as a whole, and sections or parts should not be read out of context.

This report is based on information made available to KGS Group by City of Brampton. Unless stated otherwise, KGS Group has not verified the accuracy, completeness or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith. KGS Group shall not be responsible for conditions/issues it was not authorized or able to investigate or which were beyond the scope of its work. The information and conclusions provided in this report apply only as they existed at the time of KGS Group's work.

Third Party Use of Report

Any use a third party makes of this report or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions undertaken based on this report.

Geotechnical Investigation Statement of Limitations

The geotechnical investigation findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. The findings and recommendations are based on the results of field and laboratory investigations, combined with an interpolation of soil and groundwater conditions found at and within the depth of the test holes drilled by KGS Group at the site at the time of drilling. If conditions encountered during construction appear to be different from those shown by the test holes drilled by KGS Group or if the assumptions stated herein are not in keeping with the design, KGS Group should be notified in order that the recommendations can be reviewed and modified if necessary.

Capital Cost Estimate Statement of Limitations

The cost estimates included with this report have been prepared by KGS Group using its professional judgment and exercising due care consistent with the level of detail required for the stage of the project for which the estimate has been developed. These estimates represent KGS Group's opinion of the probable costs and are based on factors over which KGS Group has no control. These factors include, without limitation, site conditions, availability of qualified labour and materials, present workload of the bidders at the time of tendering and overall market conditions. KGS Group does not assume any responsibility to City of Brampton, in contract, tort or otherwise in connection with such estimates and shall not be liable to City of Brampton if such estimates prove to be inaccurate or incorrect.

1.0 INTRODUCTION

1.1 Scope of Work and Objectives

KGS Group was retained by the City of Brampton to undertake a condition assessment of the Churchville flood protection barrier including earth dykes and floodwalls (City of Brampton Project P2023-097). The purpose of the assignment is to determine the condition of the flood barrier and to develop an asset management strategy for the City's maintenance and management of the flood protection barrier.

The scope of work summarized in this report is as follows:

- Review the available background information and historical drawings.
- Complete a visual condition assessment of the concrete floodwalls and earth dykes.
- Complete a limited topographical survey of the concrete floodwall and earth dykes.
- Complete geotechnical drilling programs and soil laboratory testing to support the engineering assessment.
- Carry out engineering analyses to assess the performance of the structures, including stability of the concrete cantilever floodwalls and slope stability and seepage analyses of the earth dykes and foundation soils.
- Develop an asset management strategy for the flood protection barrier.

1.2 Scope of Work Exclusions

- Hydrologic and hydraulic and/or river ice jam analyses

1.3 Acknowledgements

The following City of Brampton personnel contributed to the successful implementation of the field investigation program and engineering assessment:

- Kevin Thavarajah, P.Eng., PMP – Project Manager, Stormwater Infrastructure Engineer
- Olivia Sparrow, P.Eng. – Manager, Stormwater Programs

1.4 Background Reports Assessed

The following reports have been reviewed as part of the study:

- Churchville Flood Control Preliminary Engineering Study (Philips Planning Engineering Limited, 1985)
- Preliminary Feasibility Study for Alleviating Ice Jams and Associated Flooding along the Credit River (Credit Valley Conservation, 2015).
- February 2022 Churchville Flood Event: Technical Debrief (Credit Valley Conservation, 2022)
- A Soil Investigation for Proposed Residential Development (Soil-Eng Limited, 2001)
- Preliminary Servicing and Stormwater Management Report (Rand Engineering Corporation, 2002)
- Application for PTTW and Dewatering Plan Report (MacViro, 2006)

1.5 Historical Drawing Review

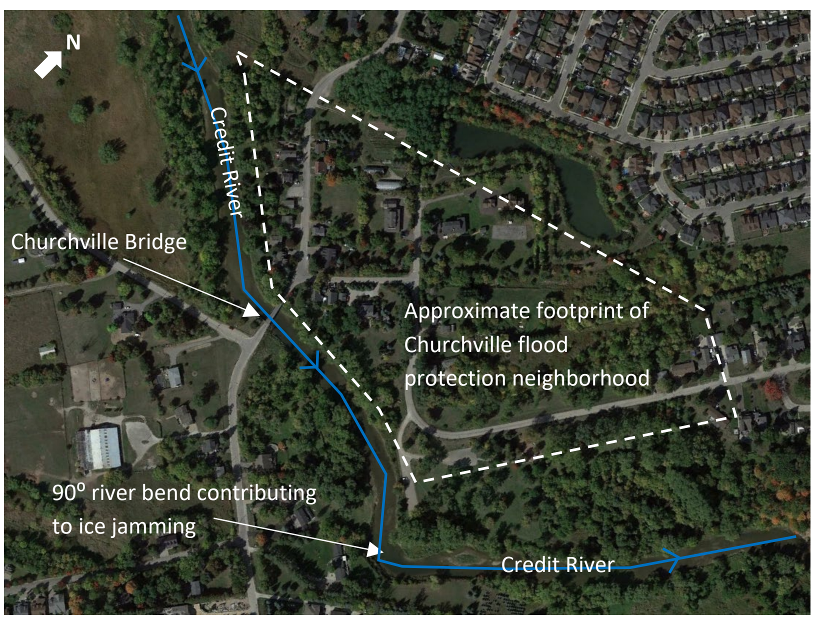
The following As-Constructed drawings (Dec 1989) have been reviewed as part of the study:

- Churchville Flood Control (DWG Nos: Sheet 1 to 19, Philips Planning Engineering Limited, 1989)
- Planning, Design and Development (DWG Nos: 35027-D to 35029-D, City of Brampton, 2006)
- Trunk Sanitary Sewer (DWG Nos: 40379-D, 40383-D, 40384-D, 40385-D, and 40391-D, Region of Peel, 2007)
- Paradise Homes Mahogany Inc., Storm Sewer (DWG Nos. D5-1-12, D5-3-6, D5-17-1, D5-17-2, D5-8-15)
- Region of Peel Public Works (DWG No. 23638-D)

1.6 Background

The Churchville neighborhood is located within the Credit River Floodplain in Brampton, between Steeles Avenue West and Hwy 407. It has a long history of flooding, including ice jam flood events and open water flood events, including a significant flood in April 1950 where the river level peaked at approximately 3.7m (12 ft.) above the normal river level. There is a constricted flow capacity below the bridge on Churchville Road during flood events based on the 1985 Philips engineering study. The geometry of the Credit River features a 90-degree bend, approximately 270 m downstream of the bridge on Churchville Road, followed by another 90-degree bend 400 m downstream (see Figure 1-1). These conditions make the river in this area prone to ice jam formation as history has confirmed.

FIGURE 1-1: AERIAL VIEW OF STUDY AREA



A flood control project was approved by the Credit Valley Conservation (CVC) board in 1983 and was completed in 1989. In the interim, a major ice jam occurred in March of 1987. No severe ice jam floods were recorded in Churchville between 1987 and 2022. In February of 2022, a breakup ice jam caused extensive flooding and flooded 22 homes. In response, 100 homes in the neighbourhood were asked to evacuate.

The existing flood protection infrastructure for the Churchville neighborhood consists of a “linked” flood barrier that includes a combination of earth dykes and concrete floodwalls. There are two (2) storm sewer outfalls through the dyke sections at Ch. 0+276 (equipped with an inline check valve, east of the bridge at Churchville Road) and Ch. 0+430 (double-box culvert, near corner of Martins Blvd). The double box culvert storm sewer outfall was constructed at a later time after the original dyke and floodwall construction (constructed in 2006 based on the available background drawings). In addition, there are several cross-culvert outfalls through the dyke and floodwall with flap gates for backflow prevention at low points to maintain the local drainage outfalls to the credit river. The top of the flood protection decreases in elevation from upstream to downstream of the flood reach to follow the profile of the design flood elevations (elevation ranges from El. 173.3 m to El. 172 m). A detailed layout of the flood protection infrastructure is provided in Appendix A.

Photo 1-1: Churchville Road Bridge following February 2022 Flood Event

A memorandum prepared by CVC after the February 2022 flood event indicated three potential major causes for the 2022 flood: the legacy of floodplain policies (and the presence of buildings in the floodplain), the breakup ice jam that formed at the first bend downstream of the Churchville Road Bridge, and backflow through select storm sewer outfalls through the earth dyke system. The post-event assessment indicated that while the flows would have been in the range of a 10-year return period, the water levels, affected by the ice jam, corresponded to those greater than a 100-year open water event. The dyke, designed for an event of that magnitude (100-year open water), was overtopped at one location.

The 1985 study determined a 100-year open water flood protection (including 0.3 m of freeboard) provided the best cost/benefit. A portion of the flood control project is located on public lands with the remaining portion located on private property. There are no agreements with private landowners or easements to carry out any maintenance activities of the floodproofing infrastructure on private property.

The original flood protection design provided in the 1985 study is slightly different from what was actually constructed at the site in 1989:

- The 1985 study indicated an earth dyke concept design on the south of Martins Blvd., rather than the as-constructed integrated road/dyke structure. The current integrated road/dyke structure may have

more flood conveyance in this section than what was estimated in the 1985 concept due to the resulting increased flood space after moving the earth dyke further in-land (away from the river).
- The 1985 earth dyke concept is shown wrapping around the residential property at 7742 Churchville Road. The existing configuration consists of the concrete floodwall tied into either side of the house, with the floodwalls connected to the house foundation walls. There are no records detailing why the as-constructed conditions of the floodwalls and earth dykes differ from those shown in the 1985 study. However, the constructed floodwalls and earth dykes generally conform to the approved design approach.
- The chainages shown in the 1985 study are different from those shown in the 1989 as-constructed drawings. It's expected the chainages were revised upon completion of the construction.

Hydraulic modelling of the Credit River through the Churchville reach was completed as part of the 1985 preliminary engineering study for the flood proofing. The overall reach was surveyed, and water surface elevation data was obtained for several cross sections under the original floodplain conditions. Table 1-1 summarized the expected river level elevations for the 100-year and the regional flood events (elevations do not include the 0.3 m freeboard protection). It is understood that elevations provided in the 1985 study report are geodetic and they are referenced to CGVD 1928 vertical datum.

TABLE 1-1: 1985 WATER ELEVATION DATA

Chainage[1]	100 Year Flood[2]	Regional Flood[2]
0+000	173.13	173.60
0+230	173.02	173.45
0+260	172.88	173.30
0+300	171.97	172.54
0+500	171.76	172.37
0+600	171.61	172.23

Notes:
[1] Chainages are based on those shown in 1989 as-constructed drawings and are approximated based on the figure provided in the 1985 report.
[2] Elevations are expected to be geodetic based on CGVD28 datum.

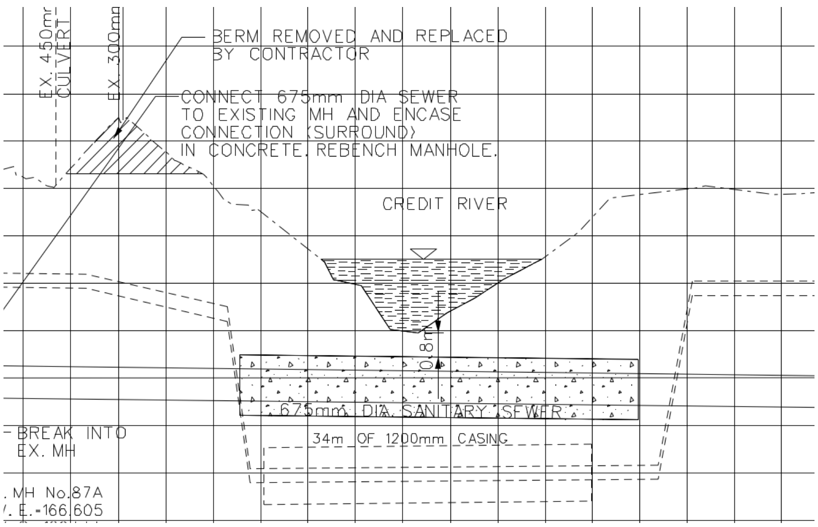
No subsequent hydrologic analyses have been completed of the Credit River since 1985. However, two hydraulic analyses have been completed since 1985 for this reach as follows:

- CVC's Environmental Water Resources Group (2007) completed hydraulic modeling and Regulatory floodplain mapping for the Credit River
- CVC (NDMP Intake No.4, 2019) also completed flood risk analysis for the Credit River, using the 2008 hydraulic modeling but with new LiDAR elevation data

Earth Dyke Section Reconstruction (Sanitary Sewer Crossing)

Based on background drawings provided by Peel Region, a section of the earth dyke east of the Churchville Road bridge was removed to facilitate the construction of a sanitary sewer crossing the Credit River in 2006 (Region of Peel Drawing #: 40379-D). The drawing indicated that the earth dyke was excavated and replaced during construction as shown in Figure 1-2. However, there was no information available detailing the foundation preparation, dyke reconstruction materials, construction methodology, etc.

FIGURE 1-2: 2006 SANITARY SEWER CONSTRUCTION ACROSS THE CREDIT RIVER



Earth Dyke Section Reconstruction (Storm Sewer Outlet)

Based on the as-built drawings, the reconstruction of the earth dyke section at the storm sewer outlet included installation of a concrete anti-seepage collar built around the culvert pipes within the earth dyke to prevent piping between the concrete-earthfill interface, as well as armour stone protection at the outlet. The dyke fill was reconstructed using silty clay fill compacted in 200mm lifts to 98% Standard Proctor Maximum Dry Density (SPMDD) under supervision of a geotechnical engineer (see Figures 1-3 and 1-4).

FIGURE 1-3: EARTH DYKE SECTION AT STORM SEWER OUTLET (1 OF 2)

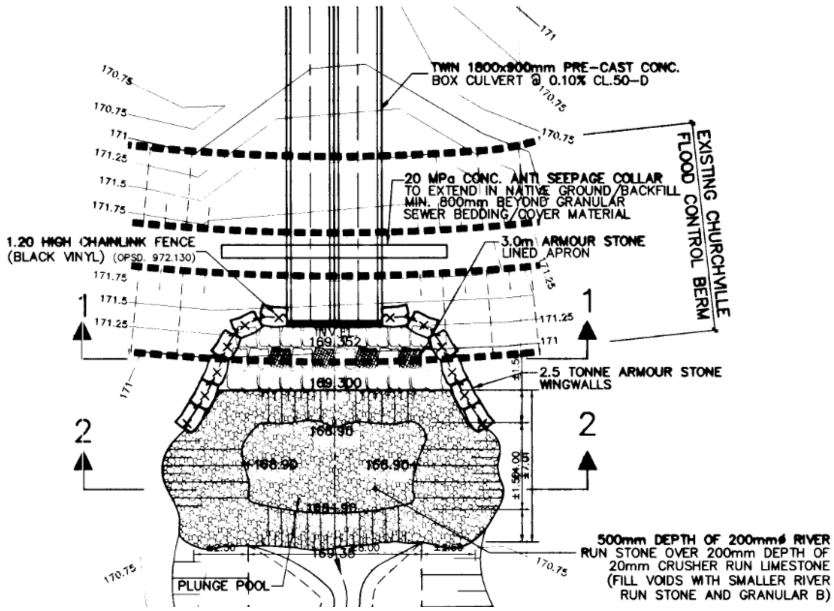
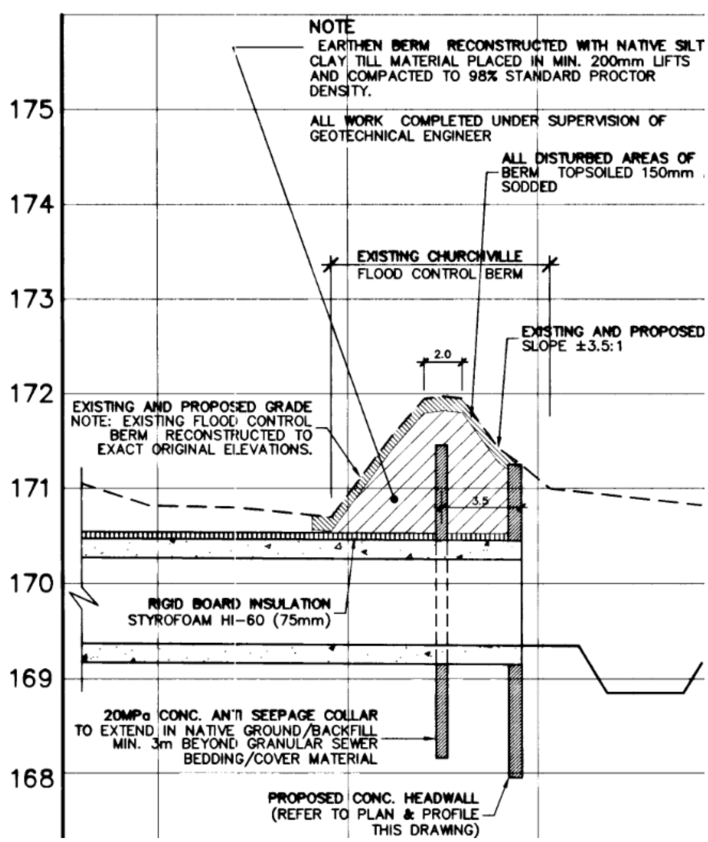


FIGURE 1-4: EARTH DYKE SECTION AT STORM SEWER OUTLET (2 OF 2)



1.7 Site Geology

The area surrounding the Churchville floodwall and earth berms is generally composed of rolling terrain formed as a result of glaciation. Information on the surface and bedrock geology at the site has been obtained from Ontario Geological Survey (OGS) maps (http://www.mndm.maps.arcgis.com) and the site reconnaissance.

The surface geology map from OGS shows that the overburden soils at the vicinity of the floodwall and dyke is mostly composed of modern alluvial deposits which contains varying amounts of gravel, sand, silt and clay. Glacial till depositions are also locally mapped near the site area and is characterized by clay to silt matrix containing varying amounts of gravel and sand (Figure 1-5). The Credit River has resulted in a broad river valley having a fairly flat floodplain.

The bedrock geological map obtained from OGS identified that the site is underlain by sedimentary rock in the form of shale, limestone, dolostone and siltstone of the Queenston Formation (Figure 1-6). Based on available drift thickness mapping, the bedrock depth is estimated to be greater than 10m below ground surface at the site and surrounding area.

FIGURE 1-5: SURFACE GEOLOGY FROM ONTARIO GEOLOGICAL SURVEY (OGS) – CHURCHVILLE

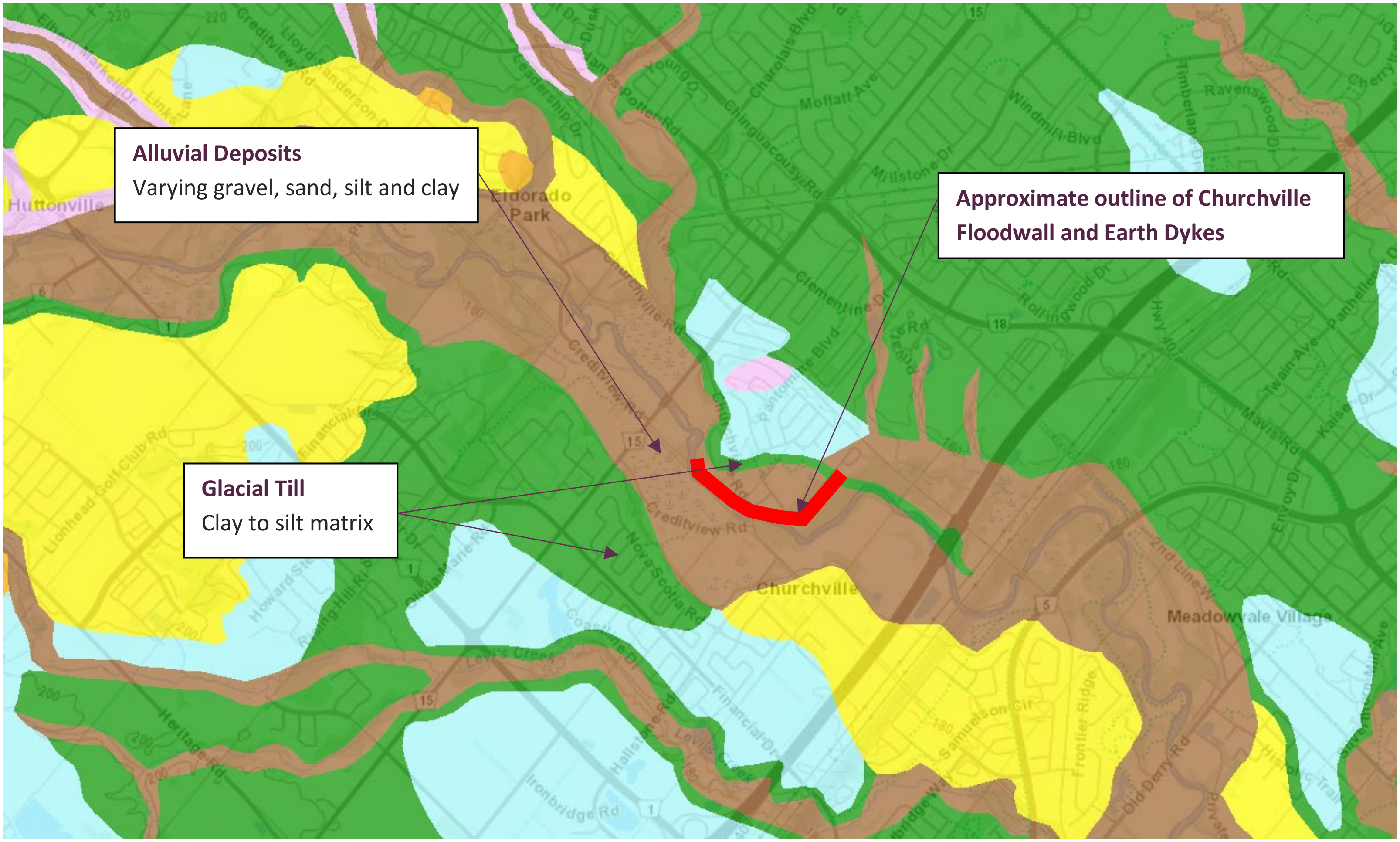
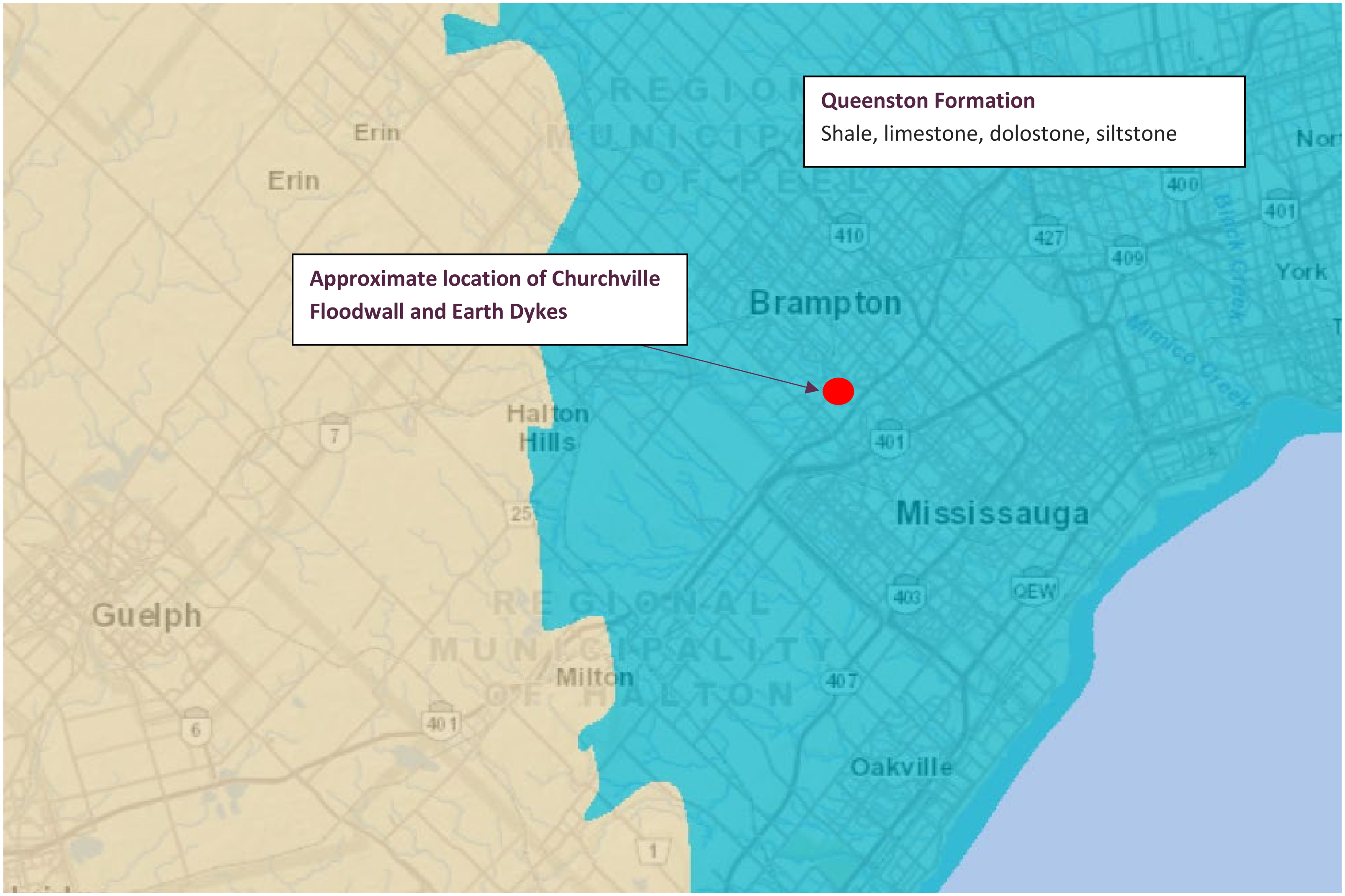


FIGURE 1-6: BEDROCK GEOLOGY FROM ONTARIO GEOLOGICAL SURVEY (OGS) – CHURCHVILLE



2.0 EARTH DYKE AND FLOODWALL INSPECTION

For the current study, the overall length of flood barrier was divided into four (4) segments based on location, geometry and composition (see Table 2-1 and Figure 2-1). The indicated chainages of the segments are based on the original 1989 as-constructed drawings. Based on the limited 2023 topographical survey completed as part of the study, a detailed layout of the earth dyke and floodwall is shown in the drawings (DWG Nos: G01 to G05) provided in Appendix A. Typical cross sections of the dykes and floodwall as shown in the as-constructed drawings are provided in Figure 2-2.

There are several cross-culvert outfalls through the earth dyke (450 mm diameter CSP culverts) and floodwall (150 mm diameter) with inline check valves and flap gates, respectively, for backflow prevention at low points to maintain the local drainage outfalls to the Credit River. In addition, there are two (2) storm sewer outfalls through the dyke sections at Ch. 0+276 (equipped with inline check valves, east of the bridge at Churchville Road) and Ch. 0+430 (double-box culvert, near corner of Martins Blvd). The double box culvert storm sewer outfall was constructed around 2004 (i.e. after the original dyke and floodwall construction) as part of the Paradise Homes subdivision.

The elevations specified in the reports are geodetic, and they are referenced to CGVD 1928 (vertical datum).

TABLE 2-1: SEGMENTS OF THE CHURCHVILLE EARTH DYKE AND FLOODWALL

Segment 1 (STA. 0+000 to 0+048)	Shallow height earth dyke north of floodwall
Segment 2 (STA. 0+048 to 0+326)	Concrete cantilever floodwall with earth dykes located in between floodwall sections to allow access ramp (driveway) over the flood structure
Segment 3 (STA. 0+326 to 0+444)	Shallow height earth dyke extending from southeast end of floodwall to Martins Blvd
Segment 4 (STA. 0+444 to 0+718)	Earth dyke extends from the corner of Victoria Street and Martins Blvd to the residential area at the east end of Martins Blvd

FIGURE 2-1: LAYOUT OF THE CHURCHVILLE EARTH DYKE AND FLOODWALL

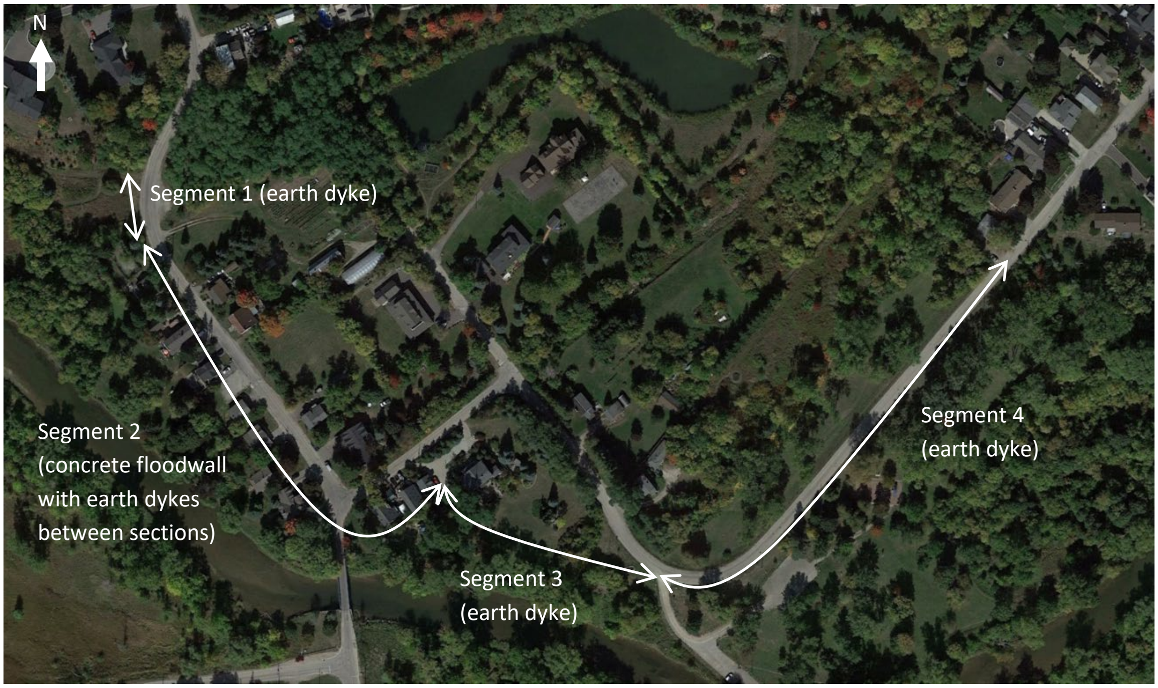
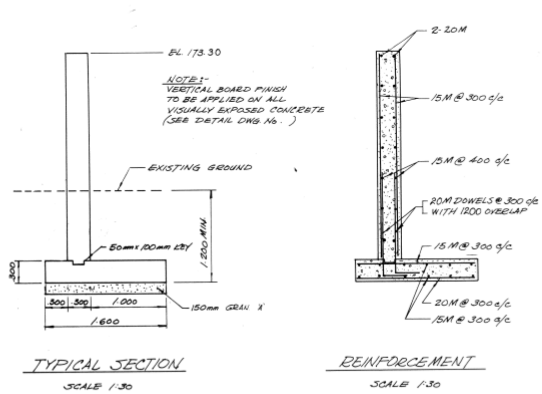
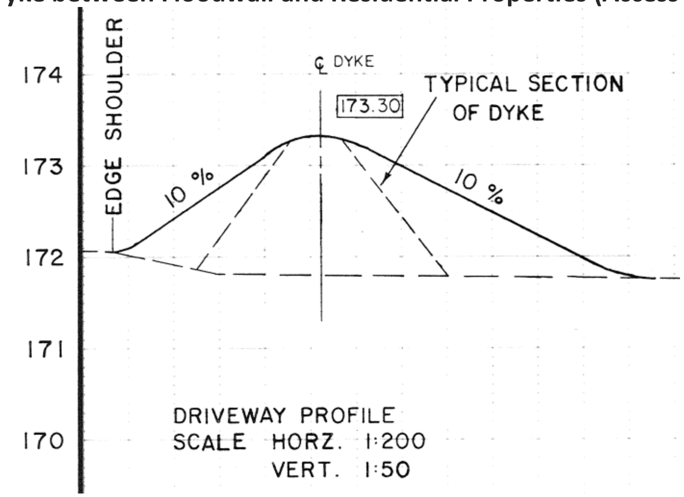


FIGURE 2-2: TYPICAL DETAILS OF EARTH DYKES AND FLOODWALL FROM 1989 AS-CONSTRUCTED DRAWINGS

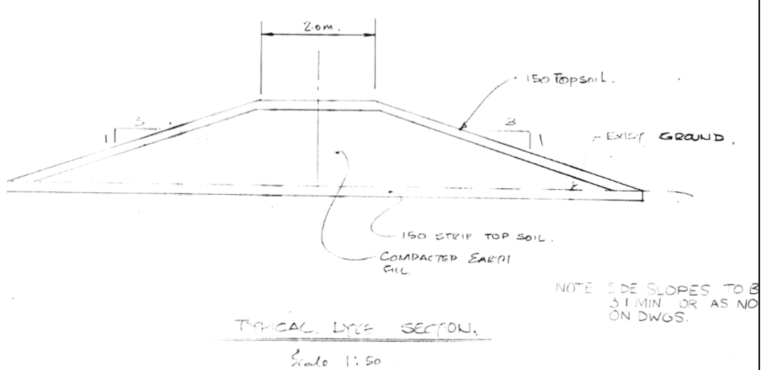
Floodwall Sections



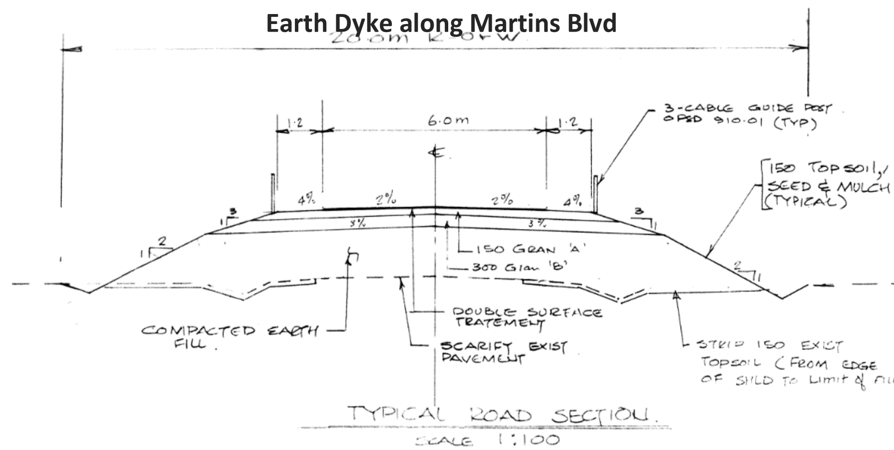
Earth Dyke between Floodwall and Residential Properties (Access Ramp)



Earth Dyke between Floodwall and Martins Blvd



Earth Dyke along Martins Blvd



2.1 Condition Assessment

A visual site inspection was carried out on June 23, 2023, under sunny conditions. Earth dyke sections were inspected by geotechnical engineers Shan Gnanasunthar, P.Eng. and Doug Dubeau, P.Eng. The concrete floodwall sections were inspected by structural engineers Yongbo Fu, P.Eng. and Jayden Levy, EIT. The visual inspection followed the Terms of Reference and criteria specified within the LRIA and CDA/OMNRF Guidelines and included assessing the dyke relative to slope stability, seepage, erosion protection, depressions/animal burrows and settlement/deformation.

The condition assessment rating system for the dyke used for the visual inspection is described in Table 2-2. The general arrangement plan for the floodwalls and earth dykes with the indicated chainages is provided in Appendix A (Drawing G01).

At the time of the inspection, the Credit River elevation was below the wet side toe of the dyke for the entire length of the dyke and floodwalls (normal sunny day river level). At Segment 1 and parts of Segments 2 and 3, the tree and vegetation cover were generally dense at the wet-side of the earth dyke and floodwalls which impeded the visual inspection.

TABLE 2-2: CONDITION ASSESSMENT RATING SYSTEM

Description	Details
Good/Satisfactory	Minimal wear or deterioration, like new condition. No repairs required.
Fair	Normal material wear or deterioration. Functionally adequate for intended uses. Annual maintenance will maintain serviceability throughout design life.
Poor	Abnormal material wear, deterioration, or local defects. Component may not fulfill intended uses. Major maintenance or repairs advisable to restore component to satisfactory condition. If maintenance or repairs are not carried out, the design life of the component may be severely limited, and the component may become unsafe.
Unsatisfactory	Severe material wear, deterioration, or local defects. Component will not fulfill uses. Immediate repair or replacement required. Present situation threatens the structural integrity of the project and represents an unsafe condition.

2.1.1 SEGMENT 1 – EARTH DYKE (CH. 0+000 TO 0+048)

Segment 1 is located at the northern most section of the Churchville flood protection, just beyond 7780 Churchville Road. The dyke at this section is relatively low in height. Based on the visual inspection, the dyke was generally in satisfactory condition. There were no observed slope movements or signs of significant erosion that would suggest significant concerns related to the slope stability and performance of the dyke. Some key observations included the following:

- The crest elevation of the dyke was surveyed and ranged between El. 173.23 m and El. 173.34 m and was 2 m wide.
- This section of the dyke is approximately 1 m high at both the wet and dry sides and is approximately 48 m long.
- The wet side slope of the dyke was approx. 3H:1V and the dry side was approx. 2.5H:1V based on the 2023 topographical survey.
- The north side of the dyke is blended into the existing ground. The south side of the dyke is tied into the concrete floodwall. At the tie-in abutment, no separation between the floodwall and earthfill was observed.
- Uneven areas (up to 50 mm) were observed at the crest, although there were no significant settlements/depressions or evidence of sinkholes.
- No visual signs of any active or historic slope failures/movements were observed on either the wet-side or the dry side slopes. No significant settlements or evidence of sinkholes/depression were observed on the slopes.
- A wooden fence is located at the crest of the dyke (Photo 2-1). An access ramp is located at the northern side of the dyke.
- A 450 mm diameter CSP culvert equipped with an inline check valve (OF_3982) was located at the southern end of the dyke section (Photo 2-3). The culvert appeared to be in satisfactory condition.
- Grass vegetation provides erosion protection. In general, the grass vegetation was thick which made visual observations difficult, particularly at the wet-side of the dyke (Photo 2-2). Larger bushes and trees were located near the abutments.

Photo 2-1: Dry side slope of Segment 1, looking south

Photo 2-2: Crest and wet side slope of Segment 1, looking north

Photo 2-3: 450mm dia. CSP culvert located at Segment 1 (OF_3982)

2.1.2 SEGMENT 2 – FLOODWALL AND EARTH DYKE (CH. 0+048 TO 0+326)

Floodwall Sections

The northernmost floodwall section extends from the northern earth dyke to 7764 Churchville Road. In general, the concrete wall was in satisfactory condition with no movement or significant structural deficiencies. The following key observations were noted:

- The top of the floodwall was surveyed at elevation El. 173.3 m consistent with the as-constructed drawings.
- Vertical cracking (0.3mm to >1mm) along the full height of the visible wall were located 39 m and 41.5 m from the northwest corner of the wall segment (Photo 2-4).
- Localized exposed and corroded rebar were found just above the existing ground level, located 21 m from the northwest corner of the wall segment, near Ch. 0+060 (Photo 2-5).
- Wall thickness was found to be typically 270 mm, rather than the 300 mm specified on the 1989 construction drawings.
- Flap gates (150mm dia.) were located near the northwest and southeast corners of the wall segment (OF_3983 and OF_3984, respectively). The flap gates appeared to be in satisfactory condition. The flap gate outlet at the southeast corner of the wall segment (Ch. 0+110) was partially obstructed with debris at the wet-side which may prevent it from opening (Photo 2-6).
- Several larger/mature trees were observed growing near the concrete floodwall, particularly between properties 7780 and 7772 (wet side of floodwall – Photo 2-7).
- At 7772 Churchville Road, some of the supports for the wooden stairs are anchored into the concrete floodwall (Photo 2-8). It does not appear that the concrete floodwall was originally designed to support the stairwell.

Photo 2-4: Vertical crack in concrete wall

Photo 2-5: Exposed and corroded rebar

Photo 2-6: (a) Flap gate at northwest corner of floodwall segment (OF_3983) (b) Flap gate at southeast corner of floodwall with partial blockage (OF_3984)

Photo 2-7: Larger/mature trees were observed growing near the concrete floodwall

Photo 2-8: Wooden supports for the stairs anchored into the concrete floodwall

A floodwall section is located behind 7764 Churchville Road and is tied into the earth dykes on opposite sides of the residence (Photo 2-9). In general, the concrete wall was in satisfactory condition with no movement or significant structural deficiencies. Backfill is placed to the top of the dry-side of the wall to accommodate the backyard patio of the property. The top of the floodwall was surveyed at elevation El. 173.3 m consistent with the as-constructed drawings. A large crack was observed at the wet-side toe of the north abutment wall (Photo 2-10); however, this is not considered to be a concern for the overall stability of the floodwall at this section.

Photo 2-9: Floodwall behind 7764 Churchville Road

Photo 2-10: Crack in floodwall at wet-side toe

Beyond 7764 Churchville Road, the floodwall extends to 7736 Churchville Road where the wall is tied into the earth dyke west of the bridge. In general, the wall was in satisfactory condition with no movement or significant structural deficiencies. Some key observations included the following:

- The top of the floodwall was surveyed at elevation El. 173.0 m at this segment, consistent with the as-constructed drawings.
- The thickness of the concrete wall varied throughout the length of the segment (varied between 190mm and 350mm thick) due to varying surface finishing (stone cladding, smooth finish and plank finish – Photos 2-11 and 2-12).
- At some locations along the length of the floodwall section, the backfill was near the top elevation of the concrete wall on both the wet and dry sides.
- The floodwall was tied into the residential property at 7742 Churchville Road. The contacts of the wall to the house foundation appeared to be satisfactory however some deterioration of the exposed decorative plank finish was observed at the north contact of the house (Photo 2-13).
- A flap gate (150mm dia.) was observed at the base of the floodwall behind 7752 Churchville Road (OF_4340). The flap gate appeared to be in satisfactory condition.

Photo 2-11: Floodwall section with stone cladding surface finish

Photo 2-12: Floodwall thickness transition near property at 7742 Churchville Road

Photo 2-13: Floodwall tied into structural foundation of building at 7742 Churchville Road

Photo 2-14: End of floodwall behind 7736 Churchville Road

The floodwall segments at the Churchville bridge and the segment east of the bridge were in satisfactory condition with no movement or significant structural deficiencies. Some key observations included the following:

- The top of the floodwall at the bridge walls was surveyed at elevation El. 173.0 m. At the floodwall segment east of the bridge, the top elevation was surveyed at available locations and ranged between El. 172.6 m and 172.7 m. The surveyed elevations were generally consistent with the as-constructed drawings.

- At the floodwall section east of the bridge, heavy tree and bush vegetation was observed growing beside the wall which impeded the visual inspection (Photo 2-17). Survey of the floodwall was not possible in the overgrown vegetated areas due to the overhead cover.

Photo 2-15: Floodwall at west side of the bridge on Churchville Road

Photo 2-16: Floodwall at east of the bridge behind 11 Church Street (dry side)

Photo 2-17: Floodwall at east of the bridge behind 11 Church Street (wet side)

Earth Dyke Sections

Two separate earth dykes flank (slope 3.5H:1V or flatter) either side of the residence at 7764 Churchville Road. These sections provide access over the dykes to the wet-side of the dyke/river. Based on the visual inspection, the dykes were generally in satisfactory condition. There were no observed slope movements or signs of significant erosion that would suggest significant concerns related to the slope stability and the performance of the dykes. Some key observations included the following:

- The crest elevations of the dykes were surveyed and was found to be El. 173.12 m on both dykes, with a 2 m wide crest.
- The dyke sections were approximately 2 m high at both the wet and dry sides and were approximately 12 m long on both sides of the house.
- The wet side and dry side slopes of the berms were approx. 3.5H:1V or flatter.
- The ends of the dykes are tied into the concrete floodwalls. No separations between the floodwall and earthfill was observed.
- Grass vegetation provides erosion protection along the berm located north of the house, and along the dry side slope of the south berm. The wet side of the south berm is protected by bush/shrub vegetation.
- There were no significant settlements/depressions or evidence of sinkholes and the crests were relatively flat.
- No visual signs of any active or historic slope failures/movements were observed on the wet side and dry side slopes. No significant settlements or evidence of sinkholes/depression were observed on the slopes.

Photo 2-18: Wet side slope of dyke flanking north side of 7764 Churchville Road (looking north)

Photo 2-19: Wet side slope of dyke flanking south side of 7764 Churchville Road (looking north)

Two separate earth dykes are located on opposite sides of the bridge on Churchville Road. These sections provide access over the dyke to the wet side of the dyke. Based on the visual inspection, the dykes were generally in satisfactory condition. There were no observed slope movements or signs of significant erosion that would suggest significant concerns related to the slope stability and performance of the dykes. Some key observations included the following:

- The crest elevation of the dykes was surveyed and generally ranged between El. 172.80 m and El. 173.0 m and the crests were 2 m wide. At the berm east of the bridge (approx. Ch. 0+282), the crest was slightly lower with the lowest elevation measured to be El. 172.58 m (approx. 200mm lower than crest, for an approximately 2 m long area). The lower elevation may be associated with settlement of the dyke after the earth dyke reconstruction following the installation of a sanitary sewer crossing the Credit River near the bridge in 2006 as described in Section 1.6.

- The heights of the berms were approximately 2 m high at the wet side and 1-1.5 m at the dry side. The berms were approximately 10 m in length.
- The wet side and dry side slopes of the berms were approx. 3H:1V.
- Both berms are tied into the concrete floodwalls. No separation between the floodwall and earthfill was observed.
- Grass vegetation provides erosion protection.
- At the crest of the berm located east of the bridge, a depression was observed near the middle portion of the dyke (Photo 2-21). The cause of the depression may be associated with settlement as described above, or from subsequent access traffic. The low spot was approximately 0.2-0.3 m lower than the overall crest elevation.
- No visual signs of any active or historic slope failures/movements were observed on either the wet side or the dry side slopes. No significant settlements or evidence of sinkholes/depression were observed on the slopes.
- A few large, mature trees were located on the dykes.
- A 450 mm diameter CSP culvert equipped with an inline check valve (OF_2870) was located at the southern end of the dyke section (Photo 2-22). The culvert appeared to be in satisfactory condition. A double stormwater catch basin (Photo 2-21) is located at the dry side toe of berm east of the bridge which discharges through the culvert.

Photo 2-20: Crest and wet side slope of berm west of bridge, looking south

Photo 2-21: Dry side slope of berm east of bridge, looking south

Photo 2-22: 450mm dia. CSP culvert (OF_2870) located at Segment 3 (east of bridge)

2.1.3 SEGMENT 3 – EARTH DYKE (CH. 0+326 TO 0+444)

Segment 3 is composed of a relatively shallow dyke extending from the end of the concrete floodwall to Martins Blvd. Based on the visual inspection, the dyke was generally in satisfactory condition with the exception of a low spot (at approx. Ch. 0+410) at the crest of the dyke (location of 2022 overtopping event). There were no observed slope movements or signs of significant erosion that would suggest significant concerns related to the slope stability and the performance of the dyke. Some key observations included the following:

- The crest elevation of the dyke was surveyed and ranged between El. 171.88 m and El. 172.45 m and was generally 2 m wide.
- The crest elevation of the dyke near the box culvert was lower than the other dyke sections with a minimum elevation of El. 171.76 m, which is approx. 0.3 m lower than the surrounding crest elevation.
- This section of the dyke was approximately 1 m high at both the wet and dry sides and is approximately 117 m long.
- The wet and dry side slopes of the dyke were approx. 5H:1V or flatter based on the topographical survey. At the stormwater box culvert outlet, the wet side of the dyke is protected with large armour stones (Photo 2-25).
- The north end of the dyke section is tied into the concrete floodwall and the south end of the dyke section is tied into the road berm at Martins Blvd.
- Uneven areas (up to 50 mm) were observed at the crest, although there were no significant settlements/depressions or evidence of sinkholes.
- No visual signs of any active or historic slope failures/movements were observed on the wet-side and dry side slopes. No significant settlements or evidence of sinkholes/depression were observed on the slopes.
- The double-box stormwater sewer culvert outlet (OF_3977 and OF_3978) and armour stones appeared to be in satisfactory condition, however the area was difficult to inspect due to the dense tree cover. The outlet of the box culverts had protective steel grates. The City confirmed that backflow prevention is provided by inline check valves installed on each catch basin lead connecting to the box culvert on Victoria St.
- A 450 mm diameter CSP culvert equipped with an inline check valve (OF_3979) was located near the corner of Martins Blvd and Victoria Street (Photo 2-26). The culvert appeared to be in satisfactory condition. A gabion basket was partially visible at the outlet. Minor erosion of the overburden around the outlet was observed. It is suspected the erosion may be due to local poor vegetation cover at the outlet.
- Grass vegetation provides erosion protection. Between approx. Ch. 0+400 and 0+435, the tree and vegetation cover was dense.

Photo 2-23: Crest of dyke at Segment 4, located on 45 Church Street

Photo 2-24: Wet side slope of dyke at Segment 4, near box culvert (area of 2022 overtopping)

Photo 2-25: Stormwater box culvert outlet (OF_3977 and OF_3978)

Photo 2-26: CSP culvert with inline check valve (OF_3979) near corner of Martins Blvd and Victoria Street

2.1.4 SEGMENT 4 – EARTH DYKE (CH. 0+444 TO 0+718)

The road structure along Martins Blvd serves as the flood dyke in this section, which extends from the corner of Victoria Street and Martins Blvd to the residential area at the east end of Martins Blvd. The earth berm is composed of a wider crest (2-lane asphalt road) and is relatively low in height at both the wet and dry sides. Based on the visual inspection, the dyke was generally in satisfactory condition. There were no observed slope movements or signs of significant erosion that would suggest significant concerns related to the slope stability and the performance of the dyke. Some key observations included the following:

- The crest elevation of the dyke was surveyed and ranged between El. 171.95 m and El. 172.08 m and was generally 10 m wide throughout the length of the section.
- This section of the dyke is approximately 1.5 m high at both the wet and dry sides and is approximately 270 m long.
- The wet side and dry side slopes of the dyke were approx. 3H:1V based on the 2023 topographical survey.
- Asphalt pavement provides erosion protection at the crest which was in good condition. A 0.3 m wide gravel shoulder is located on either side of the asphalt pavement and grass vegetation provides erosion protection along the slopes.
- There were no significant settlements/depressions or evidence of sinkholes.
- No visual signs of any active or historic slope failures/movements were observed on either the wet side or the dry side slopes. No significant settlements or evidence of sinkholes/depression were observed on the slopes.
- A 450 mm diameter CSP culvert equipped with an inline check valve (OF_3980) was located at the end of the earth dyke section (Photo 2-29). The culvert appeared to be in satisfactory condition. Standing water was observed at the wet-side of the culvert at the time of the inspection. The source of the water appears to be from poor drainage in the channel from the outlet to the Credit River.

Photo 2-27: Crest of dyke along Martins Blvd (looking east)

Photo 2-28: Wet side slope of Segment 5 (looking east)

Photo 2-29: CSP culvert with inline check valve (OF_3980) at end of earth dyke section along Martins Blvd

3.0 GEOTECHNICAL INVESTIGATION PROGRAM

The 2023 KGS Group site investigations consisted of the following:

- Completion of cone penetration testing (CPTu) to obtain continuous subsurface condition through the dyke and foundations soils;
- Completion of porewater dissipation testing to obtain the groundwater conditions and the permeability of the foundation soils;
- Completion of test hole drilling using hand and drill rig auguring to characterize the dyke fill and foundation soils;
- Completion of soil index laboratory testing for use in material identification and characterization; and
- Completion of a limited topographical survey to obtain the dyke/floodwall layout and cross sections for the assessment.

A summary of the 2023 CPT/drilling program is presented in Table 3-1 and a location plan of the approximate test hole/CPT locations is shown in Appendix A. The investigation methodologies are described in the following subsections. Results of the investigations are provided in Section 4.

TABLE 3-1: SUMMARY OF 2023 KGS GROUP INVESTIGATION PROGRAM

<table>
<tr><th>Location</th><th>Test Hole ID</th><th>Approx. Ground Elevation[1] (m)</th><th>Total Depth (m)</th><th>Fill (m)</th><th>Foundation Soil (m)</th></tr>
<tr><td colspan="6">Segment 1 (STA. 0+000 to 0+048)</td></tr>
<tr><td rowspan="3">Wet Side Toe</td><td>CPT23-07</td><td rowspan="3">172.15</td><td>6.2</td><td>1.2</td><td>5</td></tr>
<tr><td>TH23-07</td><td>3.0</td><td>1.2</td><td>1.8</td></tr>
<tr><td>HA23-01</td><td>1.2</td><td>1.2</td><td>-</td></tr>
<tr><td rowspan="3">Crest</td><td>CPT23-06</td><td rowspan="3">173.23</td><td>9.1</td><td>2.4</td><td>6.7</td></tr>
<tr><td>TH23-06</td><td>3.0</td><td>2.4</td><td>0.6</td></tr>
<tr><td>HA23-02</td><td>1.5</td><td>1.5</td><td>-</td></tr>
<tr><td>Dry Side Slope</td><td>HA23-03</td><td>172.60</td><td>1.5</td><td>1.5</td><td>-</td></tr>
<tr><td colspan="6">Segment 2 (STA. 0+048 to 0+326)</td></tr>
<tr><td rowspan="2">Dry Side Toe (beside floodwall)</td><td>CPT23-05</td><td rowspan="2">172.19</td><td>5.2</td><td>2.4</td><td>2.8</td></tr>
<tr><td>TH23-05</td><td>3.0</td><td>2.4</td><td>0.6</td></tr>
<tr><td colspan="6">Segment 3 (STA. 0+326 to 0+444)</td></tr>
</table>

Location	Test Hole ID	Approx. Ground Elevation[1] (m)	Total Depth (m)	Fill (m)	Foundation Soil (m)
Wet Side Toe	CPT23-01	171.14	2.5	1.5	1.0
	TH23-01		3.0	1.5	1.5
	HA23-04		1.5	1.5	-
Wet Side Slope	HA23-05	171.50	1.2	1.2	-
Crest	CPT23-02/02B	172.10	2.6 / 3.1	2.4 / 2.4	0.2 / 0.7
	TH23-02		4.6	2.4	2.2
	HA23-06		1.5	1.5	-
Dry Side Slope	HA23-07	171.50	1.5	1.5	-
Segment 4 (STA. 0+444 to 0+810)					
Wet Side Toe	CPT23-04	170.33	2.4	0.7	1.7
	TH23-04		3.0	0.7	2.3
	HA23-08		1.2	0.9	0.3
Crest	CPT23-03	171.97	9.5	3.2	6.3
	TH23-03		3.0	3.0	-
Dry Side Toe	HA23-09	170.22	1.2	0.9	0.3

Notes

[1] Elevations based on 2023 topographical survey carried out by KGS Group.

3.1 Hand Auguring

A total of nine (9) hand auger test holes were carried out at the crest and the wet and the dry side toe areas in July 2023 at key sections of the earth dyke to assess the properties of the dyke fill and foundation soils of the earth dyke. The hand auger holes were typically bored to a depth of 1.2-1.5 m. Laboratory testing was carried out on select soil samples for analysis. The findings and laboratory testing results of the hand auger holes are incorporated in the test hole logs provided in Appendix B and are summarized in Section 4.

Photo 3-1: Hand auger investigation (Segment 1 – northern dyke section)

3.2 Cone Penetration Testing (CPTu)

A cone penetration testing (CPTu) program was completed in November 2023. ConeTec Investigations Ltd. of Richmond Hill, ON, provided the CPTu sounding services using a rubber track mounted rig (M5T Marl) equipped with integrated electronic piezocone penetrometers, a data acquisition system and auger drilling capabilities as shown in Photo 3-2. The CPTu soundings were completed under the supervision and direction of KGS Group personnel.

Traffic management was carried out during drilling at Martins Blvd as a safety precaution and to maintain local traffic during the drilling operations. KGS Group retained TCI field services to provide the traffic management services. A Road Occupancy Permit was also obtained from the City of Brampton for the investigation work occurring on the road.

The CPTu sounding program consisted of seven (7) cone penetration testing locations.

Due to refusal on suspected cobbles/coarse gravel material, some areas were drilled through using a hollow stem auger to advance past the cobble/coarse gravel-rich areas. The CPTu holes were sealed immediately in accordance with provincial abandonment requirements upon completion of the work. The approximate locations of the CPTu soundings are shown in Appendix A.

As the electronic piezocone penetrometer is advanced into the ground, measurements of cone tip resistance, sleeve friction and pore water pressure are recorded every 5 cm to a purpose-built data acquisition system. Analysis of the CPTu sounding data allows an estimation of geotechnical design parameters and inference of subsurface stratigraphy from the recorded soil type behaviour characteristics.

Porewater dissipation testing was conducted at specific depths to estimate the hydraulic conductivities of the subsurface soils. To conduct the testing, the piezocone is halted at a specified depth and the data acquisition system measures and records the variation of the pore pressure with time. The pore pressure dissipation data was interpreted to estimate the groundwater conditions and hydraulic conductivities of the soils. The results of the interpretation of are summarized in Section 5.1 in Table 5-2.

The CPTu sounding results are provided in Appendix C.

Photo 3-2: CPTu drill rig set-up at Churchville flood dyke (northern section on Churchville Road)

3.3 Test Hole Drilling and Sampling

During the CPTu sounding program, a total of seven (7) test holes were advanced to depths varying between 3 m and 4.6 m using conventional augers (Photo 3-3). The subsurface stratigraphy was observed and select auger samples were retrieved for laboratory testing. All soil samples were visually classified in accordance with the Unified Soil Classification System (USCS). Selected samples were collected in sealed plastic bags for further identification and laboratory testing. The stratigraphic interpretation from the CPTu soundings was verified from the test hole drilling. Test hole logs incorporating key field observations and laboratory testing results are included in Appendix B.

Photo 3-3: Auger sample taken during CPTu program

3.4 Topographical Survey

A limited topographical survey of the earth dykes and floodwalls was completed to capture relevant topographical data as part of the assessment and to develop representative cross sections for the stability and seepage analyses. The survey was completed in conjunction with the visual inspection of the flood structures on June 23, 2023 under sunny conditions. The survey was completed where the dykes and floodwalls were accessible. The presence of thicker vegetation growth prevented survey from being carried out at select areas of the dykes and floodwalls due to lack of line of sight and/or overhead GPS clearance, however, the completed survey was considered adequate for the current study.

The topographical survey capture was completed using Topcon survey grade Global Positioning System (GPS) RTK style surveying and Topcon Robotic Total Station. All topographic survey information was processed using Topcon's Magnet Tools software and was completed in Universal Transverse Mercator (UTM) NAD83 CSRS and NAD83 Zone 21 projection and elevations in the Canadian Geodetic Vertical Datum (CGVD1928). A static occupation was performed by L1/L2 dual constellation (GPS and GLONASS) GPS receivers using post-processing procedures. The static data was then post-processed using data from Canadian Active Control System (CACS) data available on the Canadian Spatial Reference System (CSRS) website to give a more accurate geodetic location. The accuracy of these networks is anticipated to be +/- 25 mm in horizontal and vertical.

The topographical survey plan and cross section drawings are provided on the drawings in Appendix A.

3.5 Laboratory Testing

Diagnostic laboratory testing was performed on select soil samples for use in material identification and characterization, as well as estimation of shear strength parameters of the embankment fill materials and foundation soils. The laboratory testing was completed at the EXP Soil Laboratory in Brampton, Ontario. The following soil testing standards used were developed by the American Society for Testing and Materials (ASTM):

- ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils;
- ASTM D1140 – Test Method for Amount of Material in Soils Finer than the No. 200 Sieve;
- ASTM D2216 – Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock;
- ASTM D4318 – Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils;

The laboratory results from the diagnostic testing are summarized in Table 4-1 as well as on the detailed test hole log records presented in Appendix B.

4.0 GEOTECHNICAL INVESTIGATION RESULTS

The subsurface conditions at the Churchville floodwall and earth dyke were inferred from the information obtained from the CPTu soundings, exploratory test holes, laboratory test data and our understanding of the site geology. The boundaries are intended to signify a transition from one geological unit to another and are not necessarily an exact plane of geological change. The actual stratigraphic sequence of the soil materials between the test holes may differ from those inferred from drilling.

The stratigraphy and engineering properties of the subsurface soils are described in this section. Detailed test hole logs are provided in Appendix B, and laboratory test data are summarized in Table 4-1.

4.1 Subsurface Characterization

4.1.1 DYKE FILL

The low permeability dyke fill was encountered at the following test hole locations (Segments 1, 3 and 4):

- Test holes TH23-01 to 04, TH23-06 and TH23-07 (crest and toe areas)
- CPTu soundings CPT23-01 to 04, CPT23-06 and CPT23-07 (crest and toe areas)
- All hand auger holes (located through the crest, slopes and toe areas)

Approximately 300 mm of dark brown topsoil was encountered at the ground surface of the earth dyke (grassed surface areas). At Segment 4 (located on Martins Blvd), 300 mm of well graded sand and gravel fill (gravel shoulder) overlies the dyke fill.

Silty Clay Fill (CL-CI) – (Low Permeability Dyke Fill)

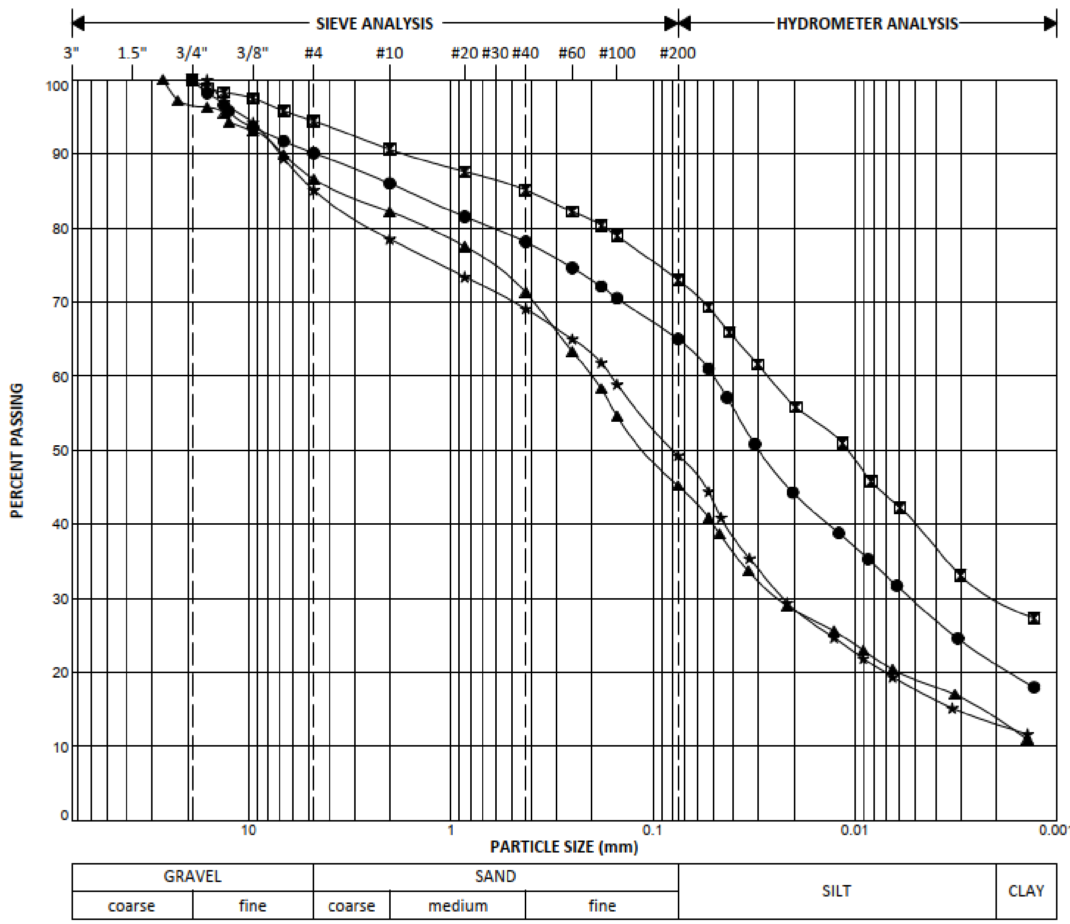
Silty clay fill was encountered below the surface protection described above. At the wet-side toe areas, the fill extended to depths between 0.8m and 1.5m below the ground surface. At the crest, the fill extended between 2.1m and 3.25m below the top of the crest based on the test hole drilling and CPT results. At the dry-side toe, the fill extended 0.9m below the ground surface. The silty clay fill was brown to reddish brown, moist, firm to stiff, of low to intermediate plasticity (CL-CI) and contained some sand and gravel. The presence of trace organics was observed at one (1) location (TH23-03) Segment 4 – Martins Blvd, 1.5m below the crest).

Grain size and hydrometer analyses (4 samples) of the silty clay fill (Figure 4-1) resulted in grain size compositions of 6% to 15% gravel, 21% to 42% sand, 32% to 44% silt and 13% to 30% clay. Atterberg Limits testing (4 samples) on the fine fraction of the silty clay fill (Figure 4-2) indicated the fill was of low to intermediate plasticity with a measured Liquid Limit between 24 and 40, Plastic Limit between 13 and 20, and Plasticity Index between 11 and 21. Moisture contents for the samples ranged from 17% to 23%.

The presence of the silty clay fill was confirmed by the CPTu soundings completed at the crest and toe areas of the dyke. The fill mainly consisted of clay and silt mixtures and occasional increased sand and gravel contents, which is consistent with the test hole observations (silty clay till material). However, due to the presence of larger gravel within the fill, the CPT results indicate higher sand and gravel content than the representative dyke fill observed during the test hole drilling and sampling. The typical stratigraphy interpreted by the CPTu soundings through the dyke crest is presented on Figure 4-3.

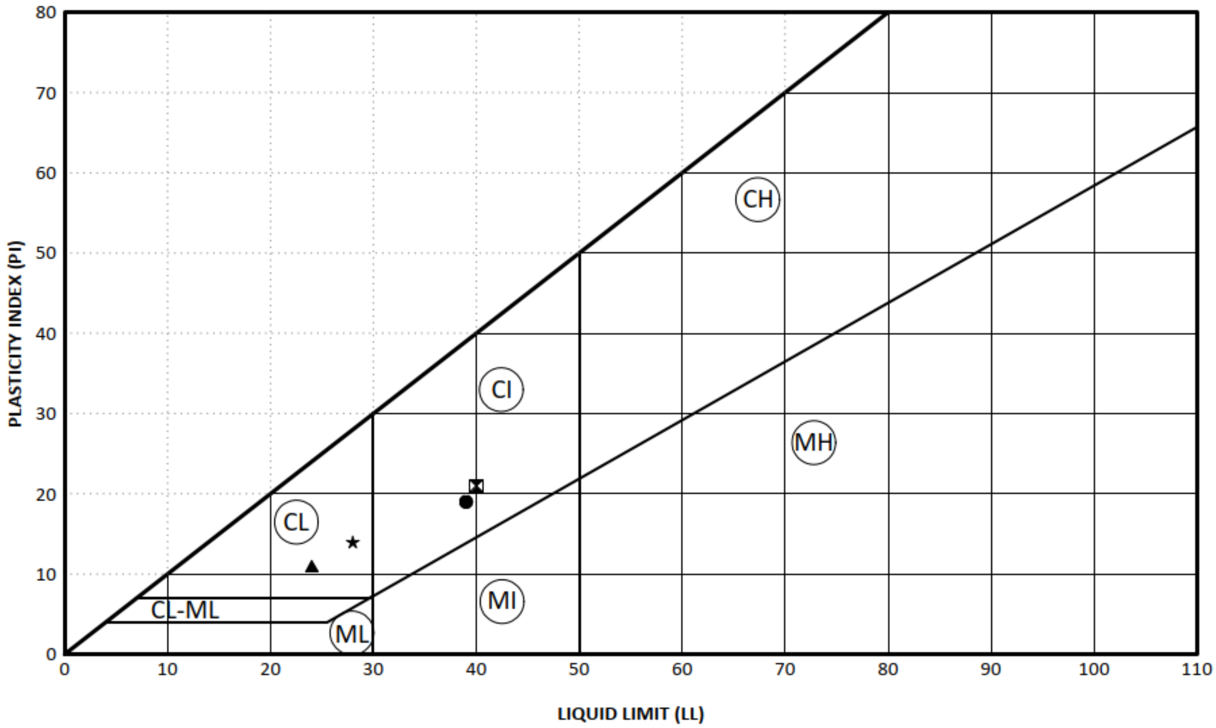
An effective friction angle of Phi = 28° and an effective cohesion of c' = 0 kPa was estimated for the fill based on the CPTu data, index testing and previous experience with similar soil materials.

FIGURE 4-1: GRAIN SIZE DISTRIBUTION, SILTY CLAY FILL – CL-CI (DYKE FILL)



	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	HA23-03	1.2	HA1	10	25	44	21	65			CI
⊠	HA23-06	1.2	HA1	6	21	43	30	73			CI
▲	TH23-02	2.1	CPT1	13	42	32	14	45			SC
★	TH23-03	2.7	CPT1	15	36	36	13	49			SC

FIGURE 4-2: ATTERBERG LIMITS, SILTY CLAY FILL – CL-CI (DYKE FILL)



	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	MC (%)	CLASSIFICATION
●	HA23-03	1.2	HA1	39	20	19	25	44	21	65	21	CI
⊠	HA23-06	1.2	HA1	40	19	21	21	43	30	73	17	CI
▲	TH23-02	2.1	CPT1	24	13	11	42	32	14	45	14	SC
★	TH23-03	2.7	CPT1	28	14	14	36	36	13	49	23	SC

FIGURE 4-3: TYPICAL STRATIGRAPHY INTERPRETED BY CPTU SOUNDINGS (CPT23-02B)

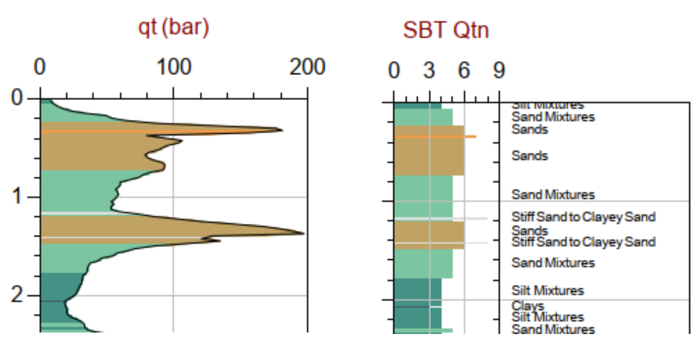


Photo 4-1: Typical dyke fill recovered during geotechnical investigation

4.1.2 FLOODWALL BACKFILL

The floodwall backfill was encountered in Segment 2 at TH23-05 and CPT23-05, located at the dry-side toe beside the floodwall. Approximately 300 mm of dark brown topsoil was encountered at the ground surface beside the wall.

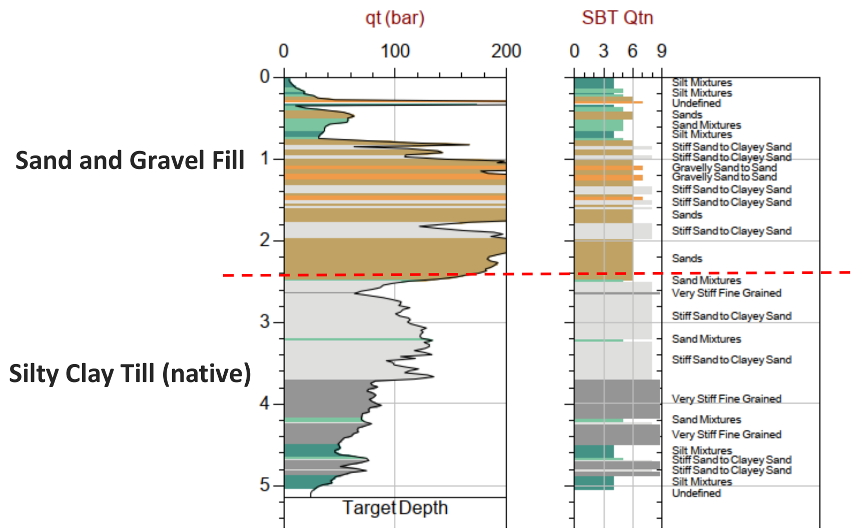
Sand and Gravel Fill

Sand and gravel fill was encountered to a depth of 2.4m (El. 169.7 m) below the dry-side toe of the wall at TH23-05. The sand and gravel fill was brown and grey, moist, compact, medium to coarse sand, medium to coarse gravel, and trace fines.

The sand and gravel fill was also determined from CPT23-05, and consisted of gravelly sand to clayey sand mixtures (Figure 4-4).

An effective friction angle of Phi = 35° and an effective cohesion of c' = 0 kPa was estimated for the sand and gravel fill based on the CPTu data, index testing and previous experience with similar soil materials.

FIGURE 4-4: STRATIGRAPHY INTERPRETED BY CPTU SOUNDING AT DRY-SIDE TOE OF FLOODWALL (CPT23-05)



4.1.3 FOUNDATION SOILS

Poorly Graded Sand and Gravel with Cobbles

Sand and gravel with cobbles (native alluvium deposit) was encountered below the low-permeable dyke fill at Segments 1, 3 and 4 based on the test hole drilling and CPTu soundings:

- Segment 1:
  - 2.4m below the crest (El. 170.8 m) to 3.2m (El. 170 m)
  - 1.26m below the wet-side toe (El. 170.9 m) to 2.2m (El. 170 m)
- Segment 3:
  - 2.4m below the crest (El. 169.7 m) to 4m (El. 168.1 m)
  - 1.5m below the wet-side toe (El. 169.6 m) to the test hole termination depth (El. 168.1 m)
- Segment 4:
  - 3.2m below the crest (El. 168.8 m) to 4.2m (El. 167.8 m)
  - 0.75m below the wet-side toe (El. 169.9 m) to 2.4m (El. 167.9 m)

The sand and gravel with cobbles was brown, moist to wet, compact, poorly graded, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, and some fines.

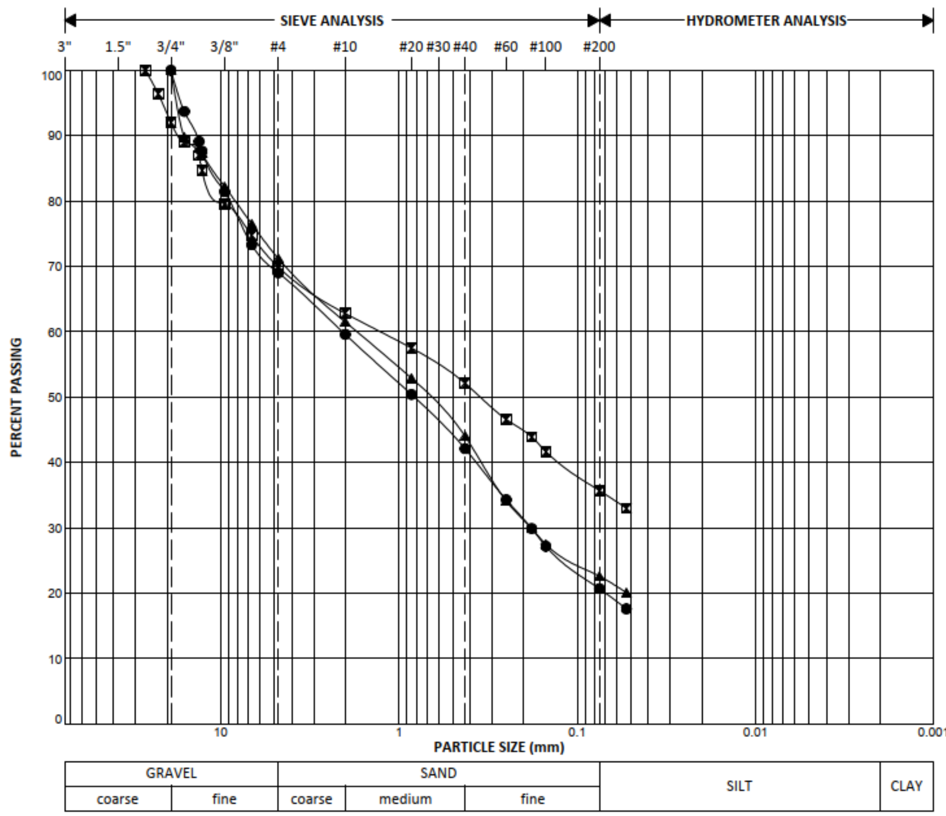
Grain size analyses (Figure 4-5) indicated compositions of 29% to 31% gravel, 34% to 48% sand, and 23% to 36% fines. Moisture contents for the samples ranged from 11.6% to 17.6%.

Groundwater was generally observed between 2.6m and 3m below the crest of the earth dyke sections (below wet and dry side toe of the slope) in the sand and gravel material. A hydraulic conductivity of 1×10^{-4} m/s was estimated for the material based on the porewater dissipation testing.

The CPTu soundings at earth dyke Segments 3 and 4 also encountered the sand and gravel with cobbles. The material mainly consisted of gravelly sand to sand mixtures. The typical stratigraphy interpreted by the CPTu soundings is presented in Figure 4-6.

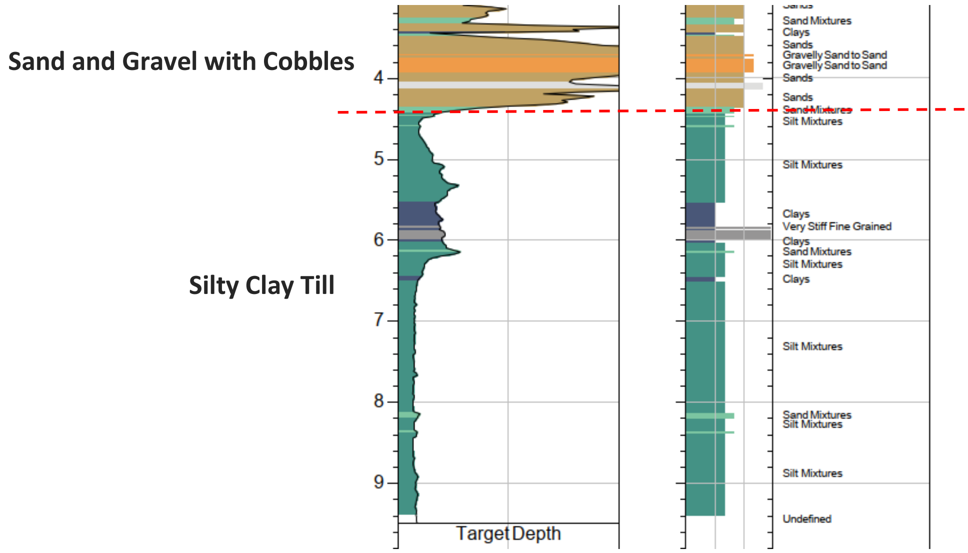
An effective friction angle of Phi = 35° and an effective cohesion of c' = 0 kPa was estimated for the sand and gravel based on the CPTu data, index testing and previous experience with similar soil materials.

FIGURE 4-5: GRAIN SIZE DISTRIBUTION, SAND AND GRAVEL (FOUNDATION SOIL)



	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	HA23-08	1.1	HA1	31	48			21			
⧖	TH23-02	3.4	CPT2	30	34			36			
▲	TH23-04	0.8	CPT1	29	48			23			

FIGURE 4-6: TYPICAL FOUNDATION STRATIGRAPHY INTERPRETED BY CPTU SOUNDINGS (CPT23-03)



Silty Clay Till (CL-CI)

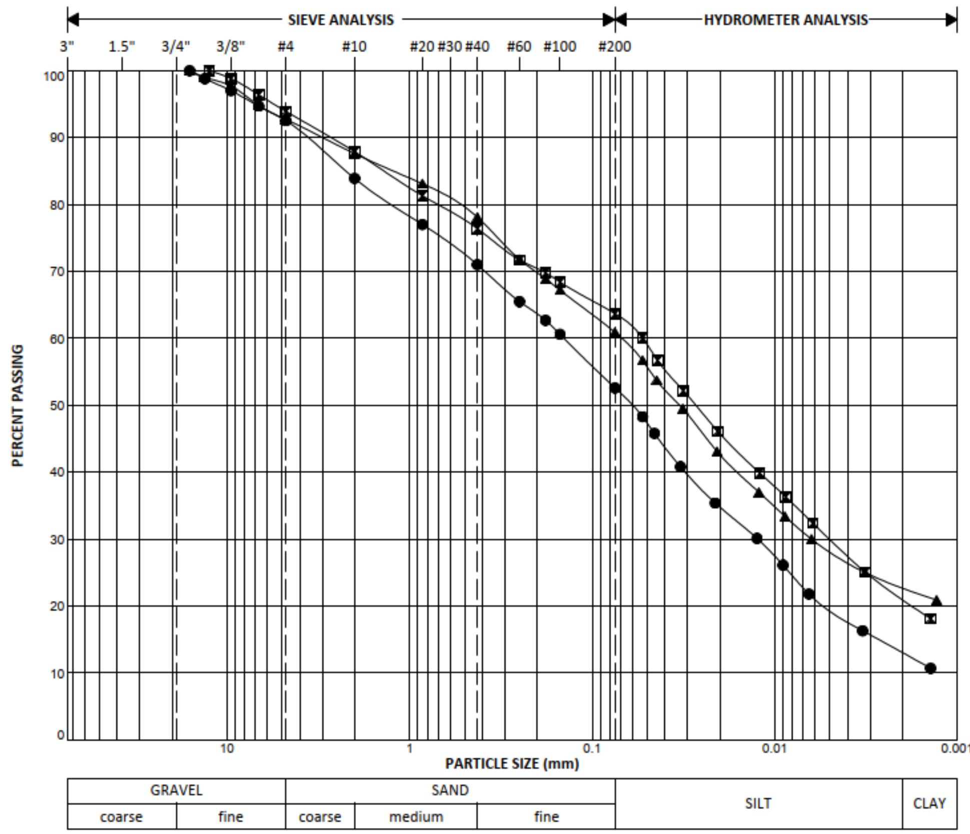
Silty clay till was encountered below the native sand and gravel at Segments 1, 3 and 4 earth dyke sections (test holes TH23-02, 04, 06 and 07, and CPT23-03, 06 and 07) and below the sand and gravel floodwall backfill (TH/CPT23-05). At the earth dyke sections, the till was encountered at El. 170 m (Segment 1), El. 168.1 m (Segment 3) and El. 167.9 m (Segment 4) and was encountered at El. 169.7 m below the dry-side toe of the floodwall. The silty clay till was grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel.

Grain size and hydrometer analyses (3 samples) of the silty clay till (Figure 4-7) resulted in grain size compositions of 6% to 8% gravel, 30% to 39% sand, 38% to 43% silt and 13% to 23% clay. Atterberg Limits testing (3 samples) on the fine fraction of the silty clay till (Figure 4-8) indicated the fill was of low to intermediate plasticity with a measured Liquid Limit between 20 and 36, Plastic Limit between 11 and 15, and Plasticity Index between 9 and 21. Moisture contents for the samples ranged from 13% to 16%.

The presence of the silty clay till was confirmed through the CPTu soundings completed at the crest and toe areas of the dyke. The fill mainly consisted of silt and clay mixtures with occasional increased sand and gravel contents, which is consistent with the test hole observations (silty clay till material). The typical stratigraphy interpreted by the CPTu soundings is presented on Figure 4-6.

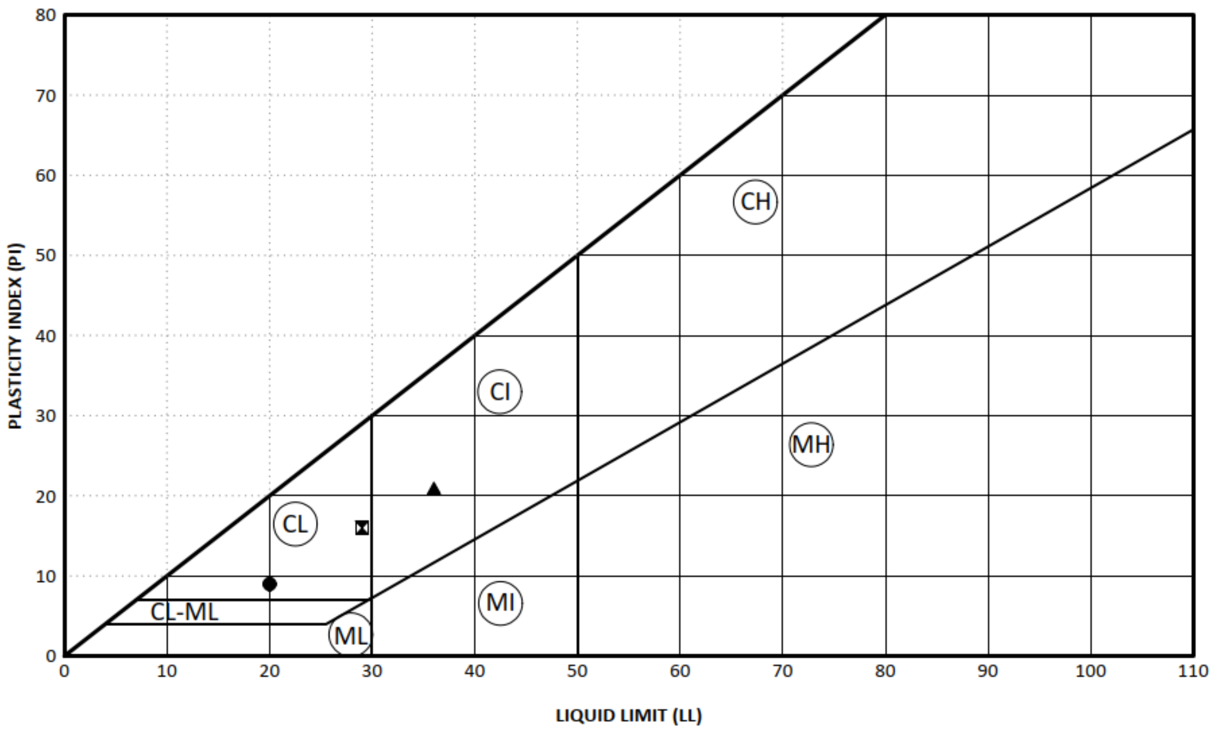
An effective friction angle of Phi = 30° and an effective cohesion of c' = 0 kPa was estimated for the silty clay till based on the CPTu data, index testing and previous experience with similar soil materials.

FIGURE 4-7: GRAIN SIZE DISTRIBUTION, SILTY CLAY TILL – CL-CI (FOUNDATION SOIL)



	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	TH23-02	4.0	CPT3	8	39	40	13	53			CL
⊠	TH23-04	2.4	CPT2	6	30	43	21	64			CL
▲	TH23-05	2.4	CPT1	7	32	38	23	61			CI

FIGURE 4-8: ATTERBERG LIMITS, SILTY CLAY TILL – CL-CI (FOUNDATION SOIL)



	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	MC (%)	CLASSIFICATION
●	TH23-02	4.0	CPT3	20	11	9	39	40	13	53	13	CL
⊠	TH23-04	2.4	CPT2	29	13	16	30	43	21	64	18	CL
▲	TH23-05	2.4	CPT1	36	15	21	32	38	23	61	16	CI

4.2 Laboratory Testing Results

The results of the index testing on the soil samples including moisture content, grain size distribution (sieve analyses and hydrometer testing) and Atterberg Limits (liquid, plastic and plasticity index) testing results are summarized on Table 4-1.

TABLE 4-1: SUMMARY OF INDEX LABORATORY TESTING

Test Hole #	Sample Depth (m) / Sample No.	Soil Description	Moisture Content (%)	Plasticity (%)			Grain Size Distribution (%)			
				LL	PL	PI	Gravel (<75 to 4.75 mm)	Sand (<4.75 to 0.075 mm	Silt (<0.075 to 0.002 mm)	Clay (<0.002 mm)
HA23-03	1.2 (HA1)	Silty Clay Fill (Dyke Fill)	21.1	39	20	19	10	25	44	21
HA23-06	1.2 (HA1)	Silty Clay Fill (Dyke Fill)	17.3	40	19	21	6	21	43	30
HA23-08	1.1 (HA1)	Sand and Gravel	17.6				31	48	21	
TH23-02	2.1 (CPT1)	Silty Clay Fill (Dyke Fill)	13.8	24	13	11	13	42	32	14
	3.3 (CPT2)	Sand and Gravel	11.6				30	34	36	
	4.0 (CPT3)	Silty Clay Till	12.7	20	11	9	8	39	40	13
TH23-03	2.7 (CPT1)	Silty Clay Fill (Dyke Fill)	23.1	28	14	14	15	36	36	13
TH23-04	0.8 (CPT1)	Sand and Gravel	14.6				29	48	23	
	2.4 (CPT2)	Silty Clay Till	17.6	29	13	16	6	30	43	21
TH23-05	2.4 (CPT1)	Silty Clay Till	16	36	15	21	7	32	38	23

5.0 SEEPAGE AND SLOPE STABILITY ANALYSES

5.1 Seepage Analyses (Earth Dykes and Floodwall Foundation)

A 2D seepage model was developed to evaluate the transient groundwater and porewater pressure response of the dyke, concrete floodwall and the foundation soils under a flood event. The seepage analysis was completed in order to:

a) Estimate the phreatic surface within the dyke during the flood event;
b) Determine stability implications on the dry side of the dyke and underneath the floodwall with respect to exit gradient and potential for blow-out/particle migration; and
c) Estimate seepage volume through the dyke and below the floodwall during the flood events.

Transient analysis provides a better understanding of the impact of a flood on the performance of the dyke and how to address any potential seepage concerns at the dry side slope toe areas during flood events.

For the seepage analysis, the river elevation was assumed to be 300 mm below the top of the earth dyke and the floodwall sections, assuming 0.3 m freeboard. As no test holes were completed at the wet-side of the floodwall, the available geotechnical information at the dry-side of the wall was used to model the wet-side. No low pervious blanket was assumed at the wet-side.

Details of the seepage model set-up and soil parameter determination are described in the following sections.

5.1.1 SEEPAGE MODELLING

5.1.1.1 Methodology

The seepage analysis was completed using the Seep/W software by GeoStudio Inc. to evaluate the seepage through the dyke and foundation, to assess the hydraulic gradients and evaluate the potential for piping failure/suffusion. Seep/W uses a finite element-based formulation to analyze groundwater seepage and excess porewater pressure dissipation problems within porous materials such as soil and rock. The software is capable of analyzing a wide range of problems from simple, saturated steady-state problems to sophisticated, saturated/unsaturated problems. The saturated/unsaturated formulation makes it possible to analyze seepage as a function of time including flow through dams/dykes and overburden/rock foundations.

5.1.1.2 Seepage Models

Three (3) representative cross sections of the earth dyke and one (1) typical section of the concrete floodwall were analyzed. The earth dyke sections were selected to evaluate the different dyke geometries and features at the site. The cross-section details and the rationale of selection are summarized in Table 5-1, and the locations of the selected cross-sections are shown in Appendix A. The geometry of each cross-section was obtained from the topographic survey carried out by KGS Group in 2023 and the concrete floodwall section was developed based on the typical section provided in the 1989 construction drawings. The stratigraphy of the dyke sections was based on the 2023 geotechnical investigation findings summarized in Section 4. The model set-up of earthfill dyke cross sections A-A´ to C-C´ and the concrete floodwall (section D-D') are shown in Figures 5-1 to 5-4. The modelled cross sections are shown in the general arrangement plan provided in Appendix A (Drawing G01).

5.1.1.3 Flood Event and Boundary Conditions

Transient analysis was carried out on the cross sections considering a flood event. Due to the absence of hydraulic model, to simulate the flood event, the river level was assumed to increase from the normal river level to the flood level (see Table 5-1) in 24 hours. Then, the river level was maintained at the regional flood level for 48 hours and gradually lowered to the normal river level after 168 hours. The total water head boundary condition (H_{total}) varying with time was assigned to the upstream slope face of the dyke.

FIGURE 5-1: SEEPAGE MODEL SET-UP OF CROSS SECTION A-A'

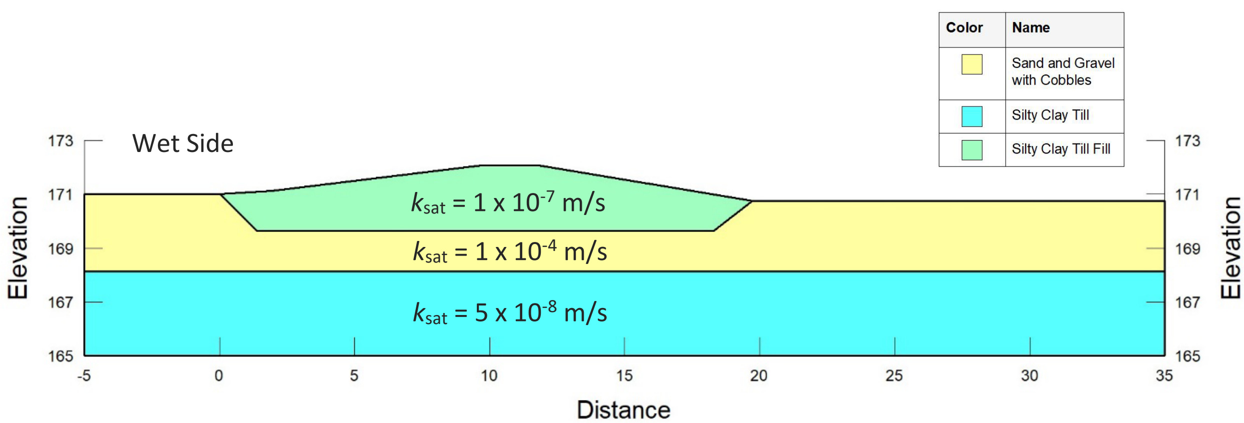


FIGURE 5-2: SEEPAGE MODEL SET-UP OF CROSS SECTION B-B'

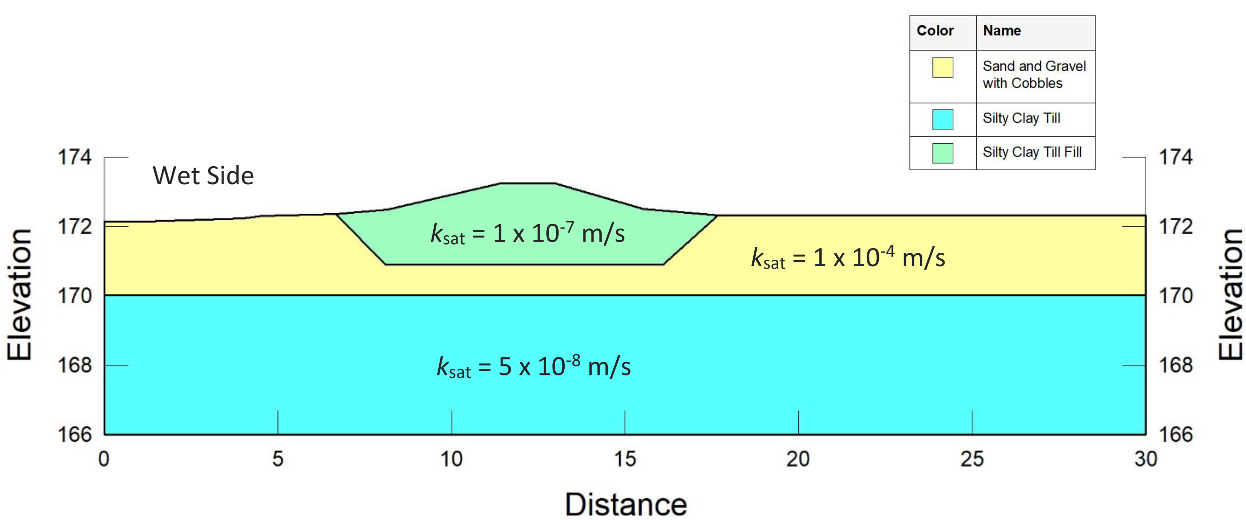


FIGURE 5-3: SEEPAGE MODEL SET-UP OF CROSS SECTION C-C'

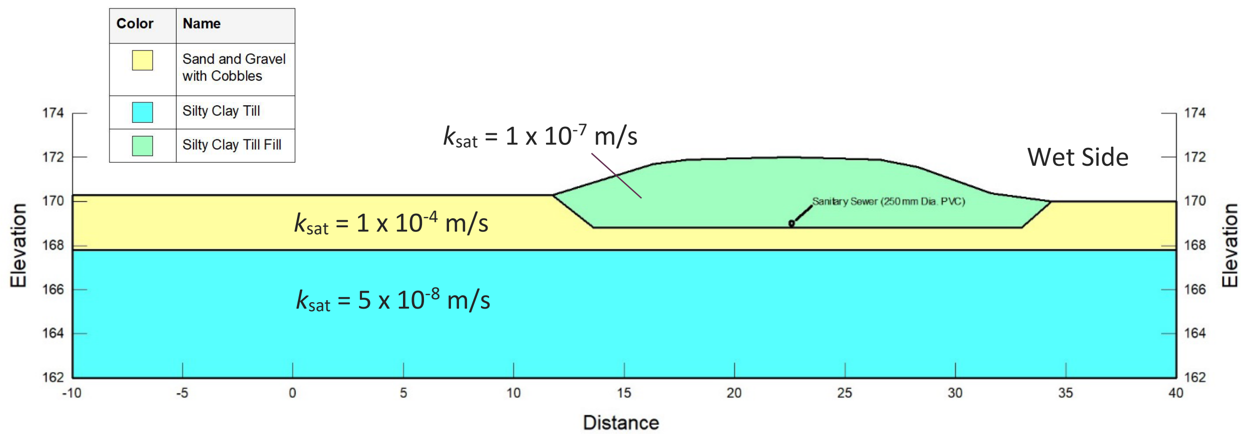


FIGURE 5-4: SEEPAGE MODEL SET-UP OF D-D' (CONCRETE FLOODWALL)

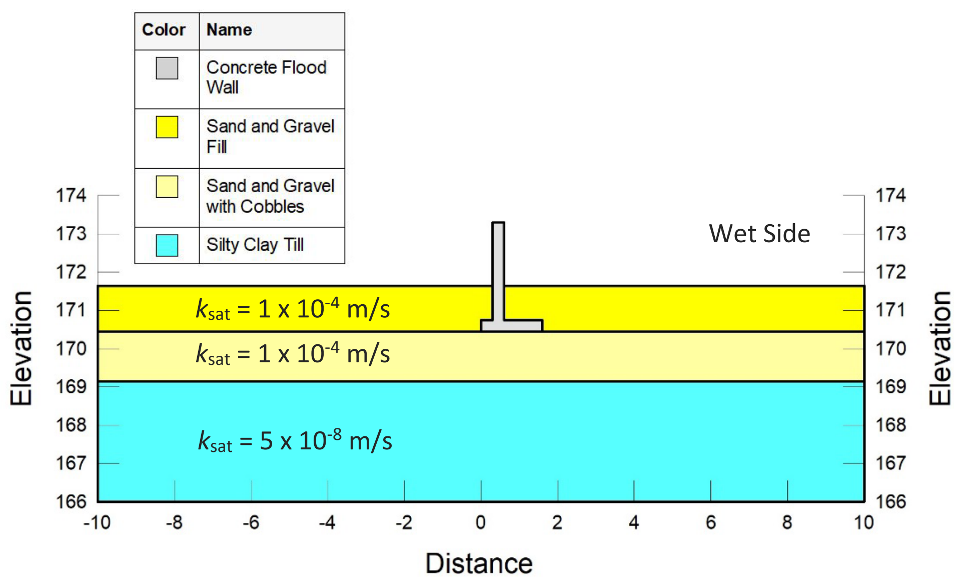


TABLE 5-1: CASE STUDY SET-UP OF SEEPAGE ANALYSIS

Cross-Section	Crest Elevation (m)	Flood Level (m)	Rationale for Selection	Related CPTs/Test Holes
A-A' (Ch. 0+038)	173.24	172.94[1] (173.13)[2]	**Segment 1** (Ch. 0+000 to 0+048): • Northern section of dyke	CPT23-06, CPT23-07, TH23-06, TH23-07
B-B' (Ch. 0+420)	172.08	171.78[1] (171.76)[2]	**Segment 3** (Ch. 0+326 to 0+444): • Area of 2022 overtopping	CPT23-01, CPT23-02/02B, TH23-01, TH23-02
C-C' (Ch. 0+640)	172.0	171.7[1] (171.61)[2]	**Segment 4** (Ch. 0+444 to 0+810): • Martins Blvd	CPT23-03, CPT23-04, TH23-03, TH23-04
D-D' (Ch. 0+160)	173.3	173.0[1] (173.02)[2]	**Segment 2** (Ch. 0+048 to 0+326) • Concrete Cantilever Wall	CPT23-05, TH23-05

Notes

[1] Assumed 0.3m freeboard as recommended in the 1985 study.

[2] Estimated 100-year flood event river level based on the 1985 study (Section 4 (Hydraulics), Calculated Water Elevation).

5.1.1.4 Hydraulic Conductivities

The soil hydraulic conductivities assigned to the fill and foundation soils were estimated based on empirical correlations (Hazen's equation) and CPTu pore pressure dissipation tests as well as previous experience with similar soil materials. The hydraulic conductivity values of the materials of different zones are provided in Table 5-2.

TABLE 5-2: MATERIAL HYDROGEOLOGICAL PROPERTIES

Material	Saturated Hydraulic Conductivity, K_{sat} (m/s)	Kx/Ky Ratio	Resource
Sand and Gravel Fill	1×10^{-4}	1	Grain size distribution and empirical correlation
Silty Clay Fill (Dyke fill)	1×10^{-7}	1	Typical value of similar material and Grain size distribution.
Sand and Gravel with Cobbles (Foundation Soil)	1×10^{-4}	1	CPTu pore pressure dissipation test
Silty Clay Till (Foundation Soil)	5×10^{-8}	1	CPTu pore pressure dissipation test
Concrete Floodwall	Impervious	-	-

5.1.1.5 Hydraulic Gradients

The key consideration for limiting potential seepage damage is to limit the hydraulic exit gradient in order to minimize the potential for internal erosion and piping failure. These potential conditions are exacerbated during flood events and particularly under high permeability foundation conditions.

When the hydraulic gradient approaches a critical level, the effective stress on any plane within the soils will converge toward zero; that is, gravitational forces on the soil particles having been negated by seepage forces. In the case of granular (cohesionless) soils, the contact forces between the soil particles may become significantly reduced and the soil may lose its shear strength. The soil may behave as a fluid (i.e., exhibit "quick" condition), and result in "boiling" as the particles are moved around with the upward flow of water, potentially leading to erosion or "piping" of the soils.

The critical vertical hydraulic gradient is defined as:

$$i_{\mathrm{cr}} = \frac{Gs - 1}{1 + e}$$

where,

G_s = Specific Gravity of Soil Element;

e = Void Ratio.

The critical horizontal hydraulic gradient is defined as:

$$i_n = tan\emptyset \cdot i_{cr}$$

where,

Ø = effective friction angle of soil.

The factor of safety against piping/internal erosion with respect to exit gradients is expressed as:

$$FS = \frac{i_{cr}}{i_{exit}}$$

Based upon index laboratory testing completed, the estimated critical vertical and horizontal hydraulic gradient of the impervious fill/foundation soils are 1.27 m/m and 0.73 m/m respectively (for estimated Gs=2.65 and void ratio of 0.3).

The LRIA geotechnical design criteria requires hydraulic gradients to be below acceptable levels but there is no formal definition of acceptable levels in the LRIA criteria. Numerous technical journals document the exit gradients at which piping is possible and these exit gradients vary drastically based on soil type. Cohesionless, fine grained, poorly graded sands are most susceptible to piping under very low gradients while cohesive clays and clay tills can generally sustain higher gradients.

The recommended engineering best practices for seepage and drainage control are consistent with Canadian Dam Safety Guidelines (CDA, 2007) and U.S Department of Interior Bureau of Reclamation (USBR 2014):

- Hydraulic gradients at the toe of the dam/dyke and in the foundation shall be low enough to prevent piping and heave in the existing material;
- For analyzing existing dams/dykes, a safety factor of 3.0 against piping/internal erosion with respect to exit gradients is considered reasonable if the structure has performed satisfactorily near normal conditions. However, a safety factor of 4.0 is recommended for a new dam/dyke or remedial repairs at an existing dam/dyke to rectify a high exit gradient situation; and

5.1.2 SEEPAGE ANALYSIS RESULTS

5.1.2.1 Simulated Phreatic Surface

The results of the transient seepage analyses of all four (4) cross-sections are shown in Figures 5-5 to 5-8, respectively. The figures show the simulated phreatic surface and seepage flow paths within the dyke at the end of the flood event (t = 72 hours).

The seepage analysis results indicated that the low-permeable dyke fill is serving to limit the seepage through the dyke at the analyzed cross sections.

FIGURE 5-5: SEEPAGE MODEL RESULT OF CROSS SECTION A-A' (T = 72 HOURS)

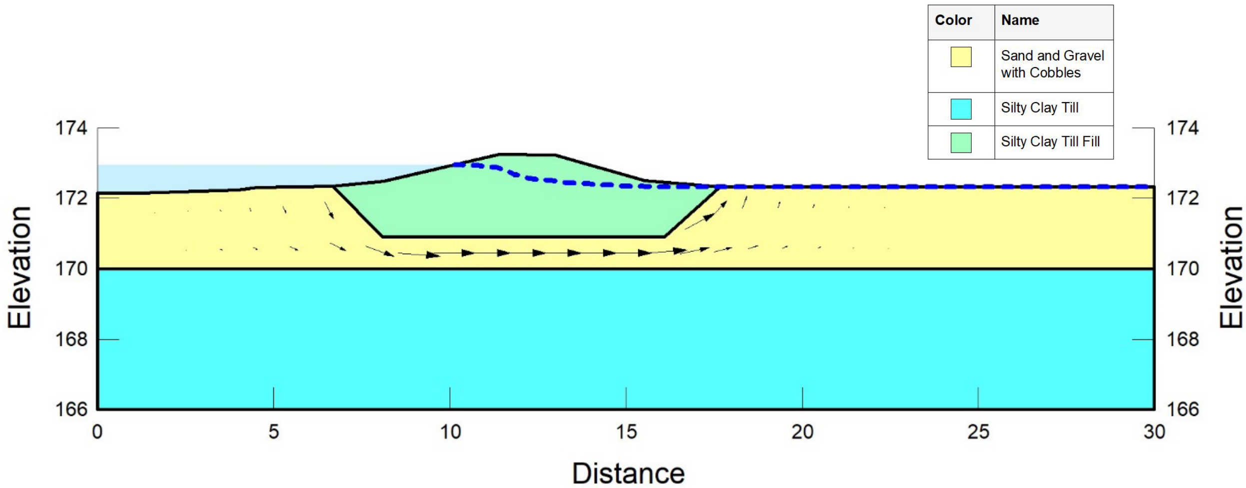


FIGURE 5-6: SEEPAGE MODEL RESULT OF CROSS SECTION B-B' (T = 72 HOURS)

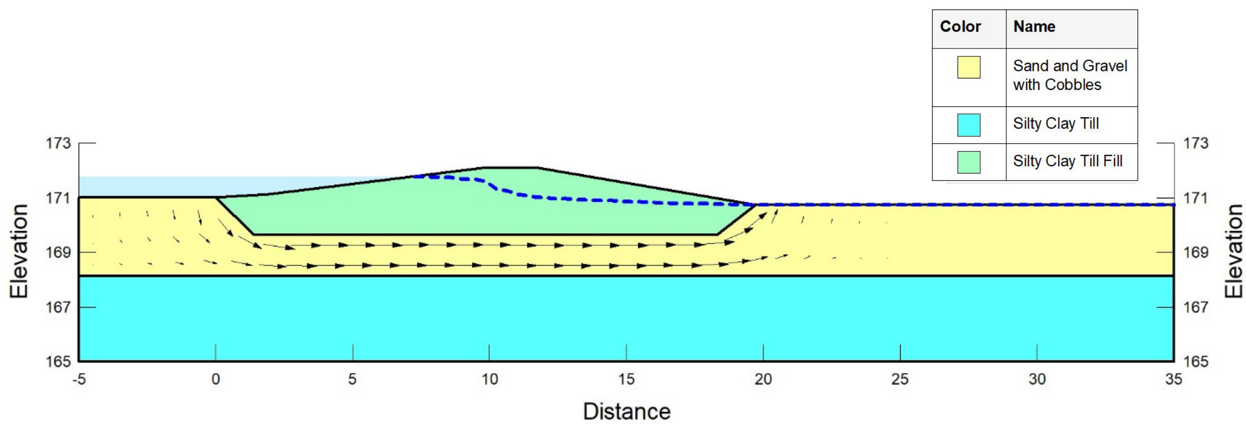


FIGURE 5-7: SEEPAGE MODEL RESULT OF CROSS SECTION C-C' (T = 72 HOURS)

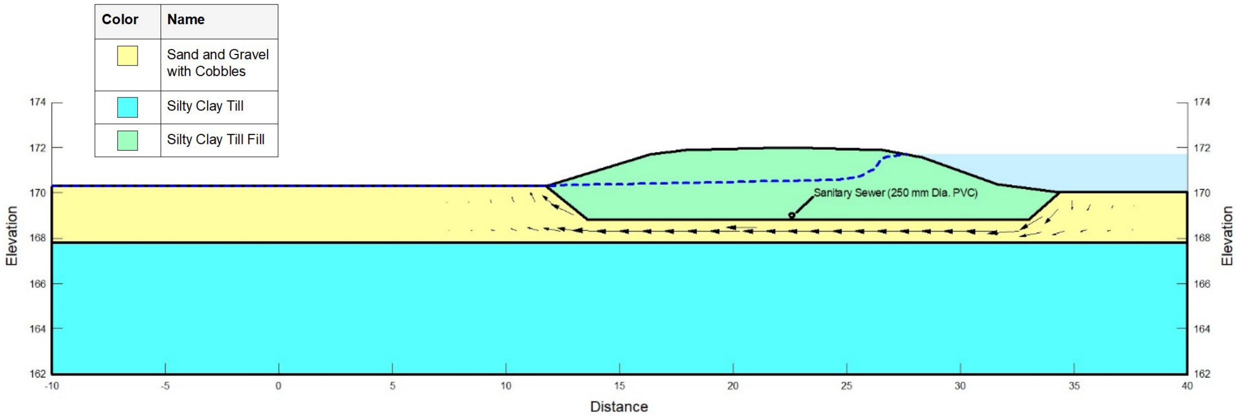
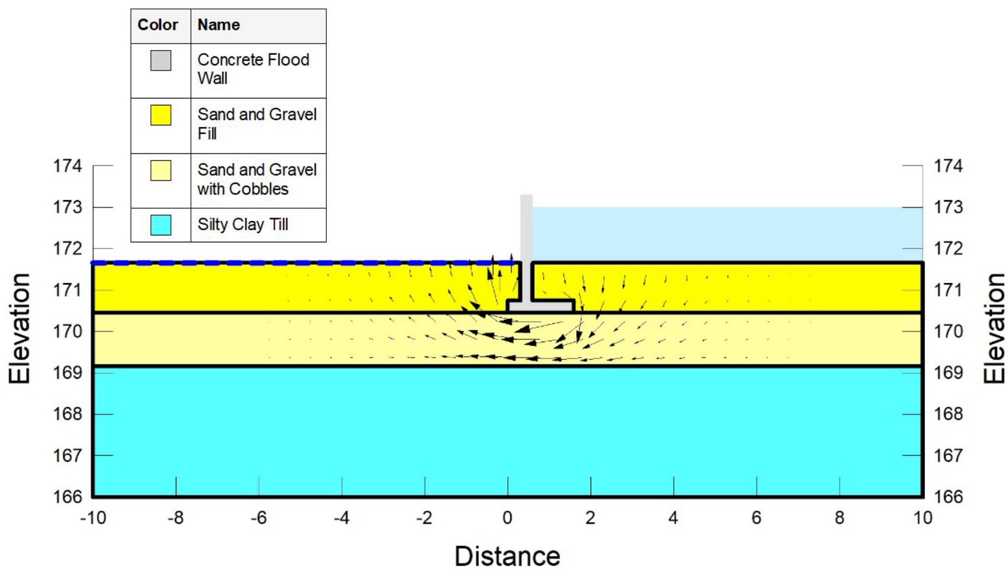


FIGURE 5-8: SEEPAGE MODEL RESULT OF CONCRETE FLOODWALL (T = 72 HOURS)



5.1.2.2 Estimated Hydraulic Gradients and Seepage Volumes

<u>Hydraulic Gradients</u>

One of the key considerations to minimize the potential for internal erosion and piping failure is limiting seepage velocities and hydraulic gradients through the dyke materials (through seepage) and foundation soils (under seepage). Piping failure/suffusion risks can be exacerbated by high seepage velocities through geological units with high permeability; therefore, controlling hydraulic exit gradients is critical to the safety of the dyke.

The estimated hydraulic gradients for the analyzed dyke sections and typical floodwall section under the assumed flood conditions were obtained from the finite element SEEP/W model and are shown on Figures 5-9 to 5-12. For the earthfill dykes (cross section A-A' to C-C') the horizontal and vertical exit gradient were examined along the ground surface at downstream of the dry slope toe. For the concrete floodwall (cross section D-D'), except for the horizontal and vertical exit gradient along the ground surface at downstream of the floodwall, the horizontal seepage gradient was also examined beneath the base of the floodwall. The factors of safety (FS) against piping (exit gradient) are summarized in Table 5-3.

Along the downstream ground surface, all the analyzed earth dyke sections met the required FS for piping/internal erosion for the assumed flood conditions (FS > 10, assuming 0.3 m freeboard).

For cross section D-D' (floodwall section), the maximum horizontal seepage gradient was 0.42 m/m below the base of the floodwall, which resulted in a FS of 1.7, assuming no impervious blanket present at the wet-side of the floodwall, which is below the recommended FS of 3. The United States Federal Emergency Management Agency (FEMA) reviewed critical exit gradients as part of their work while developing a manual for Best Practices for the Design and Construction of Filters for Embankment Dams (United States Federal Emergency Management Agency, 2011). The manual referred to laboratory tests by Schmertmann (2001), which concluded that piping can be initiated in poorly graded clean loose sands under gradients as low as 0.08; conversely, well graded sands with a Coefficient of Uniformity (Cu) greater than 6 did not exhibit piping potential even when gradients were near unity. The backfill material of the floodwall mainly consisted of sand and gravel fill, and the foundation soil is mainly poorly graded sand and gravel with cobbles.

Seepage Volume

The maximum seepage volumes through the dyke (simulated for t = 72 hours) through the cross sections A-A' to D-D' are presented in Table 5-4 for each metre-wide section of the dyke. The analyses indicated that the majority of seepage flow during a flood event occurs through the sand and gravel foundation soils. Overall, relatively low seepage quantities (0.002 to 0.12 gal/min, per metre length) were estimated at the earth dyke sections as shown in Table 5-4. A relatively higher seepage quantity (0.63 gal/min, per metre length) was estimated at the floodwall section due to the pervious sand and gravel backfill and foundation material and assuming no impervious blanket present at the wet-side of the floodwall (shorter seepage path)

TABLE 5-3: HYDRAULIC EXIT GRADIENT RESULTS

Cross-section	Exit Gradient (m/m)		Factor of Safety of Exit Gradient		Location of Max Exit Gradient with respect to Dry Side Toe/Floodwall	
	Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical
A-A'	0.01	0.03	>10	>10	D/S Toe	D/S Toe
B-B'	0.016	0.04	>10	>10	D/S Toe	D/S Toe
C-C'	0.03	0.02	>10	>10	D/S Toe	D/S Toe
D-D'	0.42[1]	0.17	1.7[1]	7.5	beneath base[1]	0.4 m D/S

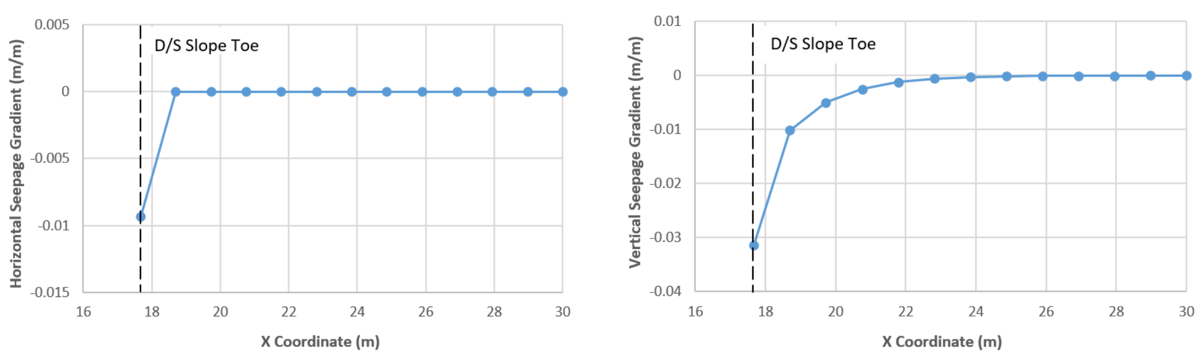
Note: [1] Horizontal seepage gradient at base of floodwall (~1.2 m below ground surface).

TABLE 5-4: SEEPAGE RESULTS

Cross-section	Seepage Volume[1] (m³/day)/(gallon/min)	Approximate Length of Segment (m)	Estimated Total Seepage Volume Through Segment (m³/day)/(gallon/min)
A-A′ (Segment 1, Earth Dyke)	0.67/0.12	35	23/4
B-B′ (Segment 3, Earth Dyke)	0.06/0.01	270	16/3
C-C′ (Segment 4, Earth Dyke)	0.01/0.002	130	1/0.3
D-D′ (Segment 2, Floodwall)	3.42/0.63	270	923/170

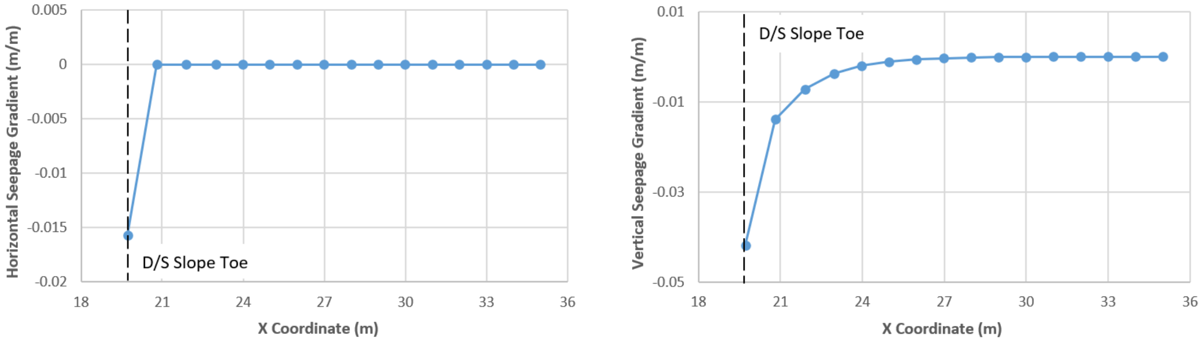
[1]Seepage volume = total volume of seepage flow through 1-m-wide section of the dam

FIGURE 5-9: SEEPAGE GRADIENT OF CROSS SECTION A-A’ (T = 72 HOURS)



(a) Horizontal Seepage Gradient (b) Vertical Seepage Gradient

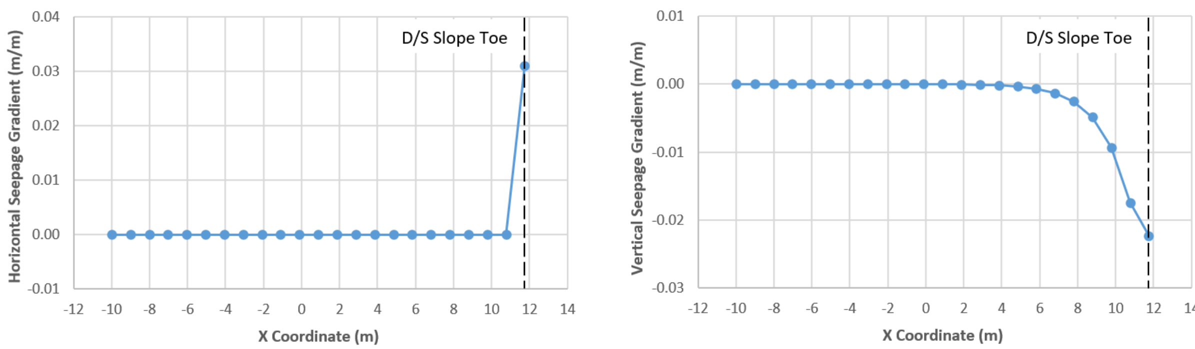
FIGURE 5-10: SEEPAGE GRADIENT OF CROSS SECTION B-B' (T = 72 HOURS)



(a) Horizontal Seepage Gradient

(b) Vertical Seepage Gradient

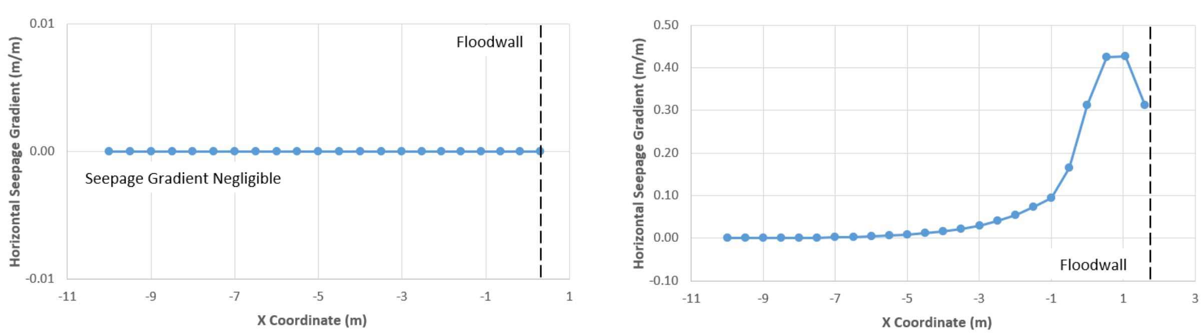
FIGURE 5-11: SEEPAGE GRADIENT OF CROSS SECTION C-C' (T = 72 HOURS)



(a) Horizontal Seepage Gradient

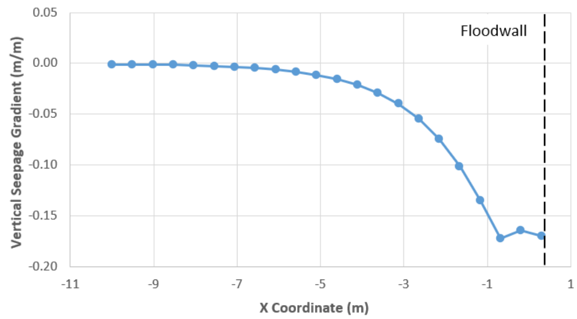
(b) Vertical Seepage Gradient

FIGURE 5-12: SEEPAGE GRADIENT OF CROSS SECTION D-D' (T = 72 HOURS)



(a) Horizontal Seepage Gradient (Ground Surface)

(b) Horizontal Seepage Gradient (Beneath Floodwall Base)



(c) Vertical Seepage Gradient (Ground Surface)

5.1.3 PIPING RESISTANCE OF DYKE FILL AND FOUNDATION MATERIALS

Another key component in determining the potential for erosion initiation is the erodibility of the earthfill and foundation material. The classification for erosion resistance of soils is outlined in the FERC Engineering Guidelines, "Risk-Informed Decision Making (Chapter R10, Internal Erosion and Piping)" (FERC) based on Sherard (1953) and is included in Table 5-5.

The dyke fill consisted of well-graded, compacted, low to intermediate plasticity with varying amounts of sand and gravel. This material can be classified as "Category 1 – Greatest Piping Resistance" according to Table 5-5. The existing dyke fill is suitable to retain water and has a low risk for piping/internal erosion.

The concrete floodwalls are tied/keyed into the earth dykes at their ends thus lowering the potential for leakage/seepage between the earthfill and concrete wall interface. The earth dykes serving as access ramps between the floodwall sections were not constructed with the walls keyed into the earth dykes. Considering the flatter earthfill slope (longer seepage path between the concrete-earthfill interface for the 1.5m of water head under flood conditions), the risk of piping between the dykes and floodwall is considered to be low and no signs of separation were observed between the earth fill and floodwalls based on the visual inspection.

The foundation soils immediately below the earth dykes consisted of poorly graded, compacted and cohesionless sand and gravel with trace to some fines, which can be classified as "Category 3 – Least Piping Resistance" according to Table 5-5. However, based on the completed seepage analyses, the Factor of Safety for piping/internal erosion was 7.5 or above, which indicates a low risk of piping failure during the flood event (0.3m freeboard at the analyzed sections of the dykes).

Considering that the flooding events are relatively infrequent, any required repair work resulting from such flooding events may be able to be addressed prior to the occurrence of any subsequent flooding events.

TABLE 5-5: EROSION RESISTANCE OF SOILS (FERC, SHERARD 1953)

Greatest Piping Resistance (Category 1)	**Plastic clay, (PI > 15), Well Compacted.**
	Plastic clay, (PI > 15), Poorly Compacted.
Intermediate Piping Resistance (Category 2)	**Well-graded material with clay binder, (6 < PI < 15), Well compacted.**
	Well-graded material with clay binder, (6 < PI < 15), Poorly compacted.
	Well-graded, cohesionless material, (PI < 6), Well compacted.
Least Piping Resistance (Category 3)	**Well-graded, cohesionless material, (PI < 6), Poorly compacted.**
	Very uniform, fine, cohesionless sand, (PI < 6), Well compacted.
	Very uniform, fine, cohesionless sand, (PI < 6), Poorly compacted.

5.2 Slope Stability Analyses (Earth Dykes)

KGS Group completed slope stability analyses to assess the suitability of the existing earth dykes for flood events. The results of the analysis were compared to the LRIA technical bulletin Geotechnical Design and Factors of Safety.

The slope stability analyses approach incorporates limit equilibrium (LE) techniques based on two-dimensional slope stability analyses using SLOPE/W software by Geo-Slope International Ltd. A seepage model (SEEP/W) that incorporates the finite element method (FEM) was set-up as part of the stability analysis to establish both the long-term (steady-state) and short-term (transient) groundwater and porewater pressure response to the design flood event hydrograph. The Morgenstern-Price method of analysis was employed for the slope stability assessment using the limit equilibrium method. This method considers both shear and normal interslice forces, and it satisfies both moment and force equilibrium.

5.2.1 DYKE GEOMETRY AND MATERIAL PROPERTIES

The geometry and zoning of the representative dyke cross sections used for the slope stability analyses were the same as used for the seepage analyses.

The key engineering parameters assigned to the various materials for the slope stability analyses are summarized in Table 5-6. The effective shear strength parameters for the dyke fill and foundation materials utilized in these analyses were estimated based on the in-situ Standard Penetration Test (SPT) data, CPT data, laboratory index testing using published and frequently used empirical correlations of shear strength versus grain size distribution and density, as well as previous experience with similar soil materials.

The slope stability models for cross sections A-A’ to C-C’ are shown in Figures 5-13 to 5-15.

TABLE 5-6: MATERIAL PARAMETERS USED IN SLOPE STABILITY ANALYSIS

Material	Effective Friction Angle, Φ' (°)	Cohesion, c' (kPa)	Saturated Unit Weight, γ_{sat} (kN/m³)	Consistency
Silty Clay Fill (CL-CI)	28	0	20	Firm to Stiff
Sand and Gravel with Cobbles (Foundation Soil)	35	0	21	Compact
Silty Clay Till (CL-CI) (Foundation Soil)	30	0	20.5	Stiff to Very Stiff

FIGURE 5-13: SLOPE STABILITY ANALYSIS OF CROSS SECTION A-A'

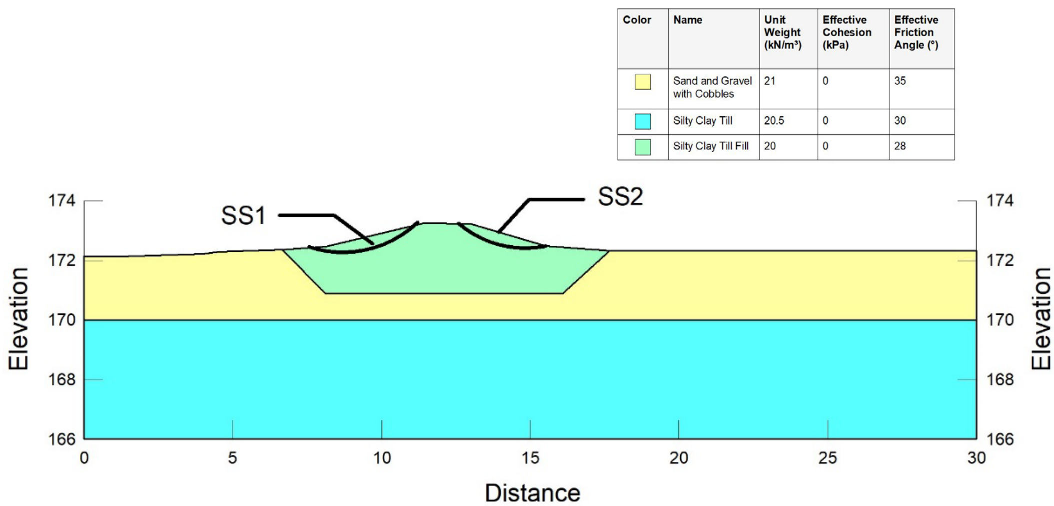


FIGURE 5-14: SLOPE STABILITY ANALYSIS OF CROSS SECTION B-B'

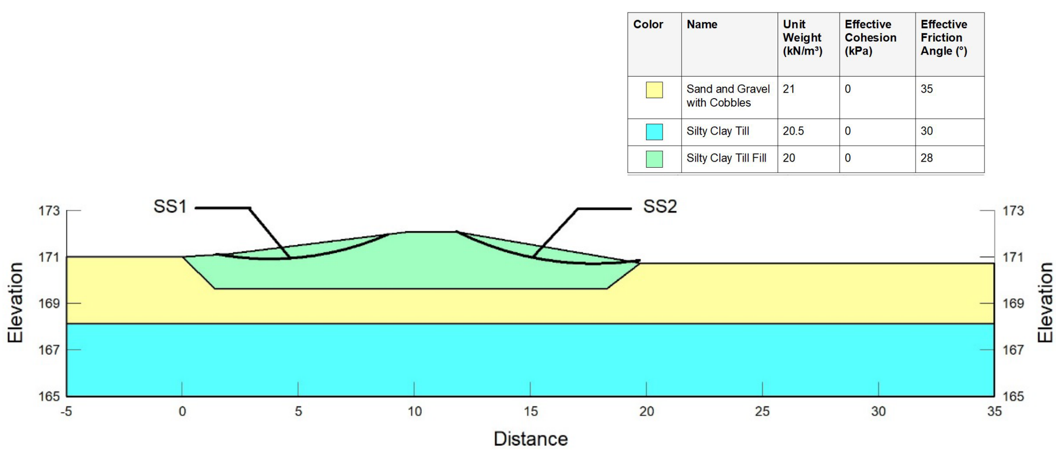
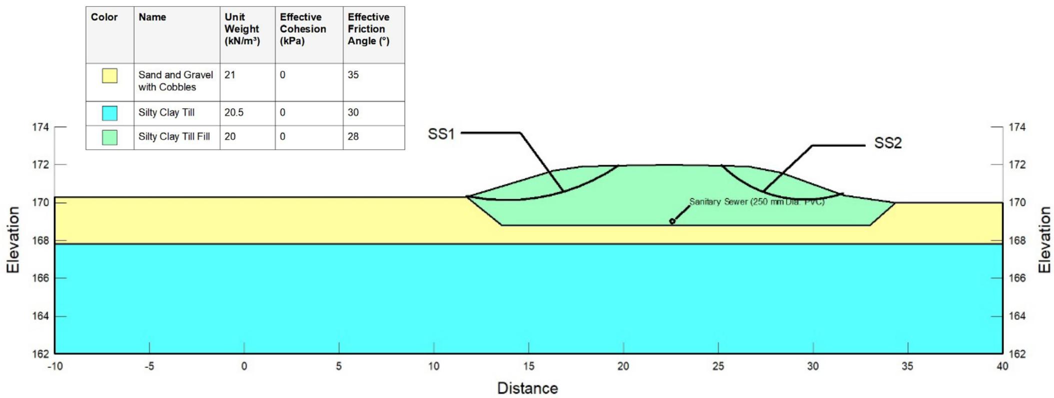


FIGURE 5-15: SLOPE STABILITY ANALYSIS OF CROSS SECTION C-C'



5.2.2 ANALYZED LOADING CASES

Load Case #1 – Long-term (Steady-State Seepage, Normal River Level)

This load case assumes that long-term steady-state seepage groundwater regime has been established within the dyke and foundation materials under the normal river level. Load Case 1, as defined by the LRIA Geotechnical Design and Factors of Safety, is most applicable to water retaining structures such as dams that see continuous impoundment at some normal maximum operating level. The expectation is that steady-state conditions might eventually develop from the wet side to the dry side of the dyke.

For the Churchville flood dyke, the river level for the normal condition is lower than both the wet and dry side toe elevations along the entire dyke, and therefore normal conditions are considered to represent 'dry' dyke conditions. The water level obtained from the 2023 KGS topographic survey (El. 169.5 m) was used as the river level under the normal condition.

Load Case #2 – Inflow Design Flood (IDF) Condition (Transient Seepage)

Load Case 2, as defined by the LRIA Geotechnical Design and Factors of Safety, is applicable to any water retaining structure that may see temporary impoundment above the normal maximum operating level. Even though the increased level is temporary, the criteria states that steady-state conditions for the IDF event should be assumed to eventually develop from the wet-side to the dry side of the dyke. As a result, a steady-state analysis assuming a flood level at the wet side of the dyke and a transient analysis considering the river level fluctuation during the flood event was carried out. The critical Factor of Safety of the dyke under both the steady-state and transient conditions were summarized and discussed (see Table 5-7).

Load Case #3 – Rapid Drawdown Condition

The rapid drawdown condition was assessed during the transient analysis and is represented by the lowest FS of the wet side slope occurring during the receding flood level.

Other LRIA Load Cases

The LRIA also lists minimum safety factors for end of construction conditions, earthquake and post earthquake conditions. The end of construction condition was not considered applicable since the dyke was constructed more than 30 years ago.

For the Churchville flood barrier, the concept of a 'Sunny Day' Hazard Potential Classification (HPC) is not applicable as defined in the LRIA/CDA/MNRF guidelines since the dykes/floodwalls are not retaining water during under normal conditions. Therefore, the earthquake loading conditions are not considered necessary. Under normal conditions, the river is typically 20m or greater from the wet-side toe of the dykes.

Table 5-7 provides a summary of the stability conditions that were assessed for the current condition of the Churchville flood dyke.

TABLE 5-7: LOADING CASES ANALYZED

Case	Loading Conditions	Minimum Factor of Safety (FS)	Slope
I	Long-term (steady-state seepage, normal river level)	1.5	Wet (River) side and Dry side
II	IDF loading condition (Transient and steady-state Seepage, regional flooding)	1.3	Wet (River) side and Dry side
III	Rapid Drawdown	1.2-1.3	Wet (River) Side

5.2.3 SLOPE STABILITY ANALYSIS RESULTS

The results of the slope stability assessment are summarized in Table 5-8. The estimated FS for the analyzed loading cases meet the LRIA/CDA/USACE criteria. Figures showing the slope stability model outputs and the potential slip surfaces that were evaluated are provided in Appendix D.

TABLE 5-8: SLOPE STABILITY ANALYSES RESULTS

Case	Description	River Level (m)	LRIA Design Criteria (FS)	Estimated Factor of Safety (FS)	
				Wet Side (SS1)	Dry Side (SS2)
Cross section A-A´					
1	Long-term Normal Conditions (Sunny day)	169.5 (Wet side slope dry)	1.5	2.83	2.47
2	Flood Event Steady-State Seepage[1]	172.94	1.3	2.26	2.13
	Flood Event Transient Seepage at 72 hrs[1]	172.94	1.3	2.21	2.09
3	Rapid Drawdown Conditions	172.94 to 169.5	1.2-1.3	2.05 (at 120 hrs)	-
Cross section B-B´					
1	Long-term Normal Conditions (Sunny day)	169.5	1.5	4.44	3.31
2	Flood Event Steady-State Seepage[1]	171.78	1.3	4.09	2.75
	Flood Event Transient Seepage at 72 hrs[1]	171.78	1.3	4.52	2.76
3	Rapid Drawdown Conditions	171.78 to 169.5	1.2-1.3	3.60 (at 138 hrs)	-
Cross section C-C´					
1	Long-term Normal Conditions (Sunny day)	169.5	1.5	1.84	1.94
2	Flood Event Steady-State Seepage[1]	171.7	1.3	2.05	1.76
	Flood Event Transient Seepage at 72 hrs[1]	171.7	1.3	2.01	1.83
3	Rapid Drawdown Conditions	171.7 to 169.5	1.2-1.3	1.49 (at 144 hrs)	-

Note 1: Assumed 0.3 m freeboard at the analyzed cross section.

6.0 STRUCTURAL STABILITY AND STRENGTH ANALYSES

The design review of the existing flood wall including the stability and strength assessments of the concrete reinforced structures has been conducted based on drawings and reports provided by the City of Brampton, as well as visual site observations. The structures were analyzed in accordance with the following documents:

- MNRF Technical Bulletins of "Lake and River improvement Act", 2011
- National Building Codes 2020
- CSA Standard A23.3

6.1 Structure Geometry

The typical geometry of the floodwalls were obtained from the 1989 as constructed drawings and 2023 topographical survey results was used to calibrate the analyses. The typical geometry of the floodwall has a stem and base thickness of 0.3 m, with a base elevation of El. 170.3 m and a top elevation of El. 173.3 m. The toe of the base slab has a length of 0.3 m and is located away from the river. The heel of the base slab was a length of 1.0 m and is located towards the river. The major steel reinforcement for the stem wall and bottom of the base slab is 20M bar spaced at 300 mm center to center. The major steel reinforcement for the top of the base slab is 15M bar spaced at 300 mm center to center.

The strength and stability analyses of the floodwall were conducted on two sections of the floodwall that are subject to the most severe loading conditions. The first configuration assessed assumes both sides of the cantilever wall retain the minimum fill elevation, as specified in the 1989 as-built drawings. For the analysis, the top foot of soil has been removed from the dry side to account for the likely presence of organic material that will not provide any lateral resistance. The major lateral pressure is from the flood water in the first configuration. The second configuration assessed assumes a full height backfill over the toe of the wall base, with the minimum fill over the heel of the base. This loading configuration was observed in several backyards. The major lateral pressure is from the earthfill in the second configuration. The two configurations are shown in Figures 6-1 and 6-2.

FIGURE 6-1: CONFIGURATION 1 GEOMETRY

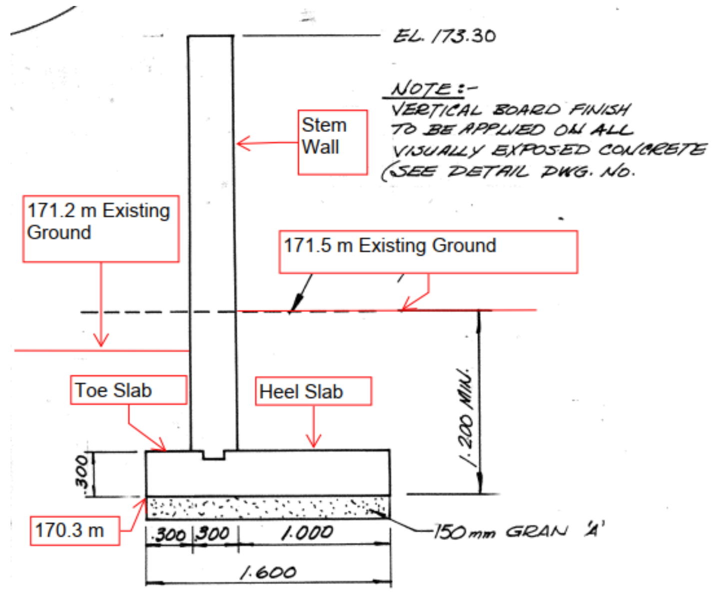
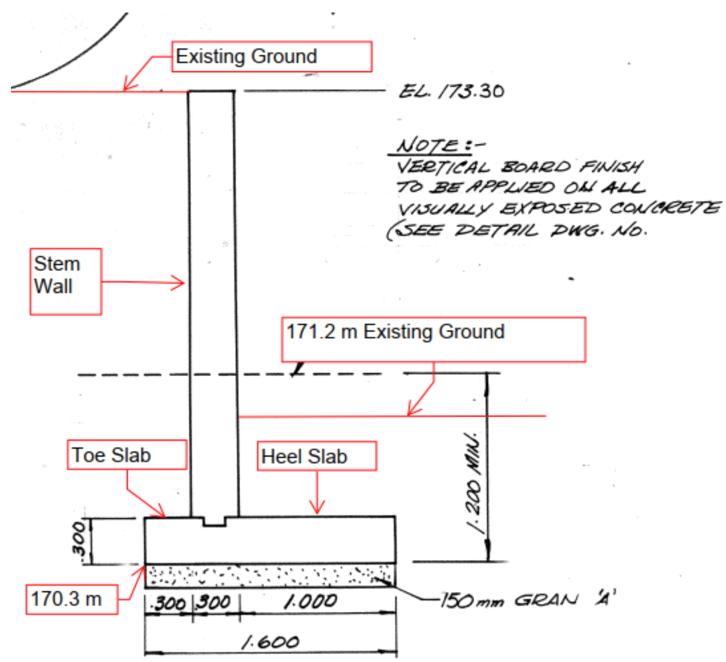


FIGURE 6-2: CONFIGURATION 2 GEOMETRY



6.2 Load Parameters

6.2.1 LOAD CASES

The floodwall configurations mentioned above have been assessed for strength and stability under three different loading conditions:

- Load Case 1: Usual conditions, no water present above the base of the floodwall.
- Load Case 2: Flood/Wet conditions, maximum expected water levels or soil pressures.

6.2.2 SOIL BEARING CAPACITY AND CONCRETE TO SOIL SHEAR STRENGTH PARAMETER

Based on the geotechnical investigations, the bearing soil material is composed of compact sand and gravel, which has an effective friction angle of 35^o with zero cohesion. It is assumed this will yield a concrete to soil effective friction angle of 23^o (2/3 of effective soil friction angle) with zero cohesion.

The following bearing capacity may be used for the stability analysis:

- Serviceability Limit State (SLS) = 175 kPa (19mm settlement)
- Ultimate Limit State (ULS) = 350 kPa

6.2.3 SOIL FILL AND LATERAL EARTH PRESSURE

The fill material on both sides of the stem wall is composed of a sand and gravel fill, which has an effective friction angle of 35 degrees and zero cohesion. The fill material was found to have a unit weight of 21 kN/m^3.

Lateral earth pressure on the floodwall was carefully considered as the assumptions have a major impact on the stability assessment results. The “at-rest” earth pressure coefficient of earth pressure, K_0 is taken as:

$$K_0 = 1 - \sin(\varphi')$$

Where:

φ' = internal friction angle (effective stress) of the backfill material.

To maintain water-tightness and water stop integrity, the deflection/movement at the top of the concrete gravity sections should be insignificant and, therefore, not be able to initiate the “active” earth-pressure wedge under lateral earth pressure. However, in the event that the existing floodwall structure goes into an unstable mode and some rotation/translation takes place, the lateral pressure state will be reduced to the “active” pressure state. The stability assessment of the floodwall was based initially on the at-rest pressures. However, if the assessment results indicate the wall is sliding and/or overturning under the at-rest pressures, the lateral pressure reduction towards the active pressure state was taken into consideration. Thus, for sensitivity purposes, the walls were also assessed for the active pressures if the walls did not meet the 2011 MNRF criteria using at-rest pressures. The estimated coefficients for the lateral pressures used in the stability analyses were calculated based on the site-specific material properties, the slope geometry of the backfill, and the geometry of the walls.

Earth loads below groundwater level was calculated using buoyant unit weights acting in conjunction with the associated water loads.

6.2.4 LIVE LOAD SURCHARGE

Live load surcharges have been included in several of the load cases, during flood condition, it is expected that no live load will be present on the ground above the floodwall. For Load Case 1, it is assumed that a live load surcharge of 4.8 kPa may be present above the driving side due to human assembly or similar. In Load Case 2 for the second configuration, it is assumed that a live load of 2 kPa may be present due to residential use.

6.2.5 WATER PRESSURE

The first floodwall configuration has flood water levels of 173.0 m (0.3 m below the top of the floodwall) on the heel side. It was assumed no water is present above the base of the foundation from the toe side. There was no data to estimate the groundwater within the backfill for the second floodwall configuration.

6.2.6 UPLIFT

Full uplift, varying linearly from 100% headwater pressure at the upstream face to 100% tail water pressure at the downstream face was assumed.

6.3 Stability Analysis

The overall stability of the retaining walls was reviewed at the soil/concrete wall interface. Where the groundwater table is present within backfill, the hydrostatic lateral and any uplift forces were taken into account at the base.

In general, this assessment has adopted the dam acceptance criteria of the 2011 MNRF for the retaining walls. A summary of the general loading parameters used for the stability analyses is provided in Table 6-1.

TABLE 6-1: GENERAL PARAMETERS USED FOR STABILITY ANALYSES

Parameter	Value
Water Unit Weight	9.81 kN/m^3
Effective Friction Angle at Concrete to Soil Foundation	23°
Cohesion at Concrete to Soil Interface	0.0 kPa
Ultimate Soil Bearing Capacity (assumed)	250 kPa
Concrete Unit Weight (assumed)	23.5 kN/m^3
Concrete Compressive Strength (from 1989 as constructed drawings)	25 Mpa
Backfill Materials Used in Analysis Internal Friction Angles of Soil Backfill Materials Unit Weight of Soil Backfill Materials	Sand and Gravel fill 35° 21 kN/m^3

6.3.1 SLIDING STABILITY

The sliding stability at each horizontal analysis plane is verified by using the following shear friction equation:

$$FS_{SLIDING} = \frac{V\tan(\emptyset)+cA}{H}$$

where:

V = Sum of vertical forces including uplift (kN);

H = Sum of horizontal forces (kN);

A = Area in compression (m^2)

Tan(ϕ) = Friction coefficient

c = cohesion (Mpa)

6.3.2 LOCATION OF THE RESULTANT

The location of the resultant force is calculated by applying all the vertical and horizontal loads and calculating the sum of the moments caused by these forces relative to the toe of the structure. The location of the resultant force on the base is calculated using the following equation:

$$\text{Resultant Location} = \frac{M}{V}$$

Where:

M = Sum of moments (kN*m)

V = Sum of vertical forces including uplift (kN)

The portion of the base in compression is calculated as follows:

$$\% = \frac{Area\ of\ the\ base\ in\ compression}{Total\ base\ area}$$

The area of the base in compression is calculated by subtracting the portion of the base that is in tension from the initial total area. This is an iterative process since stresses are calculated using the area and the inertia of the base that is in compression only, when cracking occurs. At the end of the iterative process, if the solution converges, there should be no tension and the part that has no compression is referred to as the "cracked base". If a cracked plane was indicated by a portion of the base not being in compression, the stress distribution and shear-friction safety factor were calculated and distributed along the uncracked portion.

For a rectangular base, the following base compression percentages can be associated with the location of the resultant force:

- 100%: Resultant is within the middle third of the base;
- 75%: Resultant is within the middle half of the base;
- > 0%: Resultant is within the base.

6.3.3 ACCEPTANCE CRITERIA

The acceptance criteria based on the 2011 MNRF Technical Bulletin for dam structures with cohesion assumed to be zero are summarized in Table 6-2.

TABLE 6-2: MNRF ACCEPTANCE CRITERIA

Loading Case	Load Combinations	
	USUAL (LC1)	UNUSUAL (LC2)
Sliding Stability Factor (SSF)	1.5	1.3
Location of Resultant	Within Middle-third of the base /1/	Within Middle-half of the base

/1/ For existing structures, it may be acceptable to allow a small percentage of the base to be under 0 compression if sliding factors of safety are met, the resultant is within the base of the dam and allowable bearing stresses are not exceeded.

6.3.4 STABILITY ANALYSIS RESULTS

Table 6-3 and Table 6-4 show the summary of the results of the stability analysis at the base of the concrete floodwall. The detailed calculations are provided in Appendix E.

TABLE 6-3: RESULTS OF STABILITY ANALYSIS – CONFIGURATION 1

Loading Case		Case 1 Dry	Case 2 Flood
Sliding Stability Factor (SSF)	MNRF Required	**1.5**	**1.3**
	Computed	3.16	1.35
Location of the Resultant	MNRF Required	**Within Mid-Third**	**Within Base**
	Computed	Within Mid-Third	Within Base
Resultant Location from Toe End (m)		0.71	0.19
Percentage of Base in Compression		100%	36%
Maximum Bearing Stress (kPa)		47.8	162.3

TABLE 6-4: RESULTS OF STABILITY ANALYSIS – CONFIGURATION 2

Loading Case		Case 1 Dry	Case 3 Assumed Groundwater within Backfill
Sliding Stability Factor (SSF)	MNRF Required	**1.5**	**1.3**
	Computed	1.95	1.33
Location of the Resultant	MNRF Required	**Within Mid-Third**	**Within Base**
	Computed	Within Mid-Third	Within Mid-Half
Resultant Location from Heel End (m)		0.68	0.43
Percentage of Base in Compression		100%	80%
Maximum Bearing Stress (kPa)		55.2	67.8

Based on the calculations, both representative sections of the floodwall were found to meet the minimum required sliding factors of safety for all loading cases (1 and 2). By conducting a sensitivity analysis, it was determined that the second configuration of the stem wall analyzed is able to withstand groundwater up to El. 172.0 m prior to no longer meeting the required sliding factor of safety. As mentioned in Table 6-2, it is acceptable for the resultant to be located outside the middle-third of the base during the flood/wet condition, since the floodwall is an existing structure with an allowable sliding factor of safety and maximum bearing stress.

6.4 Strength Analysis

6.4.1 METHOD OF ANALYSIS

The strength analysis assessed the strength of the stem wall, the toe slab, and the heel slab of the floodwall. Strength analysis was conducted in accordance with the CSA A23-3, Concrete Design Handbook. The stem wall, the toe and heel slab of the floodwall were all analysed to determine if the existing concrete and reinforcement are able to support the factored shear and moment loads the members are subject to under the specified loading conditions. The reinforcement configuration has been obtained from the 1989 as constructed drawings, as shown in Figure 6-3.

FIGURE 6-3: FLOODWALL SECTION REINFORCEMENT DETAILS

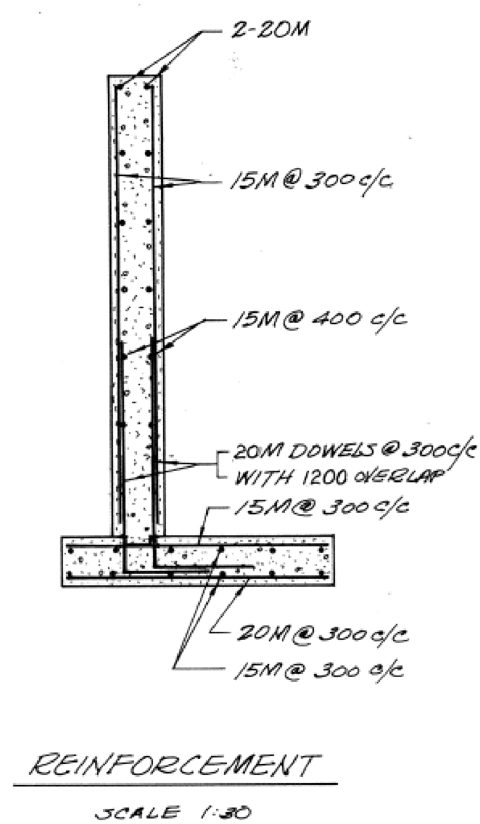


TABLE 6-5: GENERAL PARAMETERS USED FOR STRENGTH ANALYSES

Parameter	Value
Concrete Compressive Strength (from 1989 as constructed drawings)	25 Mpa
Steel Reinforcement Yield Strength	400 Mpa
Stem Wall Primary Rebar Type	20M
Base Slab Top Primary Rebar Type	15M
Base Slab Bottom Rebar Type	20M
Primary Rebar Spacing c/c (Stem Wall and Base Slab)	300 mm
Minimum Concrete Cover	75 mm

The maximum aggregate diameter used in the concrete mix could not be obtained from the available background information, it was assumed to be 20 mm, as is used in the strength calculations. To account for the uncertainty, a sensitivity analysis was performed with the maximum aggregate diameters ranging from 10 mm to 30 mm, the floodwall met the strength requirements for the entire range.

6.4.2 STRENGTH ANALYSIS RESULTS

Table 6-6 shows the summary of the results of the strength analysis for the stem wall, heel slab, and toe slab of the floodwall. The detailed calculations are provided in Appendix E.

TABLE 6-6: STRENGTH ANALYSIS RESULTS

Configuration 1				
		Stem Wall	Heel Slab	Toe Slab
Factored Forces	V_f (kN)	**31.67**	**44.61**	**51.73**
	M_f (kNm)	**31.95**	**25.53**	**8.56**
Shear Capacity	V_r (kN)	179.07	186.07	190.13
	Check	Ok	Ok	Ok
Moment Capacity	M_r (kNm)	68.52	68.52	47.48
	Check	Ok	Ok	Ok
Configuration 2				
		Stem Wall	Heel Slab	Toe Slab
Factored Forces	V_f (kN)	**31.27**	**54.04**	**24.61**
	M_f (kNm)	**33.36**	**32.02**	**4.22**
Shear Capacity	V_c (kN)	176.63	140.82	253.25
	Check	Ok	Ok	Ok
Moment Capacity	M_r (kNm)	68.52	47.48	68.52
	Check	Ok	Ok	Ok

Based on the results of the strength analysis, the stem wall, heel slab and toe slab of the floodwall have the required strength capacity to withstand the expected loads.

6.4.3 SENSITIVITY ANALYSIS

During the site visit, two sections of exposed and corroded rebar found just above the existing ground level, located 21 m from the most northern corner of the floodwall, see Figure 6-4. The floodwall at this location has ground elevations as shown in configuration 1. A sensitivity analysis was conducted on the stem wall for configuration 1 to determine if in the event of rebar failure due to increased corrosion, the stem wall will have sufficient strength to withstand the expected loads.

FIGURE 6-4: EXPOSED REBAR ON THE FLOODWALL

A summary of the results from the sensitivity analysis are provided in Table 6-7. It was determined that the stem wall at this location has the strength capacity to withstand all expected loading cases with up to two consecutive steel rebars failing. To determine the response of incremental bar failures, an equivalent bar spacing was input to the strength analysis that increases by 300 mm for the failure of one bar.

TABLE 6-7: REBAR FAILURE SENSITIVITY ANALYSIS

Number of consecutive bars failed	Equivalent c/c spacing (mm)	V_f <	V_r	M_f <	M_r	Check
1	600	31.67	112.63	31.95	35.34	Ok
2	900	31.67	86.74	31.95	23.8	Not Ok

Based on the sensitivity analysis, the stem wall will still have sufficient strength after the failure of one rebar; however, it does not have adequate strength for the failure of two consecutive rebars. It is recommended to perform a concrete repair of the section to prevent further corrosion of the rebar.

7.0 OPERATIONS, MAINTENANCE, AND SURVEILLANCE (OMS)

The City has established recurring inspection and maintenance programs for the storm sewers, culverts, and backflow prevention practices as detailed in Appendix F. These include activities such as removal of excessive vegetation growth at storm sewer and culvert outlets and recurring mowing programs. It is understood that there is no document prepared for the operations and maintenance of the earth dykes and floodwalls. The current vegetation growth does not appear to be an immediate threat to the dyke's performance, however, several larger/mature trees were observed growing in close proximity to the concrete floodwall. In addition, vegetation growth was dense at select areas of the earth dykes including the 2022 overtopping location.

In general, well established woody tree growth on water retaining structures is not desirable and could lead to detrimental impacts on performance:

- Fallen or uprooted trees could cause damage to the floodwall and/or earth dykes
- Decaying roots may create seepage paths below the floodwall that could lead to internal erosion and piping of foundation materials. Also, roots could wedge into joints or cracks in the concrete structure which could further open the joints and increase the seepage or piping potential.
- Tree and vegetation growth can cause interference with effective safety monitoring, inspection and maintenance, particularly where the cover is dense.
- Cracking, uplifting or displacing concrete structures can occur with root penetration.
- Root growth results in loosening of earth materials, which is of particular concern with tree uprooting and root decay over time.
- Tree growth can hinder establishment of more desirable vegetation cover such as grasses, which can ultimately lead to increased erosion.

Routine vegetation removal is recommended to facilitate ongoing monitoring of dyke performance so that any irregularities that could compromise the stability of the dyke slopes can be identified promptly and remedied in a timely manner. Removal of any excessive vegetation growth and prevention measures for future growth is recommended. In particular, any identified uprooted or dead trees should be promptly removed. Further assessment is required prior to removing any substantially large trees from the earth dykes.

An emergency program is necessary for major flood events in order to ensure that appropriate actions are taken in a timely manner during flood conditions. An emergency program is outlined in the City of Brampton Emergency Plan (By-Law Number 265-2014) and the City of Brampton Evacuation Plan. It is recommended to carry out periodic reviews/assessments of the current emergency action plans to ensure that the emergency programs are up to date. It is also recommended to ensure that all personnel responsible for dyke and floodwall surveillance/maintenance are trained in dyke safety and are able to recognize basic deficiencies that may lead to more serious safety issues.

8.0 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations provided in this section were provided under the assumption that the earth dykes and concrete floodwall were originally designed to protect against flooding for up to the 100-year (open water) flood event including 0.3m of freeboard based on the 1985 flood study report.

8.1 Key Background Review Findings

The following conclusions are based on the results of the background document review, limited topographical survey completed at the site and visual site observations:

- The 1985 flood study report indicated the flood protection infrastructure was originally designed to protect up to the 100-year (open water) flood event with 0.3m of freeboard.
- The top of the flood barrier decreases in elevation from El. 173.3 m to El. 172 m in the downstream direction along the flood reach to follow the profile of the design flood elevations.
- The original 1985 flood protection design concept slightly deviates from what was constructed at the site in 1989. In particular, the earth dyke was originally proposed on the south side of Martins Blvd but was ultimately constructed as an integrated road/dyke structure. The as-constructed dyke in this section may have more flood conveyance than the 1985 concept.
- A section of the earth dyke east of the Churchville Road bridge was removed to facilitate the construction of a sanitary sewer crossing the Credit River in 2006. There was no information available detailing the foundation preparation, dyke reconstruction materials, construction methodology, etc.

8.2 Condition Assessment

A visual site inspection was carried out to assess the condition of the earth dykes and concrete floodwalls, as well as a topographical survey. A summary of the key observations from the inspection and survey is provided below.

Topographical Survey

- A section of the earth dyke (Segment 1 – northern dyke section) currently does not have enough freeboard for the 100-year flood event based on the topographical survey completed at the site and the flood levels provided in the 1985 report (0.1 m freeboard).
- The existing concrete floodwall has adequate freeboard for the 100-year (open water) event.
- It’s by design that the current earth dyke and floodwall system is not suitable for the regional flood event. The 1985 study determined a 100-year+ freeboard protection provided the best cost/benefit. Based on the 1985 water levels, the regional flood would overtop the dykes and floodwall by approx. 0.15m to 0.3m.

Earth Dykes

- In general, the overall condition of the earth dykes was satisfactory. There were no observed slope movements, significant depressions or erosion that would suggest significant concerns related to the slope stability and performance of the dykes.

- The crest elevation was found to be lower than its surrounding dyke section at two (2) locations:
  - Ch. 0+282 (earth dyke east of Churchville Road bridge) – this area was partially reconstructed while installing a sanitary sewer in 2006.
  - Ch. 0+420 (near box culvert outlet) –location of overtopping during the ice jam flood in 2022.
- The culverts and inline check valves through the earth dyke sections were in satisfactory condition without any significant deficiencies. However, there will be some risk that river ice can obstruct their function as seen in Feb 2022.
- Vegetation growth was dense at select earth dyke sections which impeded the visual inspection. Particularly, the following sections contained denser vegetation growth:
  - Ch. 0+020 to 0+038 (Segment 1, northern dyke section)
  - Ch. 0+400 to 0+435 (Segment 3, from fence at property line to stormwater box culvert outlet)

Concrete Floodwalls

- In general, the concrete walls were in satisfactory condition with no movement or significant structural deficiencies.
- Localized exposed rebar was visible near the base of the floodwall (wet-side of wall near ground level) between properties 7780 and 7772.
- Several larger/mature trees were observed growing near the concrete floodwall, particularly between properties 7780 and 7772 (wet side of floodwall) and east of the bridge on Churchville Road.
- At 7772 Churchville Road, some of the supports for the stairs are anchored into the concrete floodwall. It does not appear that the concrete floodwall was originally designed to support the stairwell.
- The small flap gate outlet at the southeast corner of the wall segment (Ch. 0+110) was partially obstructed with debris.

8.3 Geotechnical Investigation Results

A geotechnical investigation was completed to assess the earth dyke, floodwall backfill and foundation materials to support the condition assessment. The investigation program consisted of CPTu soundings, exploratory test holes and index laboratory testing to characterize the subsurface soils.

A summary of the materials observed during the investigation program is provided below:

Low Permeable Dyke Fill – Silty Clay Fill

- Brown to reddish brown, moist, firm to stiff, of low to intermediate plasticity (CL-CI) and contained some sand and gravel.
- 6% to 15% gravel, 21% to 42% sand, 32% to 44% silt and 13% to 30% clay.
- Liquid Limit between 24 and 40, Plastic Limit between 13 and 20, and Plasticity Index between 11 and 21. Moisture contents for the samples ranged from 17% to 23%.
- The dyke fill is suitable to retain water and has a low risk for piping/internal erosion.

Floodwall Backfill – Sand and Gravel Fill

- Brown and grey, moist, compact, medium to coarse sand, medium to coarse gravel, and trace fines.

Foundation Soil immediately below dyke – Poorly Graded Sand and Gravel with Cobbles

- Brown, moist to wet, compact, poorly graded, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, and trace to some fines.
- 29% to 31% gravel, 34% to 48% sand, and 23% to 36% fines. Moisture contents for the samples ranged from 11.6% to 17.6%.
- Thickness of the sand and gravel (Alluvial deposit) was 0.8 to 1.6 m at the test hole locations.

Foundation Soil - Silty Clay Till

- Grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel.
- 6% to 8% gravel, 30% to 39% sand, 38% to 43% silt and 13% to 23% clay.
- Liquid Limit between 20 and 36, Plastic Limit between 11 and 15, and Plasticity Index between 9 and 21. Moisture contents for the samples ranged from 13% to 16%.

8.4 Seepage and Stability Analyses

Earth Dykes

Seepage analyses were performed to evaluate the transient groundwater and porewater pressure response of the earth dykes and their foundation soils under flood conditions. In addition, slope stability analyses were completed to assess the suitability of the existing dyke for flood events.

Based on the completed seepage analyses, all analyzed dyke sections met the required FS for piping/internal erosion for the sand and gravel foundation soil for the assumed flood conditions (0.3 m freeboard). The Factors of Safety for piping/internal erosion was >10, which indicates a low risk of piping failure through the foundation soils during the flood event.

Based on the slope stability analyses, the estimated FS for the analyzed loading cases meet the LRIA/CDA/USACE criteria.

Floodwall

Seepage analysis was performed to evaluate the transient groundwater and porewater pressure response below the floodwall (through sand and gravel foundation soil) under flood conditions. A stability and strength assessment of the concrete reinforced floodwall structures was also completed based on the typical section provided in the as-constructed drawings, using parameters obtained from the site investigation program.

Based on the completed seepage analyses, the analyzed floodwall section is estimated to have a maximum horizontal seepage gradient of 0.42 m/m below the base of the wall, which resulted in a FS of 1.7 which is below the recommended FS of 3, assuming no impervious blanket present at the wet-side of the floodwall. A relatively higher seepage quantity (0.63 gal/min, per metre length) was estimated at the floodwall section due to the pervious sand and gravel backfill and foundation material, and assuming no impervious blanket present at the wet-side of the floodwall (shorter seepage path).

Based on the calculations, the floodwalls were found to meet the minimum required sliding factors of safety for all loading cases for the assumed flood conditions (0.3 m freeboard). Based on the results of the strength analysis, the stem wall, heel slab and toe slab of the floodwall have the required strength capacity to withstand the expected loads. A sensitivity analysis was conducted on the stem wall to determine if the stem wall will have sufficient strength in the event of rebar failure due to increased corrosion of the localized exposed rebar at the base of the wall. Based on the sensitivity analysis, the stem wall will still have sufficient strength after the failure of one rebar; however, it does not have adequate strength for the failure of two consecutive rebars.

8.5 Asset Management Strategy

A list of deficiencies for the earth dykes and floodwalls was developed based on the background document review, condition assessment results and analyses completed as part of this study. Recommendations were developed to address the deficiencies and establish the City's asset management strategy for the maintenance of the earth dykes and floodwalls. The deficiencies and the associated recommendations are listed in Table 8-1.

Expected Remaining Lifespan

The remaining lifespan of the Churchville flood barriers is dependent on the continued maintenance and care of the structures. Re-evaluation of the remaining lifespan of the structures should be carried out during future engineering studies.

- **Earth Dykes** – Generally, earth dykes can be relied on indefinitely provided they continue to meet the current stability criteria and that their overall conditions are kept satisfactory (i.e., vegetation growth is controlled, prompt repair of any damage caused by erosion or external factors such as human activities or extreme weather events, etc.). As the Churchville earth dykes were found to meet the stability criteria and were found to have a low risk of piping, the earth dykes are expected to continue to perform well in the foreseeable future provided they are properly maintained as recommended in Table 8-1.
- **Concrete Floodwalls** – Typically, concrete structures have an expected life between 70-90 years if there have been so significant changes to their design assumptions, however this is dependent on their overall condition and shorter/longer lifespans may be expected. Based on the as-found condition of the floodwalls during this study, the floodwalls are expected to continue to perform well for the next 35-55 years provided they are properly maintained and repaired as necessary.

Recommendations are provided with the following Priority Ranking system:

High: Work that needs to be done to meet current regulations and safety requirements. Generally, it is the result of an identified deficiency and needs to be attended to within the next 2 years.

Medium: These deficiencies may include additional work that could improve safety or issues that may become deficiencies. These items should be addressed before the next formal condition assessment/study.

Low: These are opportunities for improvement. These issues are not currently considered to be urgent and can be scheduled at the City’s convenience.

TABLE 8-1: DEFICIENCIES AND RECOMMENDATIONS FOR THE CHURCHVILLE EARTH DYKES AND FLOODWALLS

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
1	The river levels for the flood events (100-year return period and regional flood event) considered as part of this study correspond to the original 1985 flood protection study. Segment 1 (earth dyke) does not have adequate freeboard (0.3m) for 100-year flood as recommended in the 1985 report (existing freeboard is 0.1 m). However, this location did not overtop during the ice jam in February 2022.	The flooding risk (water levels associated with annual probability) in Churchville should be further assessed both for open water (100-year up to 350-year return period) and for ice jam conditions. Evaluate the freeboard along the length of the earth dykes and floodwalls. If the freeboard deficiencies are found, consider raising the earth dyke and/or wall using suitable dyke fill material to accommodate 0.3m freeboard. Re-assess the seepage and stability analyses based on the revised hydraulic study.	Study (High)	$ 60,000	$ -	Stormwater Programs
2	There are two (2) locations where the top (crest) elevation of the earth dyke is lower than its surrounding parts. Ch. 0+282 (east of Churchville Road bridge) Ch. 0+420 (location of overtopping during 2022 flood)	Raise the earth dyke section using suitable dyke fill material. The repair should involve technical specification/design by an engineer, removal of the surficial topsoil/organic rich material, placement and compaction of new fill approved by a geotechnical engineer. Topographical surveys should be carried out before and after placement of new dyke fill to confirm crest elevation data.	Repair (Medium)	$ 15,000	$ 30,000	
3	A section of the earth dyke east of the Churchville Road bridge was removed to facilitate the construction of a sanitary sewer crossing the Credit River in 2006. There was no information available detailing the foundation preparation, dyke reconstruction materials, construction methodology, etc. A depression was observed at the crest of this dyke section which may be associated with settlement following reconstruction.	Complete a confirmatory site survey to locate the reconstructed section and carry out a drilling investigation including soil sampling to assess the dyke fill. SPT drilling and sampling is preferred to assess the soil consistency and fill quality.	Background Review / Investigation (High)	$ 10,000	$ 15,000	
4	A higher horizontal seepage gradient and relativity higher seepage quantities were estimated at the base of the floodwall under flood conditions, with the assumption that no-low pervious blanket is present at the wet-side toe of the wall.	Complete a geotechnical investigation at the wet-side of the floodwall to determine the characteristics and thickness of the pervious soil (sand and gravel — alluvium deposit). Shallow test holes, frequent sampling at several locations and lab testing should be carried out to assess the subsurface soils. Re-assess the seepage analysis with the updated geotechnical information.	Investigation / Study (High)	$ 25,000	$ 20,000	
		If no low-permeable soils are present, consider installation of a clay blanket to reduce hydraulic gradient and increase the seepage path below the floodwalls during flood conditions.	Repair (Medium)	$ 30,000	$ 120,000	

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
5	There are no agreements with private landowners or easements to carry out required maintenance activities of the floodproofing infrastructure on private properties.	Consider options for establishing access and maintenance responsibilities between the City and property owners, such as acquiring easements and/or establishing a maintenance agreement.	Study (Medium)	$ -	$ 180,000	
			Subtotal:	**$ 140,000**	**$ 365,000**	
Short Term Maintenance & Repairs						
6	Several larger/mature trees were observed growing in close proximity to the concrete floodwall, and vegetation growth was dense at select earth dyke sections.	Carry out tree clearing and brush vegetation overgrowth throughout the floodwall and earth dykes as necessary. Carry out regular mowing. Apply herbicide where required to prevent future overgrowth. Recommend clearing overgrowth 3m from toe of dykes and floodwall. Prior to any tree removal, arborist report must be obtained and a Tree Removal Application filed with the City. Obtain permission from private property owner if tree is not on public lands.	Maintenance	$ -	$ 8,000	Road Operations/Contract Services
		For substantially large tree removals, carry out the removal and restoration of the earthfill materials under the supervision of a geotechnical engineer.	Maintenance	$ -	$ 5,000	
7	At the floodwall near 7772 Churchville Road, some of the supports for the wooden stairs are anchored into the concrete floodwall. It is not clear that the concrete floodwall was originally designed to support the stairwell.	Remove or alter the stairs so as to be independent from the concrete wall.	Repair (Low)	$ -	$ 10,000	
8	There is exposed rebar near the base of the floodwall (wet-side of the wall near ground level) between properties 7780 and 7772 Churchville Rd. The rebar should have at least 2 inches (50mm) of concrete cover. Sensitivity analysis indicated that the wall does not have adequate strength for failure of two consecutive rebars.	Complete technical specification by engineer and carry out localized repair of the wall/rebar.	Repair (Medium)	$ -	$ 15,000	
			Subtotal:	**$ -**	**$ 38,000**	
Annual Inspections & Maintenance						

Item	Deficiencies	Recommendations	Category (Priority)	Study Cost Estimate	Implementation Cost Estimate	Implementation Lead
Municipal Class Environmental Assessment						
9	Flap gates and inline check valves require periodic inspections to ensure functionality is maintained.	Flap gates and inline check valves should be inspected at least annually and during/after ice jam and flood events.	Maintenance	$ -	$ -	Stormwater Programs
10	No standalone document exists for documenting the operations and maintenance requirements of the earth dykes and floodwalls.	Develop site-specific OMS procedures for the Churchville flood barrier. Ensure that all personnel responsible for dyke and floodwall surveillance/maintenance are trained in dyke safety and are able to recognize basic deficiencies that may lead to more serious safety issues.	Study (Medium)	$ 20,000	$ -	Capital Works Retaining Wall OSIM Inspections
			Subtotal:	**$ 20,000**	**$ -**	

APPENDIX A

Drawings

GENERAL ARRANGEMENT PLAN

1:750

NOTES:

1. TOPOGRAPHICAL SURVEY CONDUCTED BY KGS GROUP ON JUNE 23, 2023.
2. THE DATA PROVIDED IS GEOREFERENCED TO NAD83 (CSRS) EPOCH 2010 AND CGVD 1928 VERTICALLY.

LEGEND:

FLOODWALL

EARTH DYKE

STORMWATER OUTFALL THROUGH DYKE/FLOODWALL

25 0 25 50m

SCALE: 1:750 METRIC 24"x36"

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
A	23/10/23	ISSUED FOR REVIEW	DB	SG

REVISIONS / ISSUE

CLIENT:

PROJECT:
CHURCHVILLE DYKE

DWG. DESCRIPTION:
GENERAL ARRANGEMENT PLAN

KGS GROUP

DESIGN BY:	DB	DATE (YY/MM/DD): 23/09/12
DESIGN CHECK:	SG	DATE:
DRAWN BY:	JC	DATE: 23/09/12
DWG. CHECK:	DB	DATE:

DWG. NO.: 23-4168-001 G01 REV.: 0

FileName: D:\Data\Users\JCarlos\Documents\FMS\23-4168-001\23-4168-001.dwg -- Tab: G01 Plotted By: jcarlos 24/02/23 [Fri 5:22pm]
24"x36" PLOT SCALE: 1"=1"

NOTES:

1. TOPOGRAPHICAL SURVEY CONDUCTED BY KGS GROUP ON JUNE 23, 2023.
2. THE DATA PROVIDED IS GEOREFERENCED TO NAD83 (CSRS) EPOCH 2010 AND CGVD 1928 VERTICALLY.

LEGEND:

- FLOODWALL
- EARTH DYKE
- STORMWATER OUTFALL THROUGH DYKE/FLOODWALL
- TH00-00 TEST HOLE

10 0 10 20 30m

SCALE: 1:400 METRIC 24"x36"

GENERAL ARRANGEMENT PLAN (1 OF 3)

1:400

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
A	23/10/23	ISSUED FOR REVIEW	DB	SG

REVISIONS / ISSUE

CLIENT:

PROJECT: CHURCHVILLE DYKE

DWG. DESCRIPTION: GENERAL ARRANGEMENT PLAN

KGS GROUP

DESIGN BY:	DB	DATE (YY/MM/DD): 23/09/12
DESIGN CHECK:	SG	DATE:
DRAWN BY:	JC	DATE: 23/09/12
DWG. CHECK:	DB	DATE:

DWG. NO.: 23-4168-001 G02 REV: 0

MATCHLINE
FOR CONTINUATION SEE DWG G03

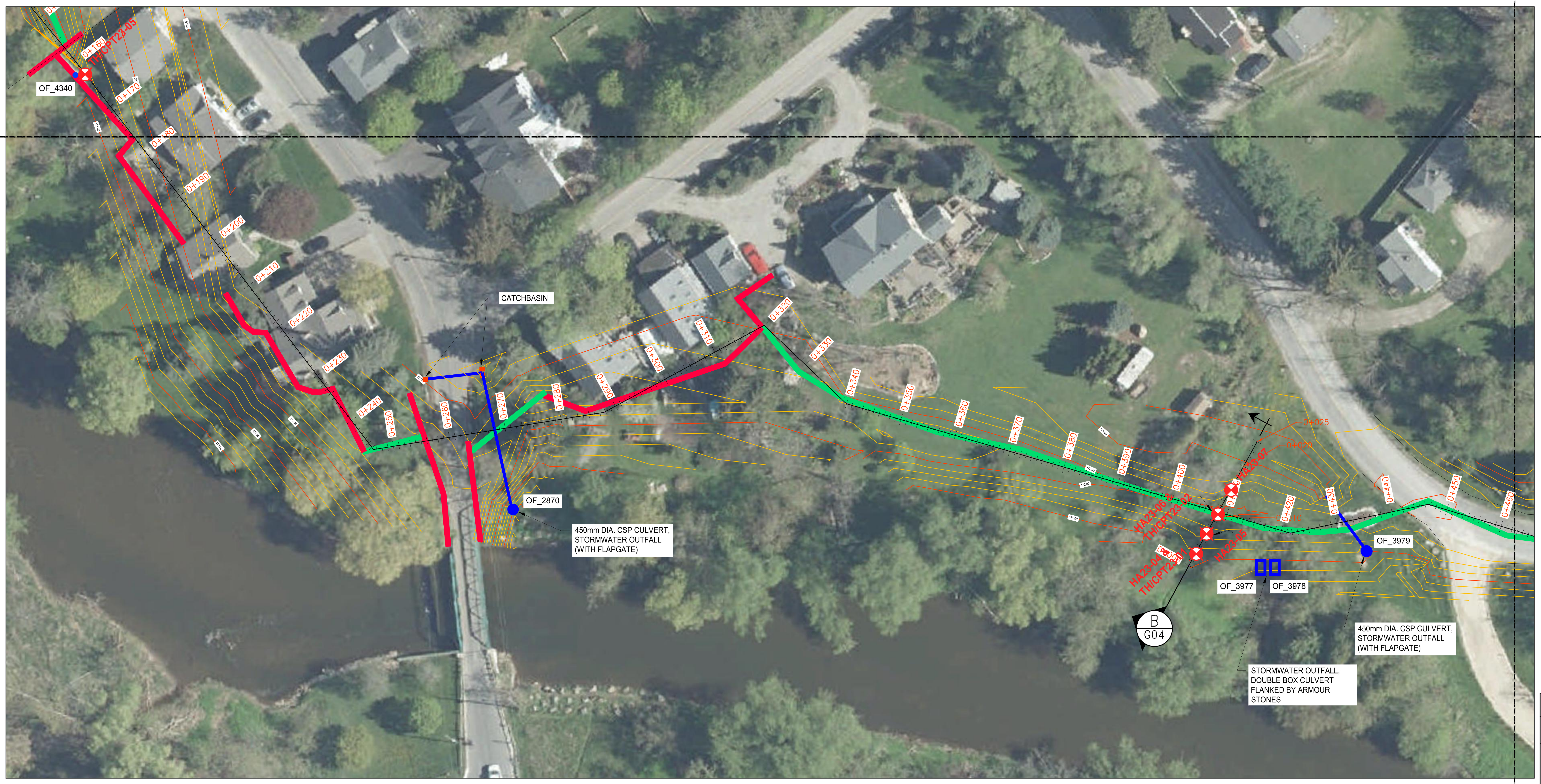
NOTES:

1. TOPOGRAPHICAL SURVEY CONDUCTED BY KGS GROUP ON JUNE 23, 2023.
2. THE DATA PROVIDED IS GEOREFERENCED TO NAD83 (CSRS) EPOCH 2010 AND CGVD 1928 VERTICALLY.

LEGEND:

- FLOODWALL
- EARTH DYKE
- STORMWATER OUTFALL THROUGH DYKE/FLOODWALL
- TH00-00 TEST HOLE

MATCHLINE
FOR CONTINUATION SEE DWG G01



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SCALE: 1:400 METRIC 24"x36"

GENERAL ARRANGEMENT PLAN (2 OF 3)

1:400

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
A	23/10/23	ISSUED FOR REVIEW	DB	SG

REVISIONS / ISSUE

CLIENT:

PROJECT:
CHURCHVILLE DYKE

DWG. DESCRIPTION:
GENERAL ARRANGEMENT PLAN

KGS GROUP

DESIGN BY:	DB	DATE (YY/MM/DD): 23/09/12
DESIGN CHECK:	SG	DATE:
DRAWN BY:	JC	DATE: 23/09/12
DWG. CHECK:	DB	DATE:

DWG. NO.: 23-4168-001 G03 REV: 0

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MATCHLINE
FOR CONTINUATION SEE DWG G02

NOTES:

1. TOPOGRAPHICAL SURVEY CONDUCTED BY KGS GROUP ON JUNE 23, 2023.
2. THE DATA PROVIDED IS GEOREFERENCED TO NAD83 (CSRS) EPOCH 2010 AND CGVD 1928 VERTICALLY.

LEGEND:

- FLOODWALL
- EARTH DYKE
- STORMWATER OUTFALL THROUGH DYKE/FLOODWALL
- TH00-00 TEST HOLE

10 0 10 20 30m
SCALE: 1:400 METRIC 24"x36"

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
A	23/10/23	ISSUED FOR REVIEW	DB	SG

REVISIONS / ISSUE

CLIENT:

PROJECT:
CHURCHVILLE DYKE

DWG. DESCRIPTION:
GENERAL ARRANGEMENT PLAN

KGS GROUP	DESIGN BY: DB	DATE (YY/MM/DD): 23/09/12
	DESIGN CHECK: SG	DATE:
	DRAWN BY: JC	DATE: 23/09/12
	DWG. CHECK: DB	DATE:

DWG. NO.: 23-4168-001 G04 REV.: 0

GENERAL ARRANGEMENT PLAN (3 OF 3)
1:400

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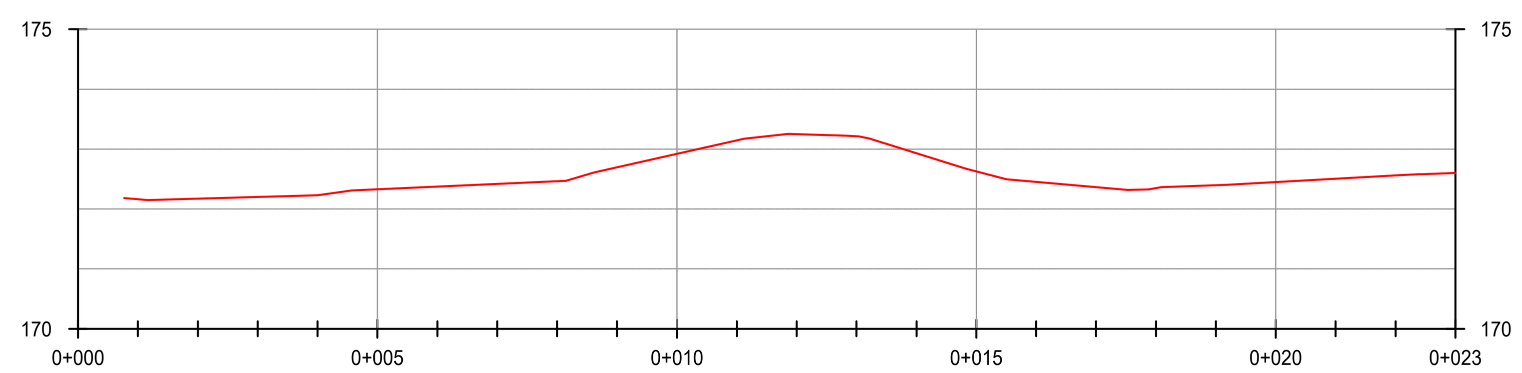
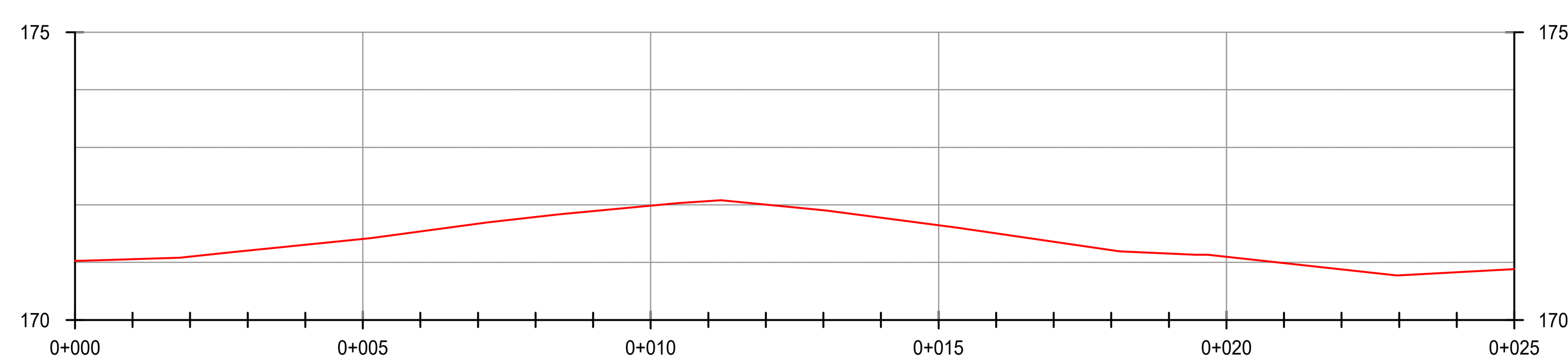


A SECTION



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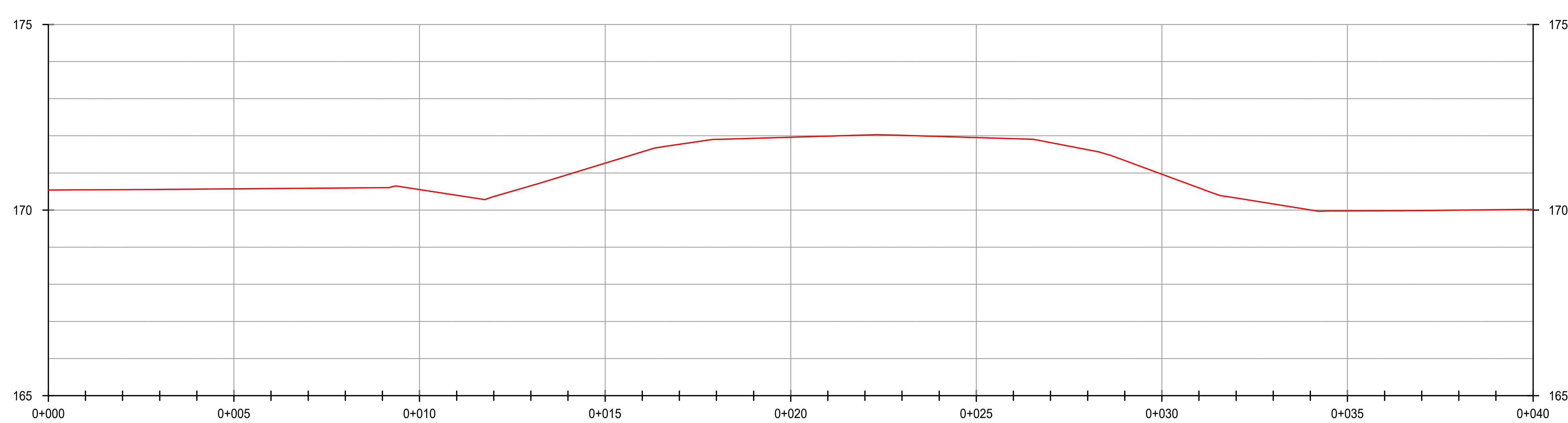
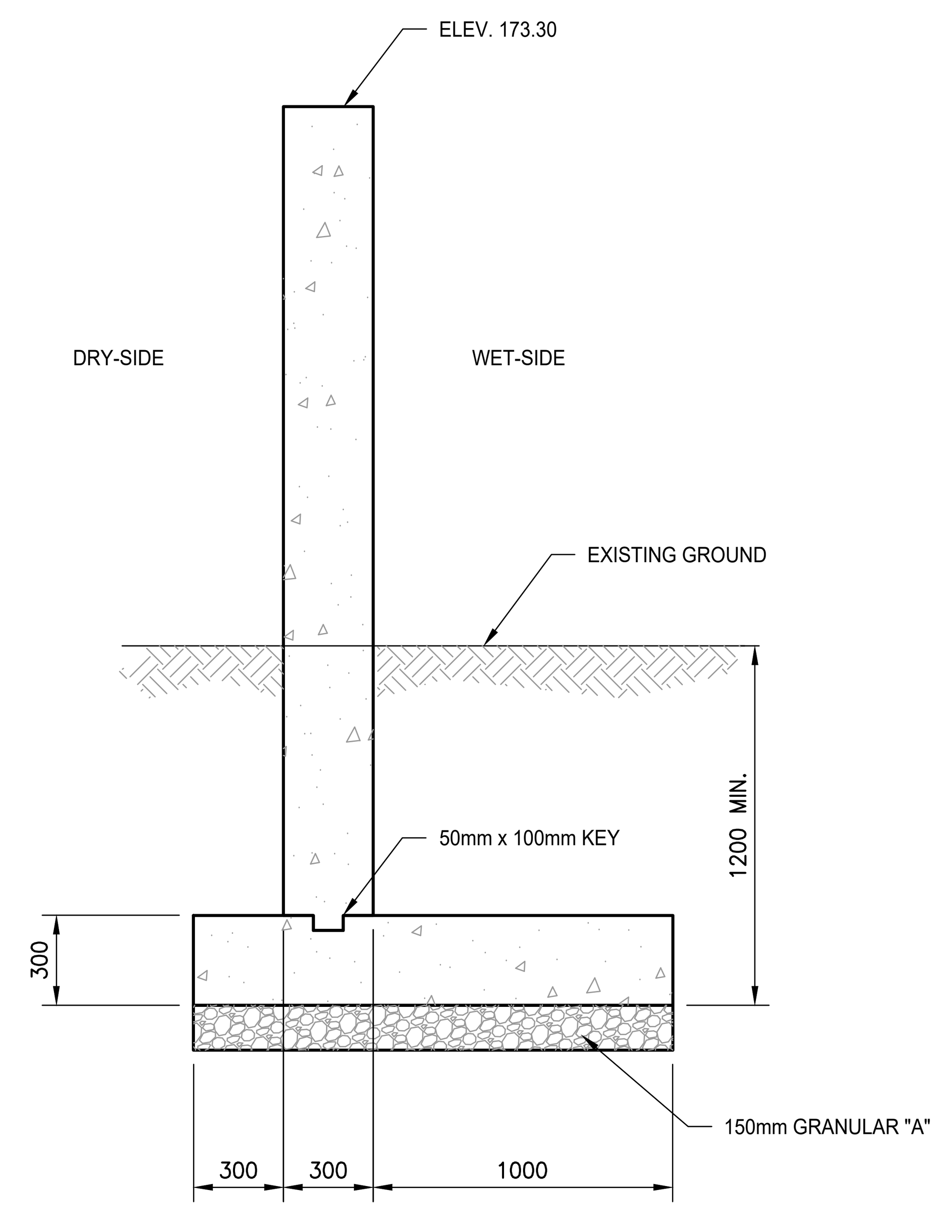
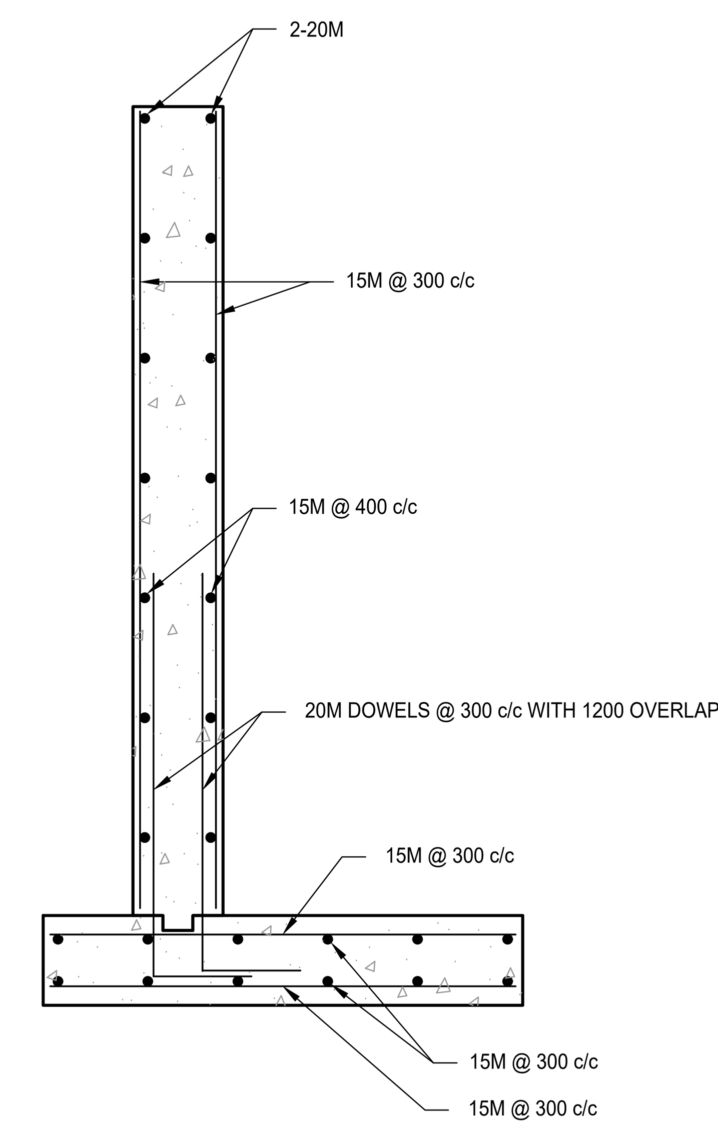


C SECTION



1 TYPICAL FLOODWALL SECTION
1:10



2 REINFORCEMENT
1:10

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
A	23/10/23	ISSUED FOR REVIEW	DB	SG

REVISIONS / ISSUE

CLIENT:

PROJECT: CHURCHVILLE DYKE

DWG. DESCRIPTION: CROSS SECTIONS

KGS GROUP

DESIGN BY: DB	DATE (YY/MM/DD): 23/11/17
DESIGN CHECK: SG	DATE:
DRAWN BY: MA	DATE: 23/09/12
DWG CHECK: DB	DATE:

DWG. NO. 23-4168-001 G05 REV: 0

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24"X36"/PLOT SCALE: 1"=1"

APPENDIX B

Test Hole Logs

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-01**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	171.14 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 3 (Earth Dyke), Ch. 0+420, Wet Side Toe	**UTM (m)**	N 4,831,845
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,240 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER
0.0	**TOPSOIL/ORGANICS** - Dark brown, moist.	170.8			
	SILTY CLAY TILL FILL - CL-CI (DYKE FILL) - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	169.6			
	POORLY GRADED SAND AND GRAVEL WITH COBBLES - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, trace to some fines. - Wet below 1.8 m.	168.1	▽		
3.0	Notes: 1. End of test hole at 3.0 m. 2. Test hole backfilled with bentonite chips.				

PL — MC — LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ 20 40 60 80

ELEVATION (m): 171, 170, 169, 168, 167, 166, 165, 164
DEPTH (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0; (ft): 5, 10, 15, 20, 25

WATER LEVELS ▽ **During Drilling/Digging** 1.83 m on 11-7-2023

CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-02**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.10 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 3 (Earth Dyke), Ch. 0+420, Crest C/L	**UTM (m)**	N 4,831,857
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,250 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

ELEVATION (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER	PL / MC / LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ (20 40 60 80)
172	0.0	**TOPSOIL/ORGANICS** - Dark brown, moist.	171.8				
171	0.3	**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel. - LL=24, PL=13, PI=11 at 2.1 m. - PSA: 13% gravel, 41% sand, 32% silt, 14% clay at 2.1 m.	169.7			CPT1	
169	2.4	**POORLY GRADED SAND AND GRAVEL WITH COBBLES** - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, trace to some fines. - Wet below 2.6 m. - PSA: 30% gravel, 34% sand, 36% fines at 3.4 m.	168.1	∇		CPT2	
168	4.0	**SILTY CLAY TILL - CL-CI** - Grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel. - LL=20, PL=11, PI=9 at 4.0 m. - PSA: 8% gravel, 39% sand, 40% silt, 13% clay at 4.0 m.	167.5			CPT3	

Notes:
1. End of test hole at 4.6 m.
2. Test hole backfilled with bentonite chips.

WATER LEVELS ∇ **During Drilling/Digging** 2.59 m on 11-7-2023

CONTRACTOR **ConeTec**

INSPECTOR **D. DUBEAU**

APPROVED DRAFT

DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-03**

SHEET 1 of 1

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	171.97 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 4 (Earth Dyke), Ch. 0+615, Wet Side Edge of the Crest	**UTM (m)**	N 4,831,910
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,386 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER	PL MC LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			WELL GRADED SAND AND GRAVEL FILL (GRAVEL SHOULDER) - Brown, moist, compact, fine to coarse sand, medium to coarse gravel, trace fines.	171.7				
171	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.					
	5		- Trace organics at 1.5 m.					
170	2.0							
169	3.0 / 10		- LL=28, PL=14, PI=14 at 2.7 m. - PSA: 15% gravel, 36% sand, 36% silt, 13% clay at 2.7 m. - Wet below 2.8 m.	169.0	▽		CPT1	
			Notes: 1. End of test hole at 3.0 m. 2. Test hole backfilled with bentonite chips.					
168	4.0							
	15							
167	5.0							
166	6.0 / 20							
165	7.0							
	25							
164								

WATER LEVELS ▽ **During Drilling/Digging** 2.80 m on 11-7-2023

CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-04**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	170.33 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 4 (Earth Dyke), Ch. 0+615, Wet Side Toe	**UTM (m)**	N 4,831,899
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,386 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			TOPSOIL/ORGANICS - Dark brown, moist.	170.0				
			SILTY CLAY TILL FILL - CL-CI (DYKE FILL) - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	169.6				
			POORLY GRADED SAND AND GRAVEL WITH COBBLES - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, trace to some fines. - PSA: 29% gravel, 48% sand, 23% fines at 0.8 m. - Wet below 1.0 m.	167.9	∇		CPT1	
			SILTY CLAY TILL - CL-CI - Grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel. - LL=29, PL=13, PI=16 at 2.4 m. - PSA: 6% gravel, 30% sand, 43% silt, 21% clay at 2.4 m.	167.3			CPT2	

Notes:
1. End of test hole at 3.0 m.
2. Test hole backfilled with bentonite chips.

WATER LEVELS ∇ **During Drilling/Digging** 0.98 m on 11-7-2023		CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
		APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-05**

SHEET 1 of 1

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.19 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 2 (Floodwall), Ch. 0+158, Dry Side Toe	**UTM (m)**	N 4,831,931
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,052 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

ELEVATION (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER	PL / MC / LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ (20 40 60 80)
172	0.0	**TOPSOIL/ORGANICS** - Dark brown, moist.	171.9				
		SAND AND GRAVEL FILL - Brown and grey, moist, compact, medium to coarse sand, medium to coarse gravel, trace fines. - Wet below 2.0 m.	169.7	▽			
		SILTY CLAY TILL - CL-CI - Grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel. - LL=36, PL=15, PI=21 at 2.4 m. - PSA: 7% gravel, 32% sand, 38% silt, 23% clay at 2.4 m.	169.2			CPT1	

Notes:
1. End of test hole at 3.0 m.
2. Test hole backfilled with bentonite chips.

Elevation scale (m): 172, 171, 170, 169, 168, 167, 166, 165
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0; (ft): 5, 10, 15, 20, 25

WATER LEVELS ▽ **During Drilling/Digging** 1.98 m on 11-7-2023

CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-06**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	173.24 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Crest C/L	**UTM (m)**	N 4,832,050
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 600,000 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
173				**TOPSOIL/ORGANICS** - Dark brown, moist.	172.9			
172	1.0	5		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	170.8			
171	2.0			**POORLY GRADED SAND AND GRAVEL WITH COBBLES** - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, trace to some fines.	170.2			
170	3.0	10		Notes: 1. End of test hole at 3.0 m. 2. Test hole backfilled with bentonite chips.				
169	4.0	15						
168	5.0							
167	6.0	20						
166	7.0	25						

WATER LEVELS

CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-07**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.16 m
LOCATION	Brampton, ON	**START DATE**	11-7-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Wet Side Toe	**UTM (m)**	N 4,832,054
DRILL RIG / HAMMER	MARL M 5T Track Mounted Drill Rig		E 599,992 Zone 17
METHOD(S)	0.0 m: 100 mm ø SSA		

DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	SAMPLE TYPE	NUMBER	PL / MC / LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ (20 40 60 80)
0.0	**TOPSOIL/ORGANICS** - Dark brown, moist.	171.9				
0.3	**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	170.9				
1.3	**POORLY GRADED SAND AND GRAVEL WITH COBBLES** - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), some cobbles up to 100mm, trace to some fines.	170.0	∇			
2.2	**SILTY CLAY TILL - CL-CI** - Grey, moist, stiff to very stiff, low to intermediate plasticity, with varying amounts of sand and gravel. - Wet below 2.2 m.	169.2				

Notes:
1. End of test hole at 3.0 m.
2. Test hole backfilled with bentonite chips.

WATER LEVELS ∇ **During Drilling/Digging** 2.19 m on 11-7-2023

CONTRACTOR **ConeTec**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-01

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.16 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Wet Side Toe	**UTM (m)**	N 4,832,054
DRILL RIG / HAMMER	Hand Auger		E 599,992 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
172			**TOPSOIL/ORGANICS** - Dark brown, moist.	171.9			
171	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	170.9			
			Notes: 1. End of test hole at 1.2 m. 2. Test hole backfilled with auger cuttings.				

Elevation scale (m): 172, 171, 170, 169, 168, 167, 166, 165
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0
Depth scale (ft): 5, 10, 15, 20, 25

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-02

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	173.24 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Crest C/L	**UTM (m)**	N 4,832,050
DRILL RIG / HAMMER	Hand Auger		E 600,000 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
173			**TOPSOIL/ORGANICS** - Dark brown, moist.	172.9			
172	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	171.7			
171	2.0 (5)		Notes: 1. End of test hole at 1.5 m. 2. Test hole backfilled with auger cuttings.				
170	3.0 (10)						
169	4.0 (15)						
168	5.0						
167	6.0 (20)						
166	7.0 (25)						

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

TEST HOLE LOG

KGS GROUP

HOLE NO. **HA23-03**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.76 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Dry Side Slope	**UTM (m)**	N 4,832,059.02
DRILL RIG / HAMMER	Hand Auger		E 600,001.46 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL / MC / LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ (20 40 60 80)
0.0–0.3	**TOPSOIL/ORGANICS** - Dark brown, moist.	172.5			
0.3–1.5	**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel. - LL=39, PL=20, PI=19 at 1.2 m. - PSA: 10% gravel, 25% sand, 44% silt, 21% clay at 1.2 m.	171.2		HA1	

Notes:
1. End of test hole at 1.5 m.
2. Test hole backfilled with auger cuttings.

Elevation scale (m): 172, 171, 170, 169, 168, 167, 166, 165
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0; (ft): 5, 10, 15, 20, 25

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-04

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	171.14 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 3 (Earth Dyke), Ch. 0+420, Wet Side Toe	**UTM (m)**	N 4,831,845
DRILL RIG / HAMMER	Hand Auger		E 600,240 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
171			**TOPSOIL/ORGANICS** - Dark brown, moist.	170.8			
170	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	169.6			
			- Hand auger refusal due to presence of coarse gravel at 1.5 m. Notes: 1. End of test hole at 1.5 m. 2. Test hole backfilled with auger cuttings.				

Elevation scale (m): 171, 170, 169, 168, 167, 166, 165, 164
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0
Depth scale (ft): 5, 10, 15, 20, 25

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

TEST HOLE LOG

KGS GROUP

HOLE NO. **HA23-05**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	171.76 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 3 (Earth Dyke), Ch. 0+420, Wet Side Slope	**UTM (m)**	N 4,831,843.92
DRILL RIG / HAMMER	Hand Auger		E 600,251.33 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL; Cu TORVANE (kPa) ◆; qu POCKET PEN (kPa) ★; SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			TOPSOIL/ORGANICS - Dark brown, moist.	171.5			
171	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	170.5			
			Notes: 1. End of test hole at 1.2 m. 2. Test hole backfilled with auger cuttings.				

Elevation scale: 171, 170, 169, 168, 167, 166, 165, 164
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0
Depth scale (ft): 5, 10, 15, 20, 25

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO. **HA23-06**

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	172.10 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 3 (Earth Dyke), Ch. 0+420, Crest C/L	**UTM (m)**	N 4,831,857
DRILL RIG / HAMMER	Hand Auger		E 600,250 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL / MC / LL; Cu TORVANE (kPa); qu POCKET PEN (kPa); SPT (N) BLOWS/0.30 m (20 40 60 80)
172	0.0	**TOPSOIL/ORGANICS** - Dark brown, moist.	171.8			
171	1.0	**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel. - LL=40, PL=19, PI=21 at 1.2 m. - PSA: 6% gravel, 21% sand, 43% silt, 30% clay at 1.2 m.	170.6	Auger	HA1	PL–LL plotted

Notes:
1. End of test hole at 1.5 m.
2. Test hole backfilled with auger cuttings.

Depth scale: 1.0–7.0 m (5–25 ft); Elevation scale: 172–165 m.

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-07

SHEET 1 of 1

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	171.51 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 1 (Earth Dyke), Ch. 0+035, Dry Side Slope	**UTM (m)**	N 4,831,855.56
DRILL RIG / HAMMER	Hand Auger		E 600,257.13 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			TOPSOIL/ORGANICS - Dark brown, moist.	171.2			
171	1.0		**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	170.0			
170	5		Notes: 1. End of test hole at 1.5 m. 2. Test hole backfilled with auger cuttings.				
169	2.0						
168	3.0 / 10						
167	4.0 / 15						
166	5.0						
165	6.0 / 20						
164	7.0 / 25						

WATER LEVELS

CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-08

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	170.33 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 4 (Earth Dyke), Ch. 0+615, Wet Side Toe	**UTM (m)**	N 4,831,899
DRILL RIG / HAMMER	Hand Auger		E 600,386 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			TOPSOIL/ORGANICS - Dark brown, moist.	170.0			
170			**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	169.4			
	1.0		**POORLY GRADED SAND AND GRAVEL WITH COBBLES** - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), trace to some fines. - PSA: 31% gravel, 48% sand, 21% fines at 1.1 m.	168.8		HA1	●
169	5		- Hand auger refusal due to presence of cobbles at 1.5 m. Notes: 1. End of test hole at 1.2 m. 2. Test hole backfilled with auger cuttings.				
	2.0						
168							
	3.0 / 10						
167							
	4.0						
166	15						
	5.0						
165							
	6.0 / 20						
164							
	7.0						
163	25						

WATER LEVELS	CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
	APPROVED DRAFT	DATE

KGS GROUP

TEST HOLE LOG

HOLE NO.
HA23-09

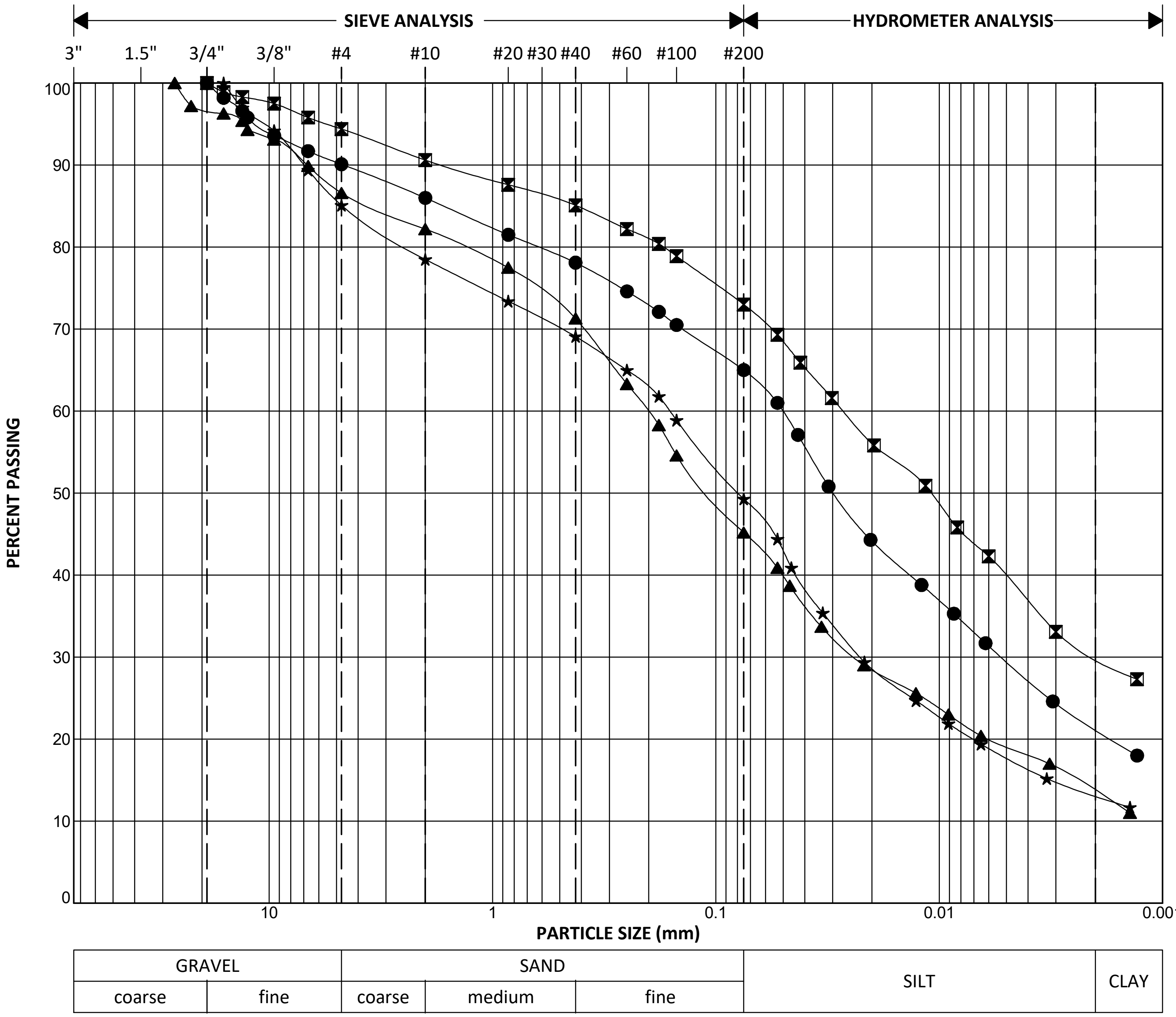
CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT	**Churchville Earth Dyke and Floodwall Condition Assessment**	**SURFACE ELEV.**	170.33 m
LOCATION	Brampton, ON	**START DATE**	7-20-2023
DESCRIPTION	Segment 4 (Earth Dyke), Ch. 0+615, Dry Side Toe	**UTM (m)**	N 4,831,913.5
DRILL RIG / HAMMER	Hand Auger		E 600,372.77 Zone 17
METHOD(S)	0.0 m: 75 mm ø Hand Auger		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
			TOPSOIL/ORGANICS - Dark brown, moist.	170.0			
170			**SILTY CLAY TILL FILL - CL-CI (DYKE FILL)** - Reddish brown, moist, firm to stiff, low to intermediate plasticity, some sand and gravel.	169.4			
169	1.0		**POORLY GRADED SAND AND GRAVEL WITH COBBLES** - Brown, moist, compact, fine to medium sand, fine to coarse gravel (rounded to sub-angular), trace to some fines.	168.8			
	5		- Hand auger refusal due to presence of cobbles at 1.5 m. Notes: 1. End of test hole at 1.2 m. 2. Test hole backfilled with auger cuttings.				

Elevation scale: 170, 169, 168, 167, 166, 165, 164, 163. Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0; (ft): 5, 10, 15, 20, 25.

WATER LEVELS	CONTRACTOR **KGS Group**	INSPECTOR **D. DUBEAU**
	APPROVED DRAFT	DATE

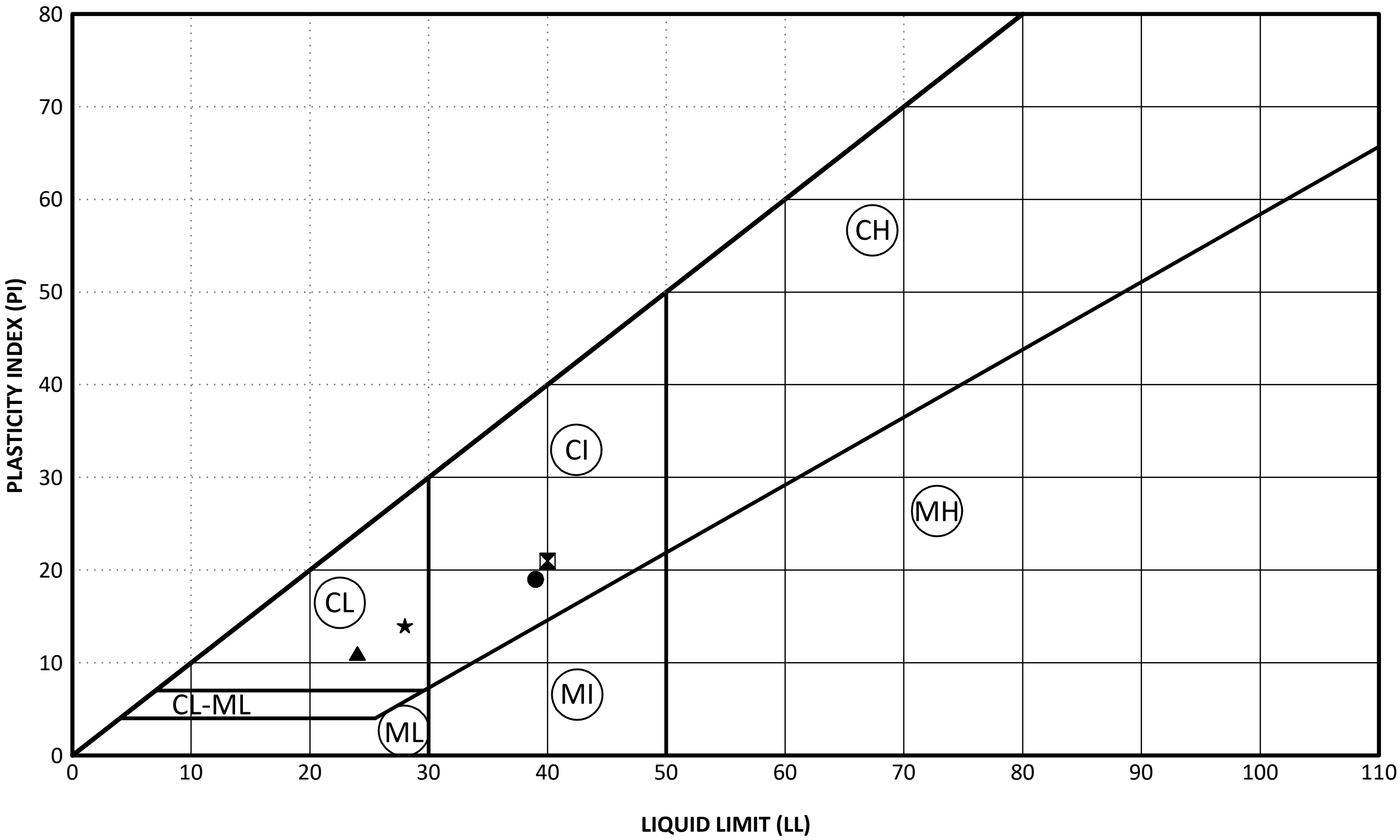
GRAIN SIZE DISTRIBUTION



	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	HA23-03	1.2	HA1	10	25	44	21	65			CI
⊠	HA23-06	1.2	HA1	6	21	43	30	73			CI
▲	TH23-02	2.1	CPT1	13	42	32	14	45			SC
★	TH23-03	2.7	CPT1	15	36	36	13	49			SC

Dyke Fill - Grain Size Distribution

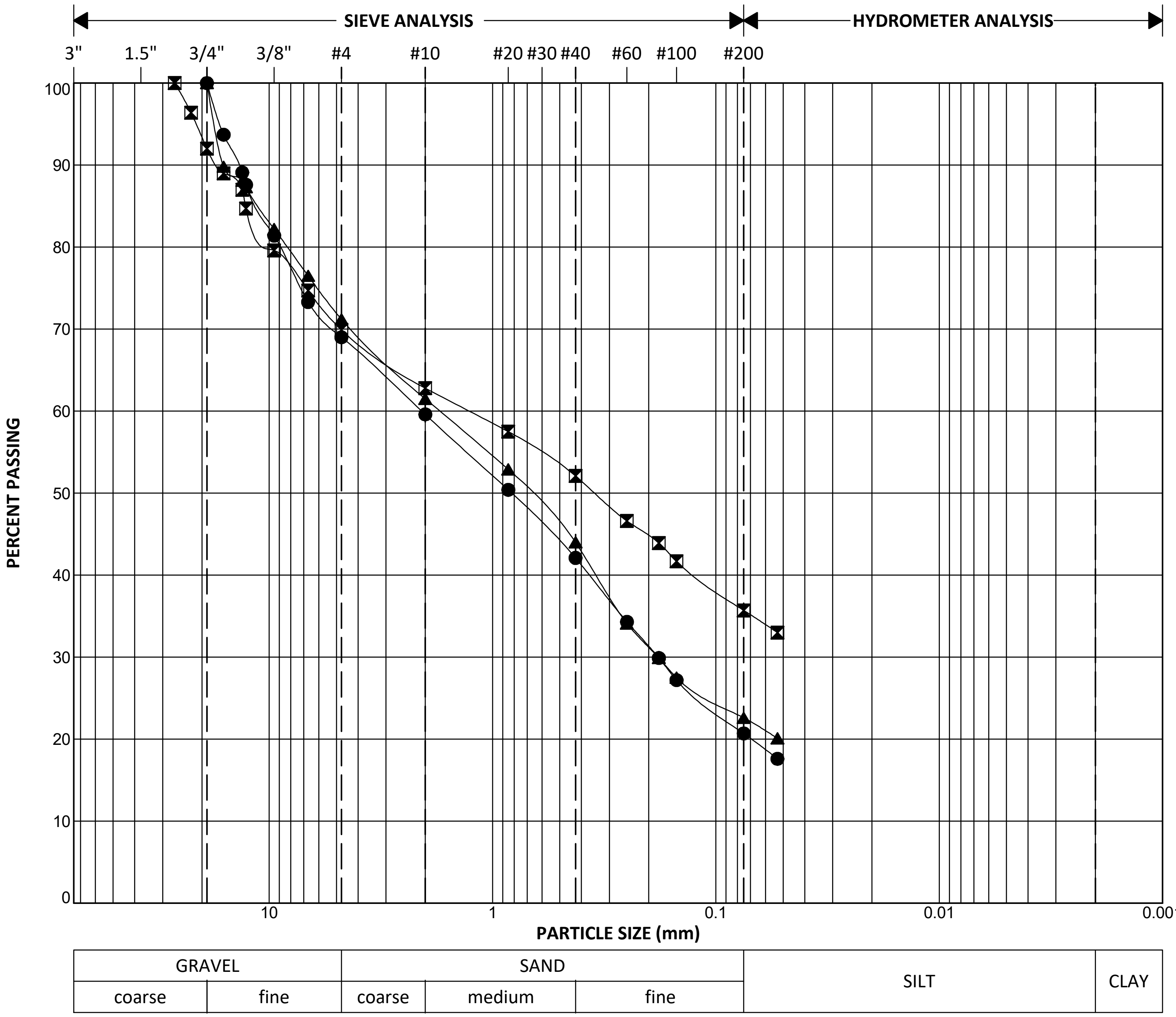
ATTERBERG LIMITS



	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	MC (%)	CLASSIFICATION
●	HA23-03	1.2	HA1	39	20	19	25	44	21	65	21	CI
⊠	HA23-06	1.2	HA1	40	19	21	21	43	30	73	17	CI
▲	TH23-02	2.1	CPT1	24	13	11	42	32	14	45	14	SC
★	TH23-03	2.7	CPT1	28	14	14	36	36	13	49	23	SC

Dyke Fill - A-Line Plot

GRAIN SIZE DISTRIBUTION



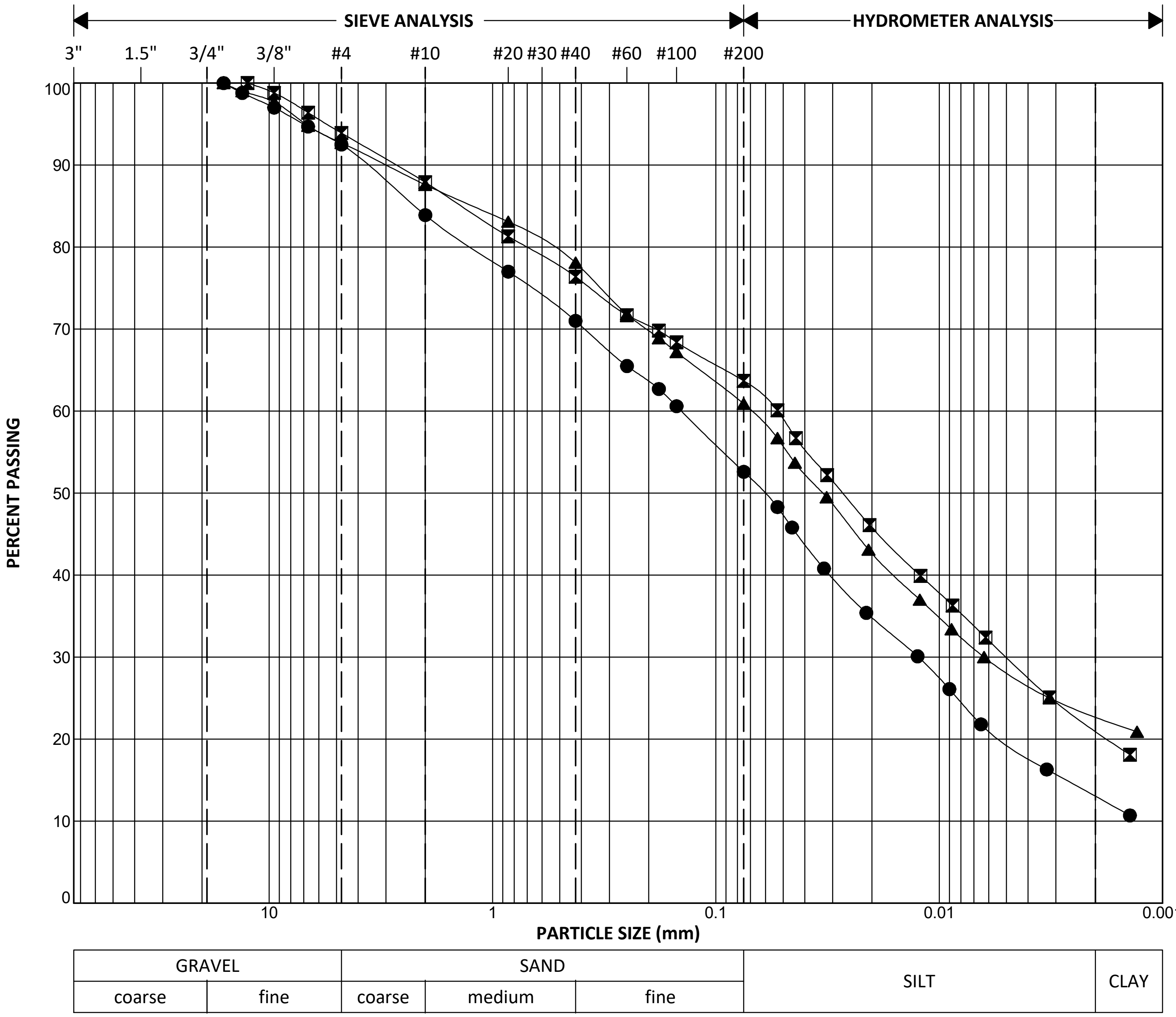
	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	HA23-08	1.1	HA1	31	48			21			
⊠	TH23-02	3.4	CPT2	30	34			36			
▲	TH23-04	0.8	CPT1	29	48			23			

Sand and Gravel - Grain Size Distribution

KGS GROUP	CLIENT	CITY OF BRAMPTON	PROJECT NO.	23-4168-001
	PROJECT NAME	Churchville Earth Dyke and Floodwall Condition Assessment	LOCATION	Brampton, ON

SIEVE ANALYSIS C:\USERS\DDUBEAU\DESKTOP\CHURCHVILLE\LOGS\23-4168-001 CHURCHVILLE_JAN 15, 2024.GPJ

GRAIN SIZE DISTRIBUTION



	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	TH23-02	4.0	CPT3	8	39	40	13	53			CL
⧗	TH23-04	2.4	CPT2	6	30	43	21	64			CL
▲	TH23-05	2.4	CPT1	7	32	38	23	61			CI

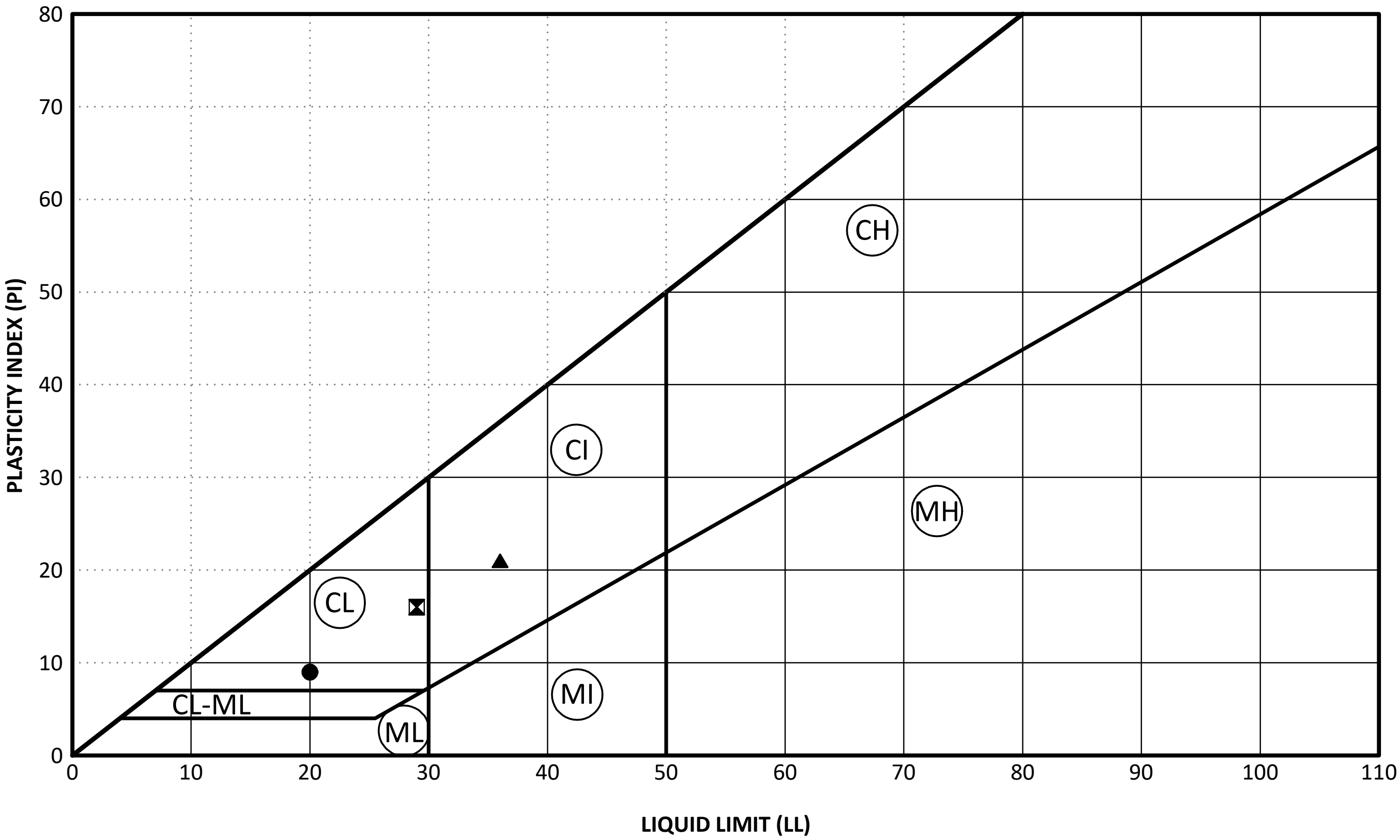
Silty Clay Till - Grain Size Distribution

SIEVE ANALYSIS C:\USERS\DDUBEAU\DESKTOP\CHURCHVILLE\LOGS\23-4168-001 CHURCHVILLE_JAN 15, 2024.GPJ

KGS GROUP

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT NAME	Churchville Earth Dyke and Floodwall Condition Assessment	**LOCATION**	Brampton, ON

ATTERBERG LIMITS



	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	MC (%)	CLASSIFICATION
●	TH23-02	4.0	CPT3	20	11	9	39	40	13	53	13	CL
⧗	TH23-04	2.4	CPT2	29	13	16	30	43	21	64	18	CL
▲	TH23-05	2.4	CPT1	36	15	21	32	38	23	61	16	CI

Silty Clay Till - A-Line Plot

A-LINE PLOT C:\USERS\DDUBEAU\DESKTOP\CHURCHVILLE\LOGS\23-4168-001 CHURCHVILLE_JAN 15, 2024.GPJ

KGS GROUP

CLIENT	**CITY OF BRAMPTON**	**PROJECT NO.**	23-4168-001
PROJECT NAME	Churchville Earth Dyke and Floodwall Condition Assessment	**LOCATION**	Brampton, ON

APPENDIX C

Cone Penetration Test (CPTu) Report

PRESENTATION OF SITE INVESTIGATION RESULTS

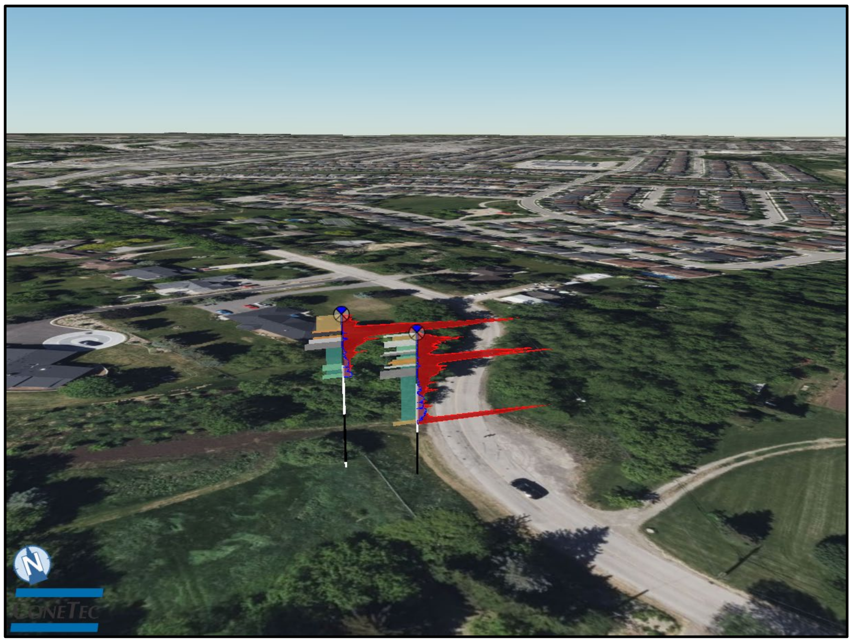
Churchville CPT

Prepared for:

KGS Group Consulting Engineers

ConeTec Job No: 23-05-26771

Project Start Date: 2023-11-07
Project End Date: 2023-11-07
Report Date: 2023-11-15



Prepared by:

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www.conetecdataservices.com

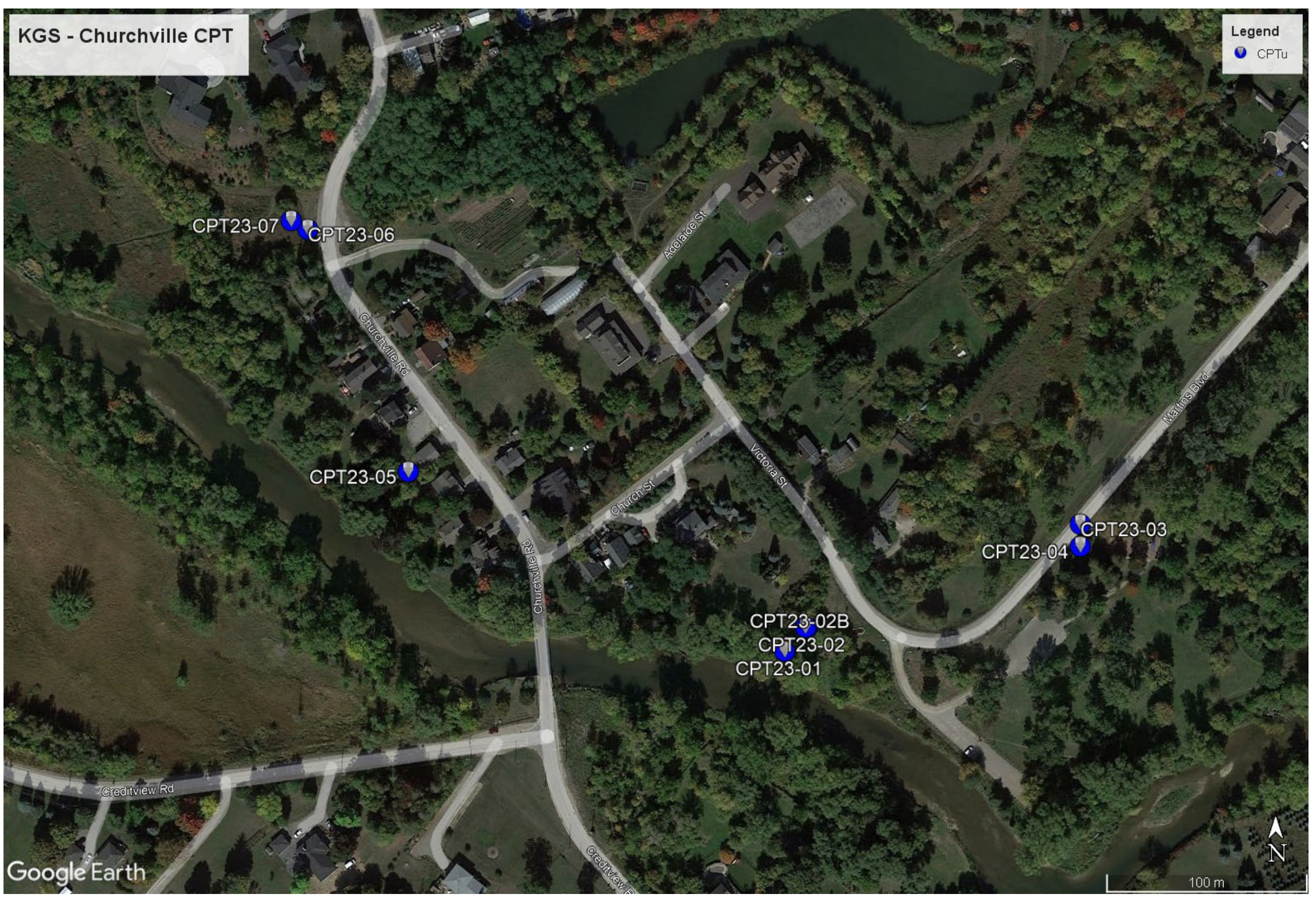
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for KGS Group Consulting Engineers in Churchville Park, Brampton, ON. The program consisted of 8 cone penetration tests (CPTu). Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information

Project	
Client	KGS Group Consulting Engineers
Project	Churchville CPT
ConeTec project number	23-05-26771

An aerial overview from Google Earth including the CPTu test locations is presented below.



Rig Description	Deployment System	Test Type
CPT track rig (M5T)	14 ton rig cylinder	CPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu	Consumer grade GPS	32617

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
766:T1000F10U35	766	10	150	1000	10	35
Cone 766 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	• Standard plots with Expanded Range • Advanced plots with Ic, Su, phi and N1(60)Ic • Soil Behaviour Type (SBT) scatter plots

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4). Equilibrium pore pressure profiles generated from the pore pressure dissipation data and assumed equilibrium points were used for the calculated parameters. Based on the dynamic pore pressure response, hydrostatic conditions were assumed after the last equilibrium pore pressure point. The equilibrium pore pressure profile points and profile line, as well as the hydrostatic line are plotted on the dynamic pore pressure for comparison.

Limitations

3rd Party Disclaimer

This report titled "Churchville CPT", referred to as the ("Report"), was prepared by ConeTec for KGS Group Consulting Engineers. The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by KGS Group Consulting Engineers to collect and provide the raw data ("Data") which is included in this report titled "Churchville CPT", which is referred to as the ("Report"). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively "Interpretations") included in the Report, including those based on the Data, are outside the scope of ConeTec's retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

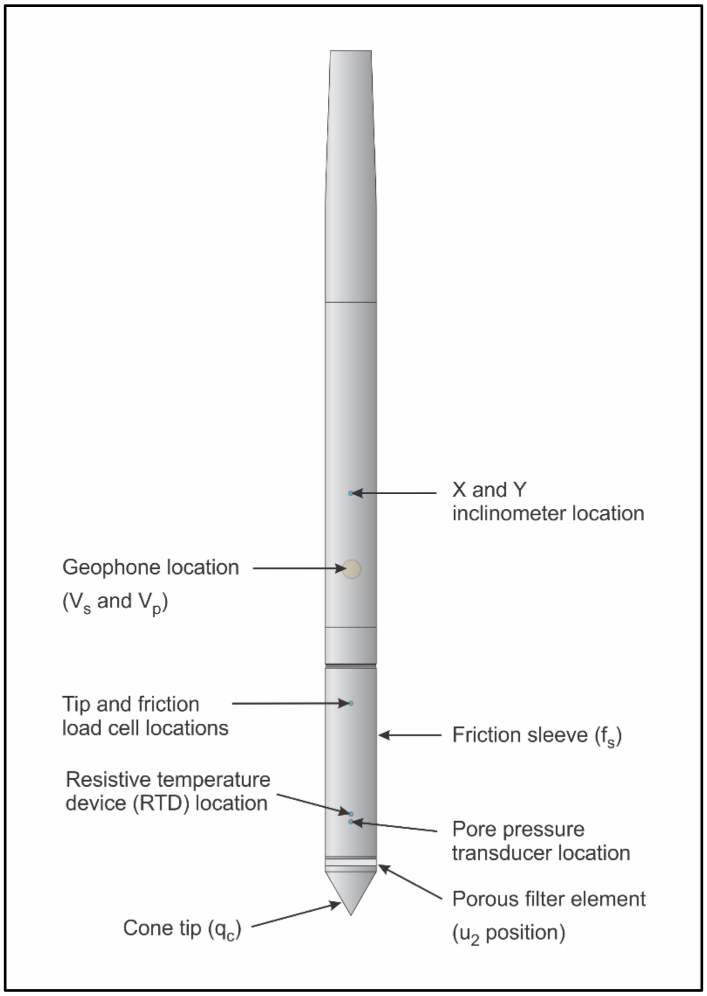


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1-a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

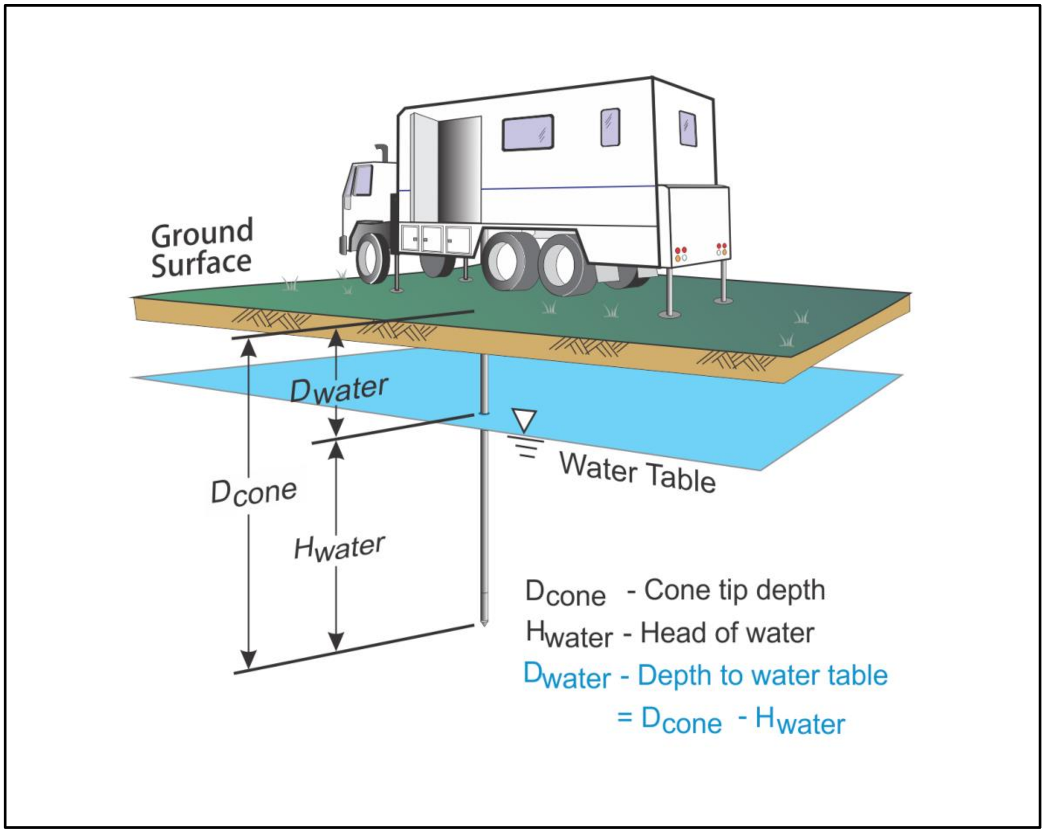


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

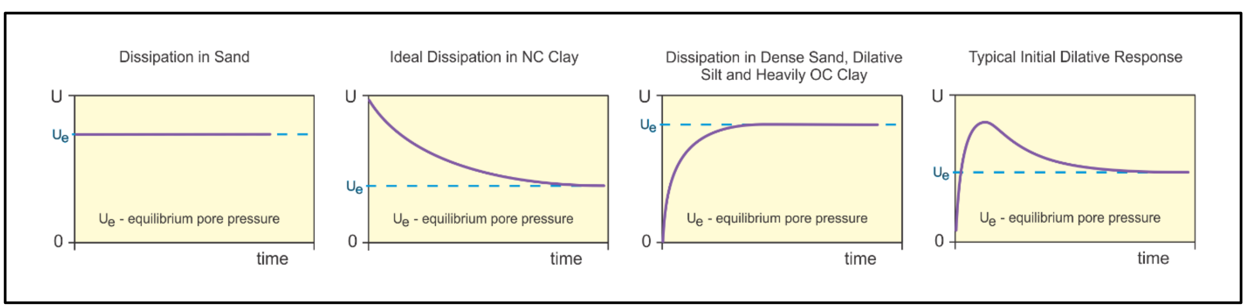


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)

a is the radius of the cone

I_r is the rigidity index

t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

References

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Range
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No:	23-05-26771
Client:	KGS Group Consulting Engineers
Project:	Churchville CPT
Start Date:	2023-11-07
End Date:	2023-11-07

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
CPT23-01	23-05-26771_CP01	2023-11-07	766:T1000F10U35	10	1.8	2.525	4831845	600240	
CPT23-02	23-05-26771_CP02	2023-11-07	766:T1000F10U35	10		2.625	4831857	600251	4
CPT23-02B	23-05-26771_CP02B	2023-11-07	766:T1000F10U35	10	2.6	3.050	4831857	600250	
CPT23-03	23-05-26771_CP03	2023-11-07	766:T1000F10U35	10	2.8	9.500	4831910	600386	
CPT23-04	23-05-26771_CP04	2023-11-07	766:T1000F10U35	10	1.0	2.350	4831899	600386	
CPT23-05	23-05-26771_CP05	2023-11-07	766:T1000F10U35	10	2.0	5.150	4831931	600052	3, 5
CPT23-06	23-05-26771_CP06	2023-11-07	766:T1000F10U35	10	3.0	9.100	4832050	600000	3
CPT23-07	23-05-26771_CP07	2023-11-07	766:T1000F10U35	10	2.3	6.175	4832054	599992	3

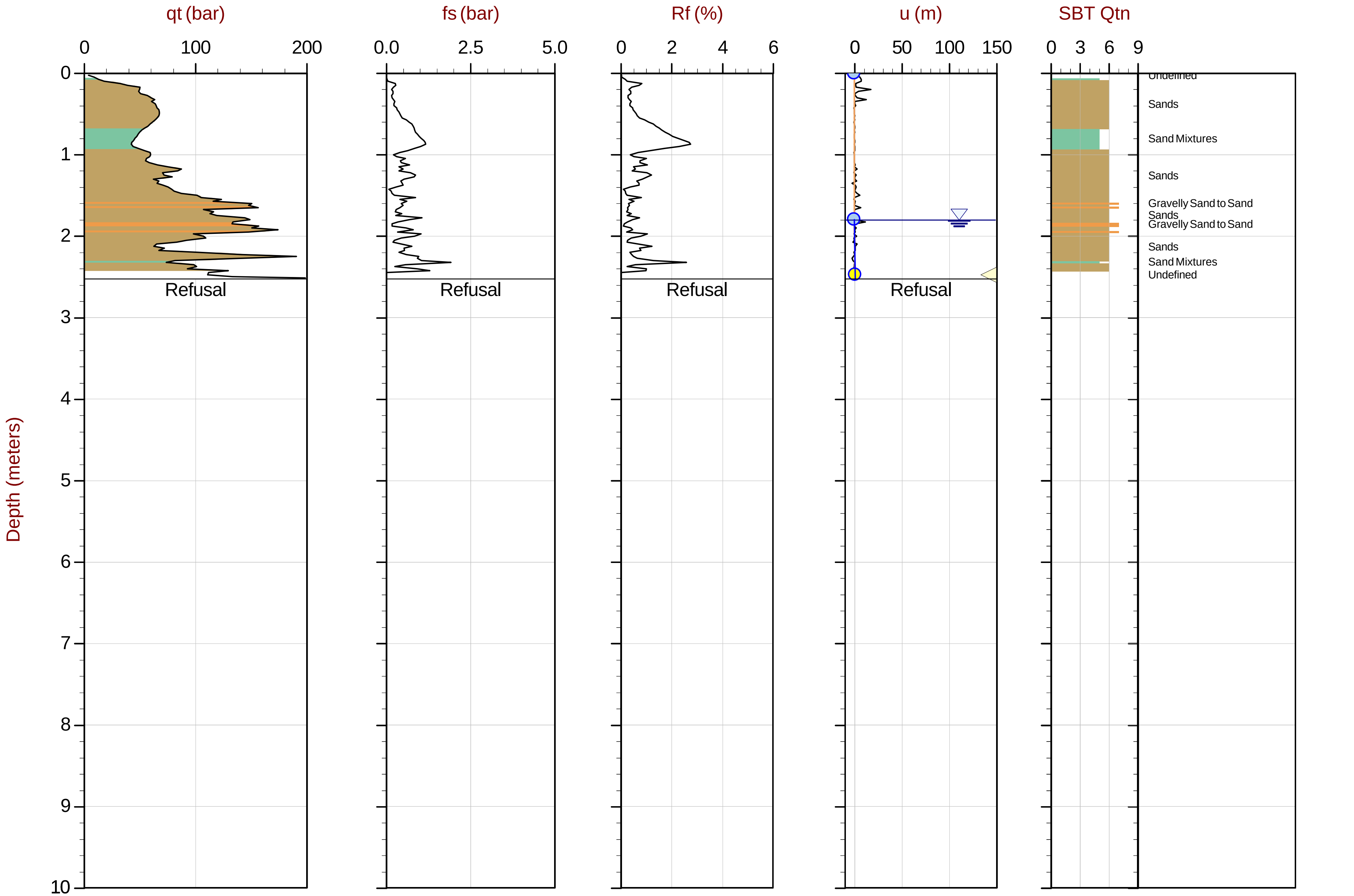
1. The assumed phreatic surface was based on a pore pressure dissipation test, unless otherwise noted. Equilibrium pore pressure profiles were used for the calculated parameters.
2. Coordinates were acquired with a consumer grade GPS device. Datum: WGS 1984 / UTM Zone 17 North.
3. The assumed phreatic surface was based on the dynamic pore pressure response.
4. No phreatic surface was detected.
5. Initial refusal occurred at 0.325 m due to an obstruction. The obstruction was drilled out and the push was continued to 5.150 m.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:13
Site: Churchville Park

Sounding: CPT23-01
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.525 m / 8.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP01.COR
Unit Wt: SBTQtn (PKR2009)

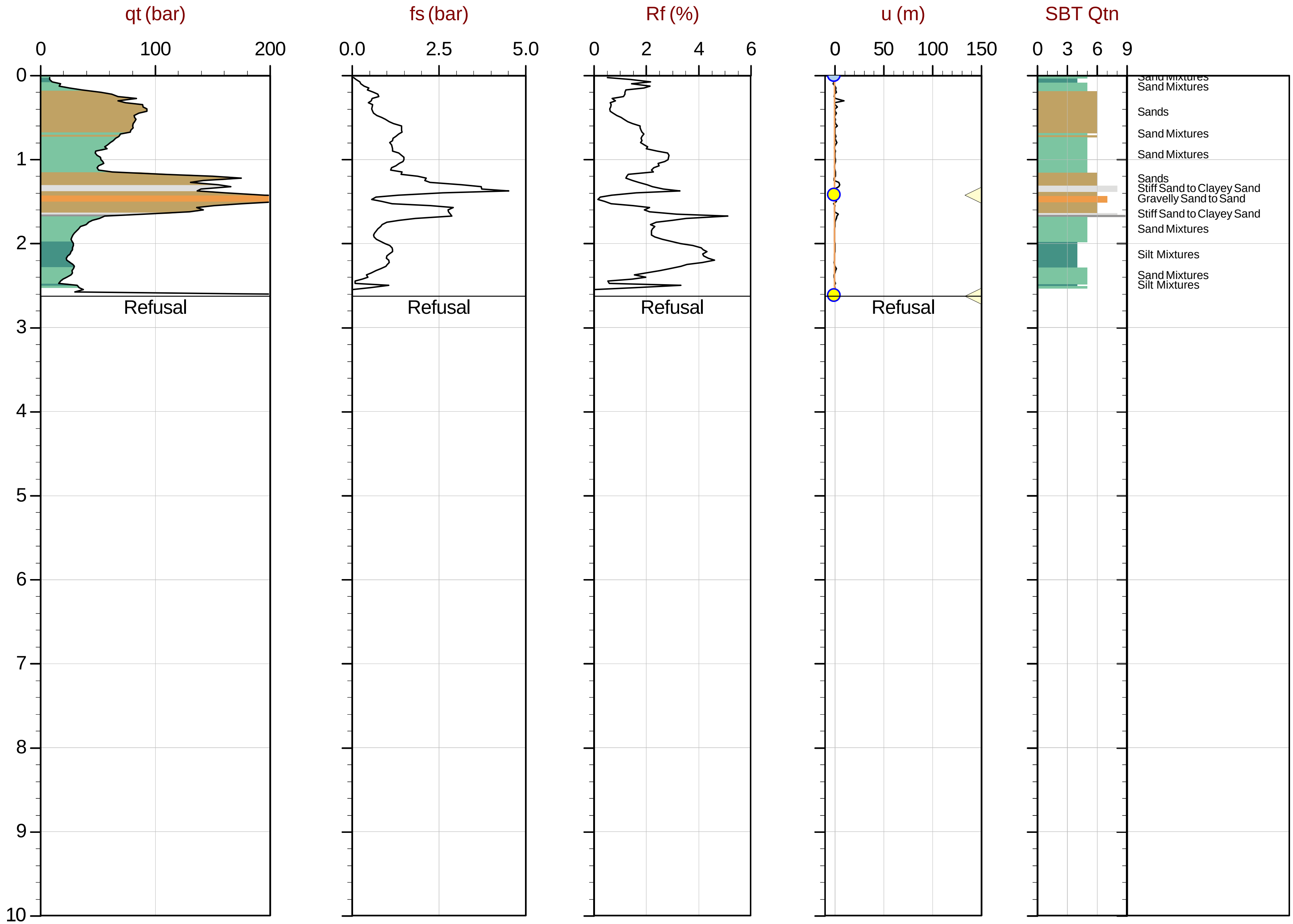
SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831845m E: 600240m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park
Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.625 m / 8.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600251m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

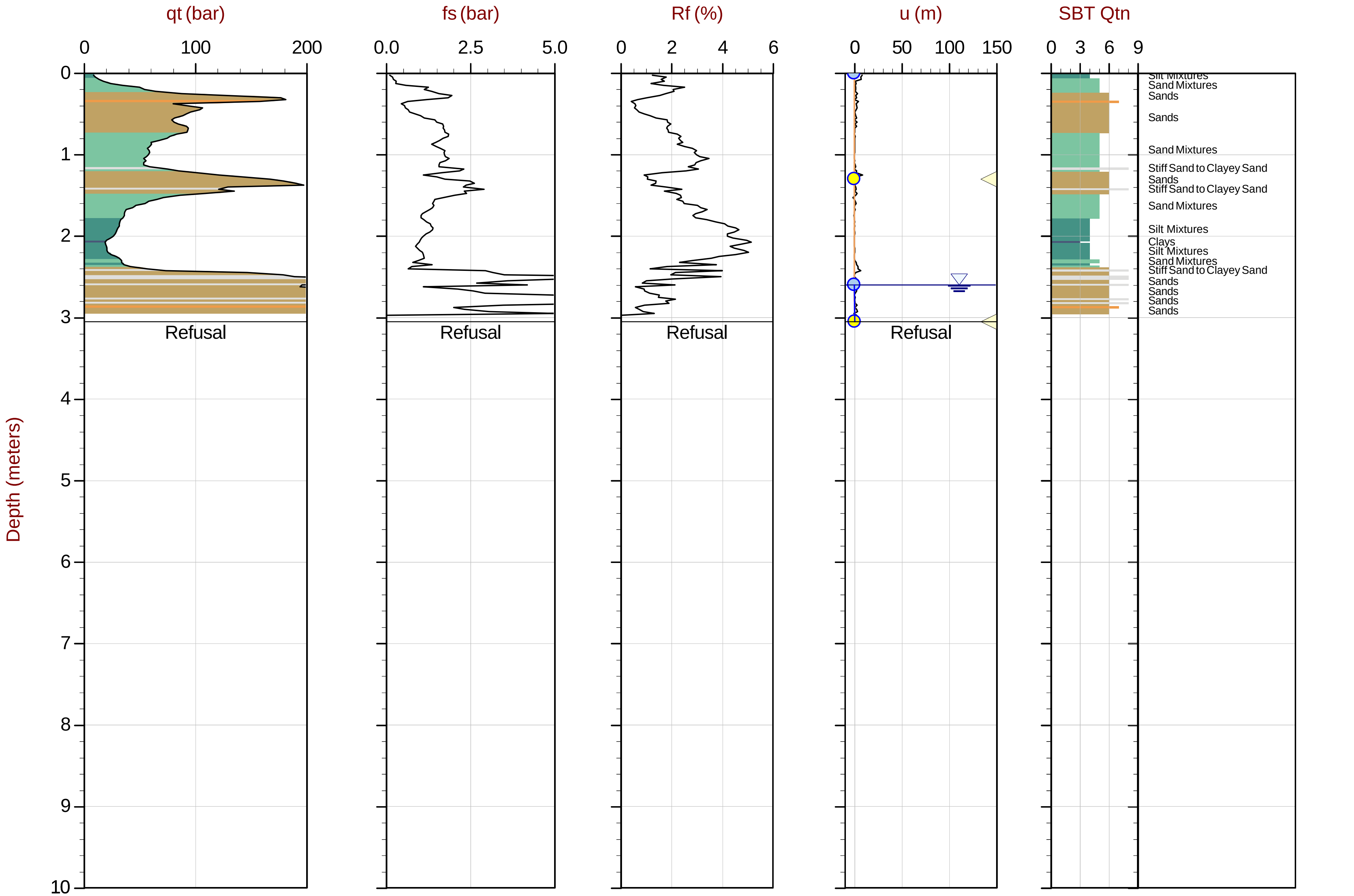
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 10:15
Site: Churchville Park

Sounding: CPT23-02B
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 3.050 m / 10.01 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600250m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

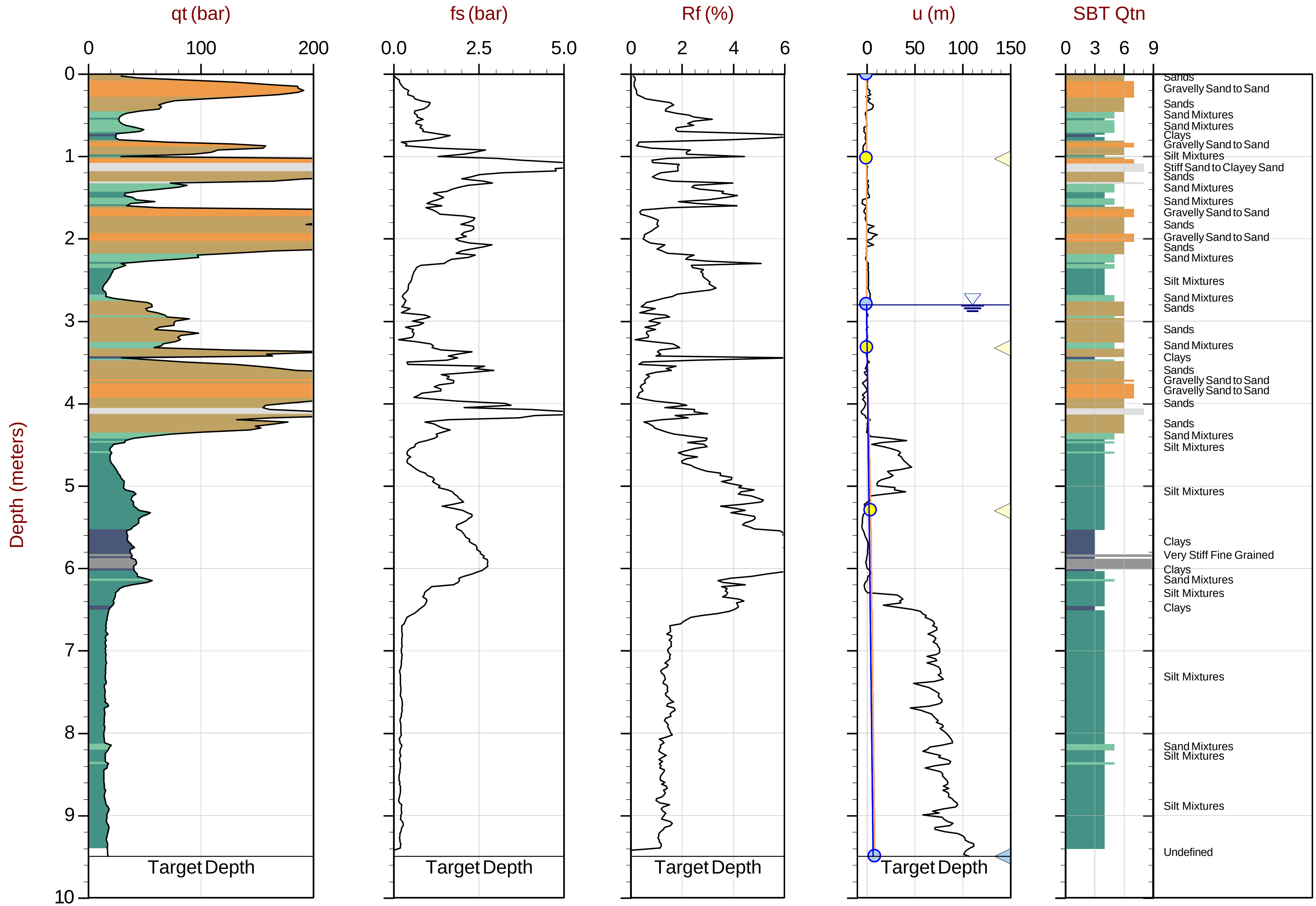
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 9.500 m / 31.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831910m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

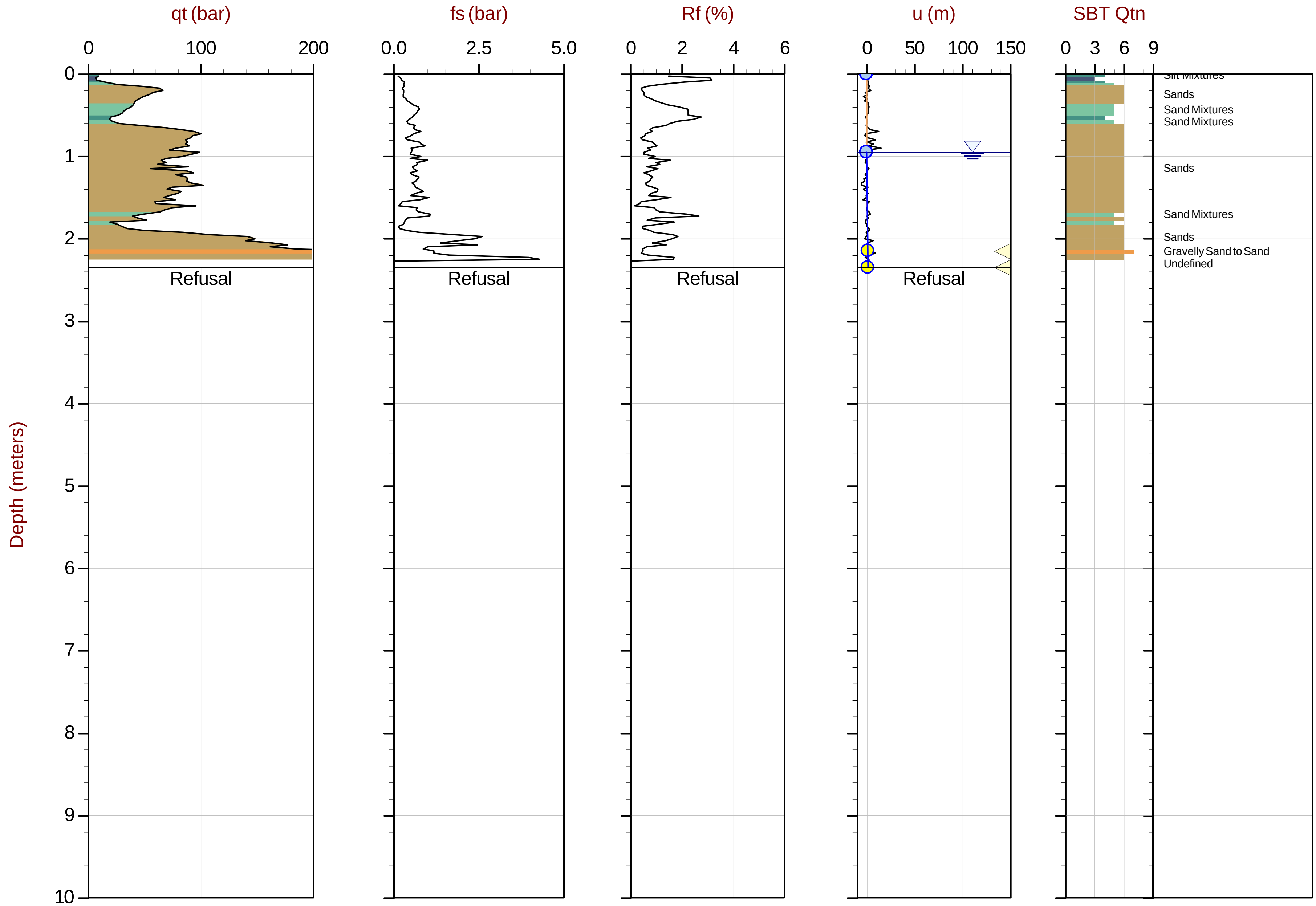
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 13:15
Site: Churchville Park

Sounding: CPT23-04
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.350 m / 7.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831899m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

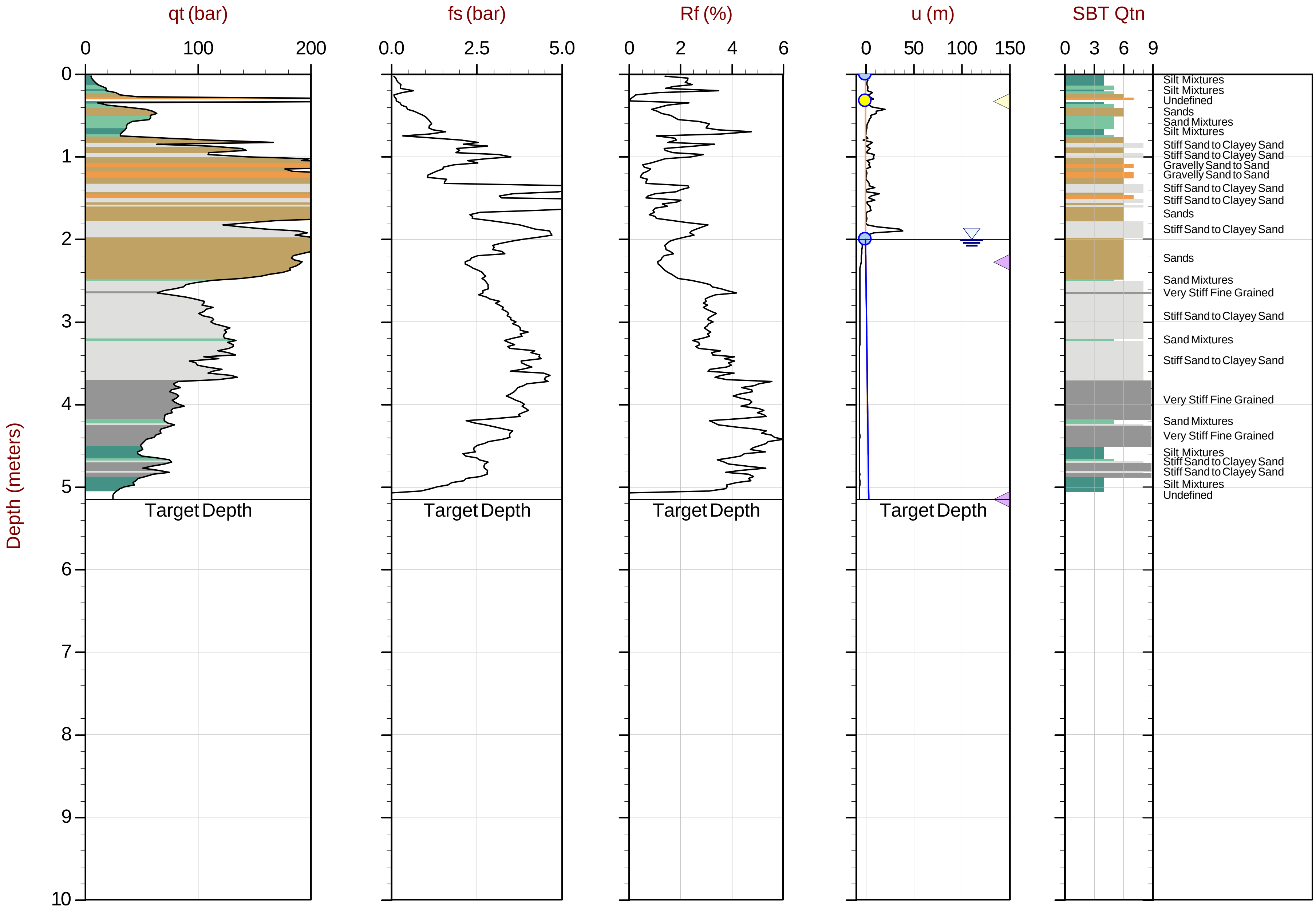
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 5.150 m / 16.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831931m E: 600052m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

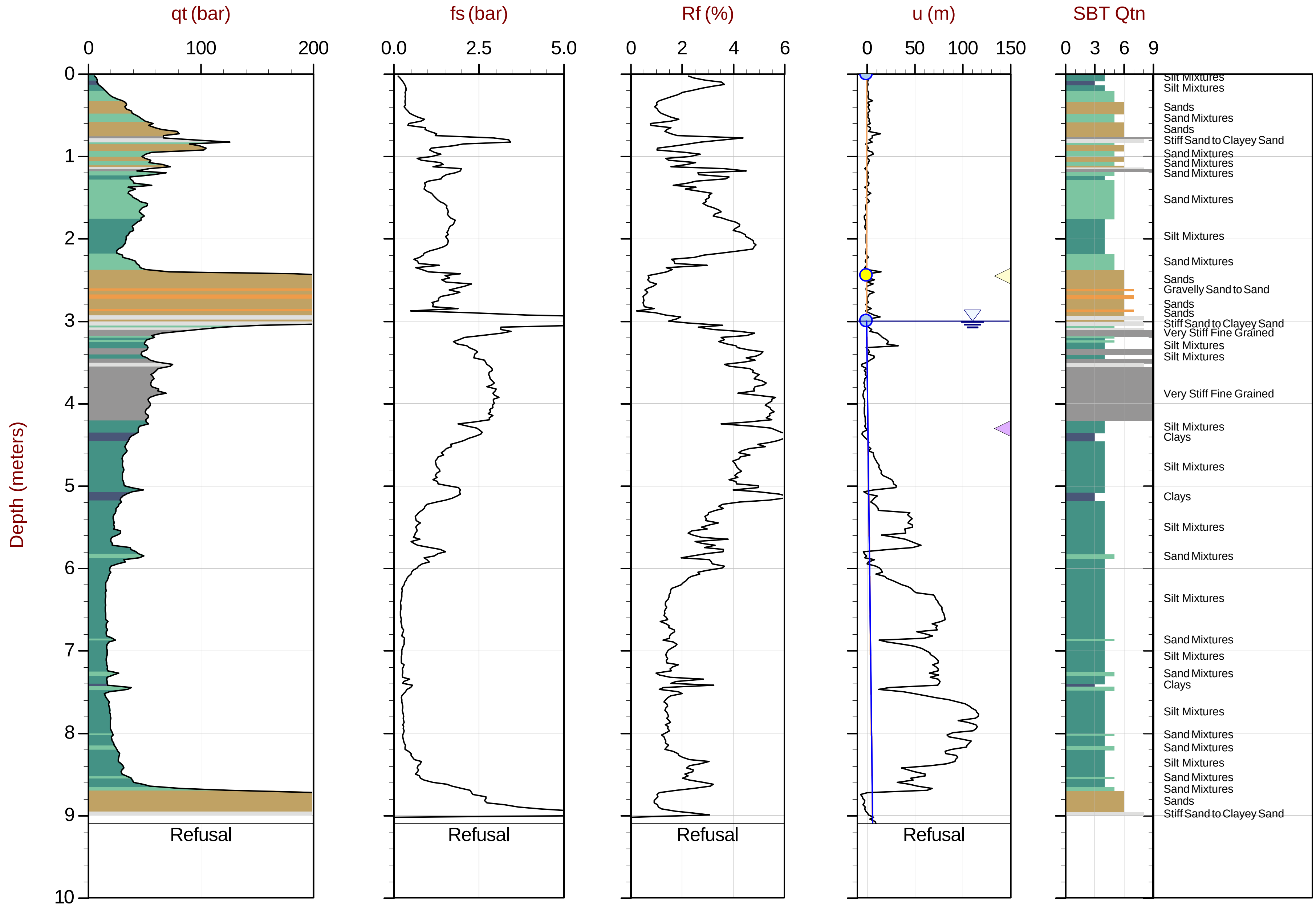
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 15:19
Site: Churchville Park

Sounding: CPT23-06
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 9.100 m / 29.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832050m E: 600000m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

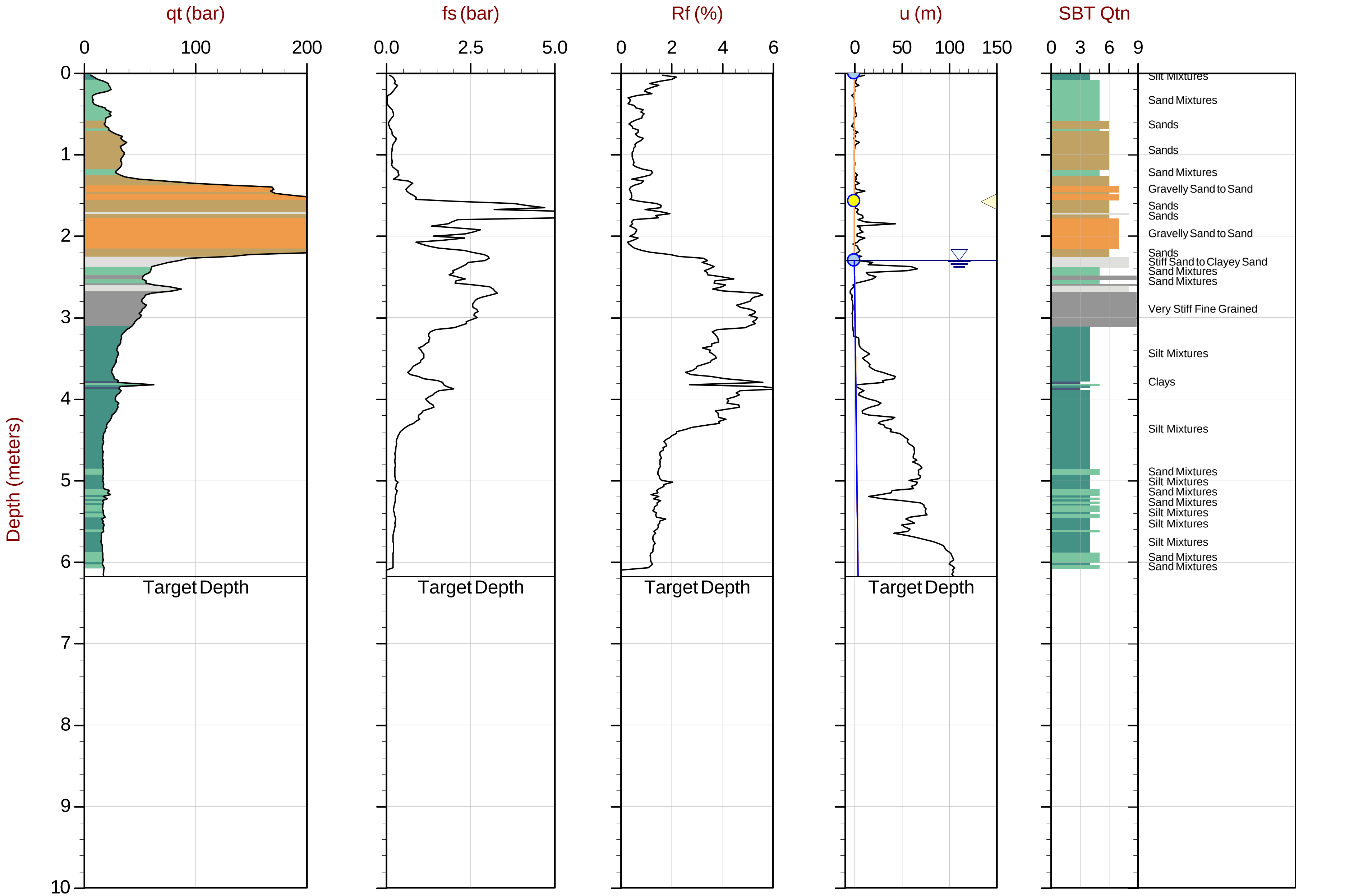
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 16:33
Site: Churchville Park

Sounding: CPT23-07
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 6.175 m / 20.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832054m E: 599992m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

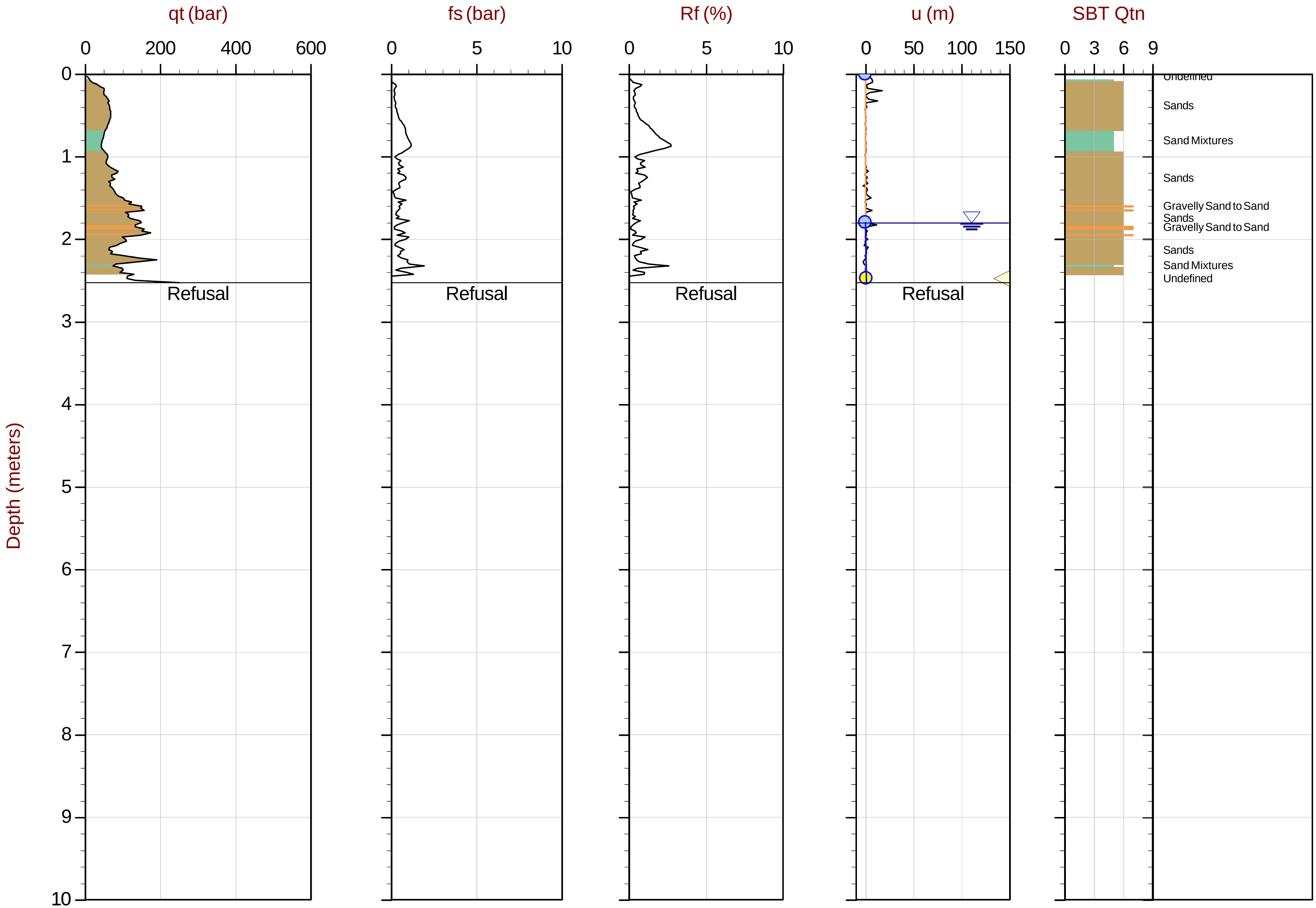
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Standard Cone Penetration Test Plots with Expanded Range

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:13
Site: Churchville Park

Sounding: CPT23-01
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.525 m / 8.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831845m E: 600240m
Sheet No: 1 of 1

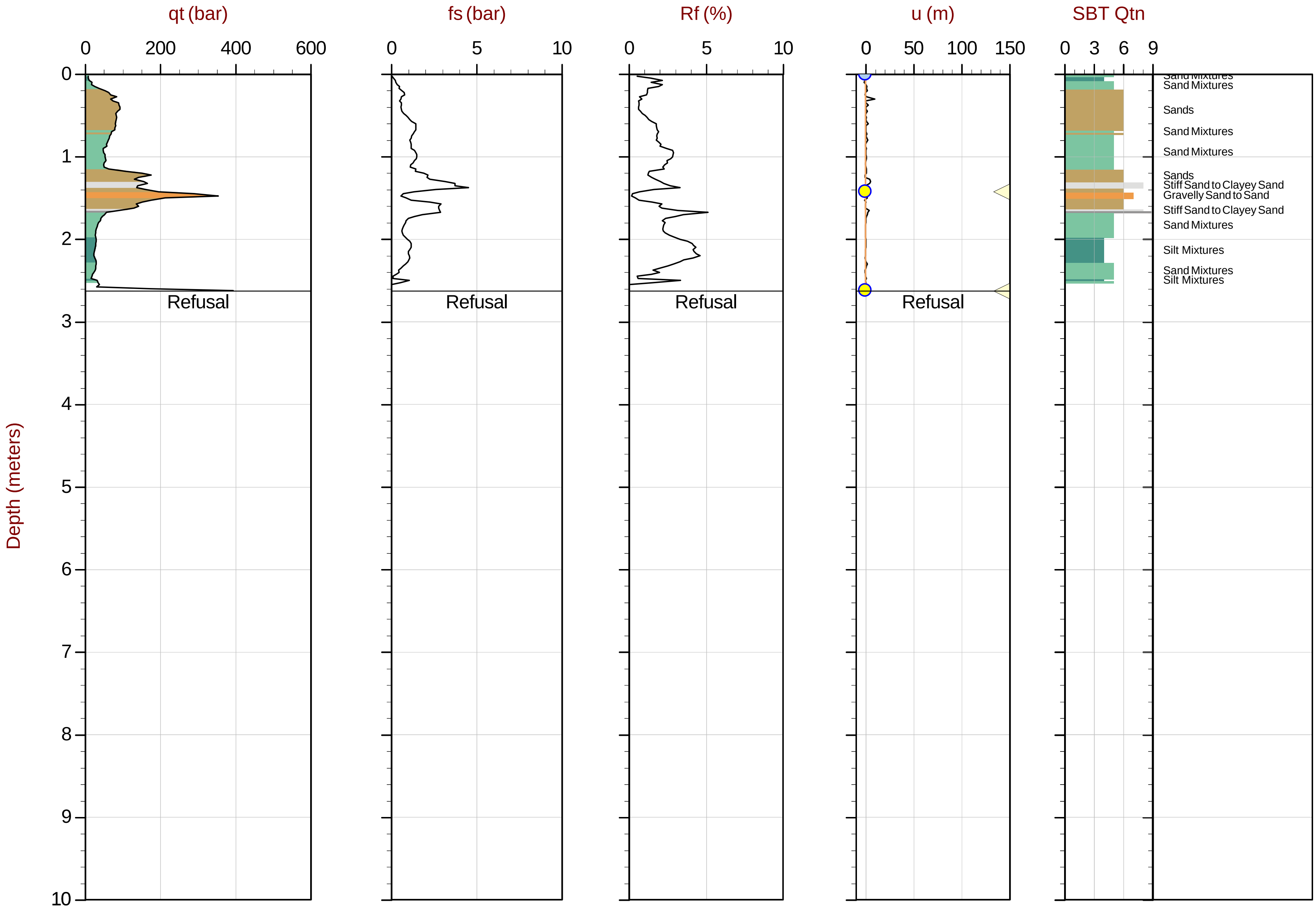
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park

Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.625 m / 8.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600251m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

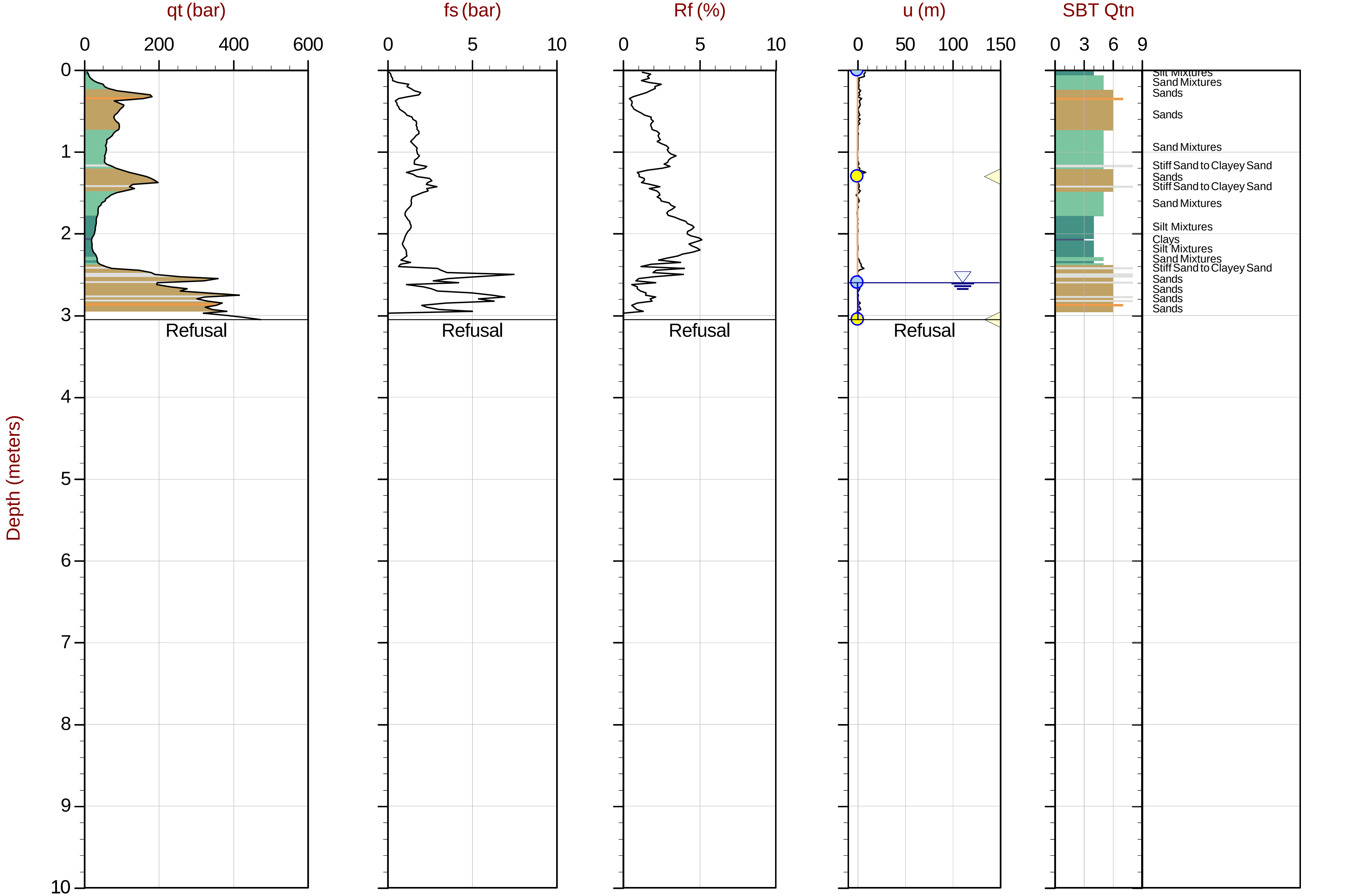
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 10:15
Site: Churchville Park

Sounding: CPT23-02B
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 3.050 m / 10.01 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600250m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

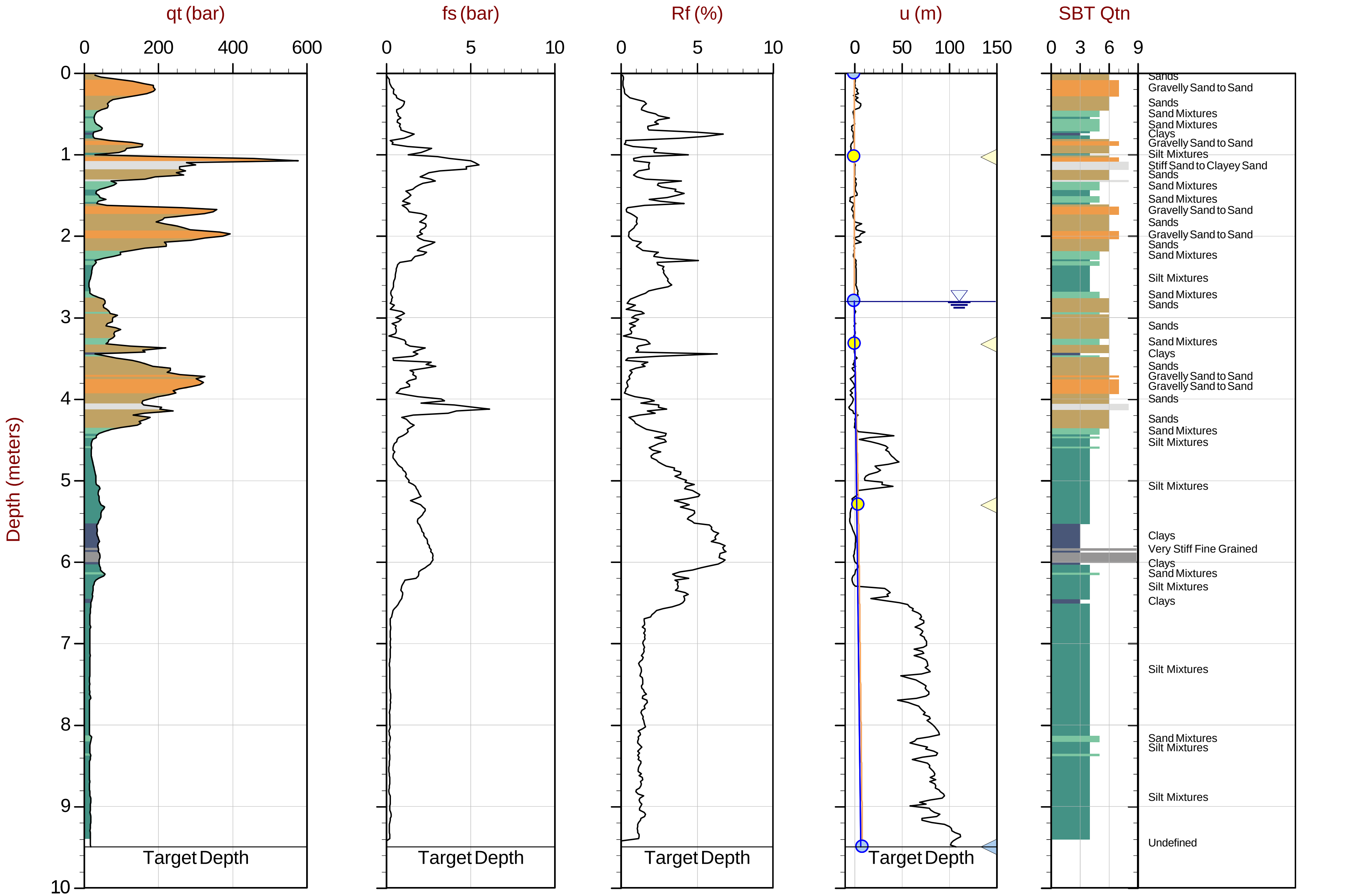
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 9.500 m / 31.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831910m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

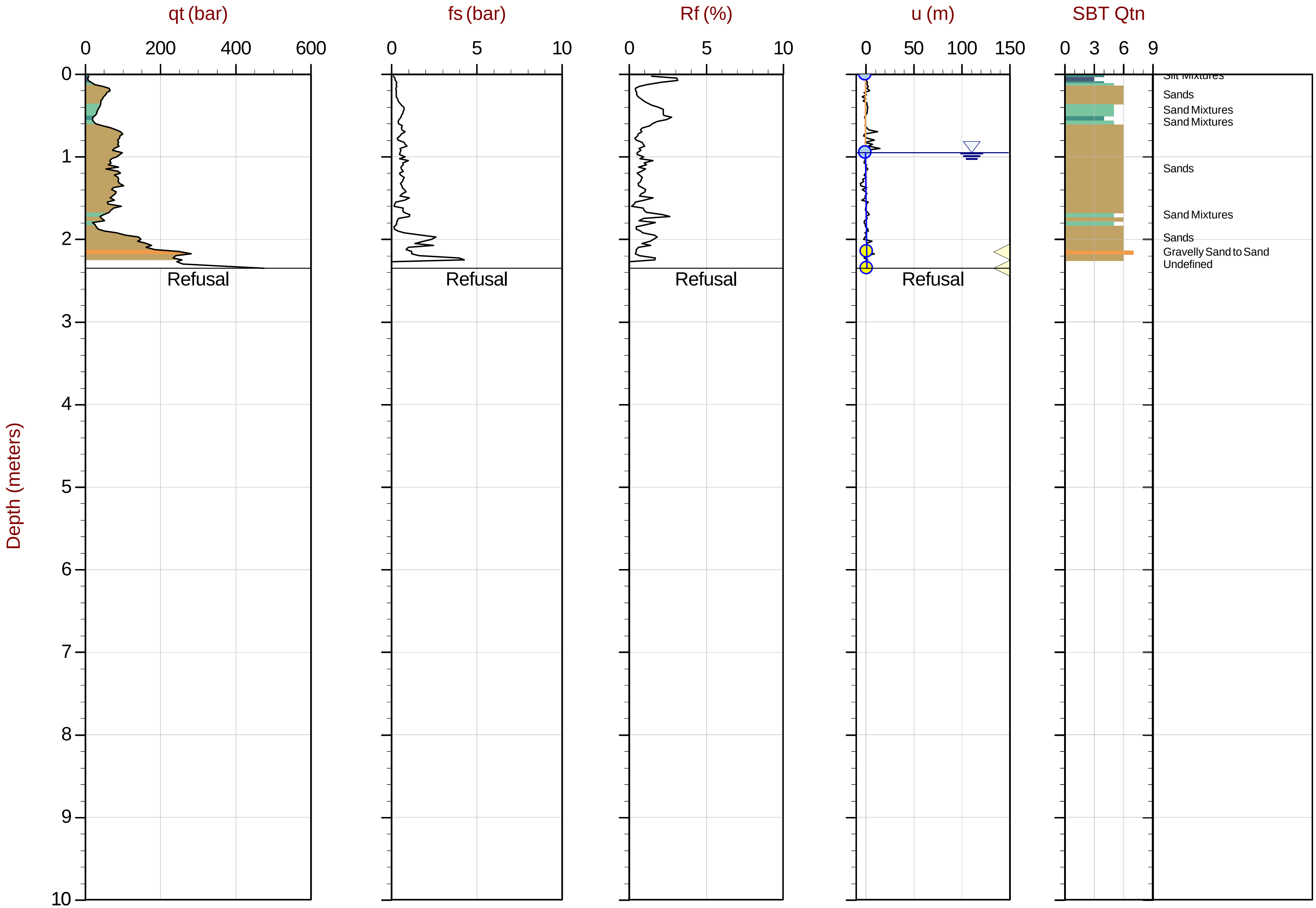
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 13:15
Site: Churchville Park

Sounding: CPT23-04
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.350 m / 7.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831899m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

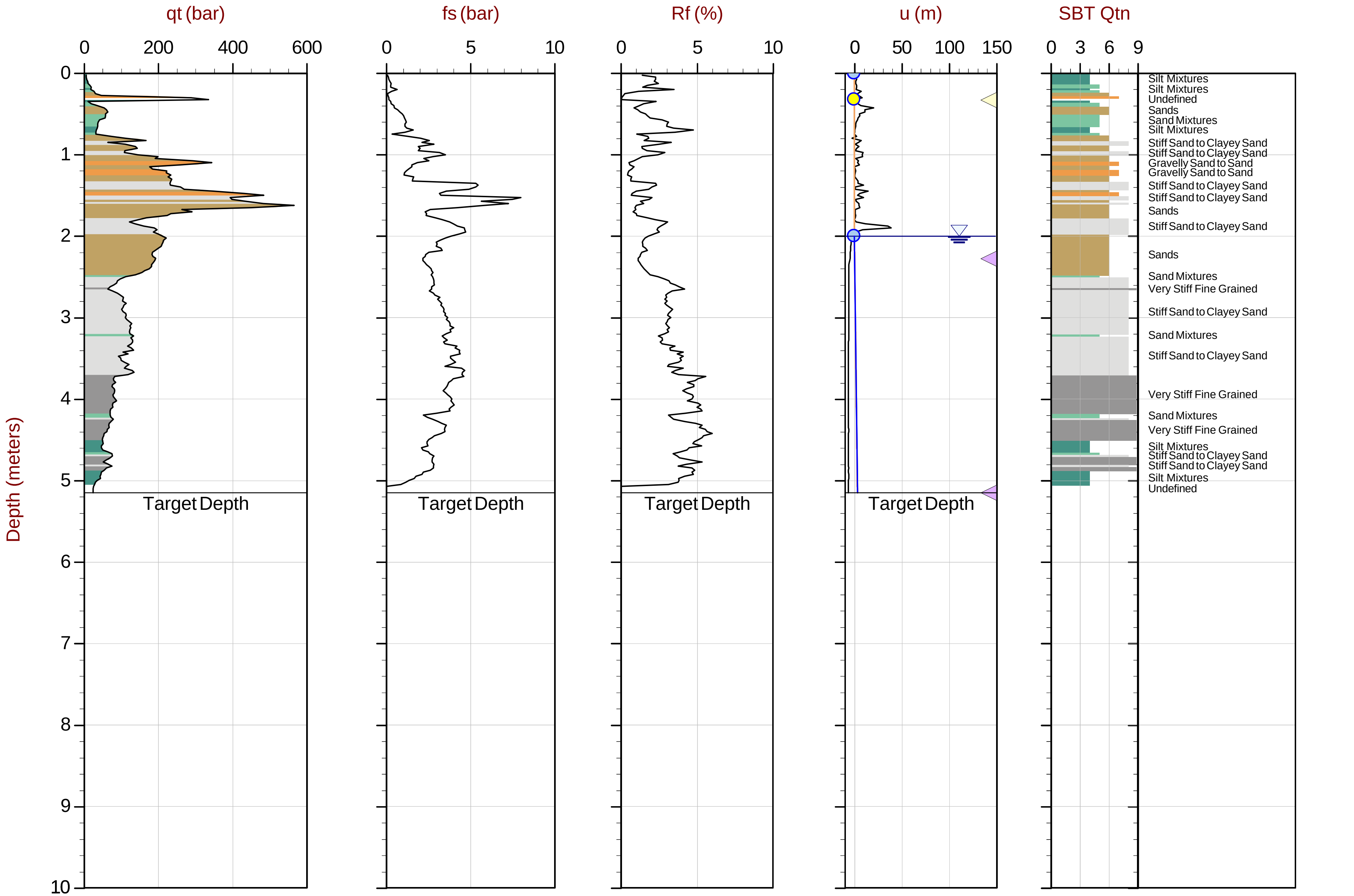
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 5.150 m / 16.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831931m E: 600052m
Sheet No: 1 of 1

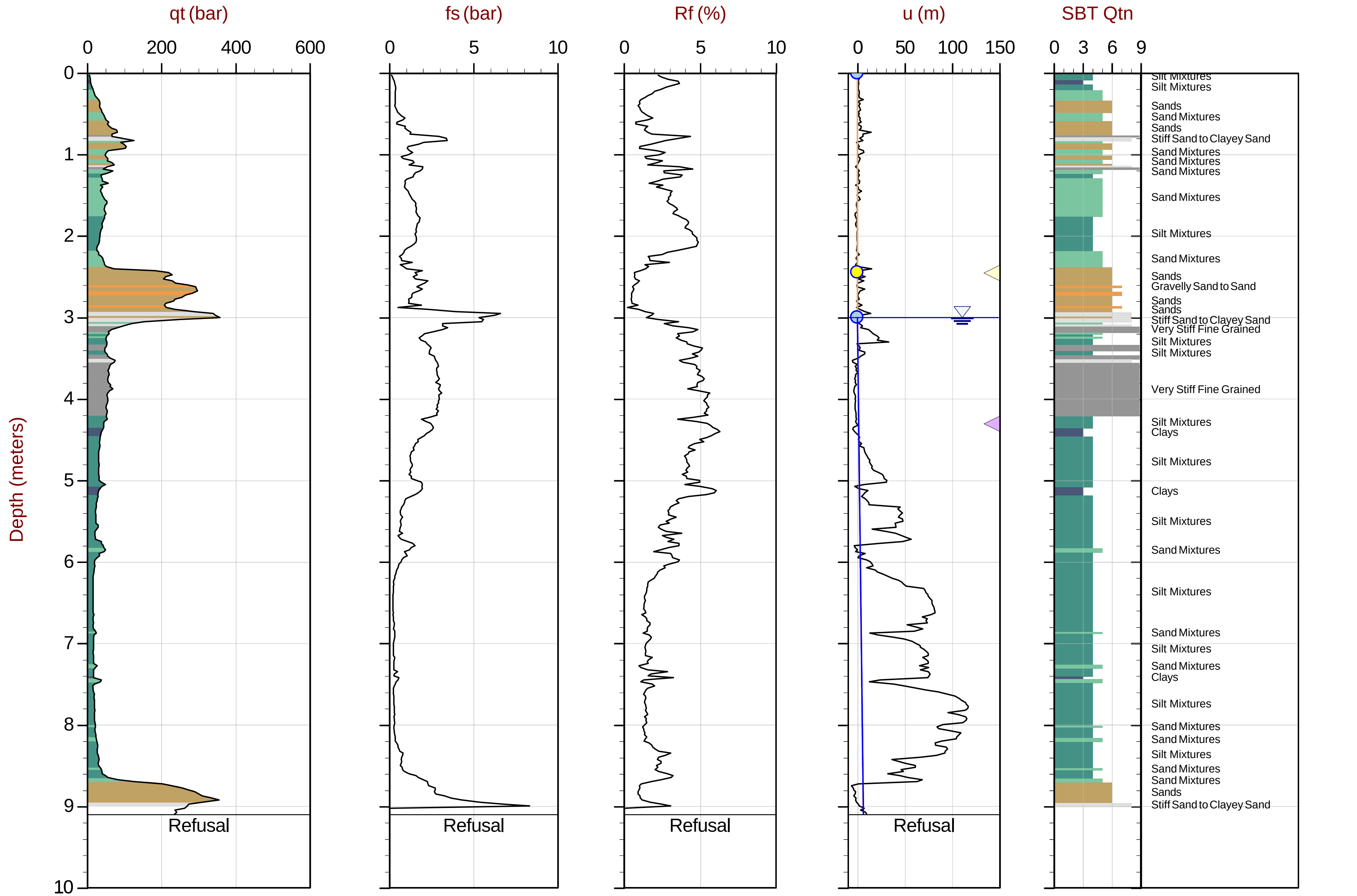
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

KGS

Job No: 23-05-26771
Date: 2023-11-07 15:19
Site: Churchville Park

Sounding: CPT23-06
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 9.100 m / 29.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832050m E: 600000m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Ueq Line, Hydrostatic Line

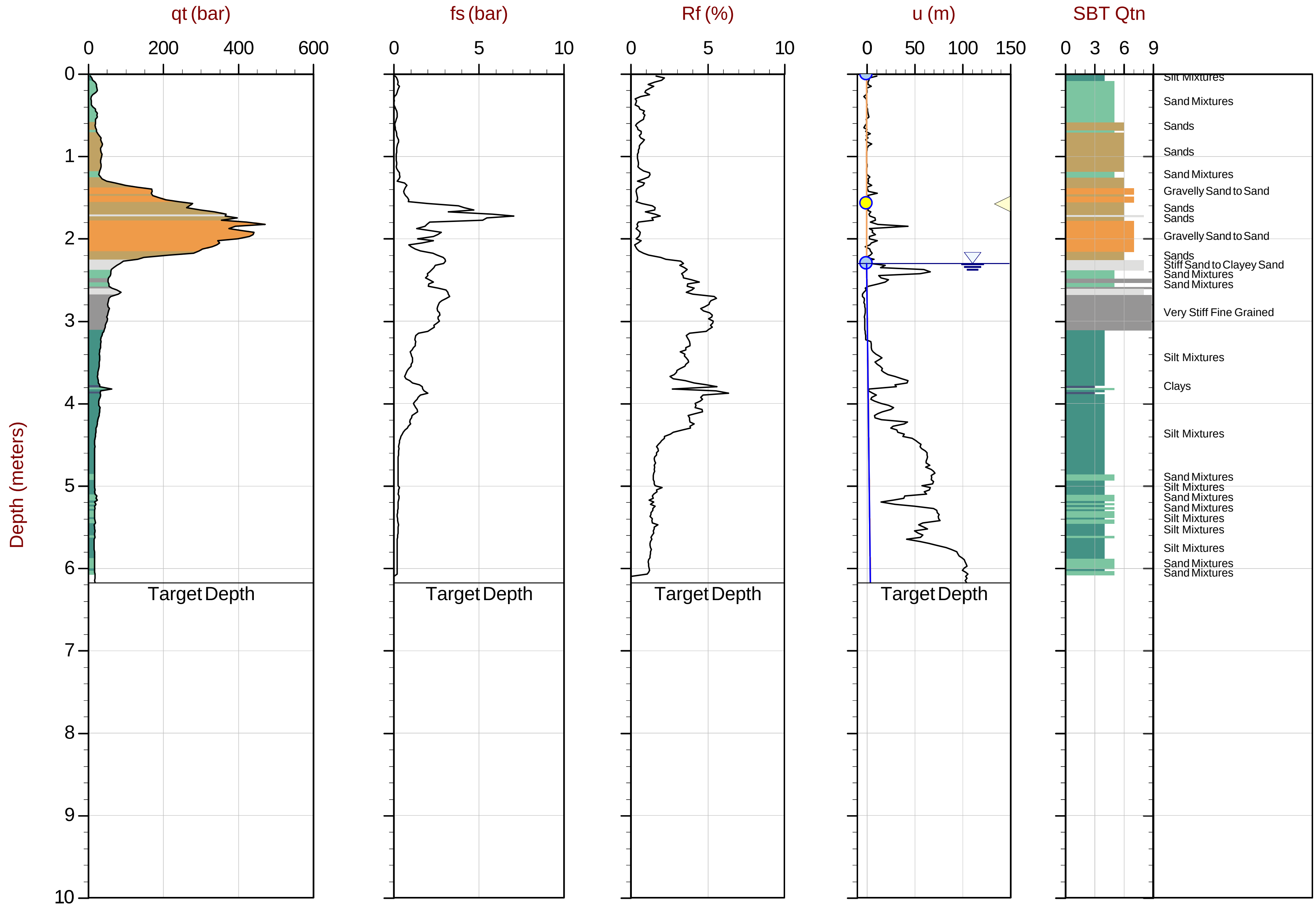
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 16:33
Site: Churchville Park

Sounding: CPT23-07
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 6.175 m / 20.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832054m E: 599992m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

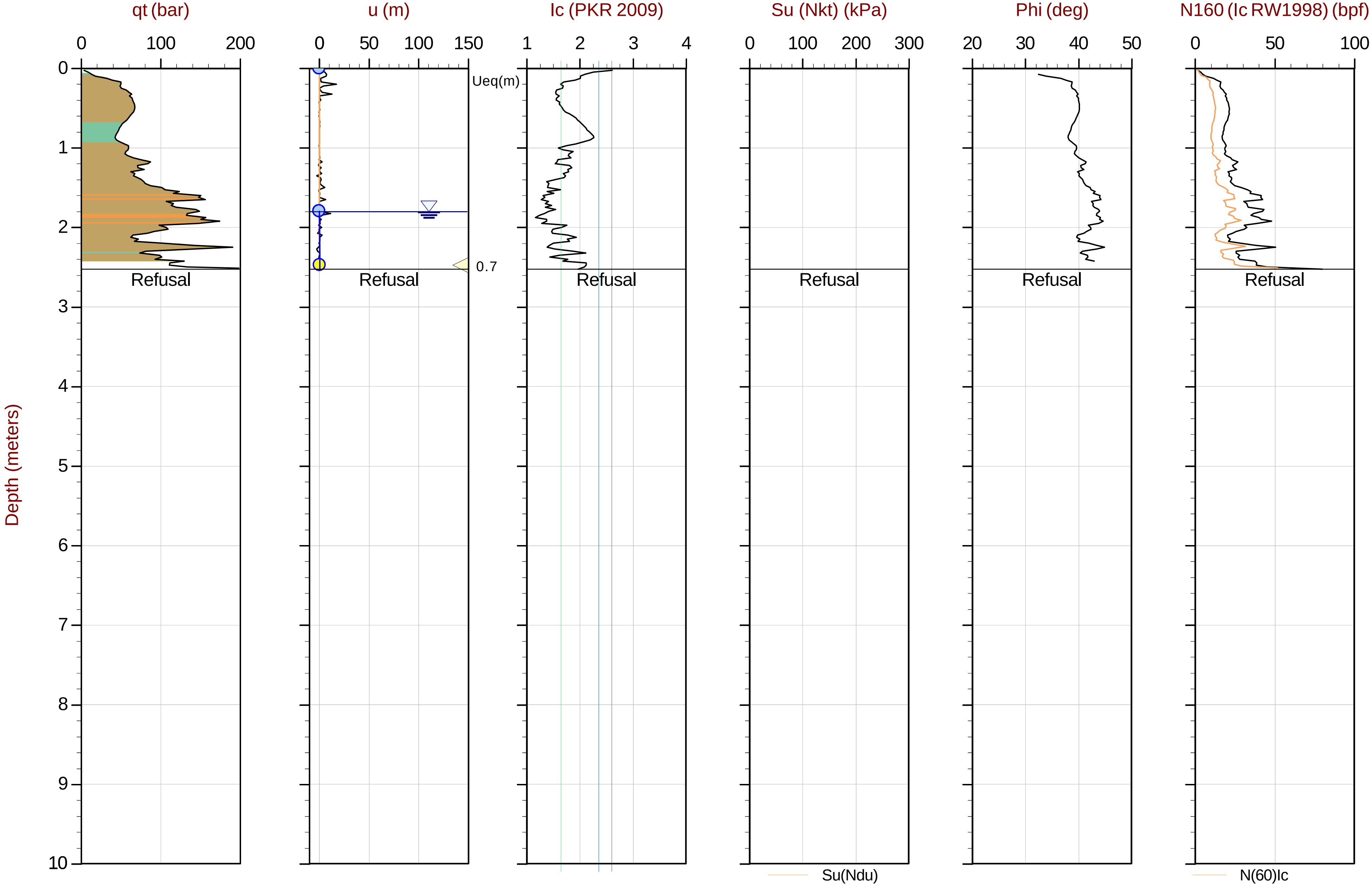
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Plots with Ic, Su(Nkt), Phi, and N1(60)lc

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:13
Site: Churchville Park
Sounding: CPT23-01
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.525 m / 8.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831845m E: 600240m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

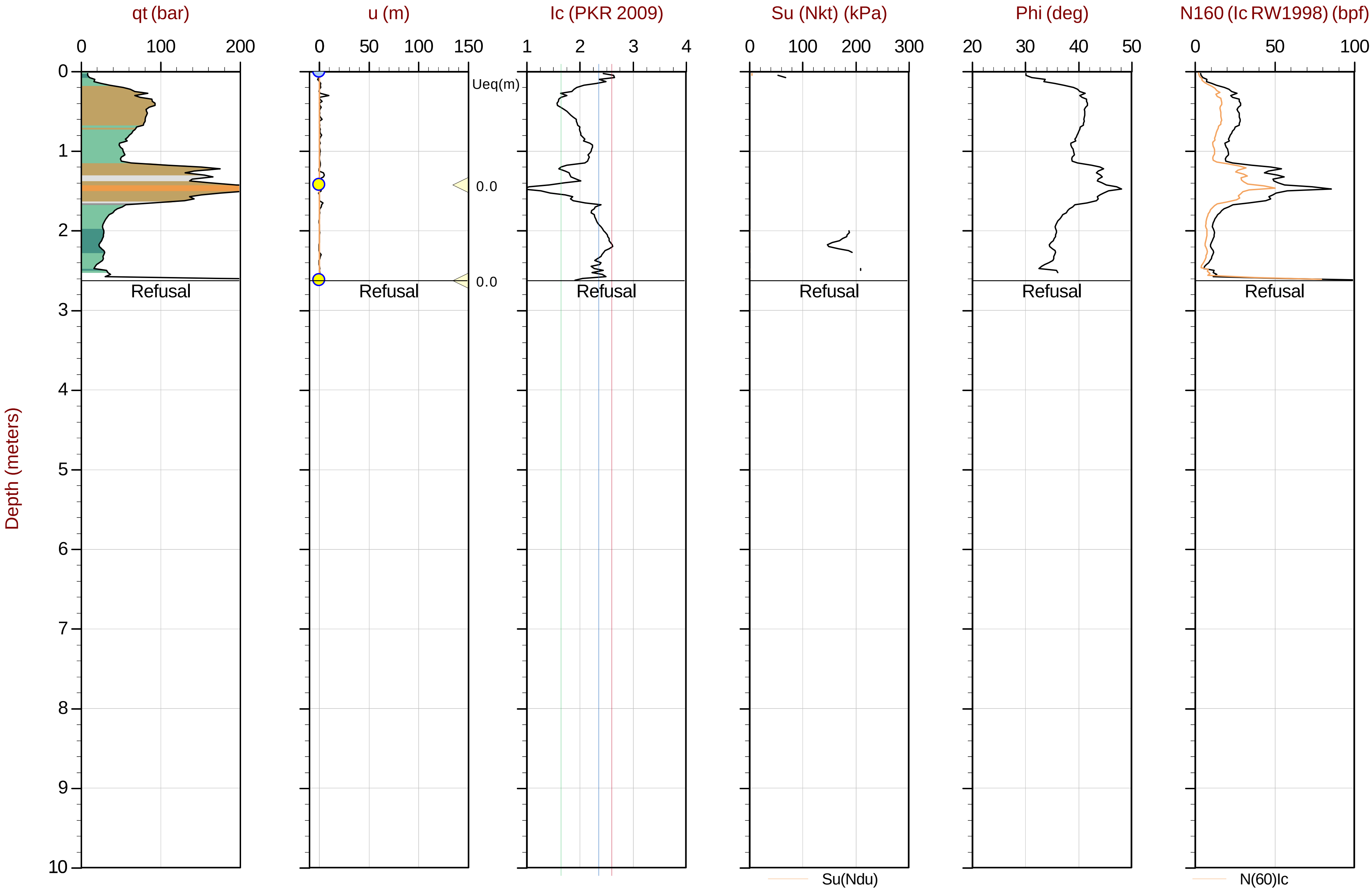
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park

Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 2.625 m / 8.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600251m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

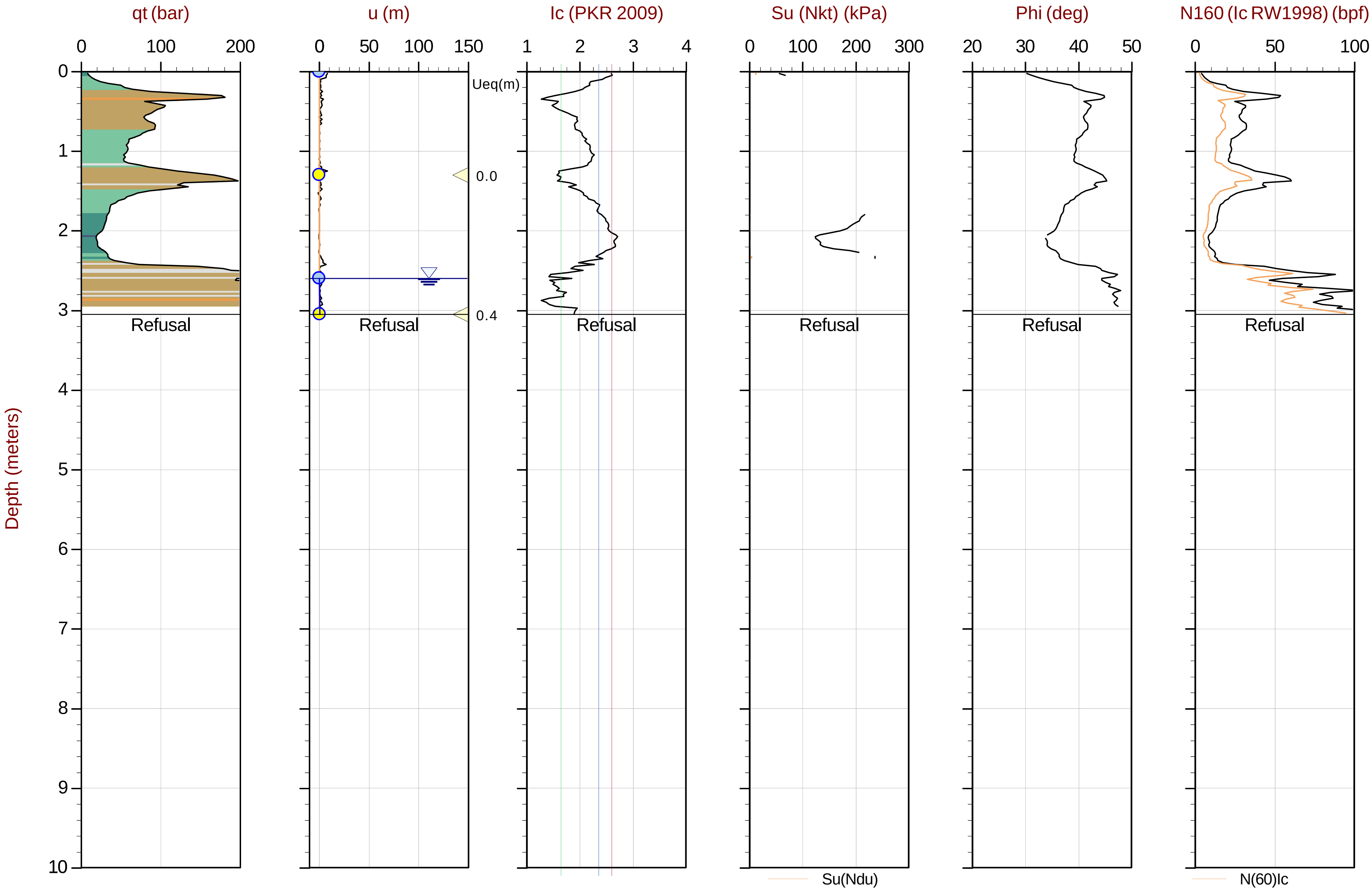
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 10:15
Site: Churchville Park

Sounding: CPT23-02B
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 3.050 m / 10.01 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP02B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831857m E: 600250m
Sheet No: 1 of 1

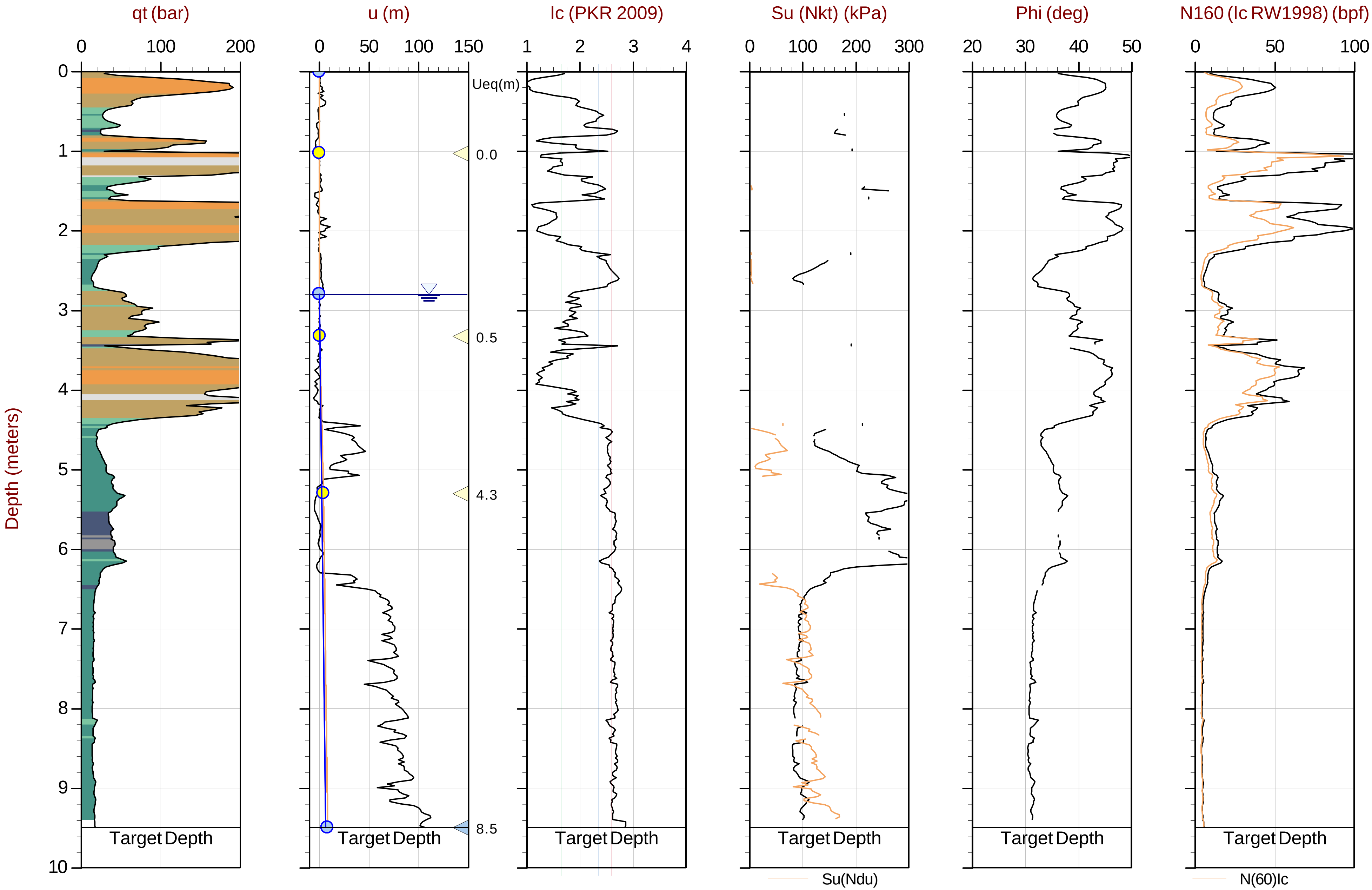
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



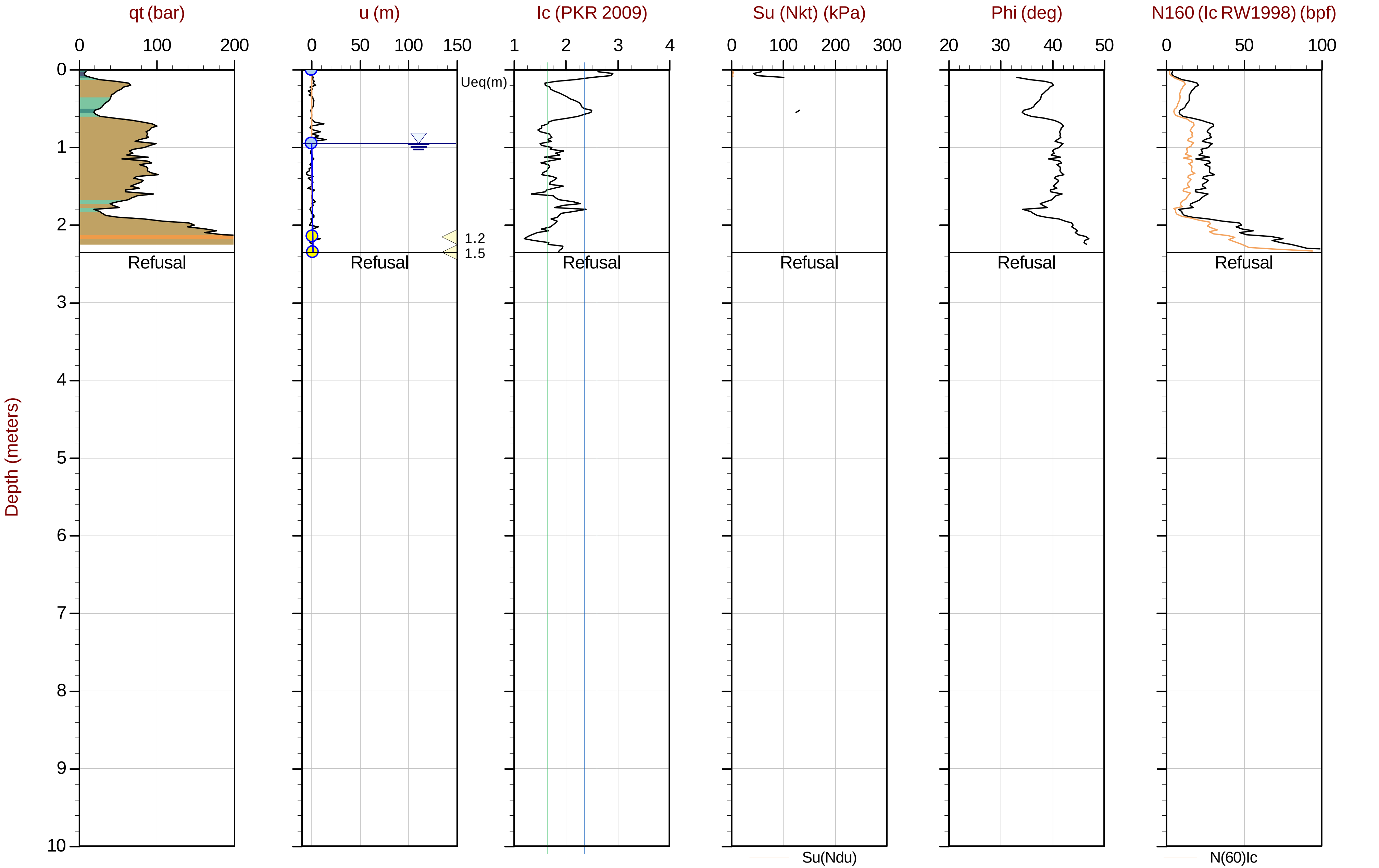
Max Depth: 9.500 m / 31.17 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831910m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Ueq Line | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 2.350 m / 7.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831899m E: 600386m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Ueq Line, Hydrostatic Line

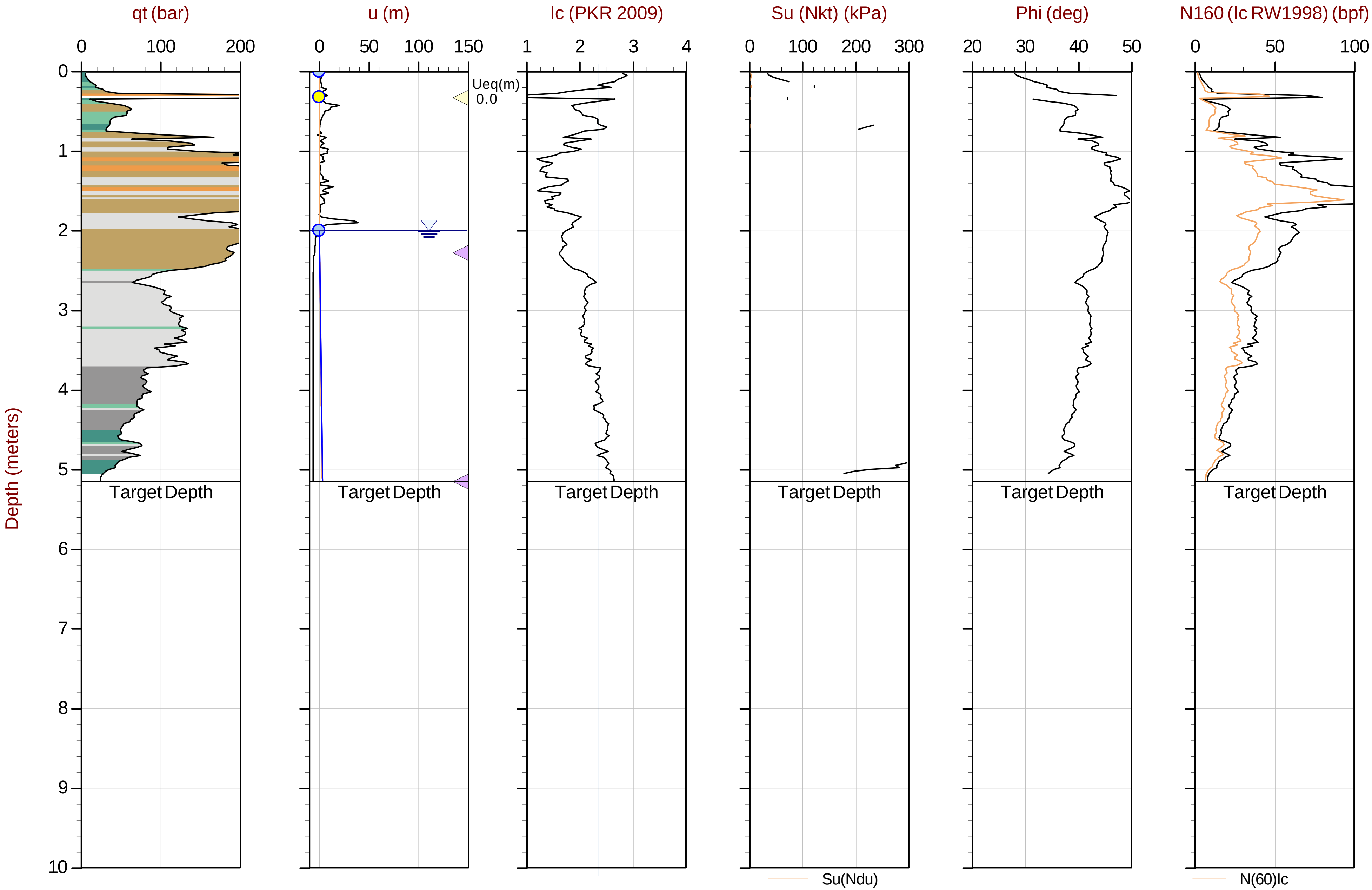
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 5.150 m / 16.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4831931m E: 600052m
Sheet No: 1 of 1

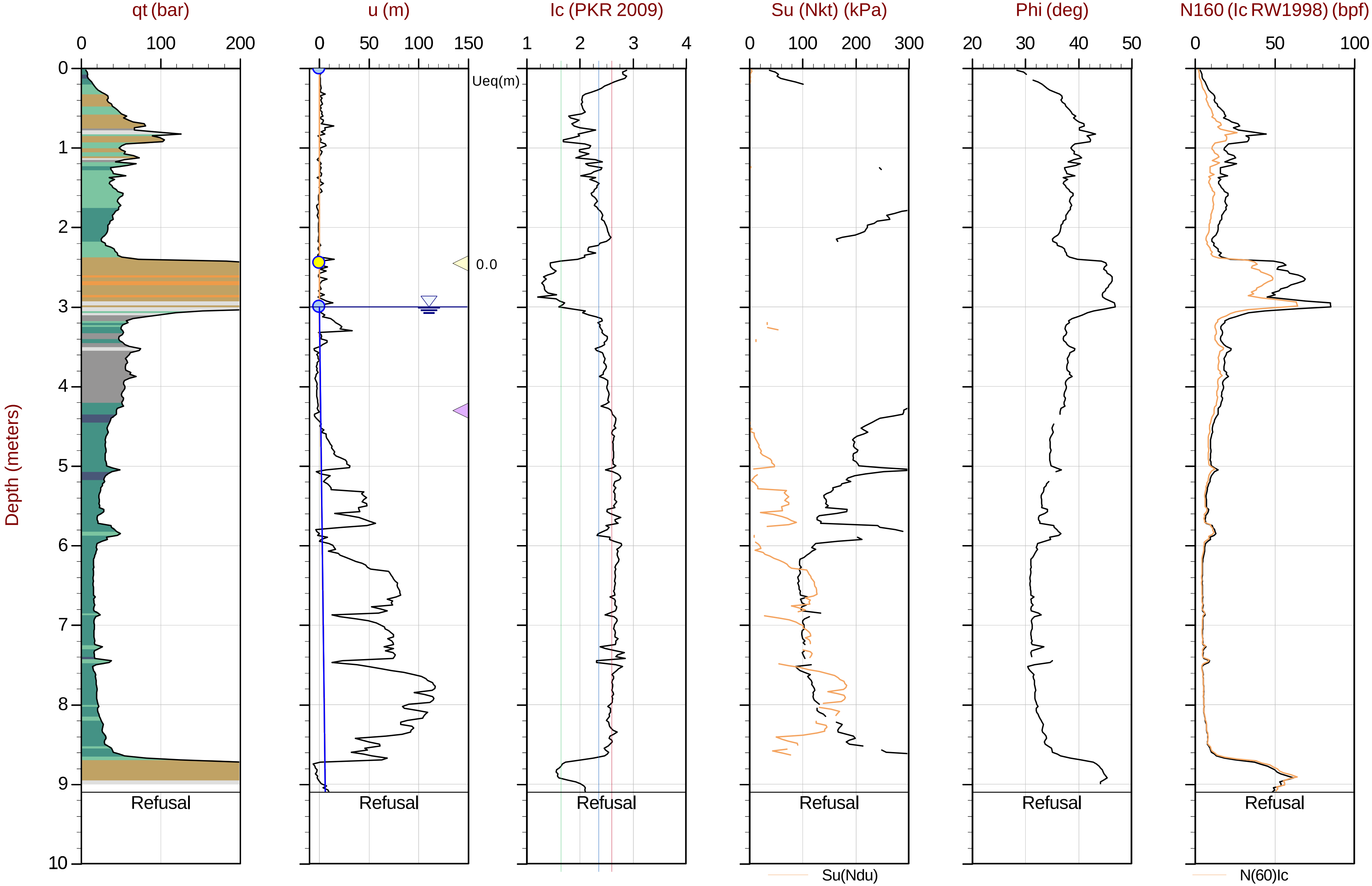
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

KGS

Job No: 23-05-26771
Date: 2023-11-07 15:19
Site: Churchville Park

Sounding: CPT23-06
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 9.100 m / 29.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832050m E: 600000m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

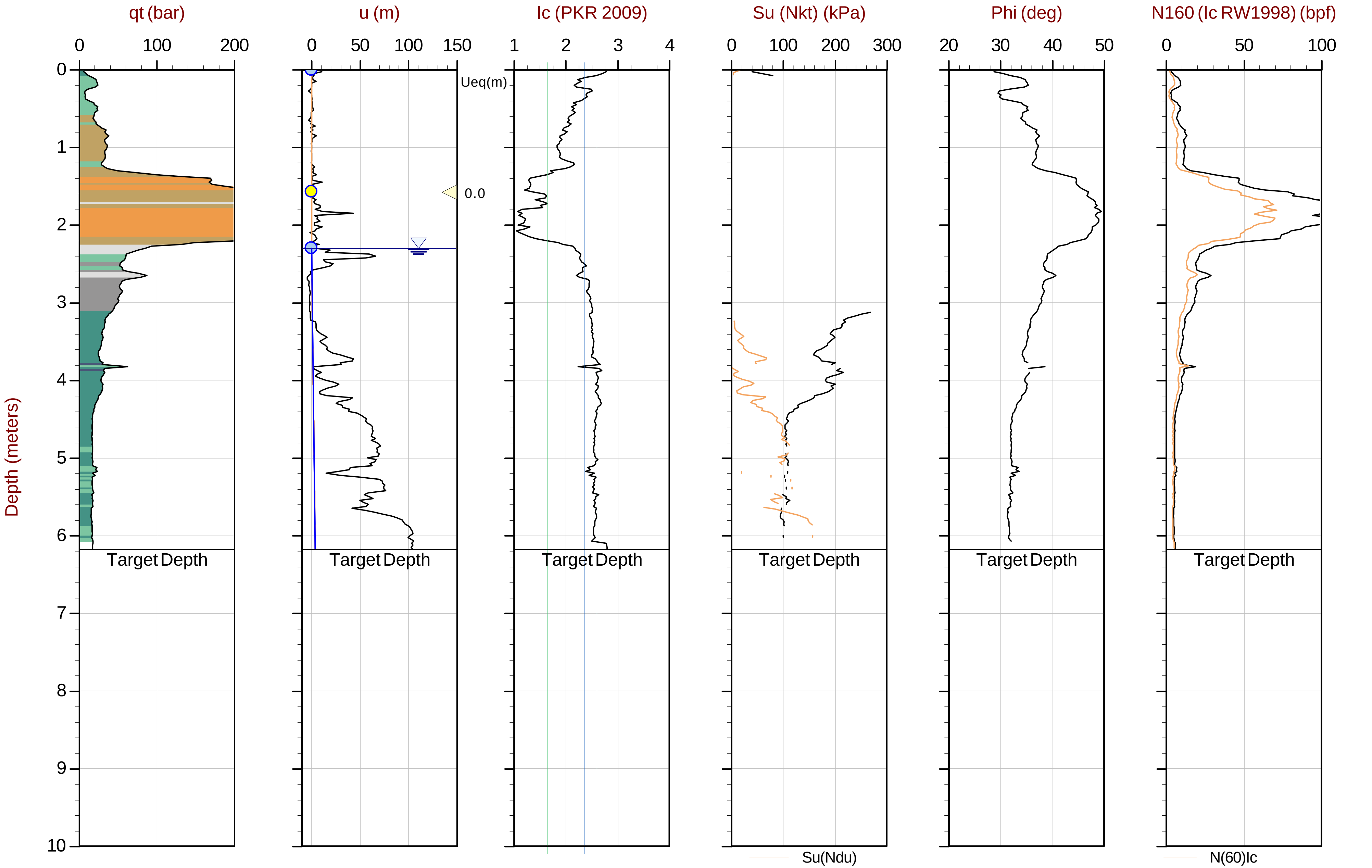
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 16:33
Site: Churchville Park

Sounding: CPT23-07
Cone: 766:T1000F10U35 Area=10 cm²



Max Depth: 6.175 m / 20.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26771_CP07.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4832054m E: 599992m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behaviour Type (SBT) Scatter Plots

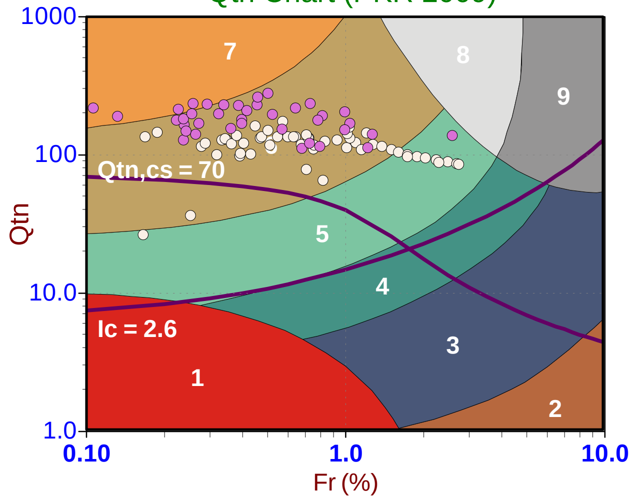
CONETEC

KGS

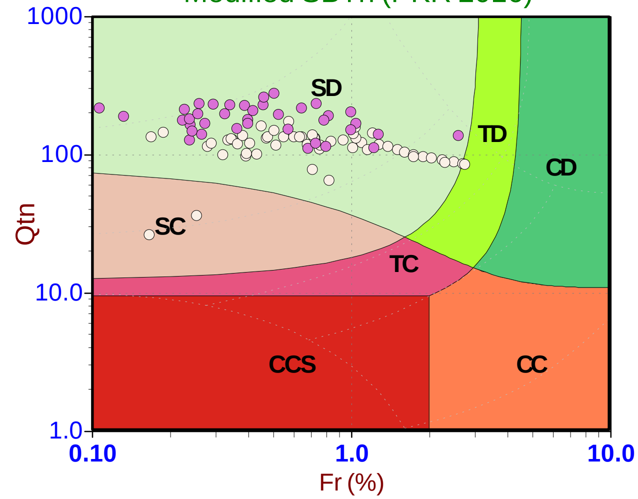
Job No: 23-05-26771
Date: 2023-11-07 09:13
Site: Churchville Park

Sounding: CPT23-01
Cone: 766:T1000F10U35 Area=10 cm²

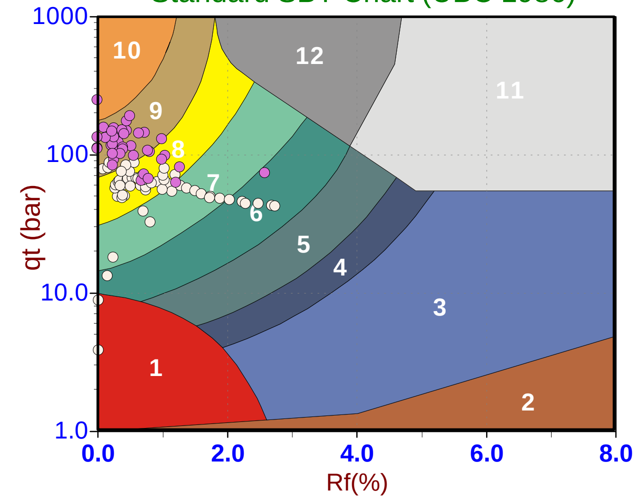
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

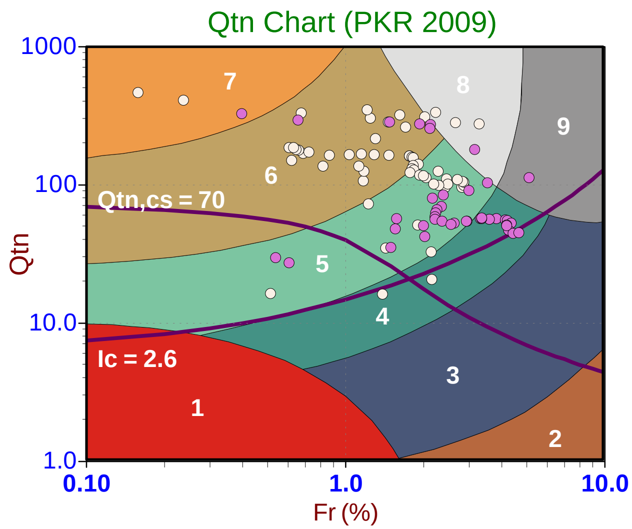
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

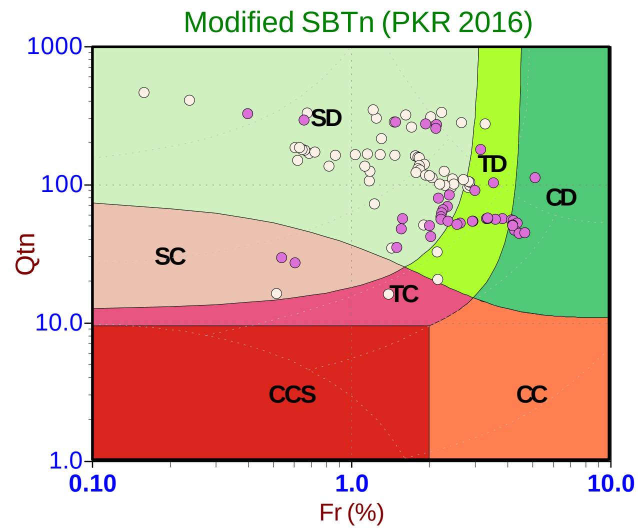
CONETEC

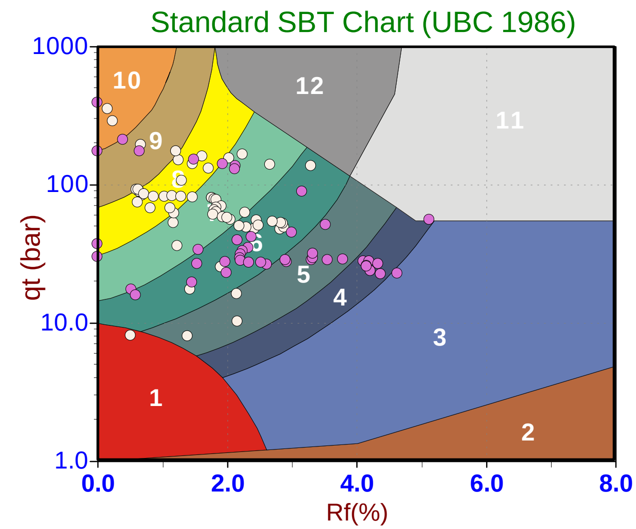
KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park

Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²







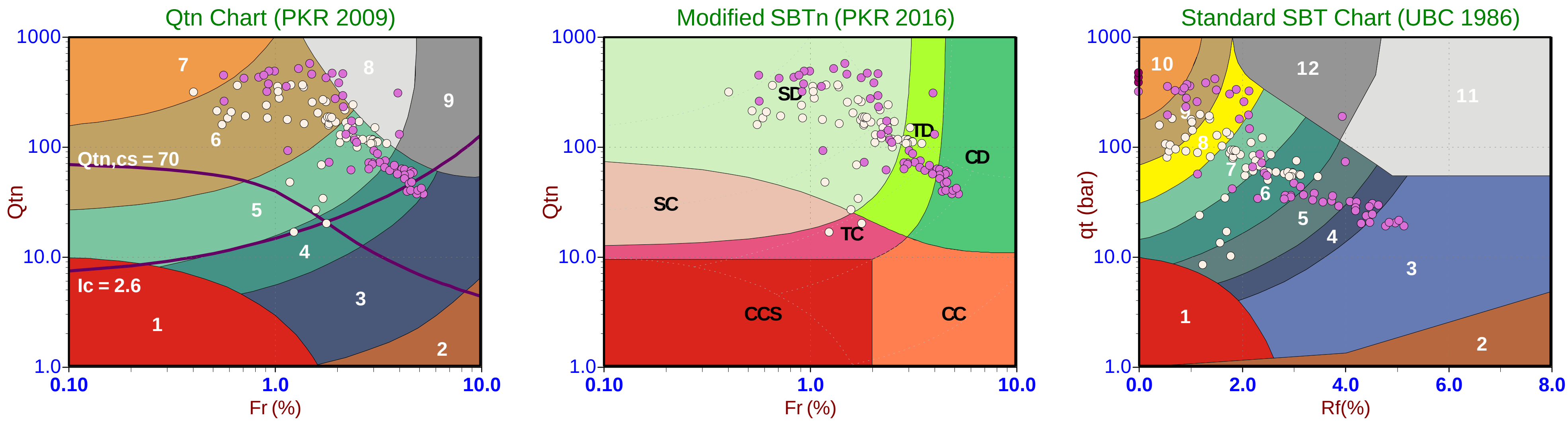
Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

KGS	Job No: 23-05-26771	Sounding: CPT23-02B
	Date: 2023-11-07 10:15	Cone: 766:T1000F10U35 Area=10 cm²
	Site: Churchville Park	



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

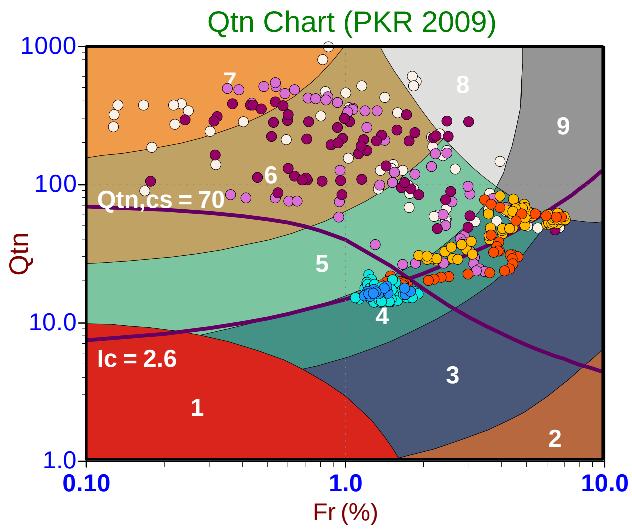
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

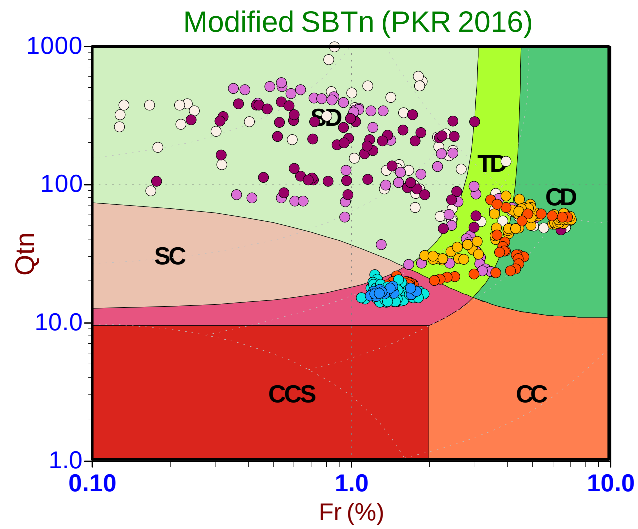
CONETEC

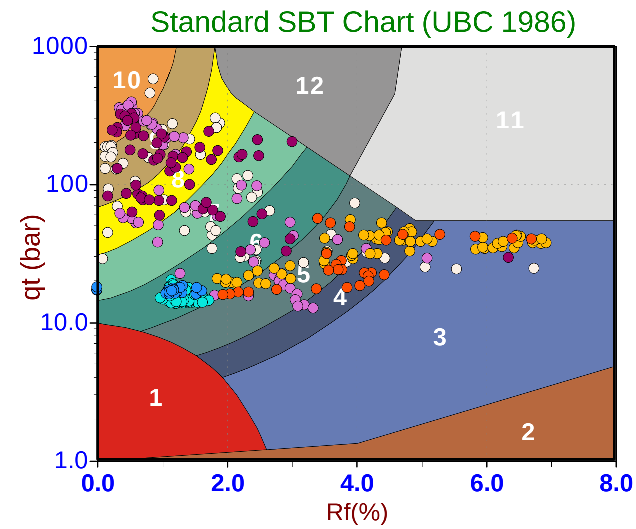
KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²







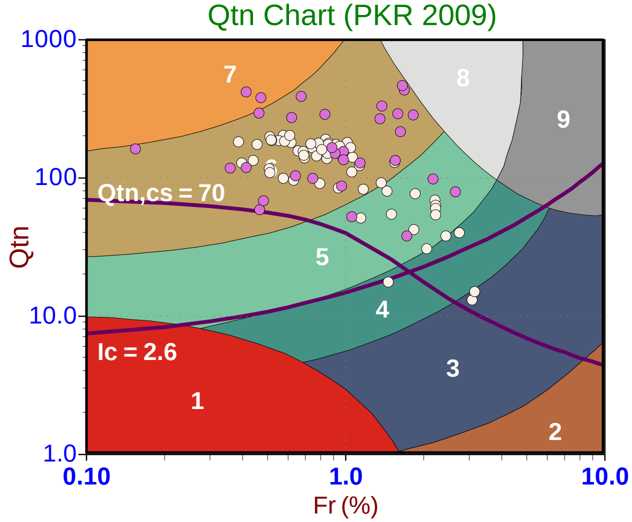
Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

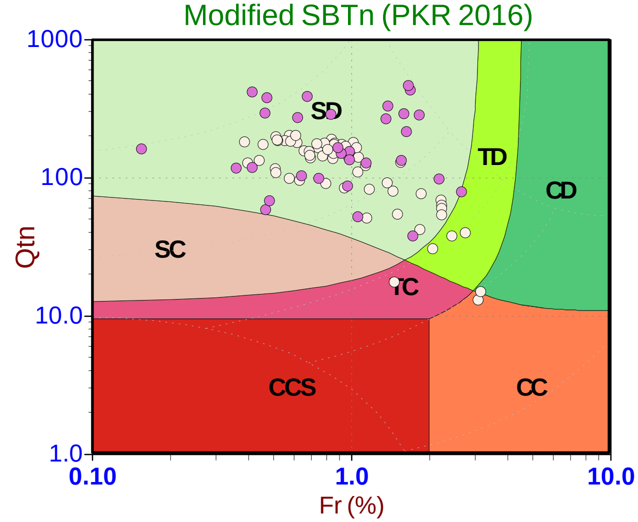
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

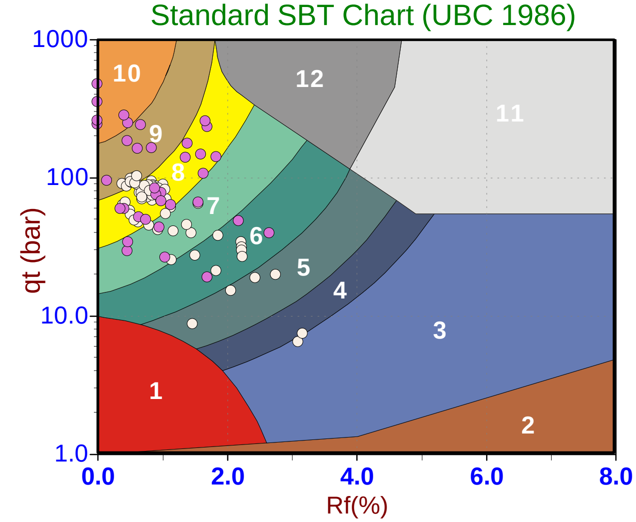
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

KGS	Job No: 23-05-26771 Date: 2023-11-07 13:15 Site: Churchville Park	Sounding: CPT23-04 Cone: 766:T1000F10U35 Area=10 cm²







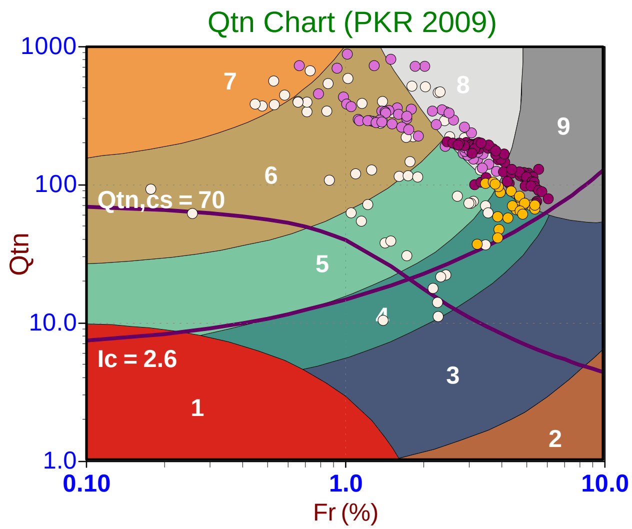
Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

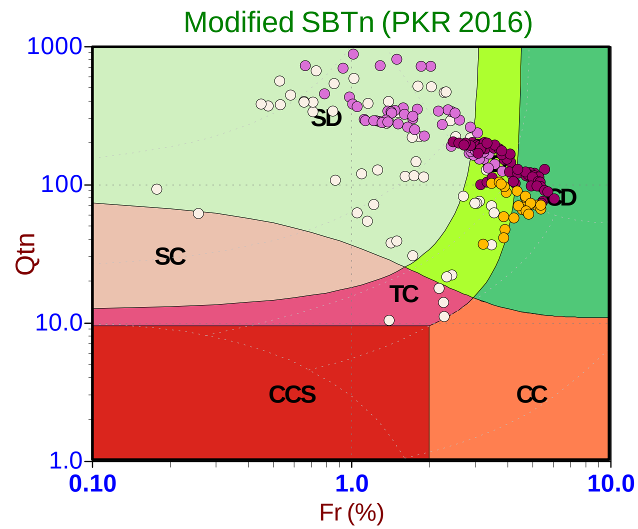
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

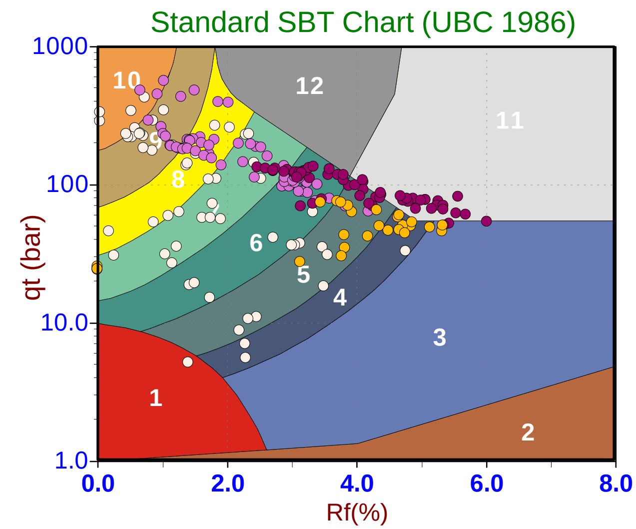
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

KGS	Job No: 23-05-26771 Date: 2023-11-07 14:29 Site: Churchville Park	Sounding: CPT23-05 Cone: 766:T1000F10U35 Area=10 cm²







Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

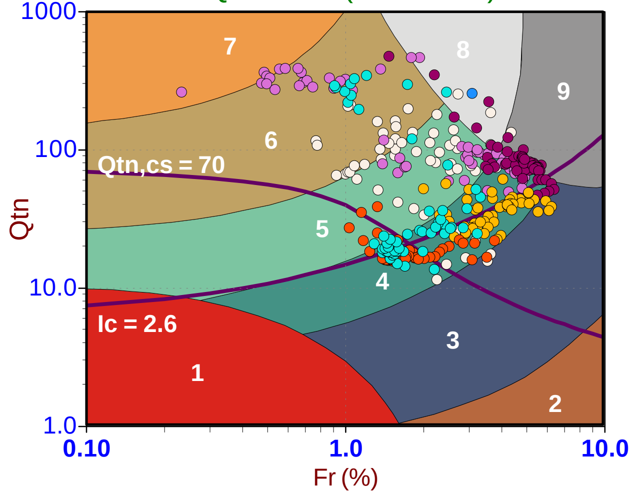
CONETEC

KGS

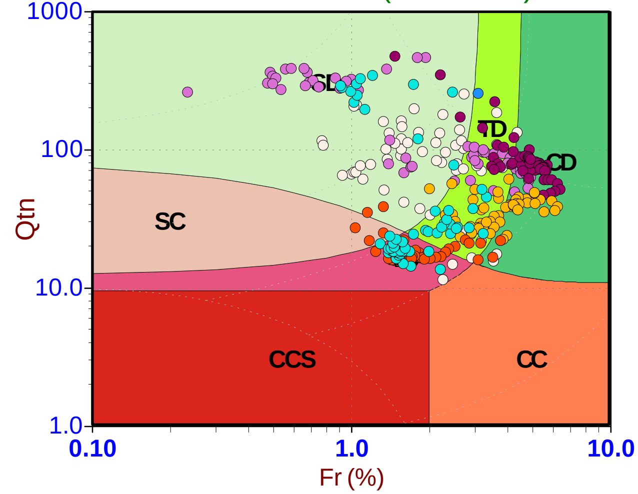
Job No: 23-05-26771
Date: 2023-11-07 15:19
Site: Churchville Park

Sounding: CPT23-06
Cone: 766:T1000F10U35 Area=10 cm²

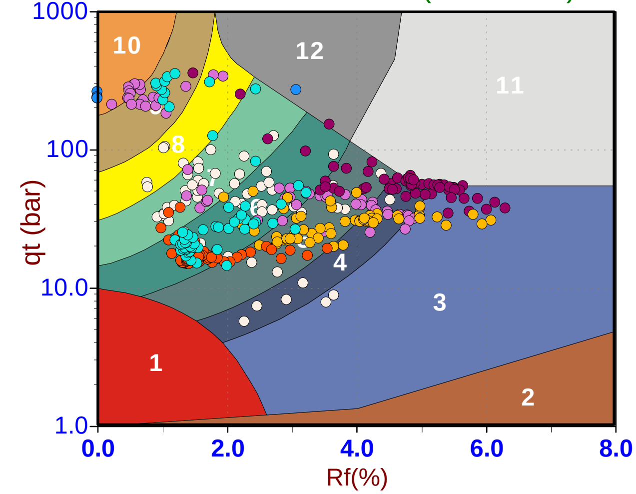
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

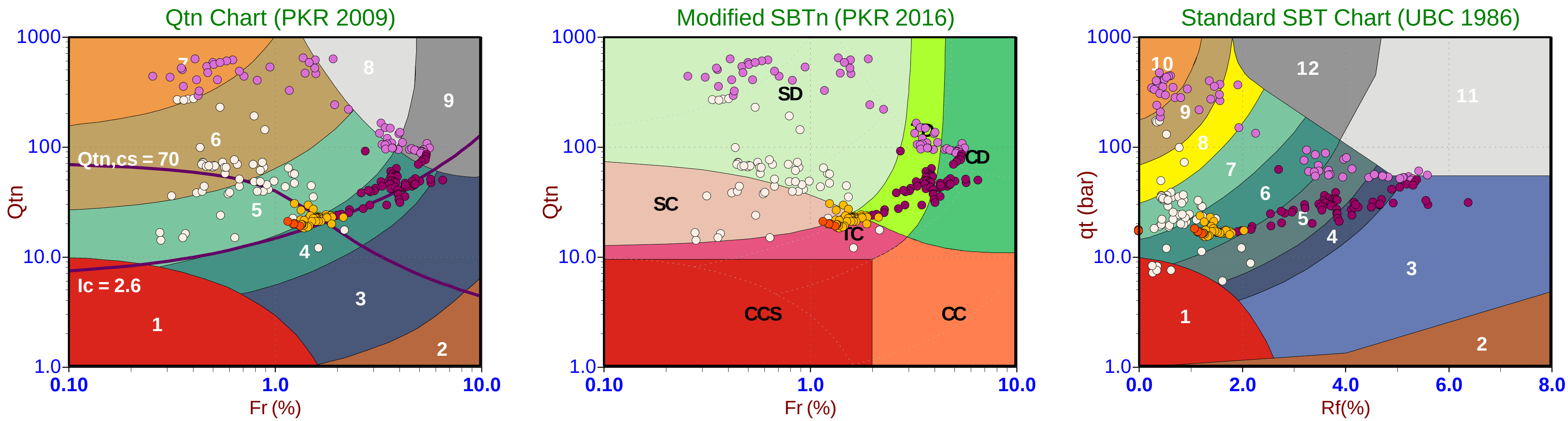
Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

KGS

Job No: 23-05-26771
Date: 2023-11-07 16:33
Site: Churchville Park

Sounding: CPT23-07
Cone: 766:T1000F10U35 Area=10 cm²



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

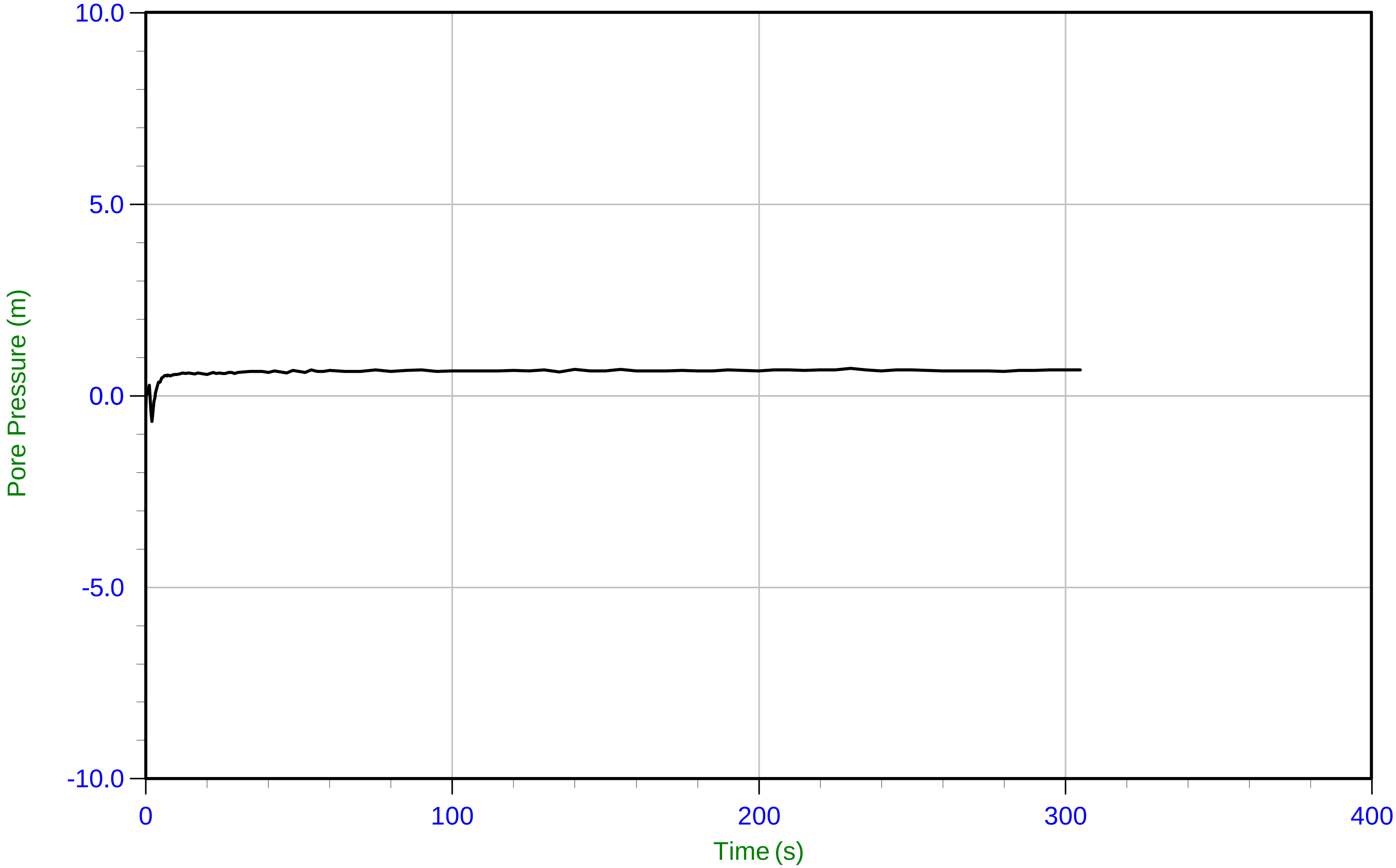
Job No:	23-05-26771
Client:	KGS Group Consulting Engineers
Project:	Churchville CPT
Start Date:	2023-11-07
End Date:	2023-11-07

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	$U_{initial}$ (m)	U_{max} (m)	U_{min} (m)	U_{final} (m)	Equilibrium Pore Pressure U_{eq} (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Percent Dissipation (%)	t_{50} (s)[1]	Assumed Rigidity Index (I_r)	c_h (cm^2/min)[2]	Refer to Notation Number
CPT23-01	23-05-26771_CP01	10	305	2.475	-0.3	0.7	-0.7	0.7	0.7		1.8					
CPT23-02	23-05-26771_CP02	10	565	1.425	2.5	2.5	-2.6	-0.1	0.0							
CPT23-02	23-05-26771_CP02	10	140	2.625	0.1	0.1	-0.9	-0.1	0.0							
CPT23-02B	23-05-26771_CP02B	10	305	1.300	0.0	0.1	-0.1	0.0	0.0							
CPT23-02B	23-05-26771_CP02B	10	380	3.050	0.8	0.8	-2.0	0.4	0.4		2.6					
CPT23-03	23-05-26771_CP03	10	300	1.025	-2.8	1.4	-2.8	0.0	0.0							
CPT23-03	23-05-26771_CP03	10	610	3.325	-0.5	0.5	-1.1	0.5	0.5		2.8					
CPT23-03	23-05-26771_CP03	10	320	5.300	0.8	5.5	0.8	4.3	4.3		1.0					
CPT23-03	23-05-26771_CP03	10	900	9.500	106.5	106.5	57.1	57.1		8.5	1.0	50	880	100	0.5	3
CPT23-04	23-05-26771_CP04	10	560	2.150	-1.2	1.2	-1.2	1.2	1.2		1.0					
CPT23-04	23-05-26771_CP04	10	310	2.350	0.8	1.7	0.8	1.5	1.5		0.9					
CPT23-05	23-05-26771_CP05	10	100	0.325	0.0	0.2	-7.0	0.0	0.0							
CPT23-05	23-05-26771_CP05	10	440	2.275	-4.7	-4.7	-5.7	-5.4								
CPT23-05	23-05-26771_CP05	10	320	5.150	-6.2	50.2	-6.3	50.2								
CPT23-06	23-05-26771_CP06	10	305	2.450	0.4	0.4	-6.4	-0.1	0.0							
CPT23-06	23-05-26771_CP06	10	325	4.300	-1.5	47.1	-2.0	47.1								
CPT23-07	23-05-26771_CP07	10	145	1.575	1.3	1.3	-2.7	-0.1	0.0							

1. Time for 50 percent dissipation based on U_{max}, U_{min}, and the applied U_{eq}. Note the time is relative to where U_{max} occurred.
2. Houlsby and Teh, 1991.
3. Equilibrium pore pressure estimated based on a hydrostatic assumption from the nearest pore pressure dissipation test that achieved equilibrium.

KGS	Job No: 23-05-26771 Date: 2023-11-07 09:13 Site: Churchville Park	Sounding: CPT23-01 Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP01.PPF2
Depth: 2.475 m / 8.120 ft
Duration: 305.0 s

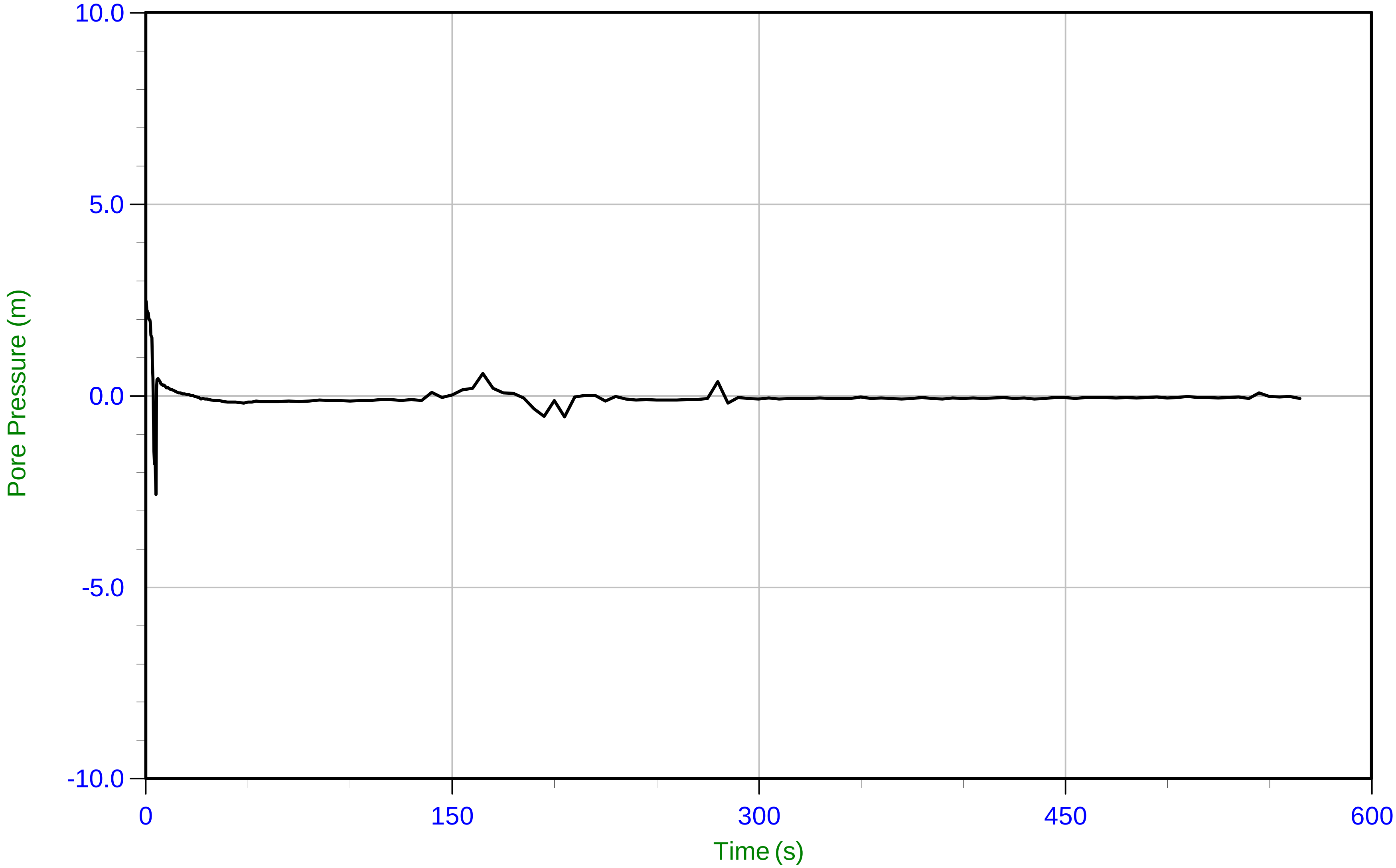
u Min: -0.7 m
u Max: 0.7 m
u Final: 0.7 m

WT: 1.8 m / 5.8 ft
Ueq: 0.7 m

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park

Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP02.PPF2
Depth: 1.425 m / 4.675 ft
Duration: 565.0 s

u Min: -2.6 m
u Max: 2.5 m
u Final: -0.1 m

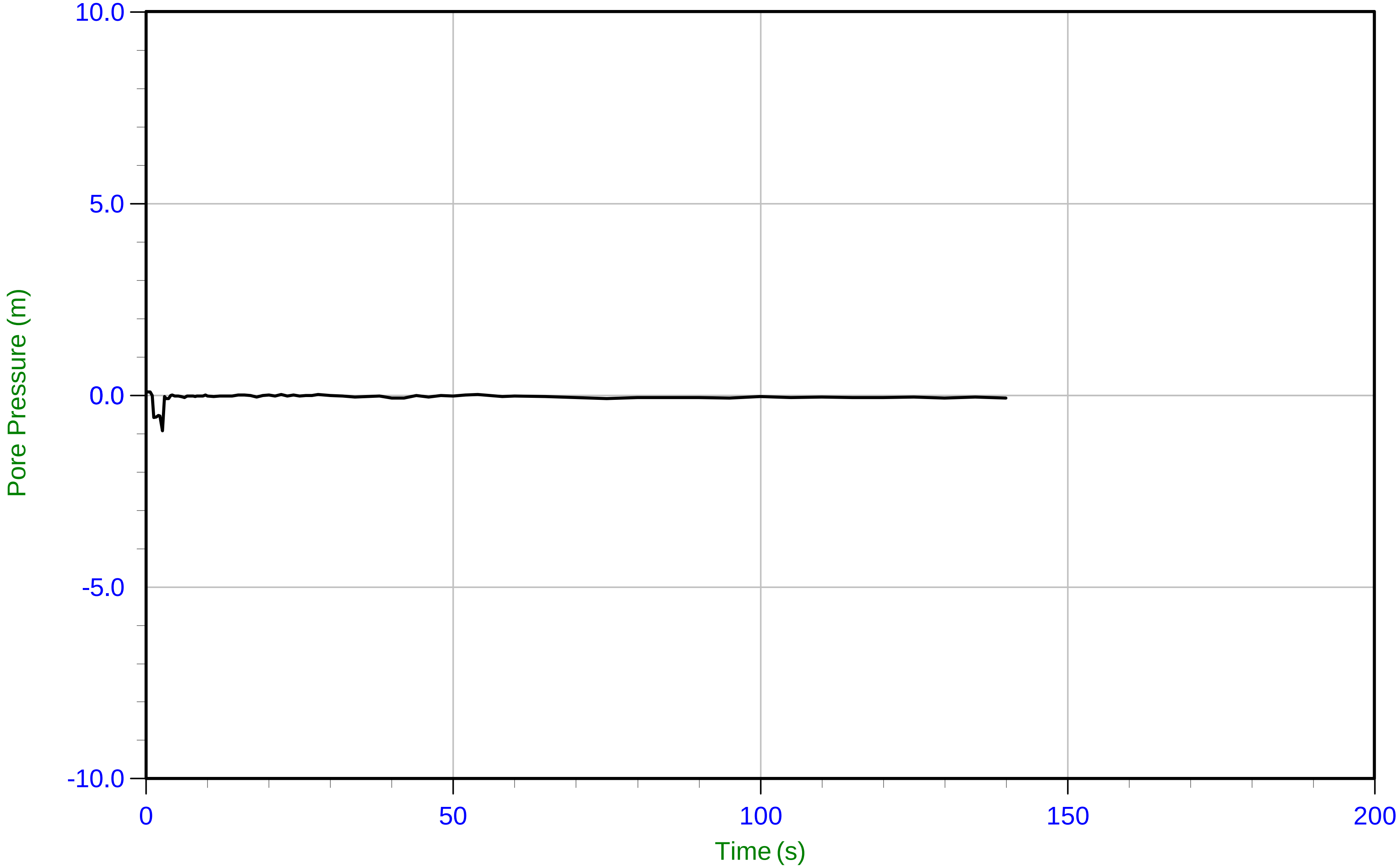
WT: 1.4 m / 4.7 ft
Ueq: 0.0 m

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 09:37
Site: Churchville Park

Sounding: CPT23-02
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP02.PPF2
Depth: 2.625 m / 8.612 ft
Duration: 140.0 s

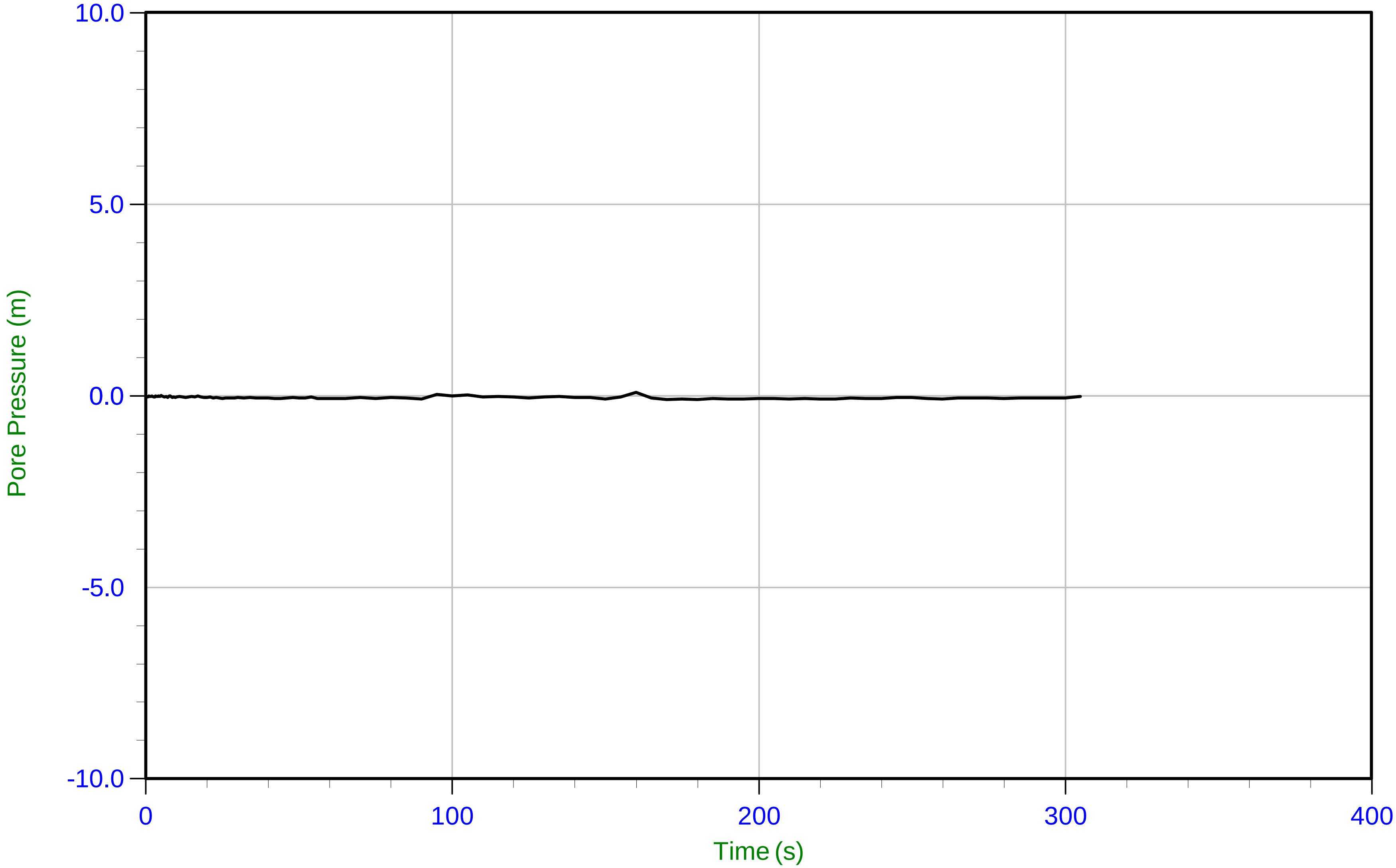
u Min: -0.9 m
u Max: 0.1 m
u Final: -0.1 m

WT: 2.6 m / 8.6 ft
Ueq: 0.0 m

KGS

Job No: 23-05-26771
Date: 2023-11-07 10:15
Site: Churchville Park

Sounding: CPT23-02B
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP02B.PPF2
Depth: 1.300 m / 4.265 ft
Duration: 305.0 s

u Min: -0.1 m
u Max: 0.1 m
u Final: -0.0 m

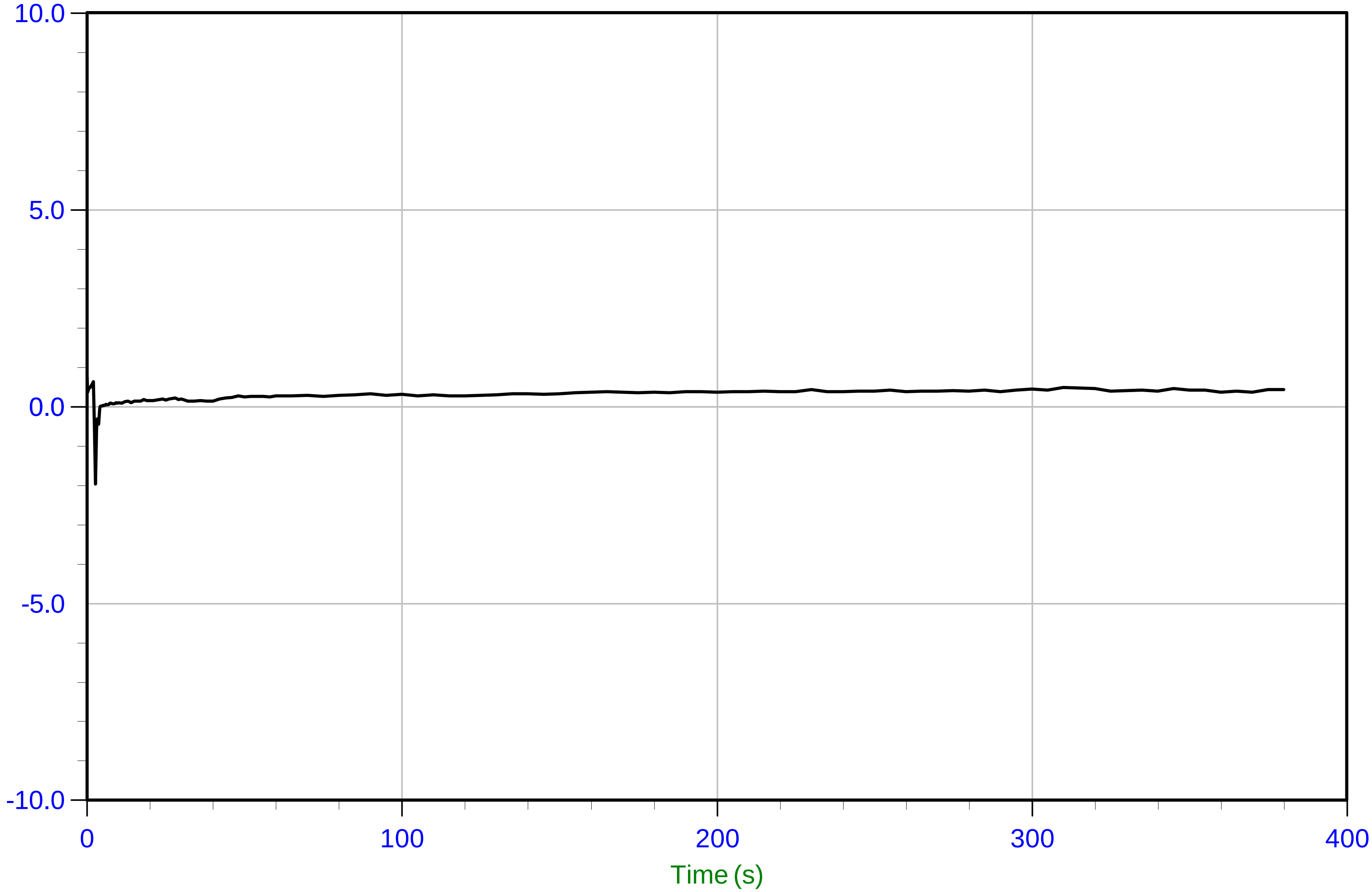
WT: 1.3 m / 4.3 ft
Ueq: 0.0 m

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 10:15
Site: Churchville Park

Sounding: CPT23-02B
Cone: 766:T1000F10U35 Area=10 cm²



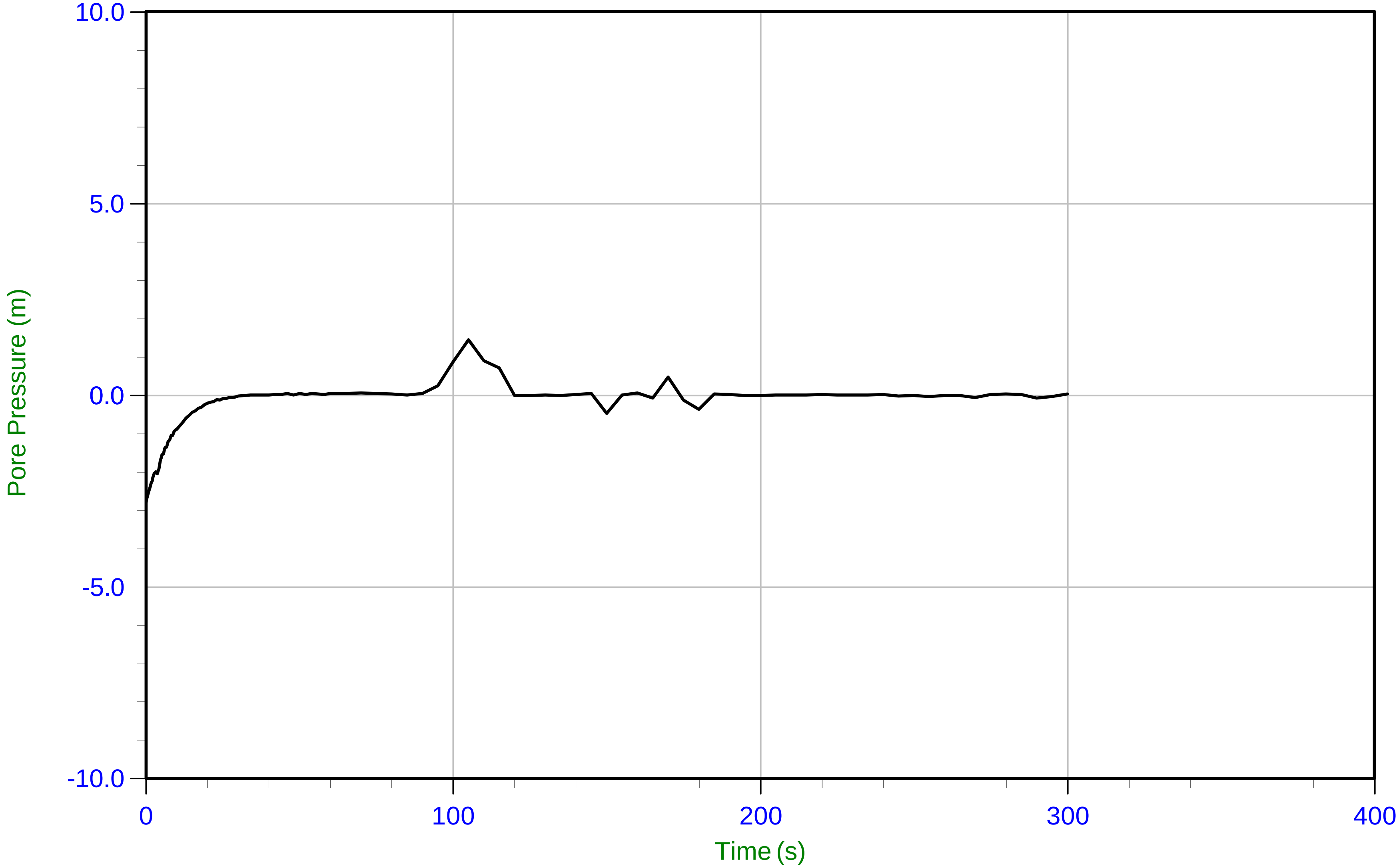
Trace Summary:

Filename: 23-05-26771_CP02B.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 380.0 s

u Min: -2.0 m
u Max: 0.8 m
u Final: 0.4 m

WT: 2.6 m / 8.7 ft
Ueq: 0.4 m

KGS	Job No: 23-05-26771 Date: 2023-11-07 11:17 Site: Churchville Park	Sounding: CPT23-03 Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP03.PPF2
Depth: 1.025 m / 3.363 ft
Duration: 300.0 s

u Min: -2.8 m
u Max: 1.4 m
u Final: 0.0 m

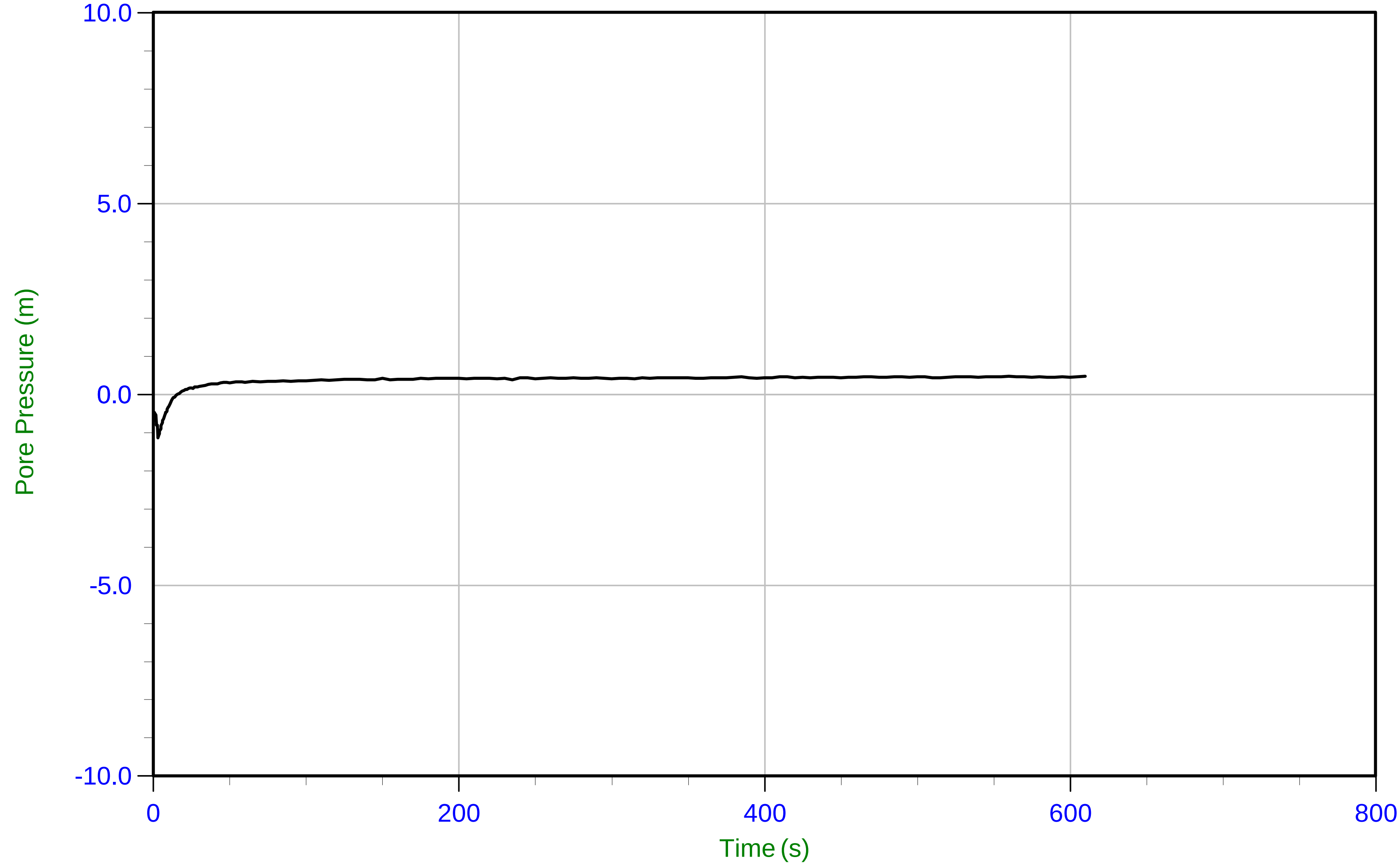
WT: 1.0 m / 3.4 ft
Ueq: 0.0 m

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP03.PPF2
Depth: 3.325 m / 10.909 ft
Duration: 610.0 s

u Min: -1.1 m
u Max: 0.5 m
u Final: 0.5 m

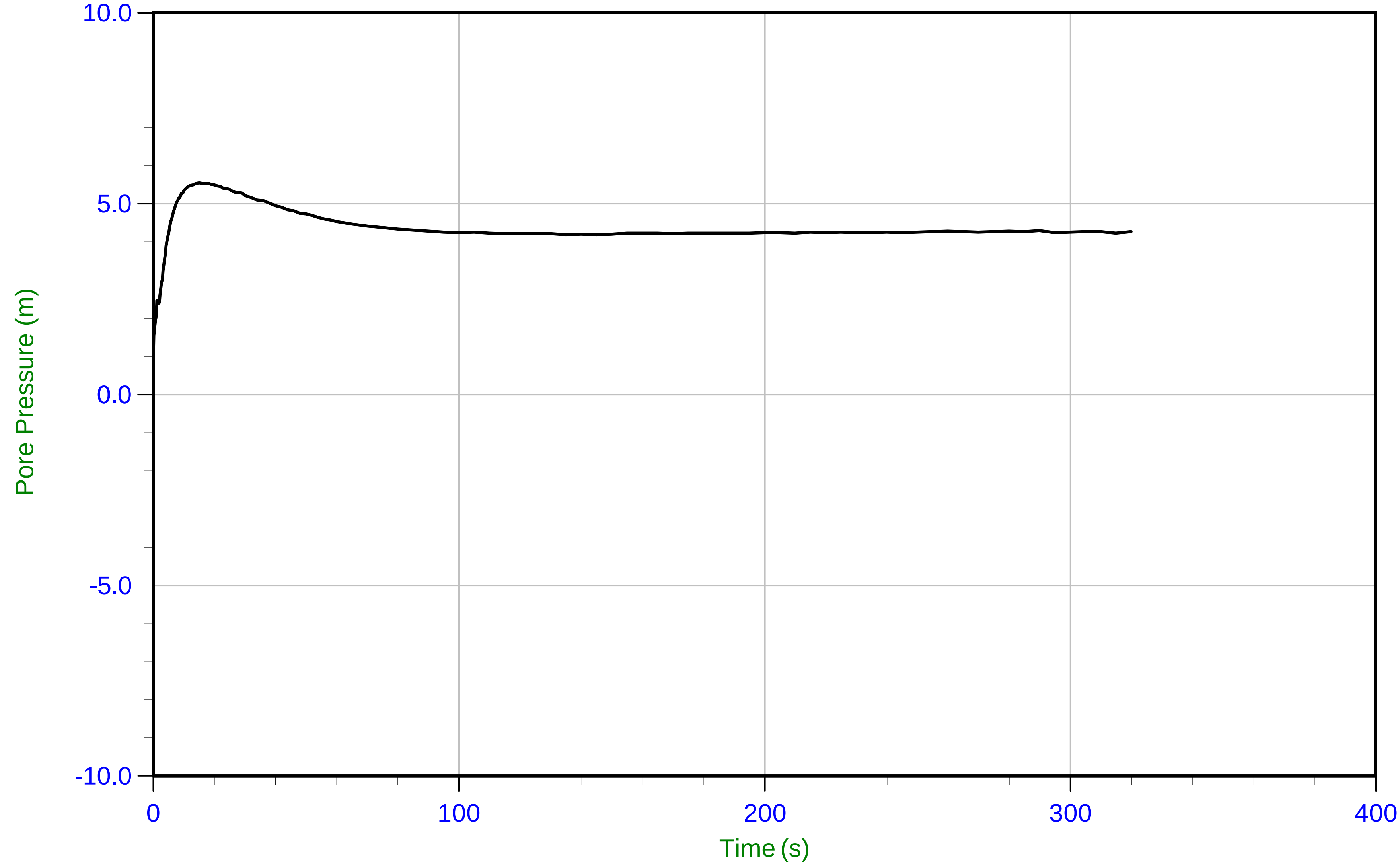
WT: 2.8 m / 9.3 ft
Ueq: 0.5 m

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP03.PPF2
Depth: 5.300 m / 17.388 ft
Duration: 320.0 s

u Min: 0.8 m
u Max: 5.5 m
u Final: 4.3 m

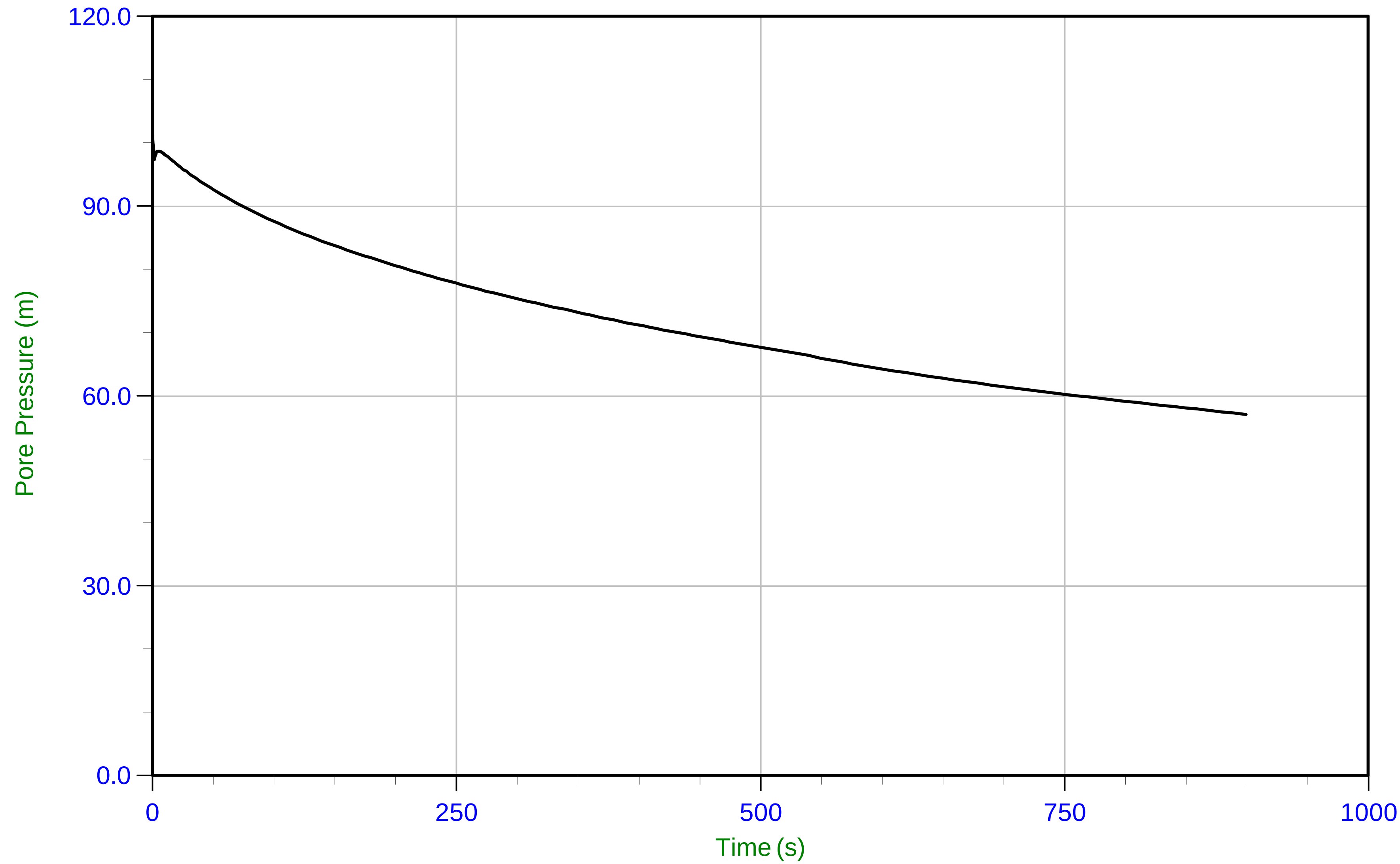
WT: 1.0 m / 3.3 ft
Ueq: 4.3 m

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 11:17
Site: Churchville Park

Sounding: CPT23-03
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP03.PPF2
Depth: 9.500 m / 31.168 ft
Duration: 900.0 s

u Min: 57.1 m
u Max: 106.5 m
u Final: 57.1 m

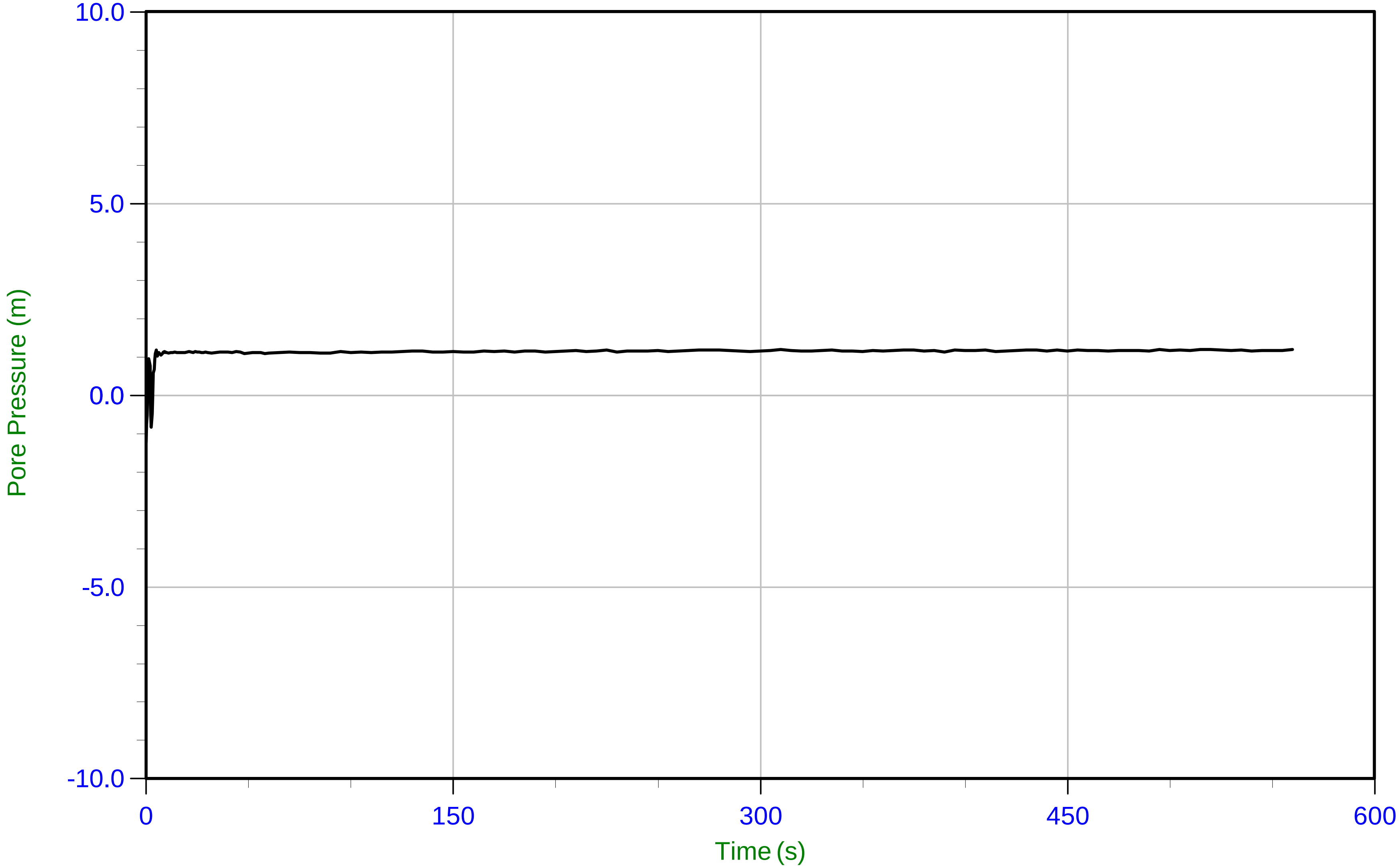
WT: 1.0 m / 3.3 ft
Ueq: 8.5 m
U(50): 57.50 m

T(50): 879.7 s
Ir: 100
Ch: 0.5 cm²/min

KGS

Job No: 23-05-26771
Date: 2023-11-07 13:15
Site: Churchville Park

Sounding: CPT23-04
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP04.PPF2
Depth: 2.150 m / 7.054 ft
Duration: 560.0 s

u Min: -1.2 m
u Max: 1.2 m
u Final: 1.2 m

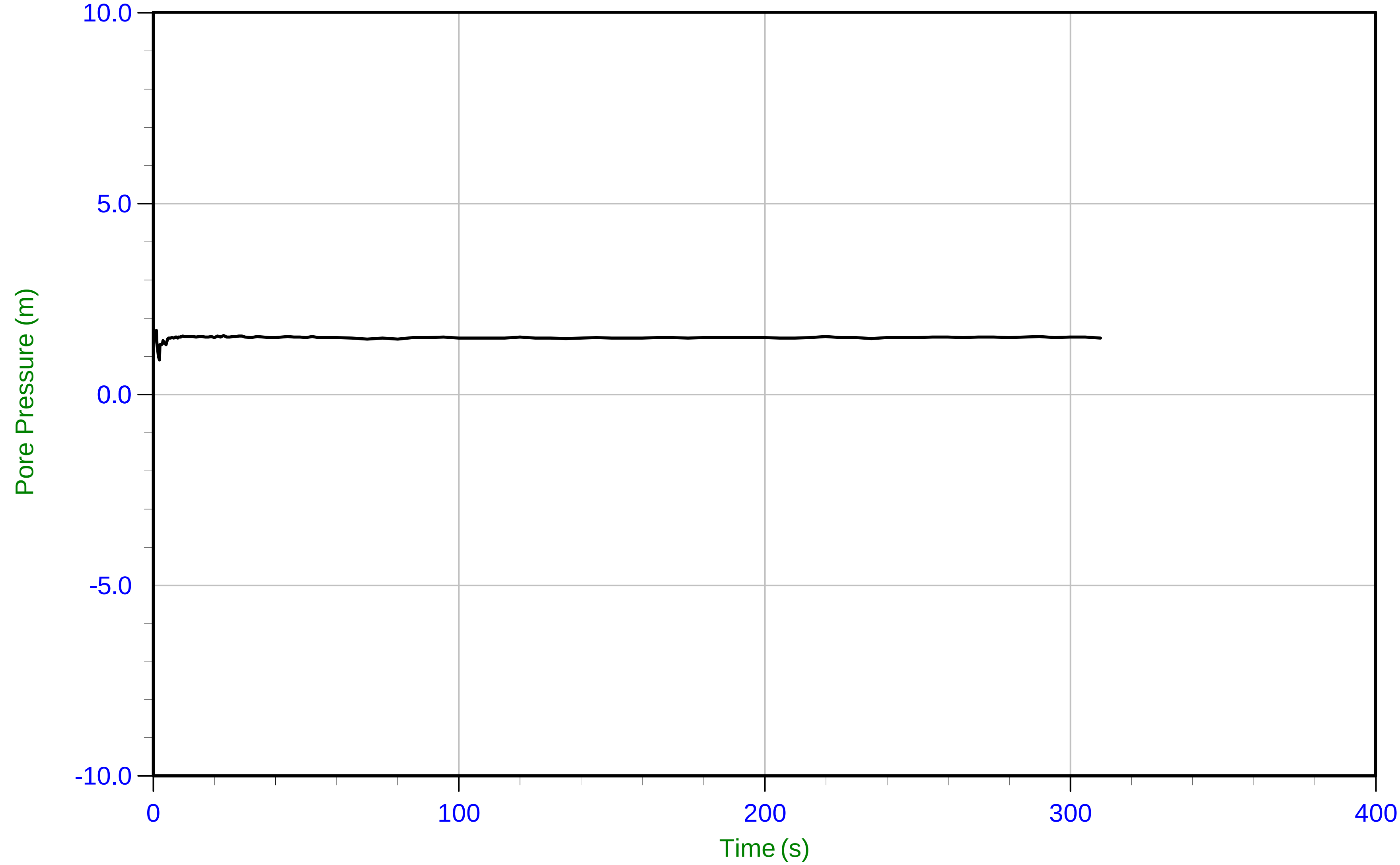
WT: 0.9 m / 3.1 ft
Ueq: 1.2 m

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 13:15
Site: Churchville Park

Sounding: CPT23-04
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP04.PPF2
Depth: 2.350 m / 7.710 ft
Duration: 310.0 s

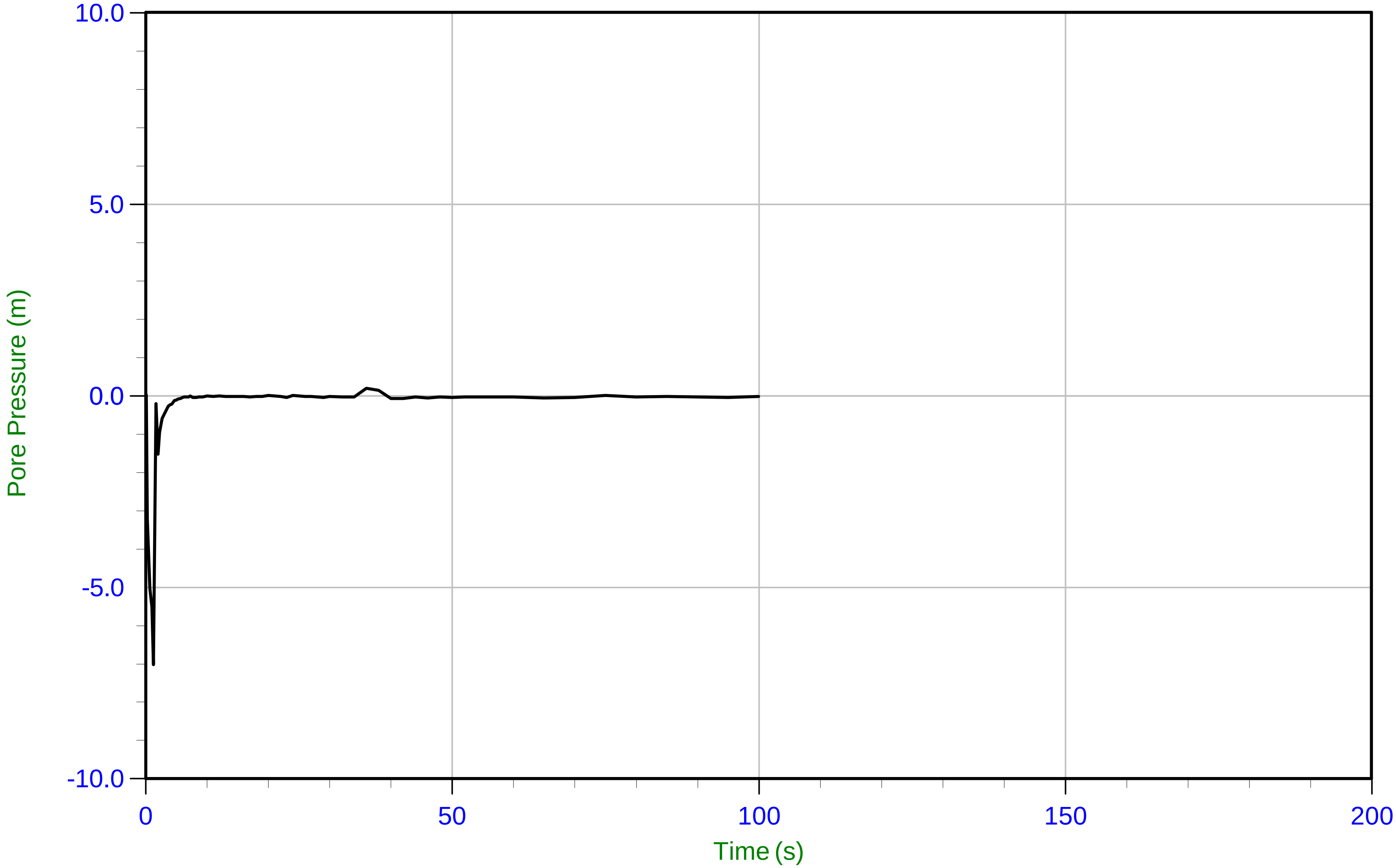
u Min: 0.8 m
u Max: 1.7 m
u Final: 1.5 m

WT: 0.8 m / 2.8 ft
Ueq: 1.5 m

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP05.PPF2
Depth: 0.325 m / 1.066 ft
Duration: 100.0 s

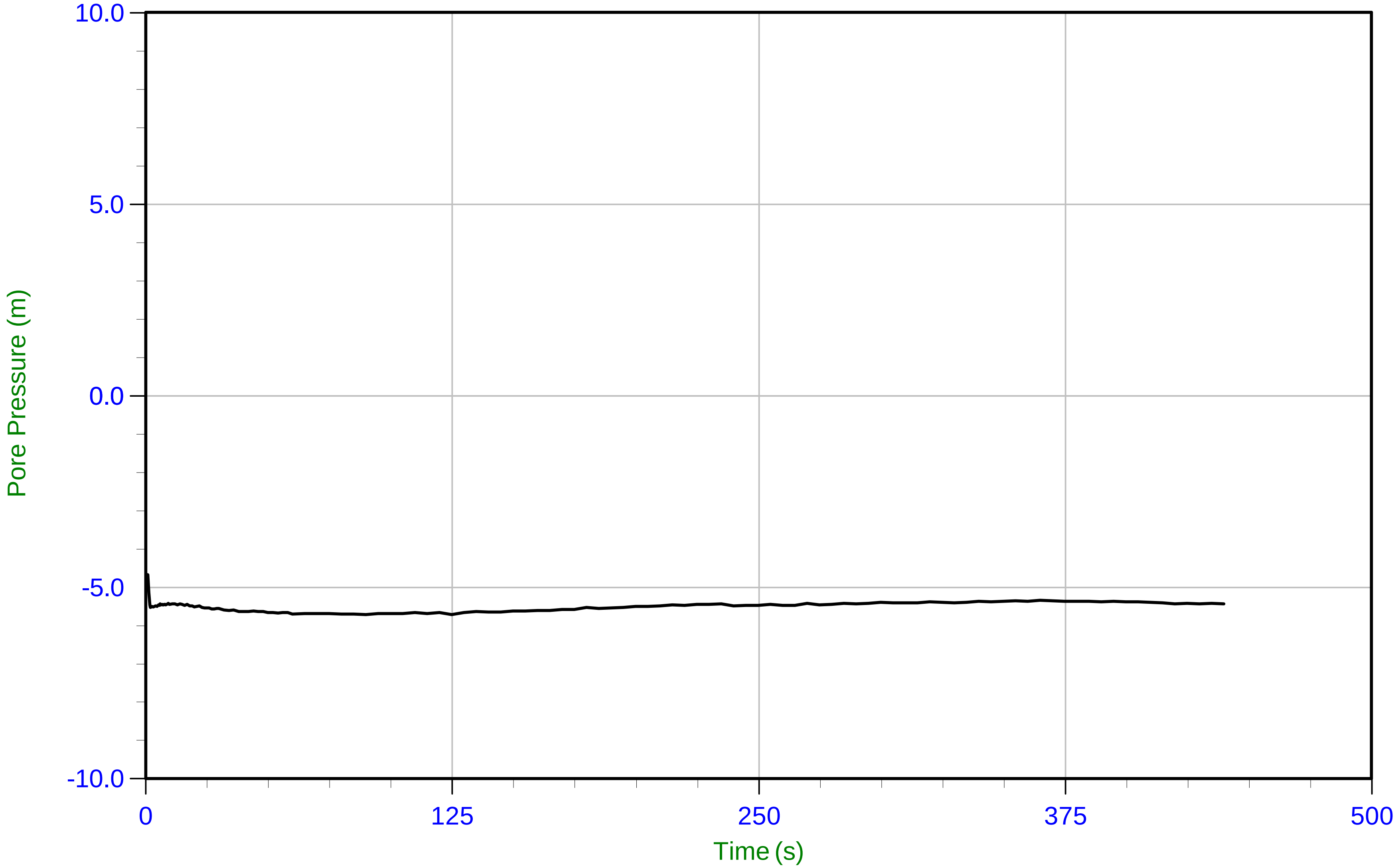
u Min: -7.0 m
u Max: 0.2 m
u Final: -0.0 m

WT: 0.3 m / 1.1 ft
Ueq: 0.0 m

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

Filename: 23-05-26771_CP05.PPF2
Depth: 2.275 m / 7.464 ft
Duration: 440.0 s

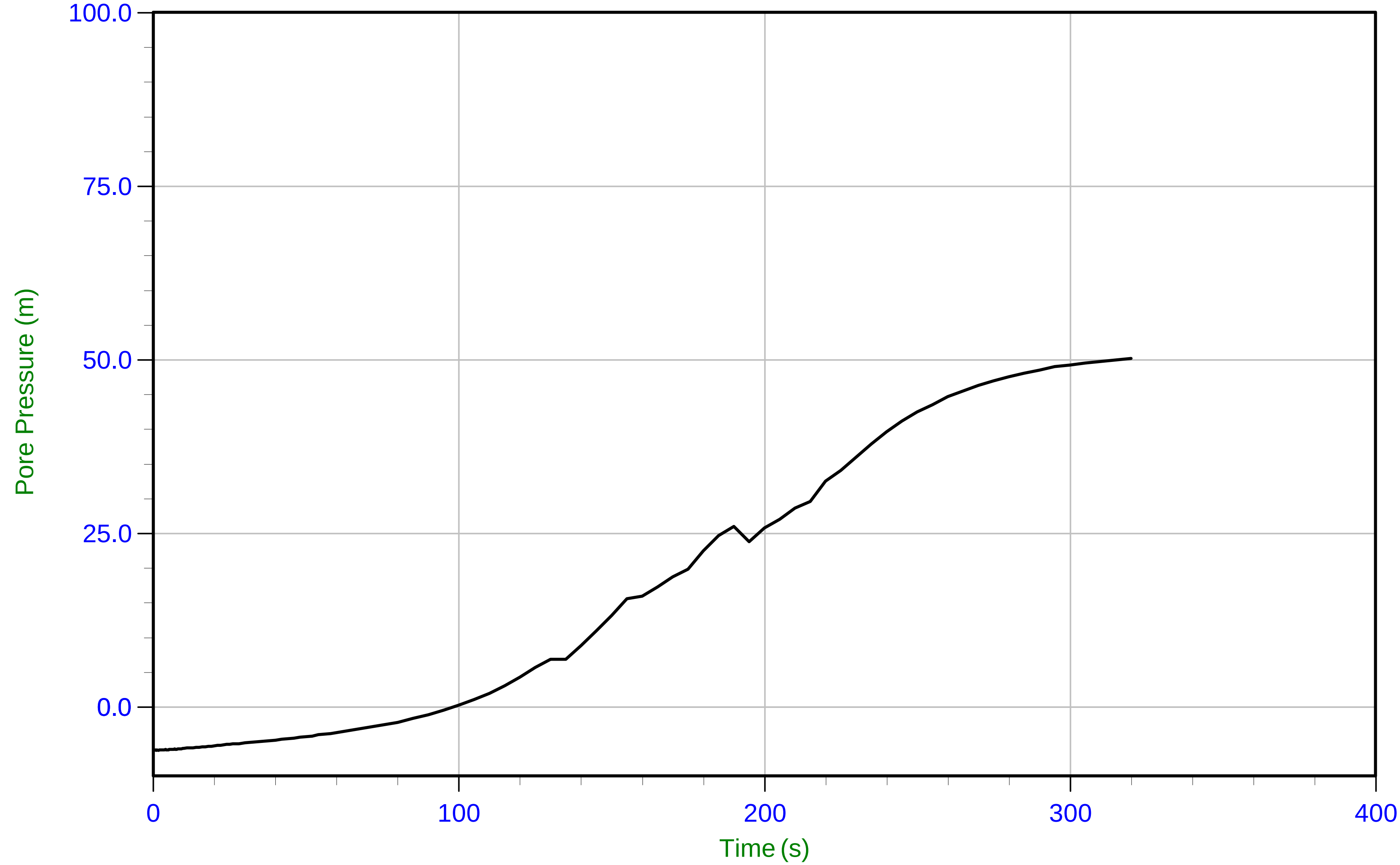
u Min: -5.7 m
u Max: -4.7 m
u Final: -5.4 m

ConeTec

KGS

Job No: 23-05-26771
Date: 2023-11-07 14:29
Site: Churchville Park

Sounding: CPT23-05
Cone: 766:T1000F10U35 Area=10 cm²

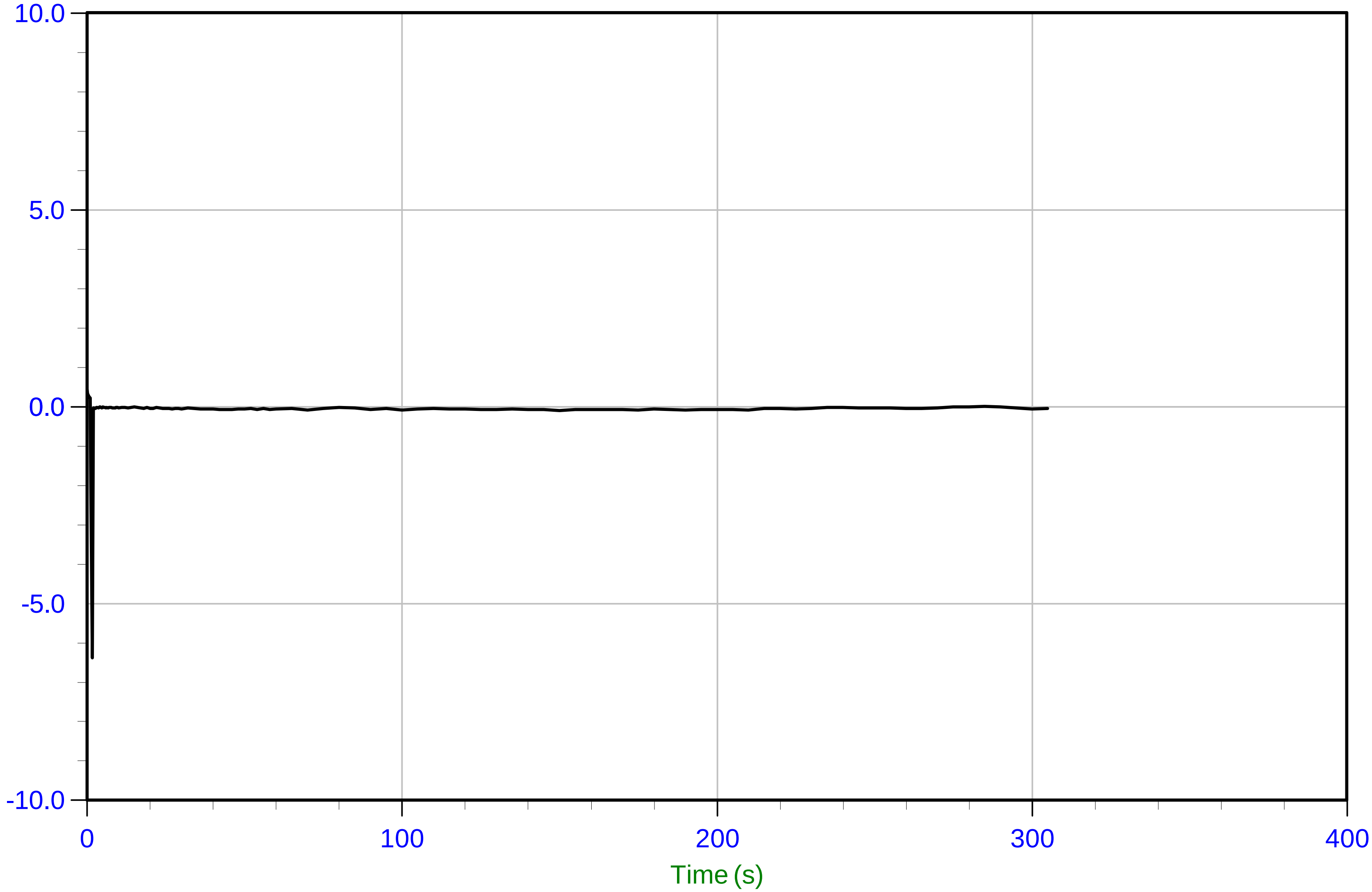


Trace Summary:

Filename: 23-05-26771_CP05.PPF2
Depth: 5.150 m / 16.896 ft
Duration: 320.0 s

u Min: -6.3 m
u Max: 50.2 m
u Final: 50.2 m

KGS	Job No: 23-05-26771 Date: 2023-11-07 15:19 Site: Churchville Park	Sounding: CPT23-06 Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

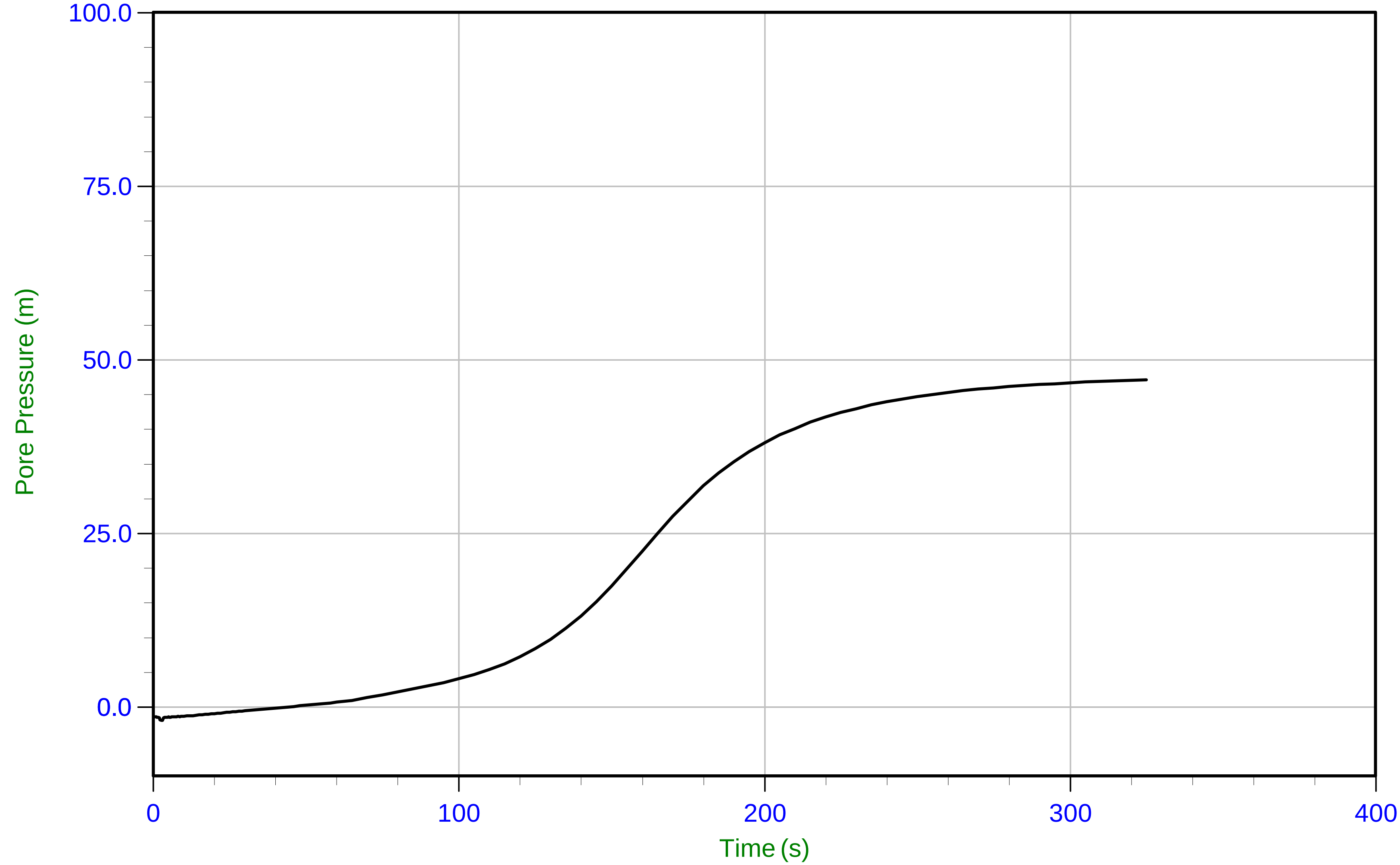
Filename: 23-05-26771_CP06.PPF2	u Min: -6.4 m	WT: 2.5 m / 8.0 ft
Depth: 2.450 m / 8.038 ft	u Max: 0.4 m	Ueq: 0.0 m
Duration: 305.0 s	u Final: -0.1 m	

CONETEC

KGS

Job No: 23-05-26771
Date: 2023-11-07 15:19
Site: Churchville Park

Sounding: CPT23-06
Cone: 766:T1000F10U35 Area=10 cm²



Trace Summary:

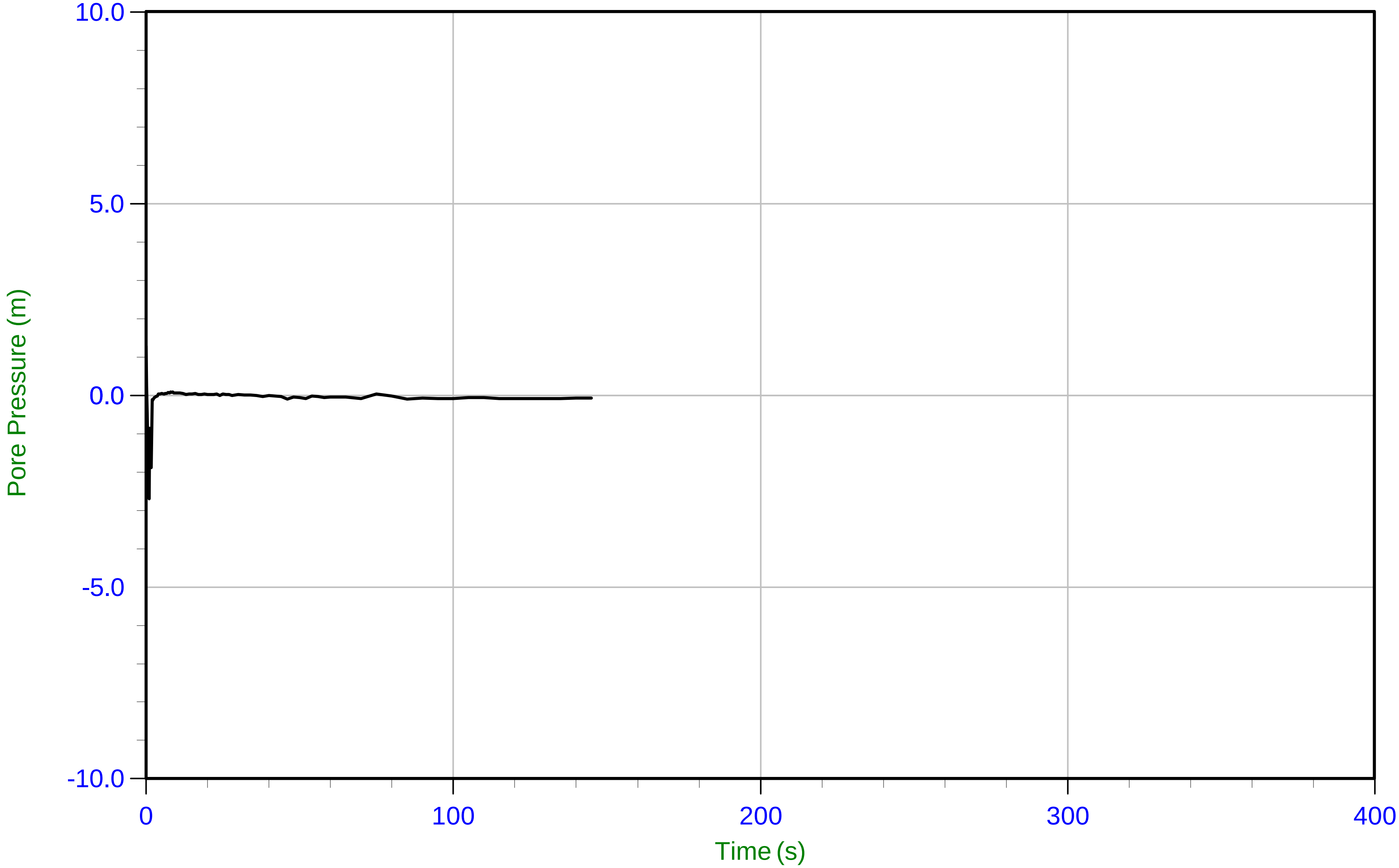
Filename: 23-05-26771_CP06.PPF2
Depth: 4.300 m / 14.107 ft
Duration: 325.0 s

u Min: -2.0 m
u Max: 47.1 m
u Final: 47.1 m

Job No: 23-05-26771
Date: 2023-11-07 16:33
Site: Churchville Park

Sounding: CPT23-07
Cone: 766:T1000F10U35 Area=10 cm²

KGS



Trace Summary:

Filename: 23-05-26771_CP07.PPF2
Depth: 1.575 m / 5.167 ft
Duration: 145.0 s

u Min: -2.7 m
u Max: 1.3 m
u Final: -0.1 m

WT: 1.6 m / 5.2 ft
Ueq: 0.0 m

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 18

Revised February 10, 2023

Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g., 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not performed.

Corrected tip resistance: $q_t = q_c + (1\text{-}a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)

a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.

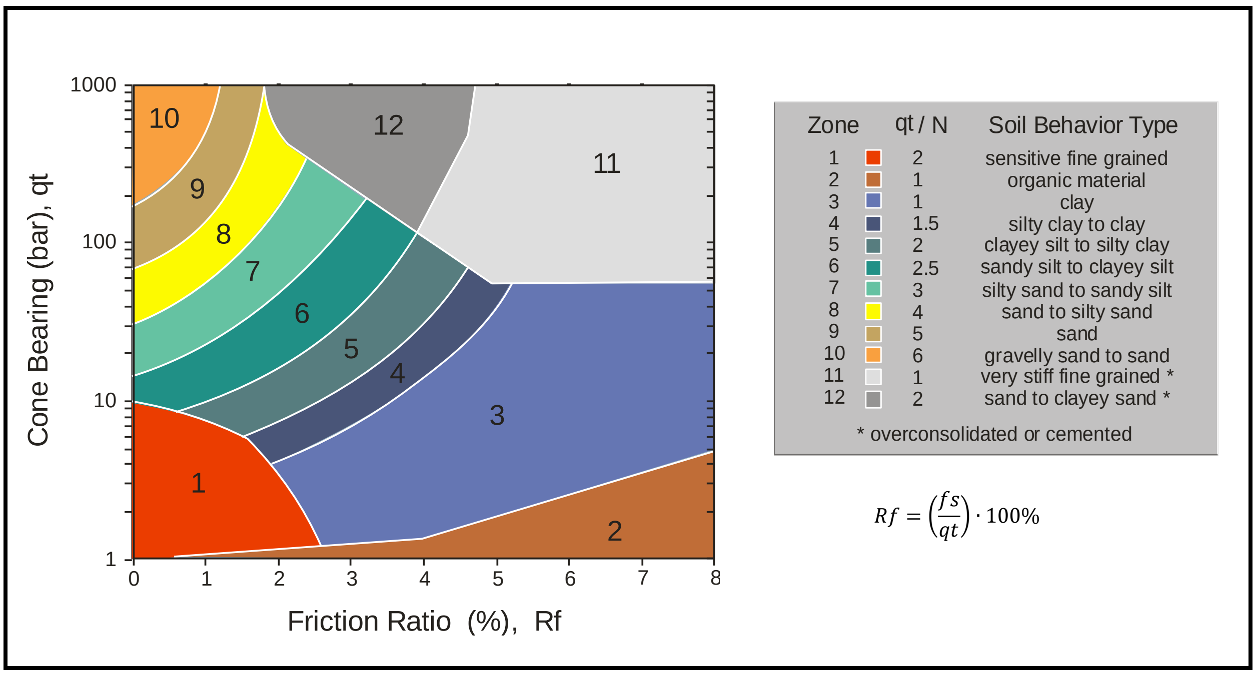


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

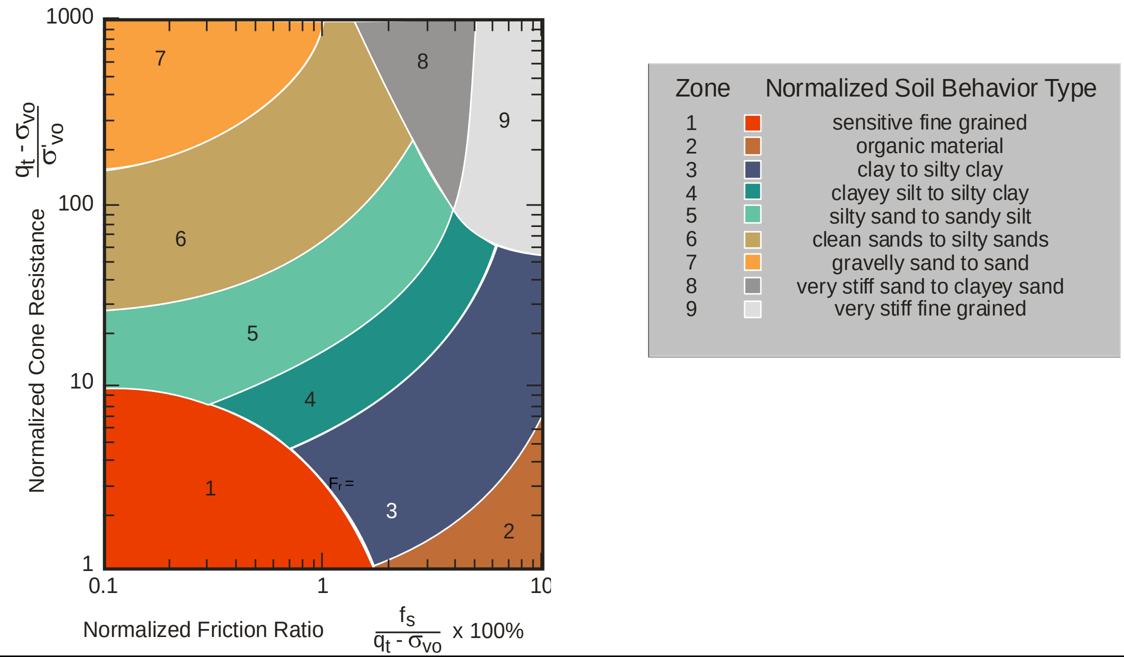


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

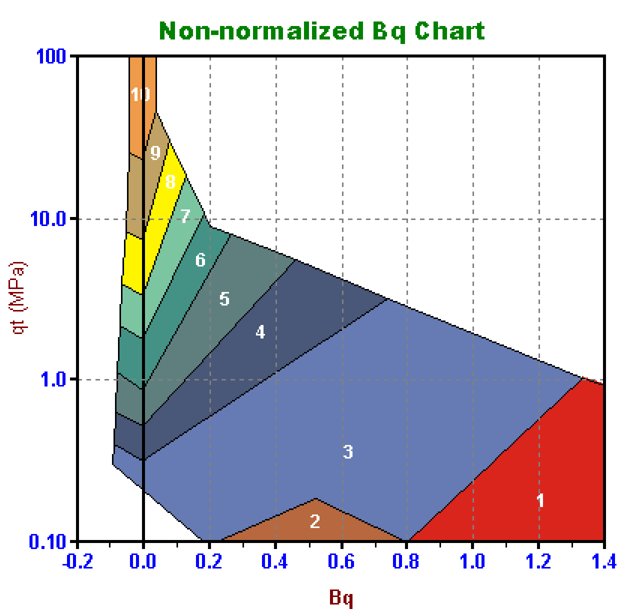


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

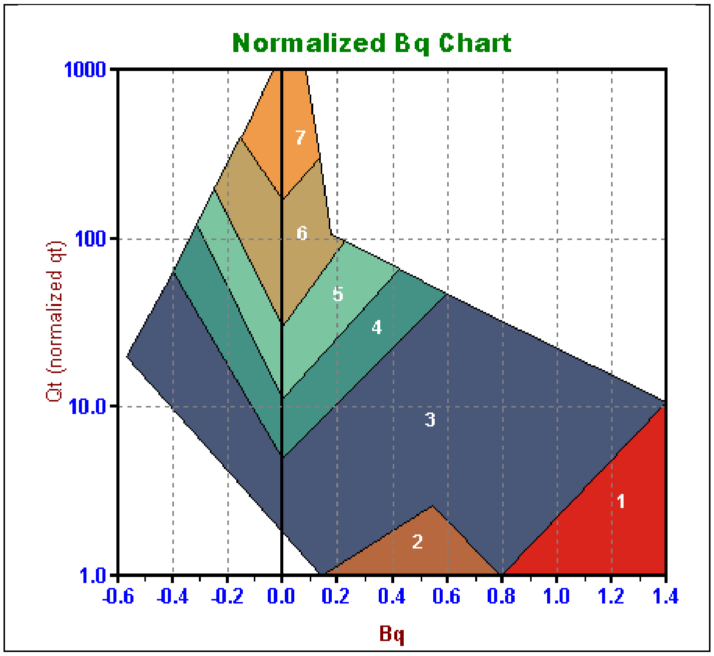


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

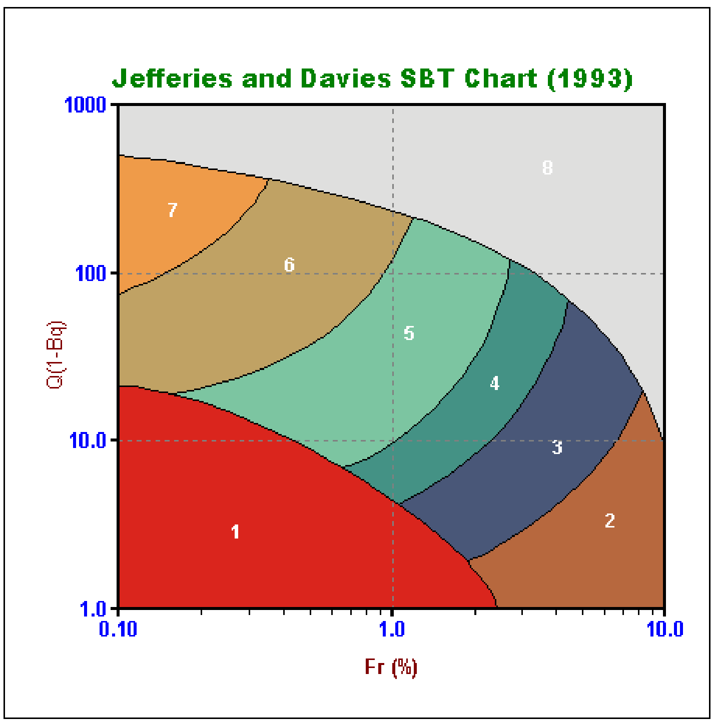


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

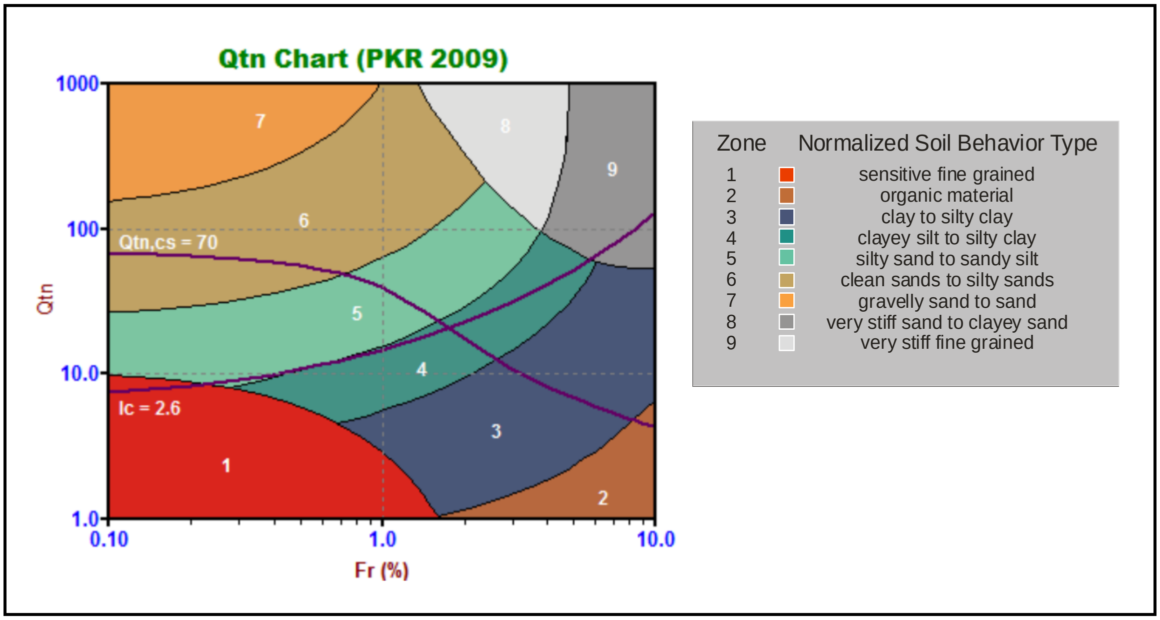


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

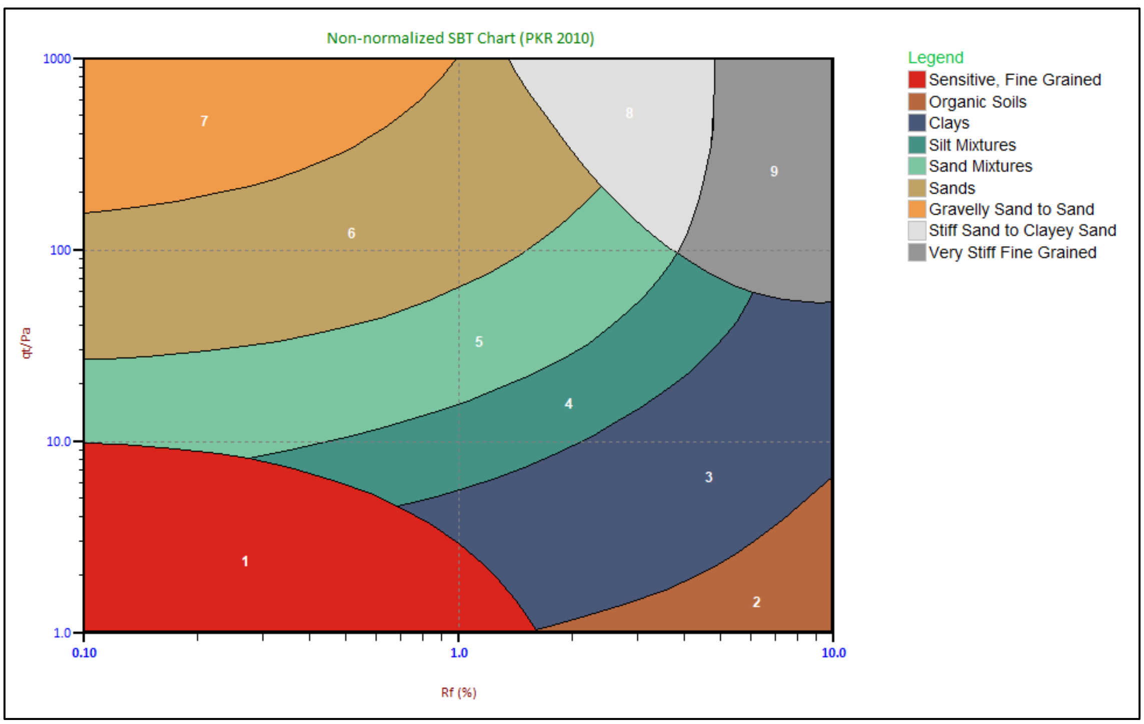


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

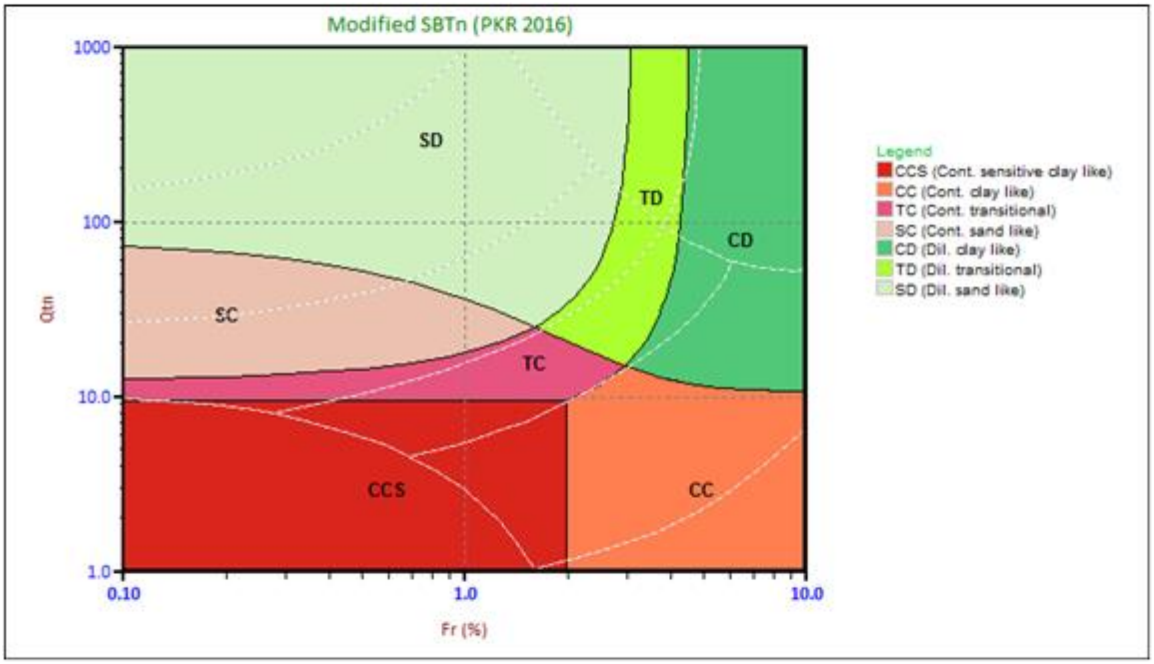


Figure 6. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary, we recommend that the user refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Tables 1and 1a may be included in the output files delivered with this report.

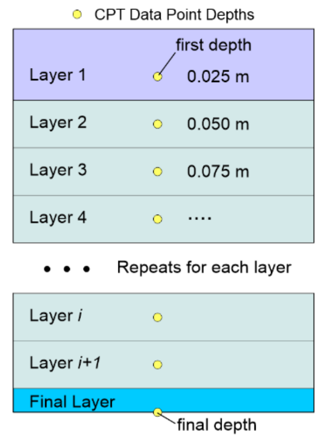
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Reference Notes: CK* - Common Knowledge, U* - Unpublished

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a)\cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s.	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

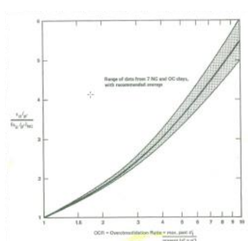
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness 	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point (“assumed value”) will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These “assumed” values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0.	$K_o = (1 - \sin\Phi') \, OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a/\sigma_v')^{0.5}$ where *0.0 < C_n < 2.0 (user adjustable, typically ranging from 1.7 to 2.0)* *P_a is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5\ (1 – I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 – 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using $N_{60}\ I_c$). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\,I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5\ (1 – I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 – 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	1, 2, 5
Net q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	36
Q_t or Norm: Qt or Q_{t1}	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5, 15
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
$Q(1-B_q)$ $Q(1-B_q) + 1$	$Q(1-B_q)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above*	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q_{c1} (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}(0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}(Cq) = C_q * q_t$ (some papers use q_c)	5, 12
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining* I_c *And* *Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	n (PKR 2009) = $0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Q_{tn} (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Q_{tn} (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs}$ = 70 line in Figure 4.	$CD = 70 = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5)(1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100 (Q_{tn} + 10) / (70 + Q_{tn} F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 2) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ 3) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60 \cdot (q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25 \cdot (Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma_v'}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma_m' = \frac{1}{3}(\sigma_v' + \sigma_h' + \sigma_h')$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress Δu / σ_v	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma_v'}$ *where:* $\Delta u = u - u_{eq}$	25, 25a
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
$qtNet/G_{max}$	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for "crane walk" accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1,68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young's Modulus, E, at a 25% working load	$E_{25} = \alpha_E\ (qtNet)$ where $\alpha_E = 0.015[10^{(0.55Ic + 1,68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D' Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D'	1) Robertson $M = \alpha_M\ (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\ [10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D\ (qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}I_c$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/(N_1)_{60cs}I_c = 8.5\,(1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta=1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\frac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\, [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\, [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\, [q_{c1ncs}/1000]^3 + 0.08$	10
Kg or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
K_g*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on $\Psi = -0.05$ curve used in SP Distance calculation		25

Table 2. References

No.	Reference
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No.	Reference
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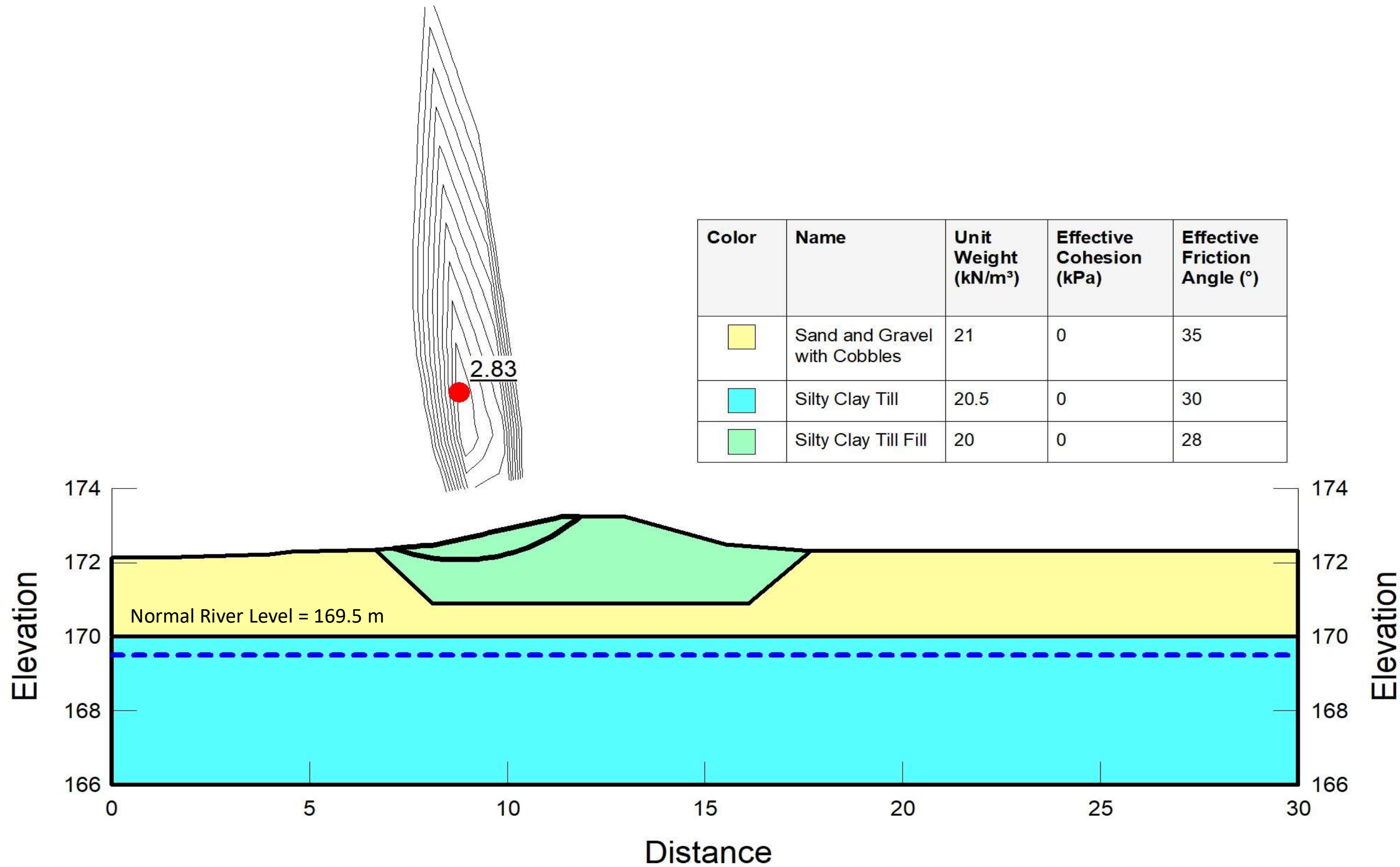
APPENDIX D

Slope Stability Results



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 2.83
2) Normal River Level = El. 169.5 m.

KGS GROUP

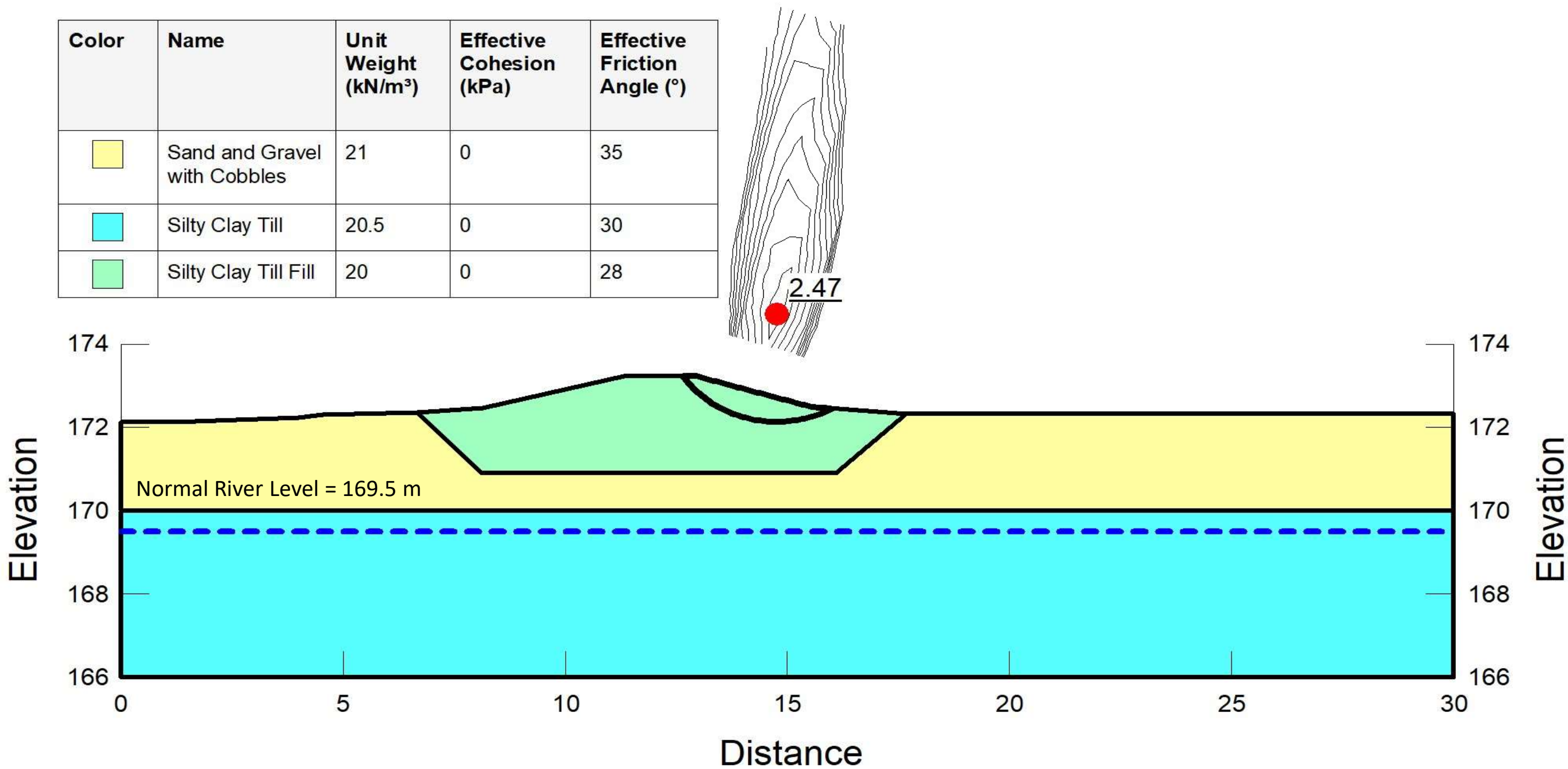
CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M UPSTREAM SIDE

January 2024	Figure D1-1	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 2.47
2) Normal River Level = El. 169.5 m.

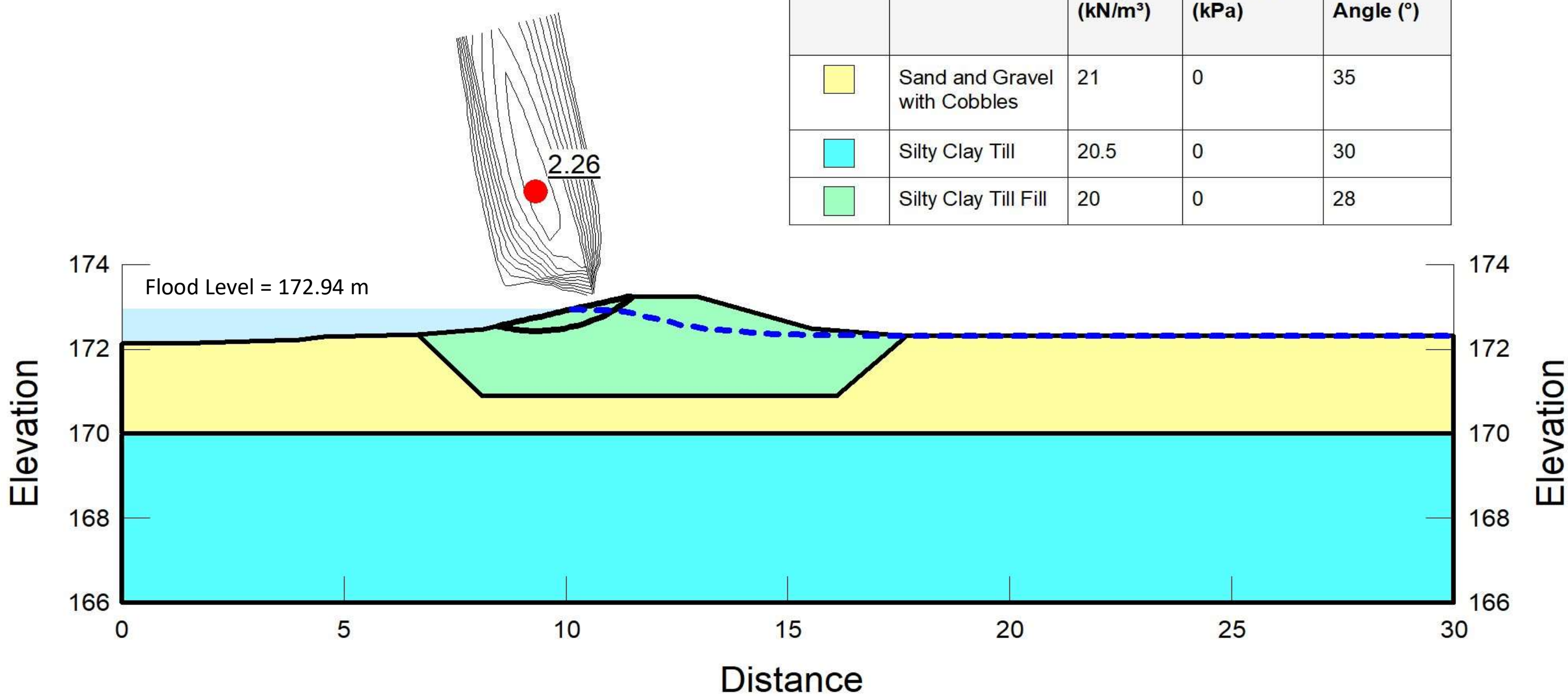
KGS GROUP

CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M DOWNSTREAM SIDE

January 2024	Figure D1-2	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 2.26
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

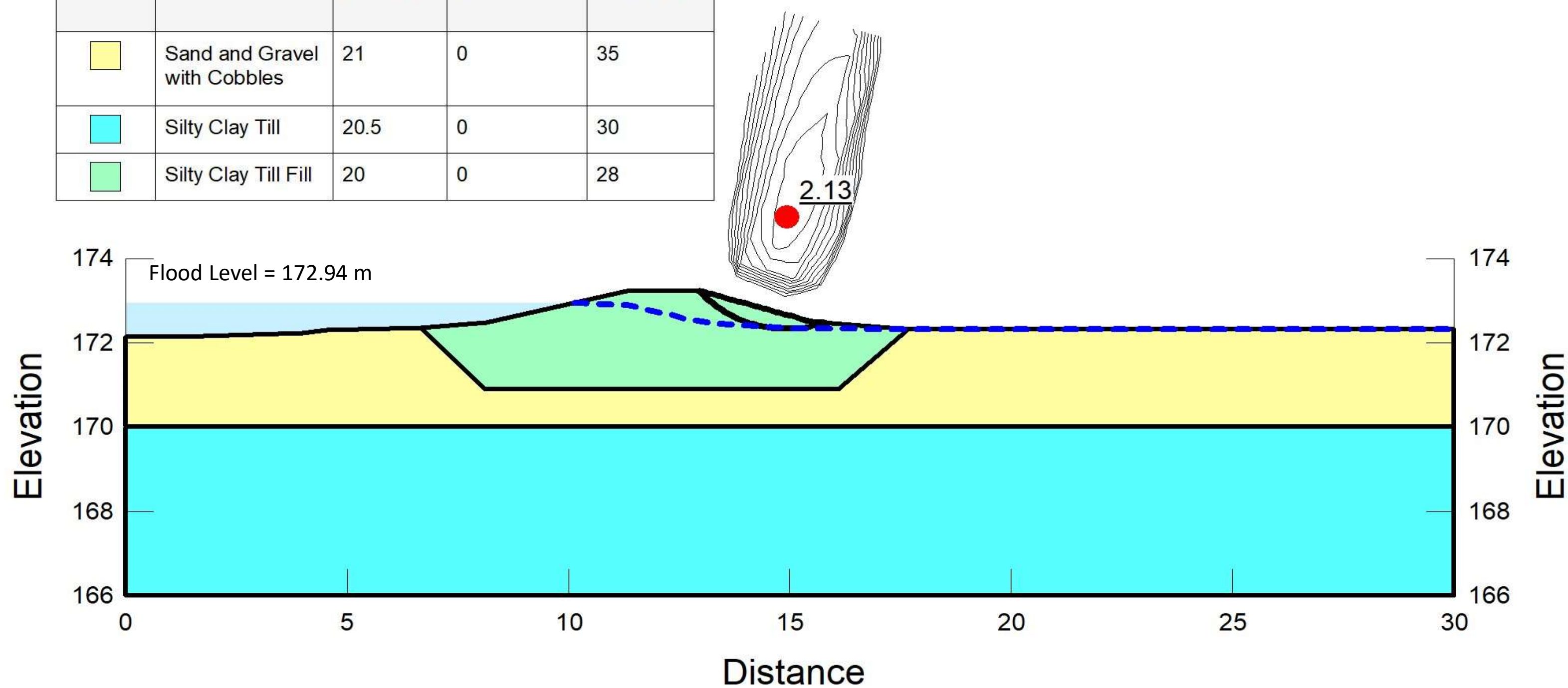
CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE) UPSTREAM SIDE

January 2024	Figure D1-3	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 2.13
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

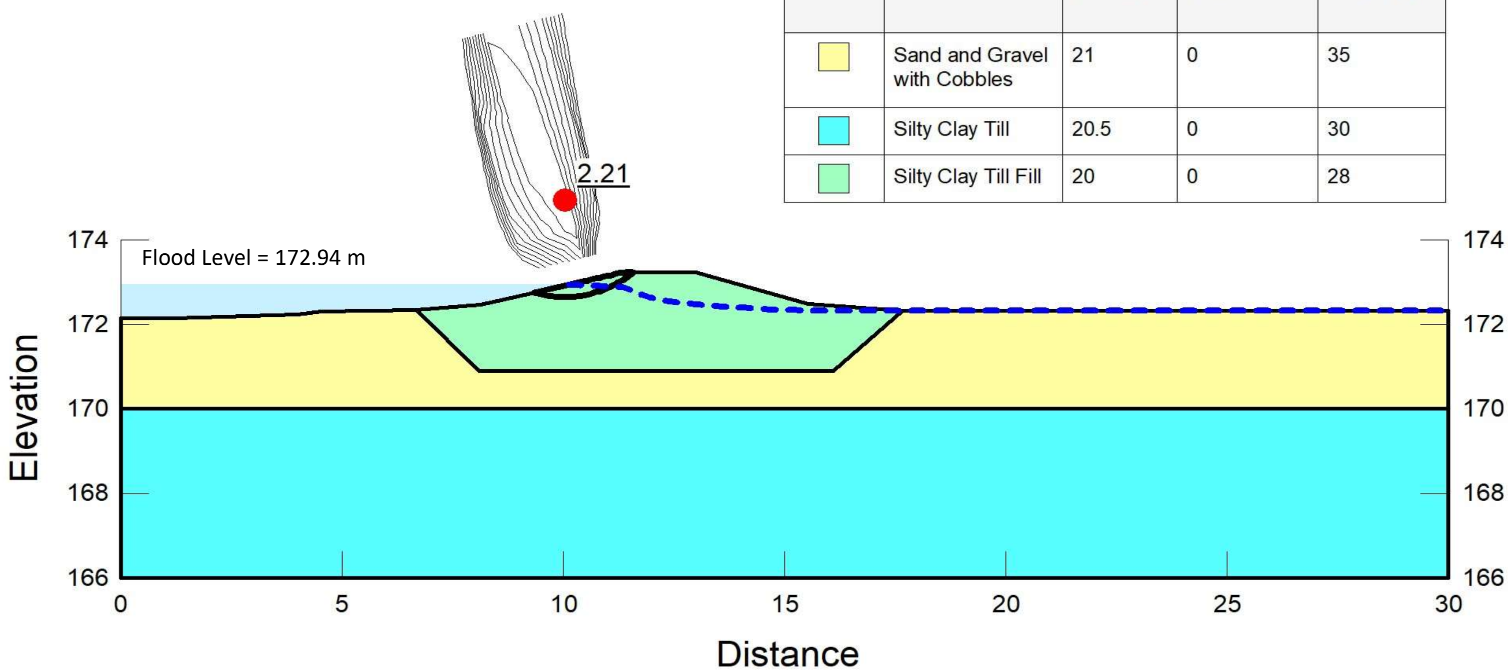
CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE)

DOWNSTREAM SIDE

January 2024	Figure D1-4	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 2.21
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

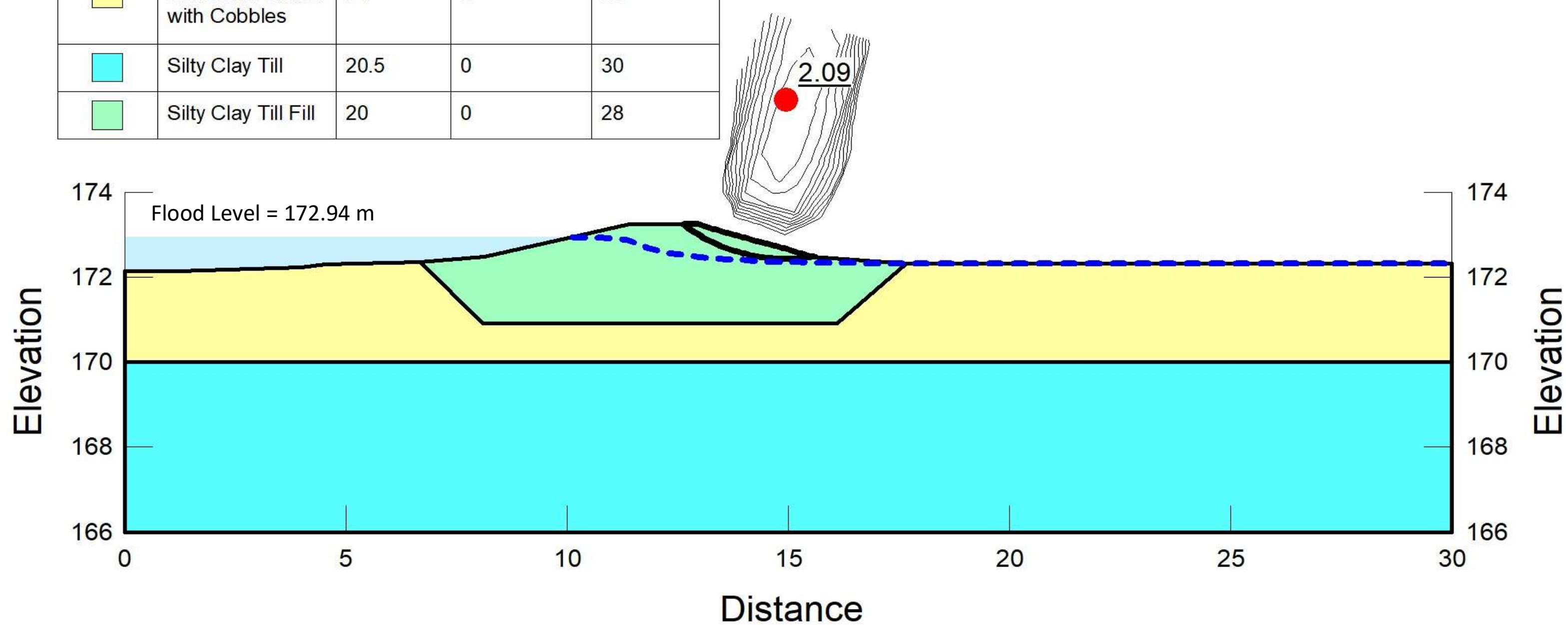
CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)
UPSTREAM SIDE

January 2024	Figure D1-5	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 2.09
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

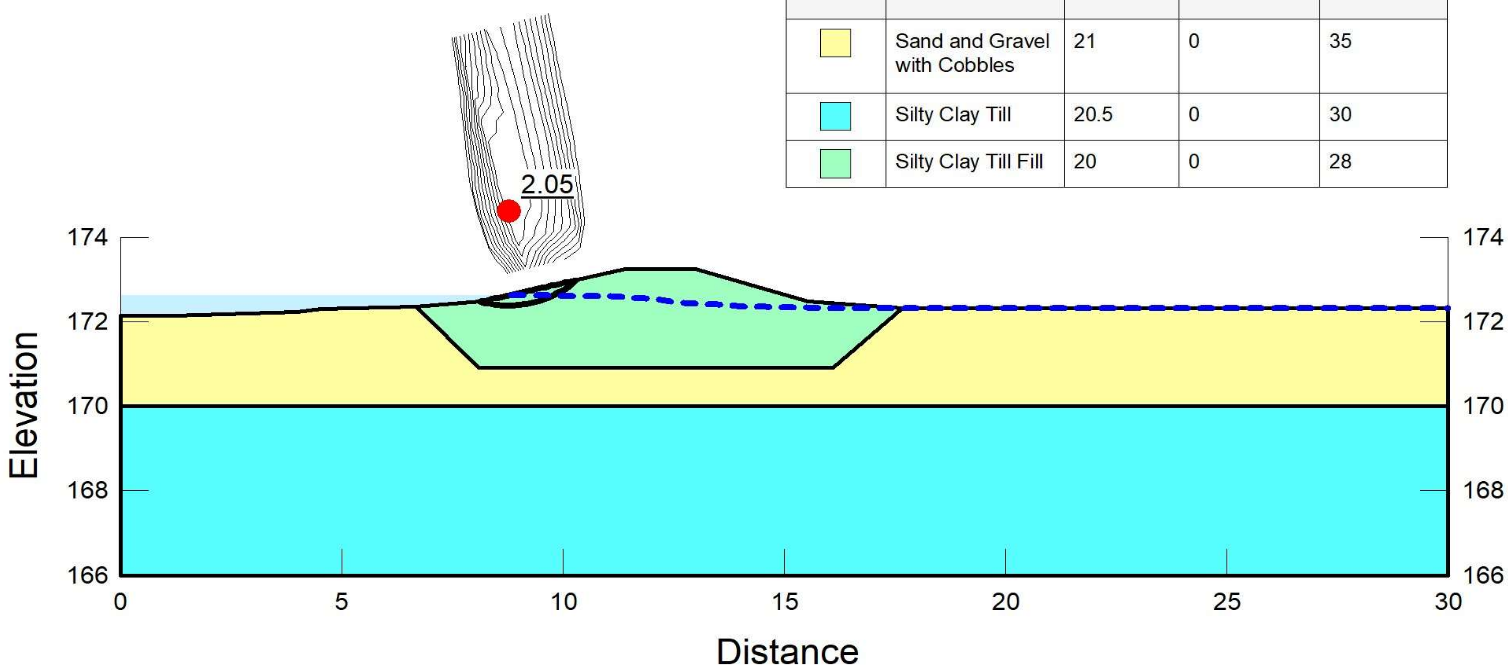
CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)

DOWNSTREAM SIDE

January 2024	Figure D1-6	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 2.05 (T = 120 Hrs)
2) Flood Level = El. 172.94 m.
3) Normal River Level = El. 169.5 m.

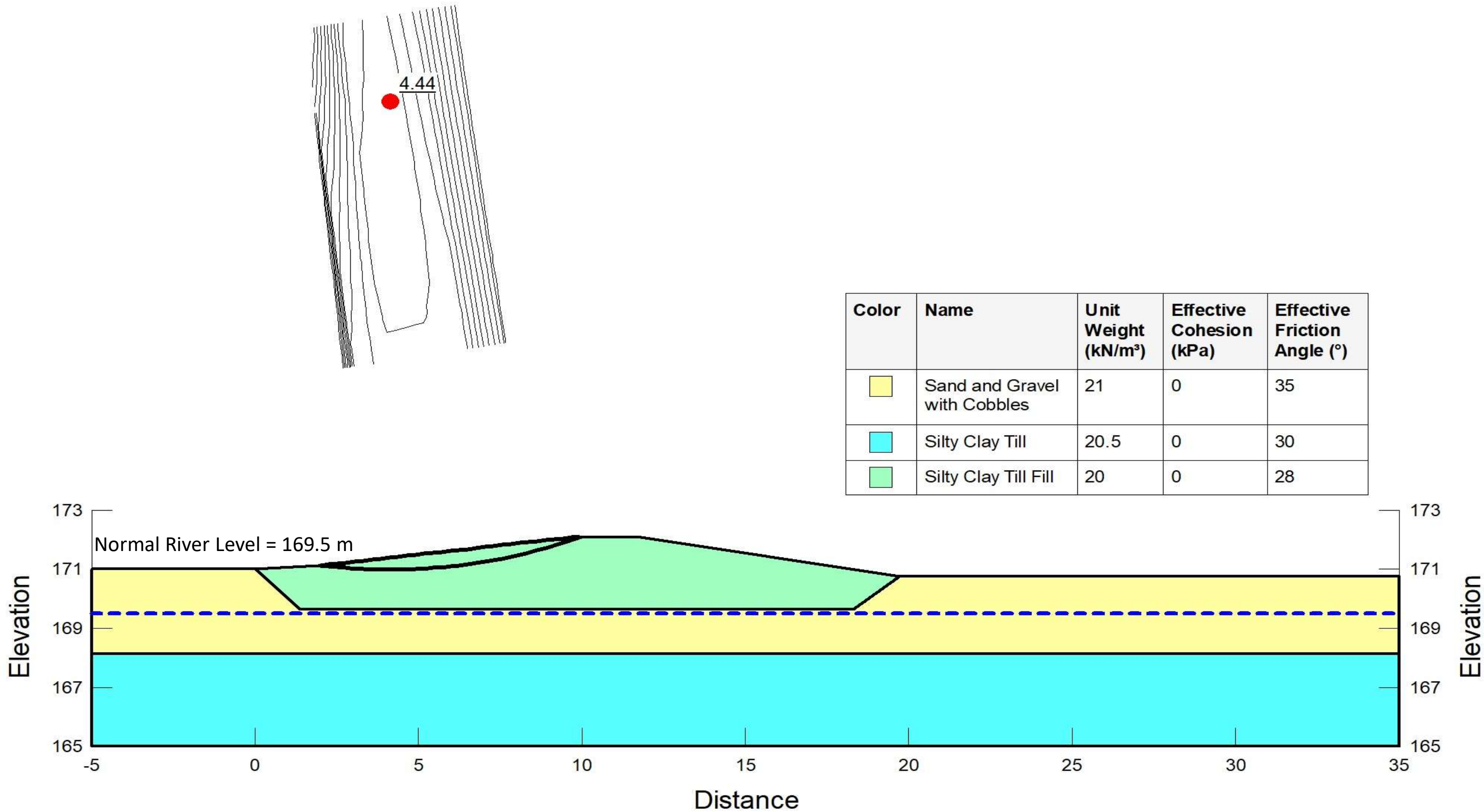
KGS GROUP

CHURCHVILLE DYKE (SECTION A-A', CH. 0+038)

SLOPE STABILITY ANALYSIS

CASE IV: FLOOD EVENT DRAWDOWN = 172.94 M to 169.5 M (TRANSIENT ANALYSIS) UPSTREAM SIDE

January 2024	Figure D1-7	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 4.44
2) Normal River Level = El. 169.5 m.

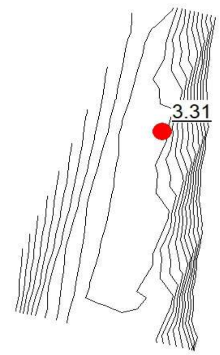
KGS GROUP

CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

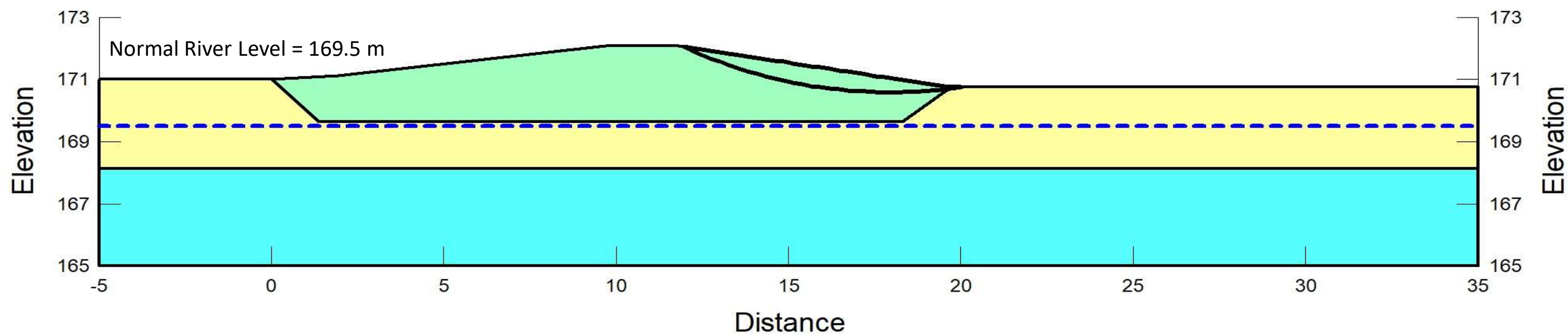
SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M UPSTREAM SIDE

January 2024	Figure D2-1	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 3.31
2) Normal River Level = El. 169.5 m.

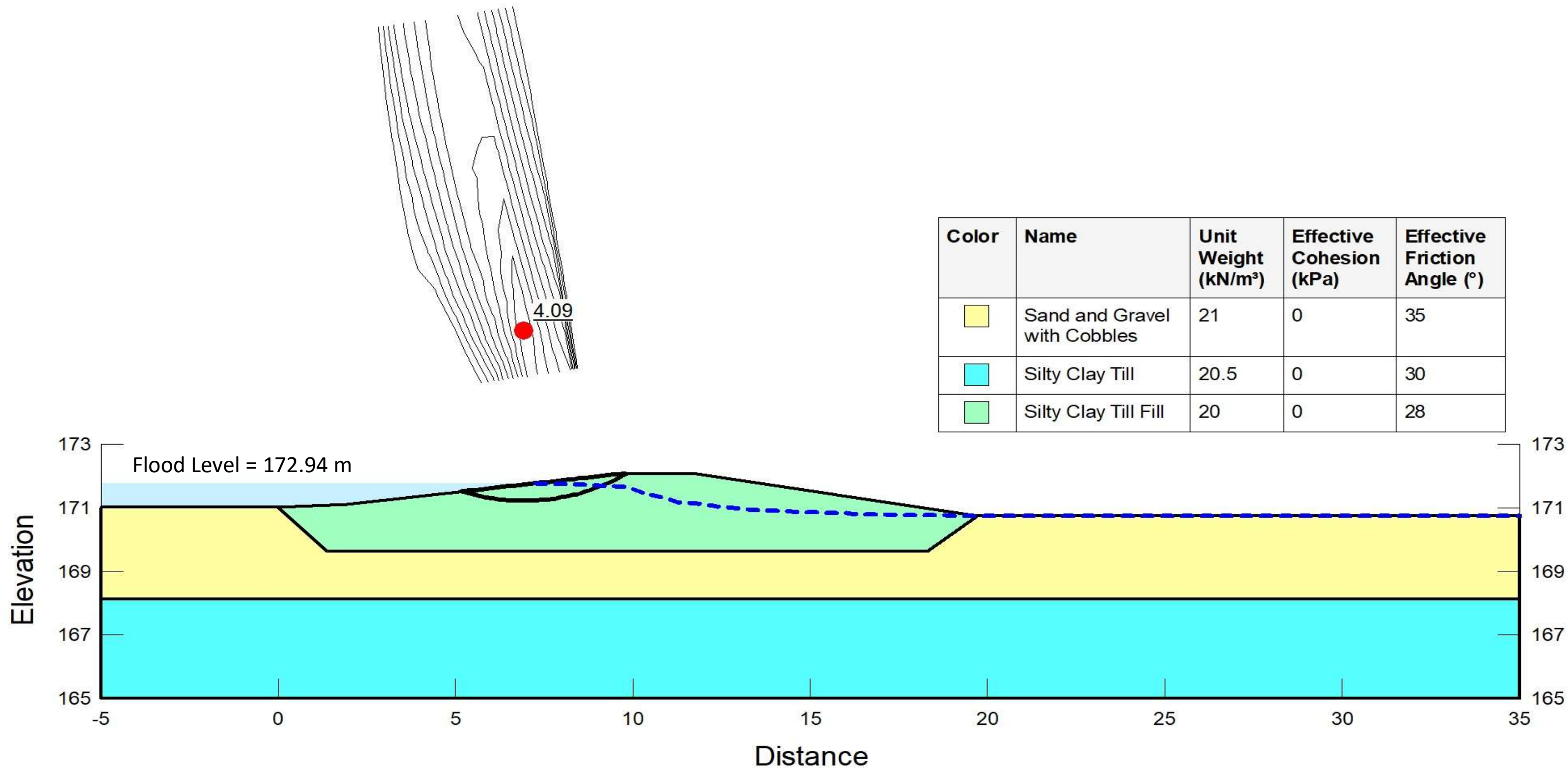
KGS GROUP

CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M DOWNSTREAM SIDE

January 2024	Figure D2-2	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
Yellow	Sand and Gravel with Cobbles	21	0	35
Cyan	Silty Clay Till	20.5	0	30
Green	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 4.09
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

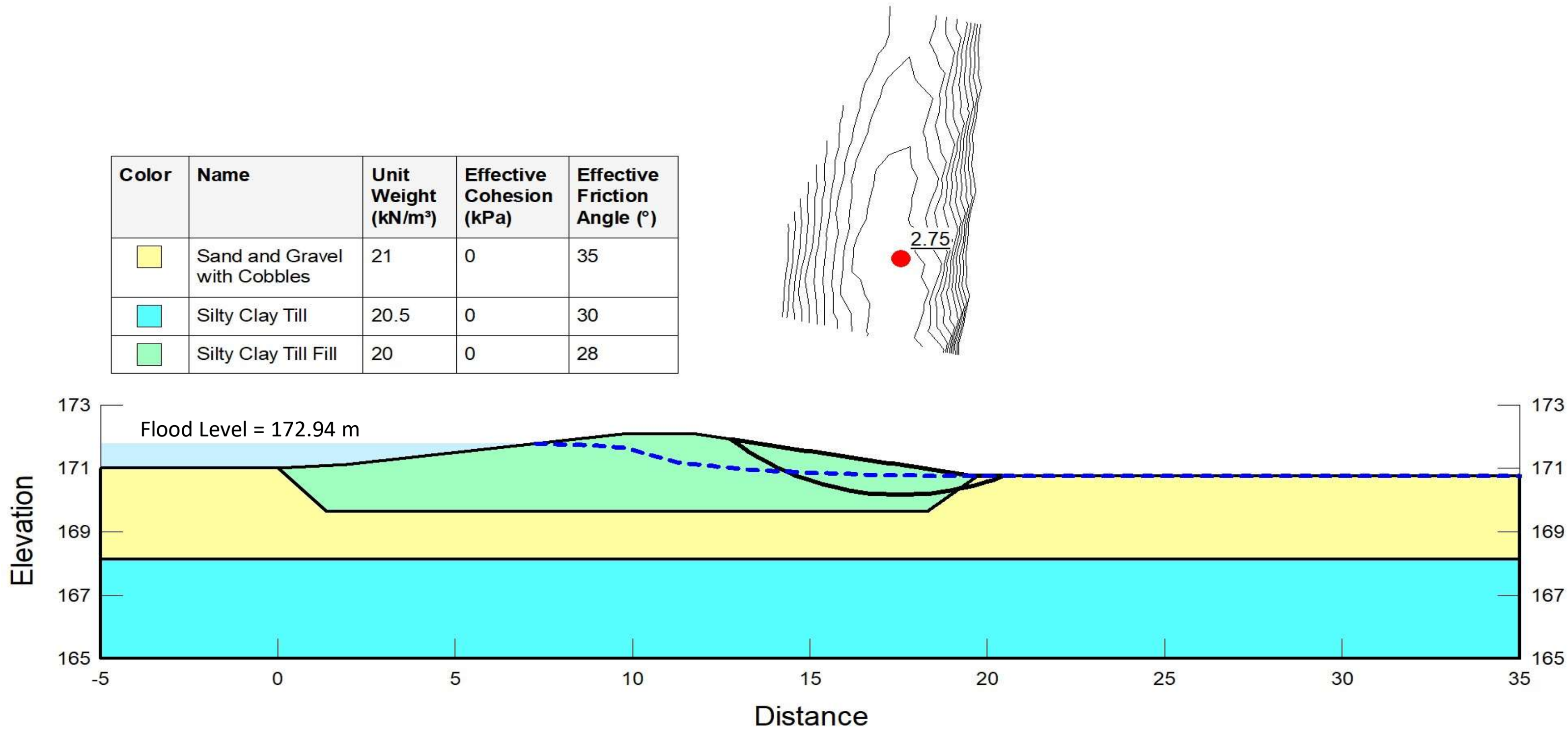
CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE) UPSTREAM SIDE

January 2024	Figure D2-3	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 2.75
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

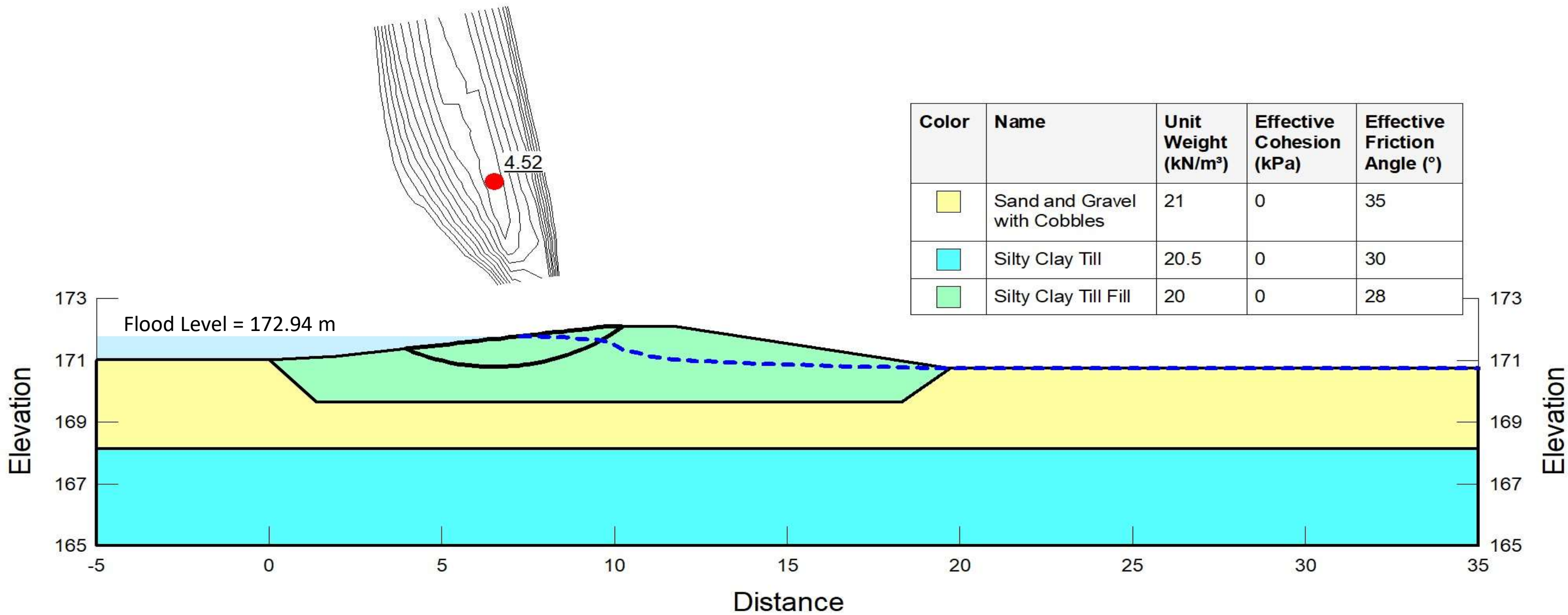
CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE)

DOWNSTREAM SIDE

January 2024	Figure D2-4	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 4.52
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

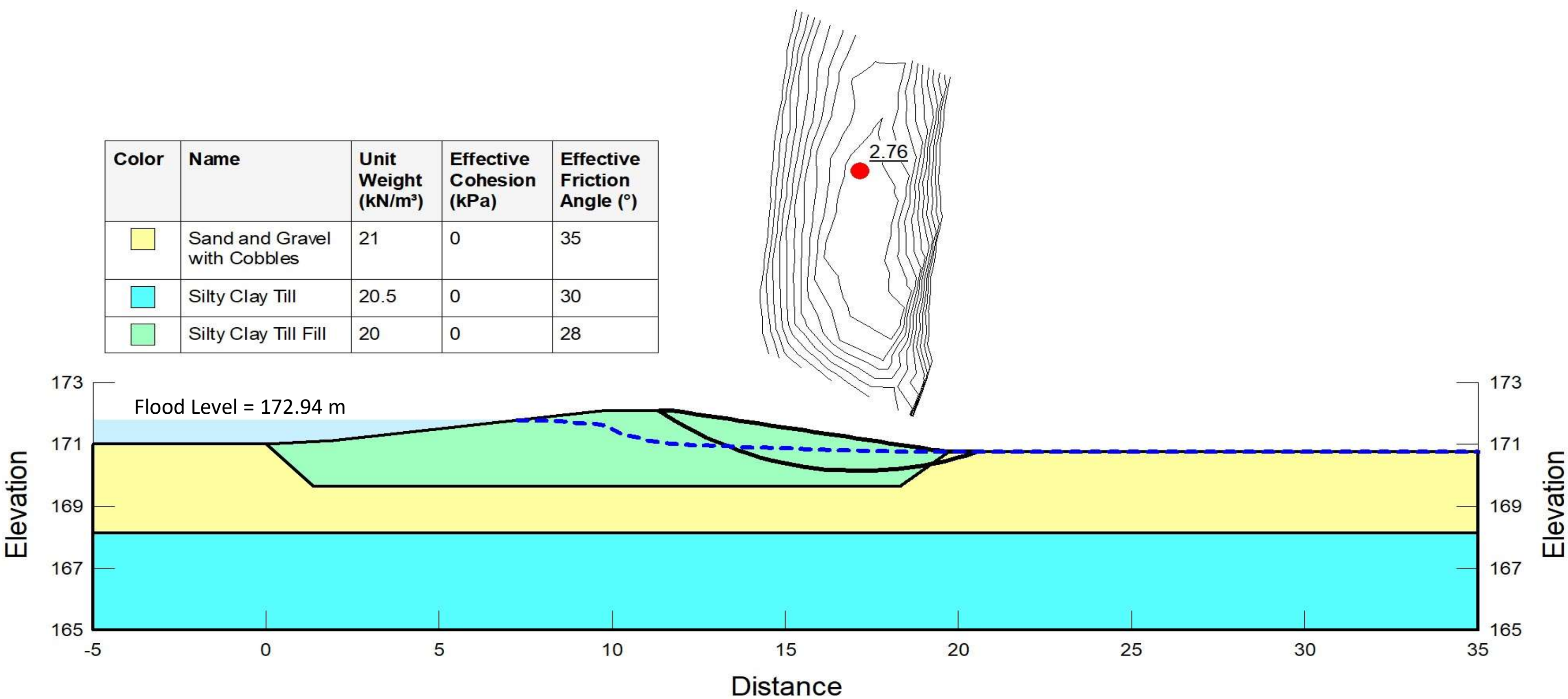
CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)
UPSTREAM SIDE

January 2024	Figure D2-5	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 2.76
2) Flood Level = El. 172.94 m (0.3 m freeboard).

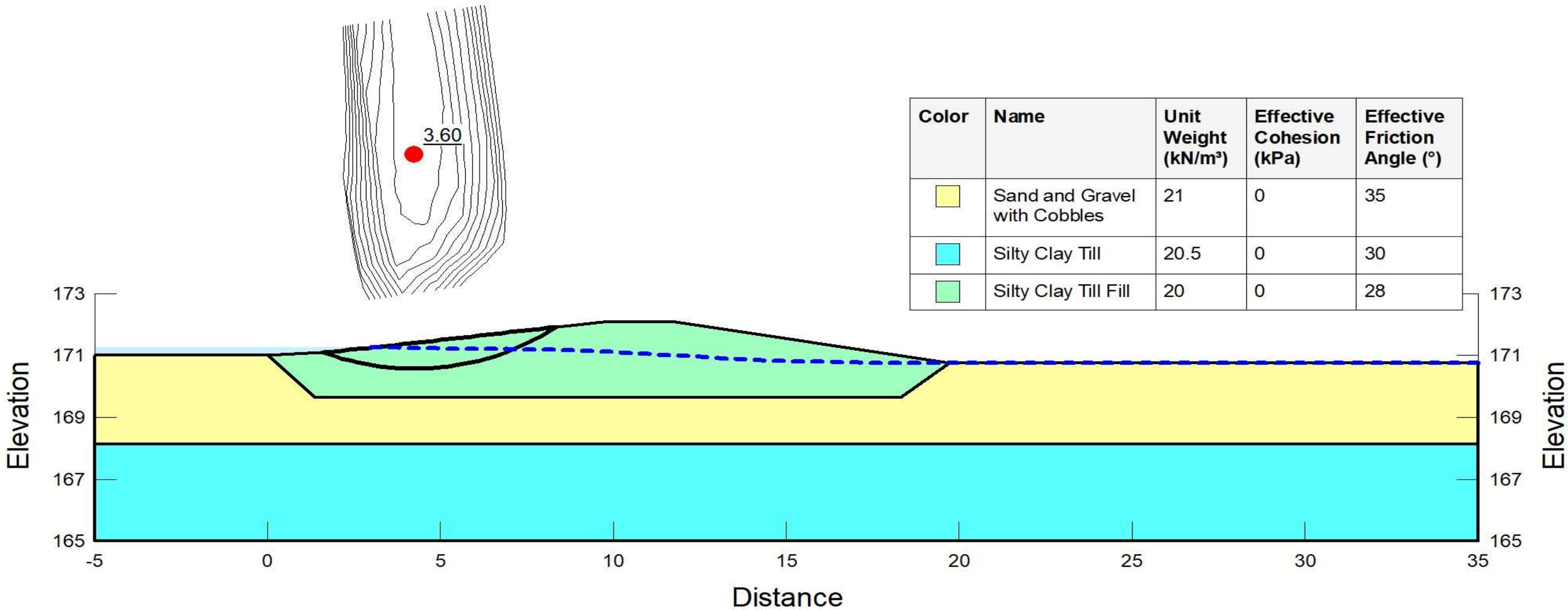
KGS GROUP

CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)
DOWNSTREAM SIDE

January 2024	Figure D2-6	Draft



NOTES:

1) Factor of Safety = 3.60 (T = 120 Hrs)
2) Flood Level = El. 172.94 m.
3) Normal River Level = El. 169.5 m.

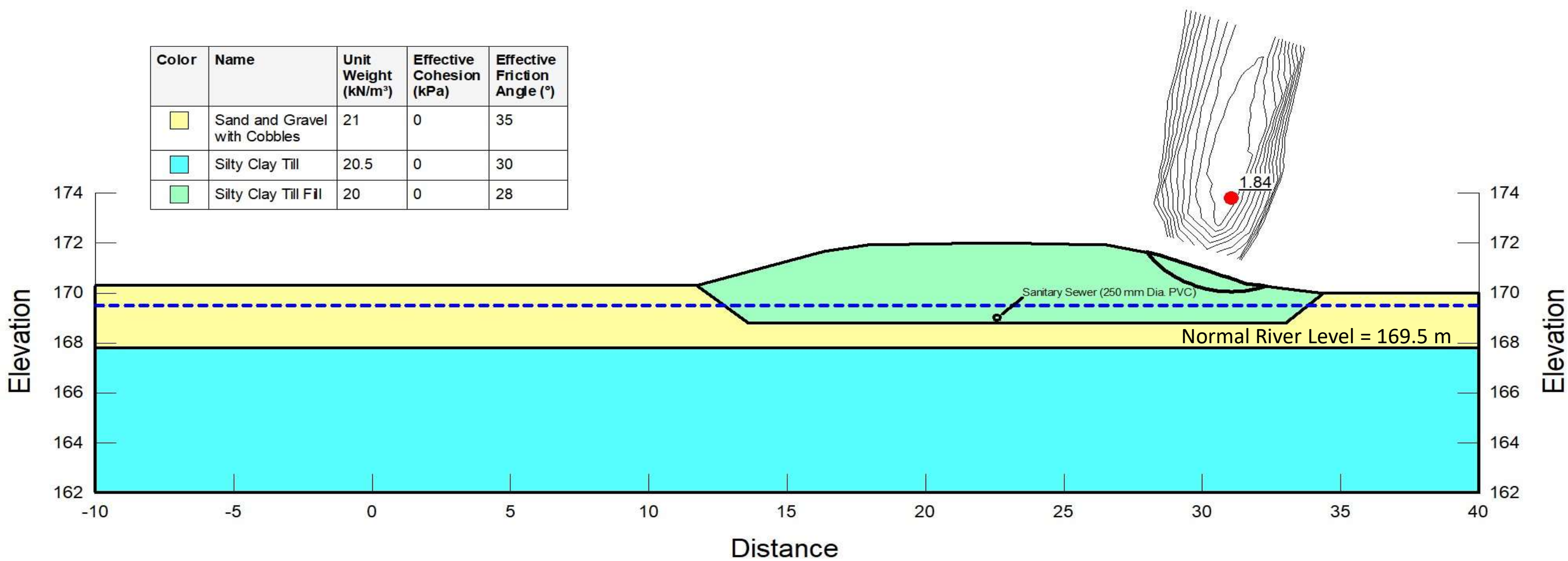
KGS GROUP

CHURCHVILLE DYKE (SECTION B-B', CH. 0+420)

SLOPE STABILITY ANALYSIS

CASE IV: FLOOD EVENT DRAWDOWN = 172.94 M to 169.5 M (TRANSIENT ANALYSIS) UPSTREAM SIDE

January 2024	Figure D2-7	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 1.84
2) Normal River Level = El. 169.5 m.

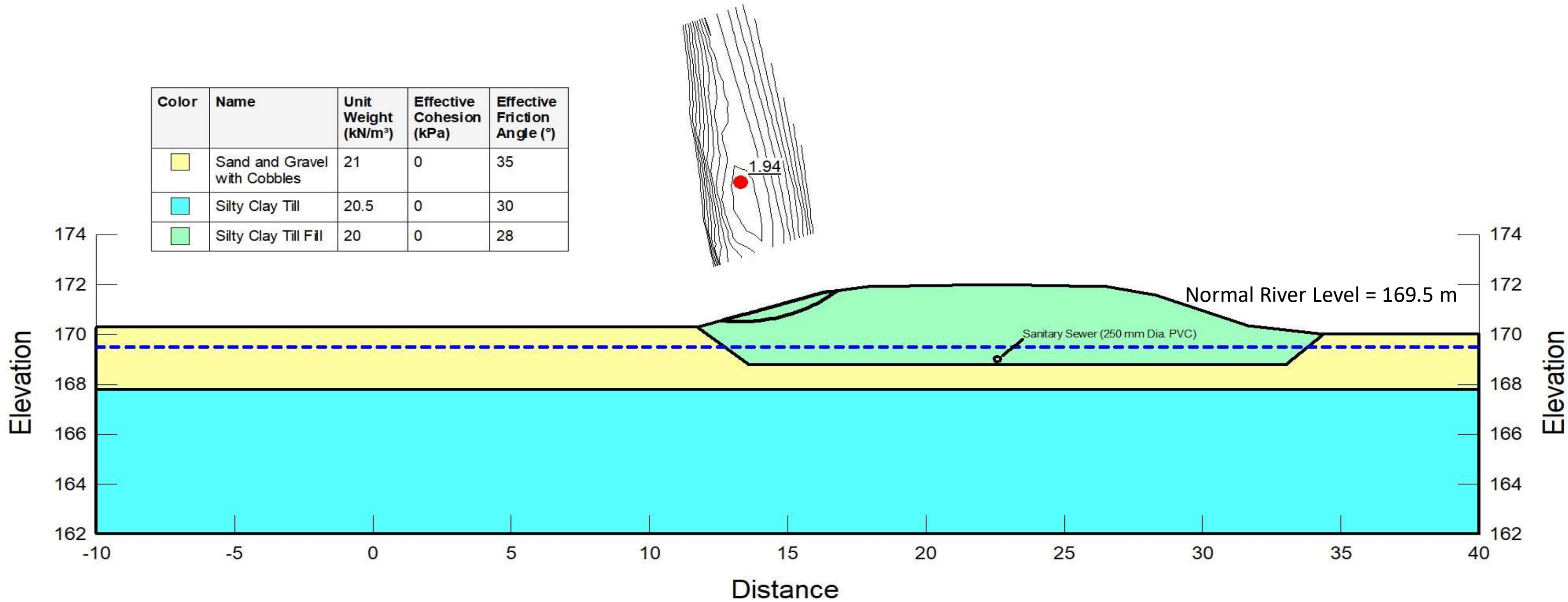
KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M UPSTREAM SIDE

January 2024	Figure D3-1	Draft



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28

NOTES:

1) Factor of Safety = 1.94
2) Normal River Level = El. 169.5 m.

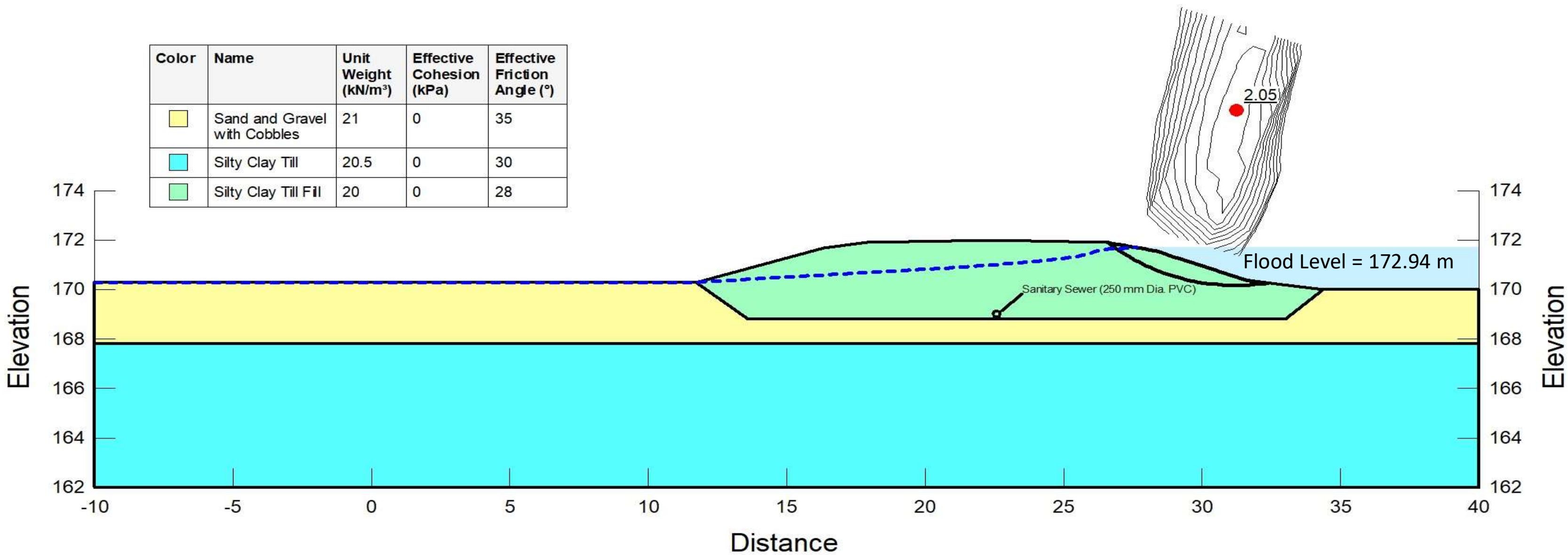
KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE I: NORMAL RIVER LEVEL = 169.5 M DOWNSTREAM SIDE

January 2024	Figure D3-2	Draft



NOTES:

1) Factor of Safety = 2.05
2) Flood Level = El. 172.94 m (0.3 m freeboard).

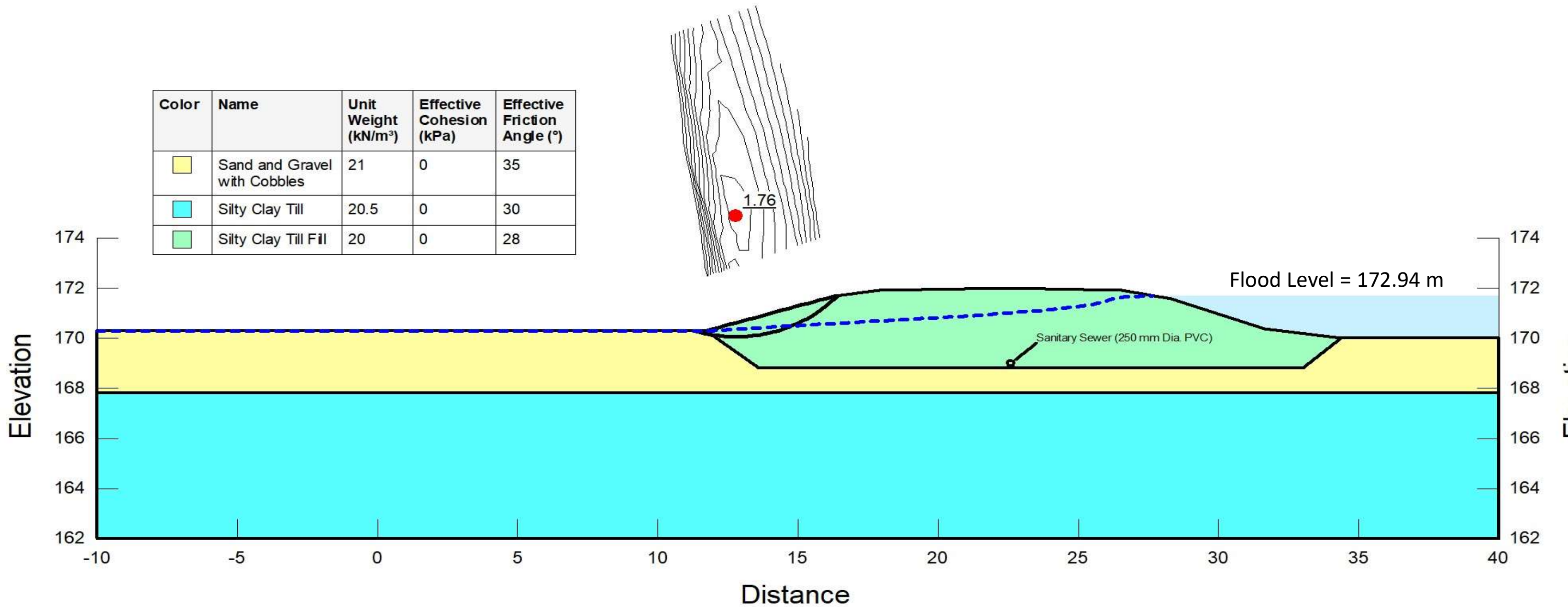
KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE) UPSTREAM SIDE

January 2024	Figure D3-3	Draft



NOTES:

1) Factor of Safety = 1.76
2) Flood Level = El. 172.94 m (0.3 m freeboard).

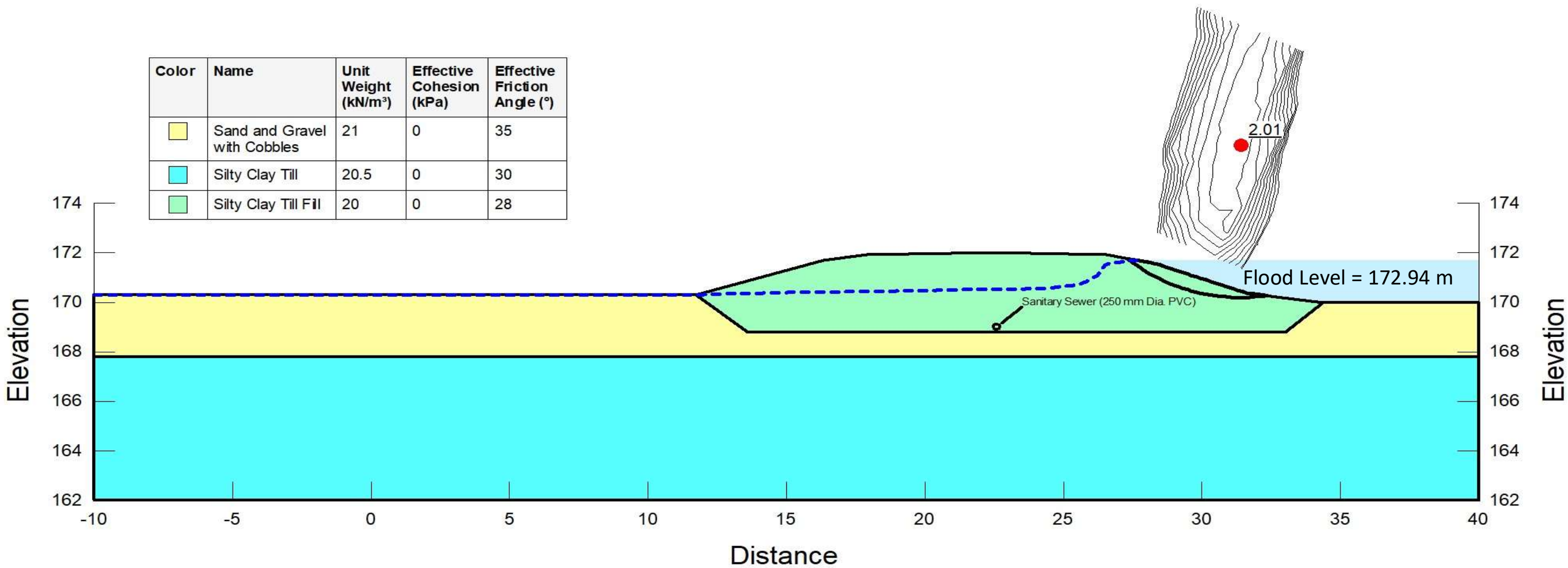
KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE II: FLOOD LEVEL = 172.94 M (STEADY-STATE)
DOWNSTREAM SIDE

January 2024	Figure D3-4	Draft



NOTES:

1) Factor of Safety = 2.01
2) Flood Level = El. 172.94 m (0.3 m freeboard).

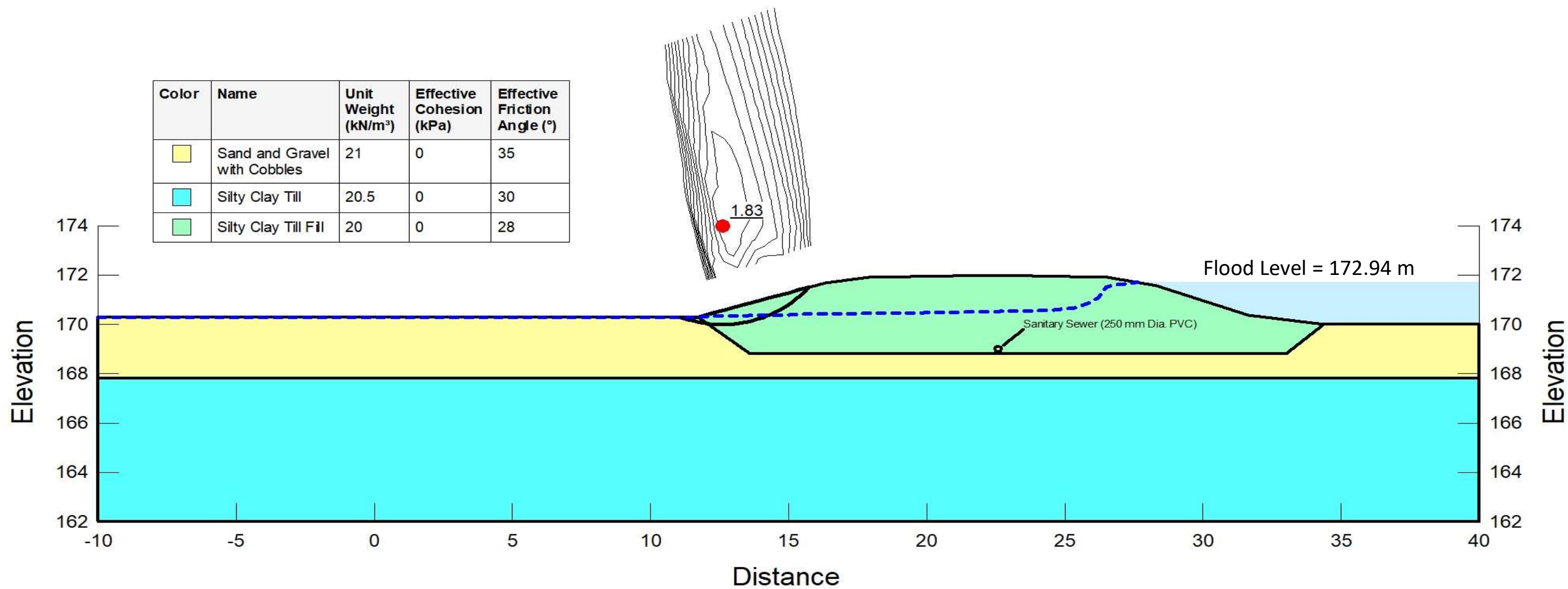
KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)
UPSTREAM SIDE

January 2024	Figure D3-5	Draft



NOTES:

1) Factor of Safety = 1.83
2) Flood Level = El. 172.94 m (0.3 m freeboard).

KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

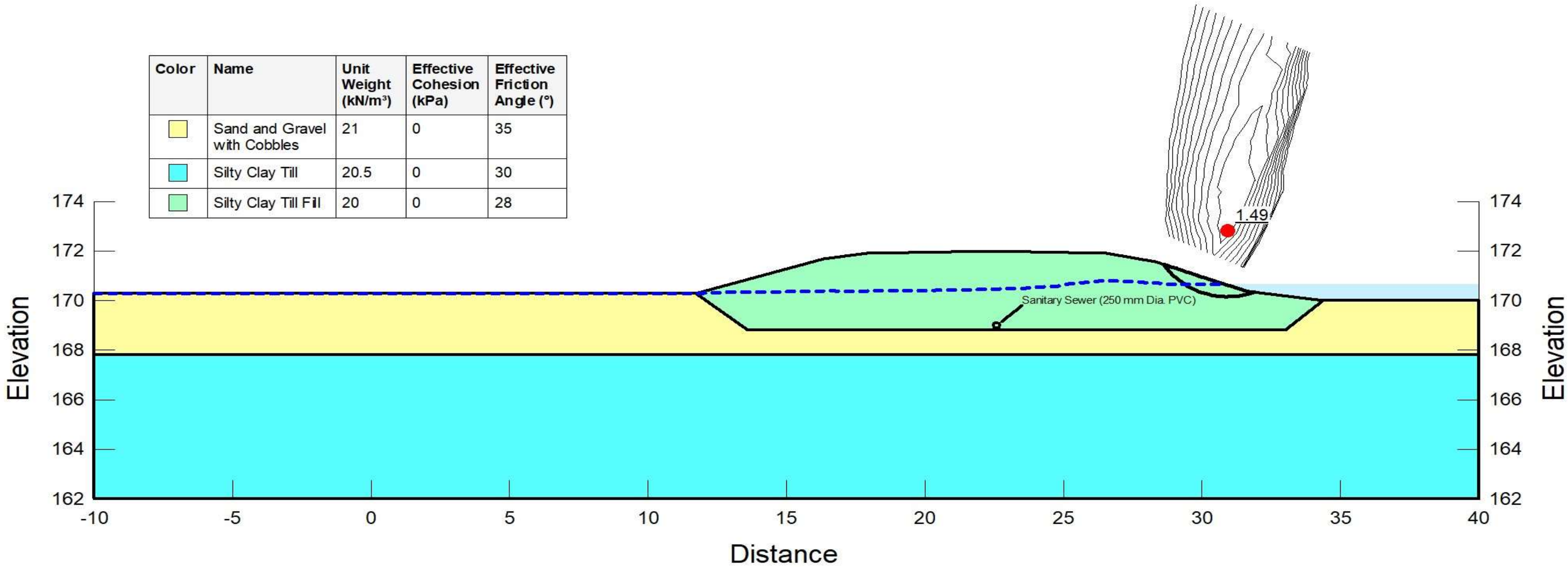
SLOPE STABILITY ANALYSIS

CASE III: FLOOD LEVEL = 172.94 M (TRANSIENT - 72 Hrs)

DOWNSTREAM SIDE

January 2024	Figure D3-6	Draft

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Sand and Gravel with Cobbles	21	0	35
	Silty Clay Till	20.5	0	30
	Silty Clay Till Fill	20	0	28



NOTES:

1) Factor of Safety = 1.49 (T = 120 Hrs)
2) Flood Level = El. 172.94 m.
3) Normal River Level = El. 169.5 m.

KGS GROUP

CHURCHVILLE DYKE (SECTION C-C', CH. 0+640)

SLOPE STABILITY ANALYSIS

CASE IV: FLOOD EVENT DRAWDOWN = 172.94 M to 169.5 M (TRANSIENT ANALYSIS) UPSTREAM SIDE

January 2024	Figure D3-7	Draft

APPENDIX E

Structural Stability Calculations

DESIGN CALCULATIONS COVER SHEET

Project No. :	23-4168-001	**Project Name :**	Churchville Condition Assessment		
File No. :		**Discipline :**	Structural Engineering		
Calculation Title :	**Floodwall Stability Analysis – Configuration 1**				
Calculation No. :	CIV-001	**Prepared by :**	JL	**Date :**	2023-12-01
No. of Sheets :		**Checked by :**	YF	**Date :**	2023-12-05
Supersedes Calc. No. :		**Approved by :**		**Date :**	

Calculation Description :

Stability analysis of the gravity retaining section.

Related Design Concept :

Reference Codes and Standards :
1. USACE – Retaining and Flood Walls EM 1110-2-2502, 1989
2. Structural Design and Factors of Safety – Technical Bulletin Ontario Ministry of Natural Resources (August 2011)

ENGINEER'S SEAL

Rev. #	**Rev. Description**	**Rev. Author**	**Date Revised**	**Checked by**	**Approved by**	**Approved Date**

Properties of Materials and Soils, Seismic Coefficients

$k_h := 0.151$ — *Unitless ratio of horizontal and vertical accelerations to the acceleration due to gravity.*

$k_v := \frac{2}{3} \cdot k_h = 0.101$

$\gamma_w := 9.81 \frac{kN}{m^3}$ — *Water density*

$\gamma_{conc} := 23.5 \cdot \frac{kN}{m^3}$ — *Concrete density*

$\phi_{cf} := 23 \cdot deg$ — *Friction angle of concrete/foundation interface*

$c := 0MPa$ — *Cohesion at concrete/foundation interface (generally set to 0)*

$f_{t.cf} := 0MPa$ — *Tensile strength at concrete/rock interface (generally set to 0). This is a negative number.*

Right (Backfill) Side

$\phi'_R := 35deg$ — *Effective friction angle*

$\gamma_{s.R} := 21 \frac{kN}{m^3}$ — *Moist (bulk) unit weight of soils*

$\gamma_{sat.R} := \gamma_{s.R}$ — *Saturated unit weight of soils*

$\gamma_{eff.R} := \gamma_{sat.R} - \gamma_w = 11.2 \cdot \frac{kN}{m^3}$ — *Submerged (effective) unit weight of soils*

$\delta_R := \phi_{cf} = 23 \cdot deg$ — *Friction angle for interface between wall and soil*

Left (Upstream) Side

$\phi'_L := 35deg$ — *Effective friction angle*

$\gamma_{s.L} := 21 \frac{kN}{m^3}$ — *Moist (bulk) unit weight of soils*

$\gamma_{sat.L} := \gamma_{s.L}$ — *Saturated unit weight of soils*

$\gamma_{eff.L} := \gamma_{sat.L} - \gamma_w = 11.2 \cdot \frac{kN}{m^3}$ — *Submerged (effective) unit weight of soils*

$\delta_L := \phi_{cf} = 23 \cdot deg$ — *Friction angle for interface between wall and soil*

Structure Geometry

Input

Note: Enter structure geometry as series of points on X-Y grid. Align structure so that upstream is on the left side. Structure outline is "closed" automatically (last point is assigned same values as first). Ensure that values of ELEusl and ELEdsl are adjusted to correspond with the lowest elevation on left and right sides.

$B := 1m$ — *Set unit width of structure*

$ELE_{top} := 173.3 \cdot m$ — *Elevation of top of wall*

$ELE_{Base.L} := 170.3m$ — *Lowest upstream elevation (left side)*

$ELE_{Base.R} := ELE_{Base.L} = 170.3\,m$ — *Lowest downstream elevation (right side)*

$ELE_{soil.R} := 171.5m = 171.5\,m$ — *Elevation of top of soil*

$ELE_{soil.L} := 171.2m = 171.2\,m$

$$X_{struct} := \begin{pmatrix} 0 \\ 0 \\ 0.3 \\ 0.3 \\ 0.6 \\ 0.6 \\ 1.6 \\ 1.6 \end{pmatrix} \cdot m \qquad Y_{struct} := \begin{pmatrix} ELE_{Base.L} \cdot m^{-1} \\ 170.60 \\ 170.60 \\ ELE_{top} \cdot m^{-1} \\ ELE_{top} \cdot m^{-1} \\ 170.60 \\ 170.60 \\ ELE_{Base.L} \cdot m^{-1} \end{pmatrix} \cdot m$$

Input X & Y coordinates

$\beta_R := 0deg$ $\quad \beta_L := 0deg$ — *Slope of soil, measured from horizontal (sloping upward is positive)*

$\theta_L := 90deg$ $\quad \theta_R := 90deg$ — *Slope of retaining wall face, measured from horizontal (90deg is vertical, battered walls are less than 90deg)*

For cantilever walls, set to 90 as soil pressure acts on a vertical plane at end of base slab

Even though backfill face is angled, Rankine cannot be used for angled face.
that is why a 90 deg angle is currently shown

$L_{hor} := \max(X_{struct}) - \min(X_{struct}) = 1.6\,m$ — *Horizontal projection of base*

$L_{slab.L} := 0.3m$ — *Length of slab beyond the stemwall on left and right sides*

$L_{slab.R} := 1m$

$t_{slab.L} := 0.3m$ — *Thickness of slab on left and right sides*

$t_{slab.R} := 0.3m$

$L_{wall.base} := L_{hor} - L_{slab.L} - L_{slab.R} = 0.3\,m$ — *Thickness of stemwall at base*

Input

Plot Functions

$X := X_{struct}$ $\qquad Y := Y_{struct}$

$i := 1 .. \text{length}(X)$ $\qquad j := 1 .. \text{length}(X) + 1$ — *Functions to automatically "close" the structure*

$X_{\text{length}(X)+1} := X_1$ $\qquad Y_{\text{length}(Y)+1} := Y_1$

$maxdim := \max[(\max(X) - \min(X)), (\max(Y) - \min(Y))] = 3\,m$ — *Sets the extents of both axis to the maximum length of the structure in the X or Y direction*

$minydim := \min(ELE_{Base.L}, ELE_{Base.R}) = 170.3\,m$

Functions for resultant plot

$Base_{heel.x} := \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ $\quad Base_{heel.y} := \begin{pmatrix} 1 \\ 0 \end{pmatrix}$

$Base_{third1.x} := \begin{pmatrix} \frac{L_{hor}}{3} \\ \frac{L_{hor}}{3} \end{pmatrix}$ $\quad Base_{third1.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$Base_{toe.x} := \begin{pmatrix} L_{hor} \\ L_{hor} \end{pmatrix}$ $\quad Base_{toe.y} := \begin{pmatrix} 1 \\ 0 \end{pmatrix}$

$Base_{half1.x} := \begin{pmatrix} \frac{L_{hor}}{4} \\ \frac{L_{hor}}{4} \end{pmatrix}$ $\quad Base_{half1.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$Base_{half2.x} := \begin{pmatrix} \frac{3L_{hor}}{4} \\ \frac{3L_{hor}}{4} \end{pmatrix}$ $\quad Base_{half2.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

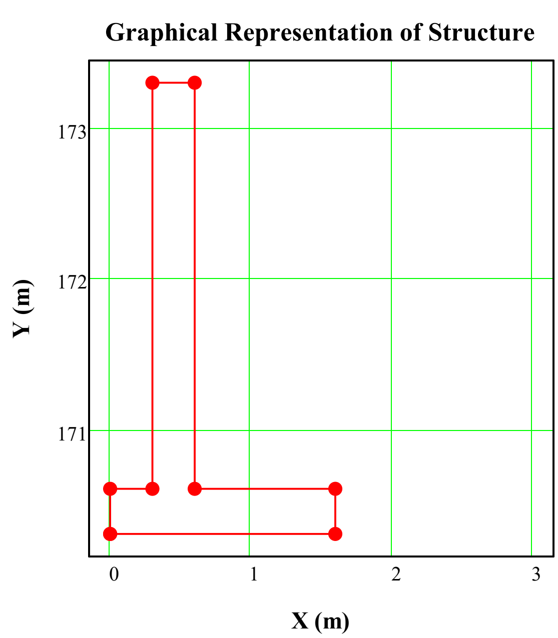
$Base_{third2.x} := \begin{pmatrix} \frac{2L_{hor}}{3} \\ \frac{2L_{hor}}{3} \end{pmatrix}$ $\quad Base_{third2.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$ground_y := \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ $\quad ground_x := \begin{pmatrix} 0 \\ L_{hor} \end{pmatrix}$

$xo_y := -.4$

Plot Functions

Graphical Representation of Structure



Computation of Area and Center of Gravity

$X_{struct} := X$ — *Assign coordinates to a seperate array to be used for plotting later in worksheet*

$Y_{struct} := Y$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3}\right]$$

$$Area := \left|\sum_i Areainc_i\right| = 1.3\,m^2$$ *Cross sectional area of the Structure*

$$Xg_{struct} := \left|\frac{\sum_i Xginc_i}{Area}\right| = 0.58\,m$$ *X coordinate of centroid of structure*

$$Yg_{struct} := \left|\frac{\sum_i Yginc_i}{Area}\right| = 171.392\,m$$ *evation of centroid of structure*

Computation of Area and Center of Gravity

Weight of Structure

$V_{conc} := Area \cdot B = 1.29 \cdot m^3$ — *Volume of concrete per unit width of structure*

$W_{conc} := V_{conc} \cdot \gamma_{conc} = 30.3 \cdot kN$ — *Dead load of concrete in structure*

$MA_{conc} := Xg_{struct} = 0.6\,m$ — *Moment arm if wall is rotating counterclockwise about toe on upstream side*

$M_{conc} := W_{conc} \cdot MA_{conc} = 17.6 \cdot kN \cdot m$

$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$
$Area = 1.3\,m^2$
$B = 1\,m$
$L_{hor} = 1.6\,m$
$Xg_{struct} = 0.58\,m$
$Yg_{struct} = 171.39\,m$

Lateral Earth Pressure Coefficients

$\beta_R = 0 \cdot deg$
$\phi'_R = 35 \cdot deg$
$\theta_R = 90 \cdot deg$
$\delta_R = 23 \cdot deg$
$\beta_L = 0 \cdot deg$
$\phi'_L = 35 \cdot deg$
$\theta_L = 90 \cdot deg$
$\delta_L = 23 \cdot deg$

$K_o(\phi) := (1 - \sin(\phi))$ *At-rest pressure*

$$K_a(\phi,\beta) := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

Rankine active/passive pressure coefficients

$$K_p(\phi) := \tan\left(45deg + \frac{\phi}{2}\right)^2$$

$$\psi(k_h, k_v) := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

$$K_{ae}(\beta,\phi',\theta,\delta,\psi) := \frac{\sin(\phi' + \theta - \psi)^2}{\cos(\psi)\sin(\theta)^2 \cdot \sin(\theta - \delta - \psi) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi') \cdot \sin(\phi' - \beta - \psi)}{\sin(\theta - \delta - \psi) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_{o.R} := K_o(\phi'_R) = 0.426$ *Calculation of lateral pressure coefficients*

$K_{a.R} := K_a(\phi'_R, \beta_R) = 0.271$

$K_{p.R} := K_p(\phi'_R) = 3.7$

$K_{o.L} := K_o(\phi'_L) = 0.426$

$K_{a.L} := K_a(\phi'_L, 0deg) = 0.271$

$K_{p.L} := K_p(\phi'_L) = 3.7$

$\psi_R := \psi(k_h, 0) = 8.587 \cdot deg$ *Seismic intertia angle (ignoring vertical component of earthquake to be conservative)*

$\psi_L := \psi(k_h, 0) = 8.6 \cdot deg$

$K_{ae.R} := K_{ae}(\beta_R, \phi'_R, \theta_R, \delta_R, \psi_R) = 0.343$ *Coefficient of (active) lateral pressure using Mononobe-Okabe method, for earthquake loading*

$$K_{oe.R} := \frac{K_{ae.R}}{K_{a.R}} \cdot K_{o.R} = 0.5$$ *Coefficient of (at-rest) lateral pressure, for earthquake loading*

$K_{ae.L} := K_{ae}(\beta_L, \phi'_L, \theta_L, \delta_L, \psi_L) = 0.343$

$$K_{oe.L} := \frac{K_{ae.L}}{K_{a.L}} \cdot K_{o.L} = 0.5$$

Weight of Material on Top of Section

Input coordinates

Reference Coordinates of Structure

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{soil.R} = 171.500\,m$
$ELE_{soil.L} = 171.200\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 0.300 \\ 0.300 \\ 0.600 \\ 0.600 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

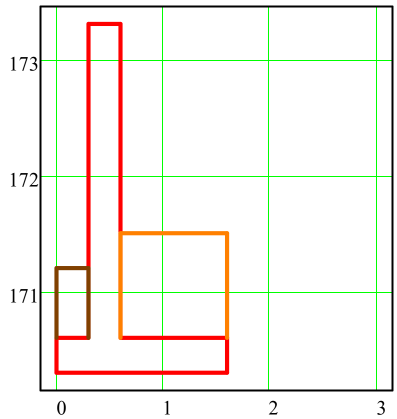
Insert coordinates of shape of material above structure

$B_{above} := B = 1.000\,m$ *Width of material*

$$X_{above.L} := \begin{pmatrix} 0 \\ 0 \\ 0.3 \\ 0.3 \end{pmatrix} m \qquad Y_{above.L} := \begin{pmatrix} 170.60m \\ ELE_{soil.L} \\ ELE_{soil.L} \\ 170.60m \end{pmatrix}$$

$$X_{above.R} := \begin{pmatrix} 0.6 \\ 0.6 \\ 1.6 \\ 1.6 \end{pmatrix} m \qquad Y_{above.R} := \begin{pmatrix} 170.60m \\ ELE_{soil.R} \\ ELE_{soil.R} \\ 170.60m \end{pmatrix}$$

Material on Top of Structure



$x_{soil.L} := \max(X_{above.L}) - \min(X_{above.L}) = 0.3\,m$

$x_{soil.R} := \max(X_{above.R}) - \min(X_{above.R}) = 1\,m$

Input coordinates

Calculations

$X := X_{above.L}$ $\qquad$ $Y := Y_{above.L}$

$i := 1 .. \text{length}(X)$ $\qquad$ $j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\qquad$ $Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\qquad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\qquad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3} \right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3} \right]$$

$$A_{above.L} := \left| \sum_i Areainc_i \right| = 0.2\,m^2$$

$$Xg_{above.L} := \begin{cases} 0 & \text{if } A_{above.L} = 0 \\ \left| \dfrac{\sum_i Xginc_i}{A_{above.L}} \right| & \text{otherwise} \end{cases} = 0.15$$

$$Yg_{above.L} := \begin{cases} 0 & \text{if } A_{above.L} = 0 \\ \left| \dfrac{\sum_i Yginc_i}{A_{above.L}} \right| & \text{otherwise} \end{cases} = 170.9$$

$X := X_{above.R}$ $\quad$ $Y := Y_{above.R}$

$i := 1..\text{length}(X)$ $\quad$ $j := 1..\text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$$

$$A_{above.R} := \left|\sum_i Areainc_i\right| = 0.9\,m^2$$

$$Xg_{above.R} := \begin{cases} 0 & \text{if } A_{above.R} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{above.R}}\right| & \text{otherwise} \end{cases} = 1.1$$

$$Yg_{above.R} := \begin{cases} 0 & \text{if } A_{above.R} = 0 \\ \left|\dfrac{\sum_i Yginc_i}{A_{above.R}}\right| & \text{otherwise} \end{cases} = 171.05$$

Calculations

Load Combination 1

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

$K_R := K_{o.R} = 0.426$ — *Coefficient of lateral stress (assign to Ko.R, Ka.R or Kp.R)*

$K_L := K_{o.L} = 0.426$

$ELE_{water.R} := 170.3m$ — *Elevation of water level on right side*

$ELE_{water.L} := 170.3m$ — *Elevation of water level on left side*

$q_{surcharge.R} := 4.8kPa$ — *Uniform surface stress , assumed to extend for an infinite length behind wall*

Calculation of Soil Forces on Right Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.R} = 0.426$
$K_{a.R} = 0.271$
$\delta_R = 23 \cdot deg$
$\theta_R = 90 \cdot deg$
$ELE_{soil.R} = 171.5\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.R} := ELE_{soil.R} - ELE_{Base.R} = 1.2\,m$ *Height of soil on right side of wall*

$$H_{above.R} := \begin{cases} (ELE_{soil.R} - ELE_{water.R}) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 1.20$$

Height of soil above water

$$H_{below.R} := \begin{cases} (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \ge ELE_{soil.R} \\ (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} > ELE_{Base.R} \wedge ELE_{water.R} < ELE_{soil.R} \\ 0 & \text{otherwise} \end{cases} = 0$$

Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_R \cdot \gamma_{s.R} \cdot H_{above.R}^2 \cdot B = 6.4\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ \left(ELE_{Base.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 170.7$$

Elevation of force above water

$F_{below1} := B \cdot K_R \cdot (\gamma_{s.R} \cdot H_{above.R}) \cdot H_{below.R} = 0\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{2}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{below2} := B \cdot K_R \cdot \frac{\gamma_{eff.R} \cdot H_{below.R}^2}{2} = 0\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{3}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{soil.R} := F_{above} + F_{below1} + F_{below2} = 6.4\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} = 170.7\,m$$

Elevation of force

$F_{soil.hor.R} := F_{soil.R} \cdot \cos(\beta_R) = 6.4\,kN$ *Horizontal component (positive is upstream/to the left)*

$F_{soil.ver.R} := F_{soil.R} \cdot \sin(\beta_R) = 0\,kN$

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.4\,m$ *Moment arm for horizontal force*

$\boxed{MA_{ver} := L_{wall.base} + L_{slab.R} = 1.3\,m}$ *Moment arm for vertical force*

$M_{soil.hor.R} := F_{soil.hor.R} \cdot MA_{hor} = 2.6\,kN \cdot m$ *Moment from horizontal component*

$M_{soil.ver.R} := F_{soil.ver.R} \cdot MA_{ver} = 0\,kN \cdot m$ *Moment from vertical component*

Calculation of Soil Forces on Right Side

Calculation of Soil Forces on Left Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.L} = 0.426$
$K_{a.L} = 0.271$
$\delta_L = 23 \cdot deg$
$\theta_L = 90 \cdot deg$
$ELE_{soil.L} = 171.2\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.L} := ELE_{soil.L} - ELE_{Base.L} = 0.9\,m$ *Height of soil on right side of wall*

$$H_{above.L} := \begin{cases} (ELE_{soil.L} - ELE_{water.L}) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0.90$$
Height of soil above water

$$H_{below.L} := \begin{cases} (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{soil.L} \\ (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{Base.L} \wedge ELE_{water.L} < ELE_{soil.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_L \cdot \gamma_{s.L} \cdot H_{above.L}^2 \cdot B = 3.6\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ \left(ELE_{Base.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 170.6$$
Elevation of force above water

$F_{below1} := B \cdot K_L \cdot (\gamma_{s.L} \cdot H_{above.L}) \cdot H_{below.L} = 0\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{2}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Elevation of force

$F_{below2} := B \cdot K_L \cdot \frac{\gamma_{eff.L} \cdot H_{below.L}^2}{2} = 0\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{3}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Elevation of force

$F_{soil.L} := F_{above} + F_{below1} + F_{below2} = 3.6\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \begin{cases} 0 & \text{if } F_{soil.L} = 0 \\ \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} & \text{otherwise} \end{cases} = 170.6$$
Elevation of force

$F_{soil.hor.L} := F_{soil.L} = 3.6\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.3\,m$ *Moment arm for horizontal force*

$M_{soil.hor.L} := F_{soil.hor.L} \cdot MA_{hor} = 1.1\,kN \cdot m$ *Moment from horizontal component*

Calculation of Soil Forces on Left Side

Hydrostatic Pressure on Right Side

$$H_{water.R} := \begin{cases} (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \geq ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0$$

Height of water acting on wall

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_R = 90 \cdot deg$

$$F_{water.R} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.R}^2 = 0\,kN$$

Hydrostatic force acting normal to face

$$ELE_F := ELE_{Base.R} + \frac{H_{water.R}}{3} = 170.3\,m$$

Elevation of resultant force

$$F_{water.hor.R} := F_{water.R} = 0\,kN$$

Horizontal component (positive is upstream/to the left)

$$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$$

Moment arm for horizontal force

$$M_{water.hor.R} := F_{water.hor.R} \cdot MA_{hor} = 0\,kN \cdot m$$

Moment from horizontal component

Hydrostatic Pressure on Right Side

Hydrostatic Pressure on Left Side

$$H_{water.L} := \begin{cases} (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \geq ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Height of water acting on wall

$$F_{water.L} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.L}^2 = 0\,kN$$

Hydrostatic force acting normal to face

$$ELE_F := ELE_{Base.L} + \frac{H_{water.L}}{3} = 170.3\,m$$

Elevation of resultant force

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.L} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_L = 90 \cdot deg$

$$F_{water.hor.L} := F_{water.L} = 0\,kN$$

Horizontal component (positive is upstream/to the left)

$$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$$

Moment arm for horizontal force

$$M_{water.hor.L} := F_{water.hor.L} \cdot MA_{hor} = 0\,kN \cdot m$$

Moment from horizontal component

Hydrostatic Pressure on Left Side

Calculation of Uplift Pressure

$$P_{U.L} := H_{water.L} \cdot \gamma_w = 0\,kPa$$

Uplift pressure at left side

$$P_{U.R} := H_{water.R} \cdot \gamma_w = 0\,kPa$$

Uplift pressure at right side

$$P_U(x) := \begin{cases} P_{U.R} + \frac{(P_{U.L} - P_{U.R})}{(L_{hor})} \cdot x & \text{if } x \leq L_{hor} \wedge x \geq 0 \\ 0 & \text{otherwise} \end{cases}$$

Uplift function. x=0 at right side.

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$ELE_{water.L} = 170.300\,m$
$ELE_{water.R} = 170.300\,m$

$$F_U := \int_0^{L_{hor}} P_U(x) \cdot B\,dx = 0 \cdot kN$$

Total uplift force assuming uncracked section.

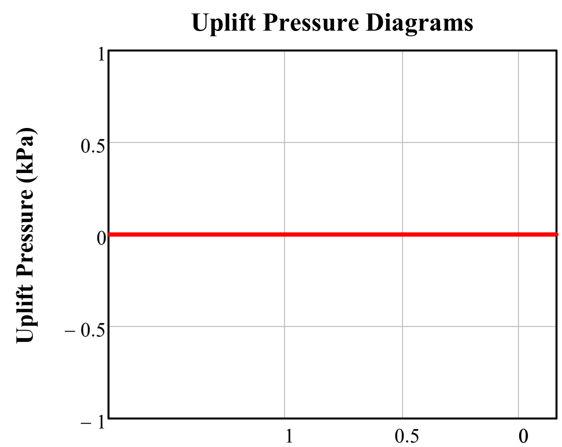
$$L_{Uf} := \begin{cases} 0 & \text{if } F_U = 0 \\ \left(\frac{1}{F_U}\int_0^{L_{hor}} P_U(x)\cdot x\cdot B\,dx\right) & \text{otherwise} \end{cases} = 0$$

Distance of uplife force from right side of base

$MA := L_{hor} - L_{Uf} = 1.6\,m$ *Moment arm of uplift force*

$M_U := F_U \cdot MA = 0 \cdot kN \cdot m$ *Moment from uplift*

$F_{U.ver} := -F_U = 0 \cdot kN$ *Vertical component (down is positive)*



Calculation of Uplift Pressure

Force from Surcharge

$F_{q.R} := q_{surcharge.R} \cdot K_R \cdot H_{soil.R} \cdot B = 2.5\,kN$ *Force on wall due to surcharge*

$ELE_F := ELE_{Base.R} + \frac{H_{soil.R}}{2} = 170.9\,m$ *Elevation of resultant force*

$F_{q.hor.R} := F_{q.R} = 2.5\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.6\,m$ *Moment arm for horizontal component*

$M_{q.hor.R} := F_{q.hor.R} \cdot MA_{hor} = 1.5\,kN{\cdot}m$ *Moment from horizontal component*

$F_{q.ver.R} := q_{surcharge.R} \cdot L_{slab.R} \cdot B = 4.8 \cdot kN$

$MA_{ver} := L_{slab.L} + L_{wall.base} + \frac{L_{slab.R}}{2} = 1.1\,m$

$M_{q.ver.R} := F_{q.ver.R} \cdot MA_{ver} = 5.3\,kN{\cdot}m$

Force from Surcharge

Weight of Water Above Section

Reference Coordinates of Structure

$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 0.300 \\ 0.300 \\ 0.600 \\ 0.600 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

$ELE_{Base.L} = 170.300m$
$ELE_{Base.R} = 170.300m$
$ELE_{top} = 173.300m$
$L_{hor} = 1.600m$
$B = 1.000m$

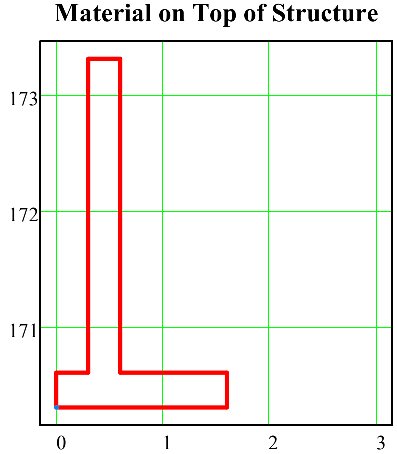
Insert coordinates of shape of material above structure

$B_{water} := B = 1.000m$ *Width of material*

$$X_{water.L} := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} m \qquad Y_{water.L} := \begin{pmatrix} ELE_{water.L} \\ ELE_{water.L} \\ ELE_{water.L} \end{pmatrix}$$

$$X_{water.R} := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} m \qquad Y_{water.R} := \begin{pmatrix} ELE_{water.R} \\ ELE_{water.R} \\ ELE_{Base.R} \end{pmatrix}$$

Material on Top of Structure



$x_{water.L} := \max(X_{water.L}) - \min(X_{water.L}) = 0$

$x_{water.R} := \max(X_{water.R}) - \min(X_{water.R}) = 0$

Calculation of Area and Centre of Gravity (Left Side)

$X := X_{water.L}$ $\quad$ $Y := Y_{water.L}$

$i := 1 .. \text{length}(X)$ $\quad$ $j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$$

$$A_{water.above.L} := \left|\sum_i Areainc_i\right| = 0\,m^2$$

$$Xg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

$$Yg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Yginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

Calculation of Area and Centre of Gravity (Right Side)

$X := X_{water.R}$ $\quad$ $Y := Y_{water.R}$

$i := 1 .. \text{length}(X)$ $\quad$ $j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$$

$$A_{water.above.R} := \left|\sum_i Areainc_i\right| = 0\,m^2$$

$$Xg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 0$$

$Yg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \dfrac{\sum_i Yginc_i}{A_{water.above.R}} & \text{otherwise} \end{cases} = 0$

Calculation of Forces

$W_{water.above.L} := (A_{water.above.L} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.L} = 0m$

$M_{water.above.L} := W_{water.above.L} \cdot MA_{Hor} = 0kN \cdot m$

$W_{water.above.R} := (A_{water.above.R} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.R} = 0m$

$M_{water.above.R} := W_{water.above.R} \cdot MA_{Hor} = 0kN \cdot m$

Weight of Water Above Section

Weight of Material Above Section

$x_{sat} := x_{water.R} = 0$

$x_{dry} := x_{soil.R} - x_{water.R} = 1m$

$\gamma_{above.L} := \gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$W_{above.L} := (A_{above.L} \cdot \gamma_{above.L} \cdot B) = 3.8 \cdot kN$ *Weight of material above base of cantilever wall on left side*

$MA_{Hor} := Xg_{above.L} = 0.2m$ *Moment arm*

$M_{above.L} := W_{above.L} \cdot MA_{Hor} = 0.6kN \cdot m$ *Overturning moment*

$\gamma_{above.R} := \gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$

$W_{above.R} := (A_{above.R} \cdot \gamma_{above.R} \cdot B) = 18.9 \cdot kN$ *Weight of material above base of cantilever wall on right side*

$MA_{Hor} := Xg_{above.R} = 1.1m$

$M_{above.R} := W_{above.R} \cdot MA_{Hor} = 20.8kN \cdot m$

$ELE_{Base.L} = 170.300m$
$ELE_{Base.R} = 170.300m$
$H_{above.L} = 0.90m$
$H_{below.L} = 0.00m$
$H_{above.R} = 1.20m$
$H_{below.R} = 0.00m$
$A_{above.L} = 0.18m^2$
$A_{above.R} = 0.9m^2$
$L_{hor} = 1.600m$
$B = 1.000m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

Weight of Material Above Section

LC.1 - Summary of Forces

Dead Load:

$W_{conc} = 30.3 \cdot kN$ $M_{conc} = 17.6 \cdot kN \cdot m$

Soil:

$F_{soil.hor.R} = 6.4 \cdot kN$ $M_{soil.hor.R} = 2.6 \cdot kN \cdot m$

$F_{soil.ver.R} = 0 kN$ $M_{soil.ver.R} = 0 kN \cdot m$

$F_{soil.hor.L} = 3.6 \cdot kN$ $M_{soil.hor.L} = 1.1 kN \cdot m$

Material Above Section:

$W_{above.L} = 3.8 kN$ $M_{above.L} = 0.6 kN \cdot m$

$W_{above.R} = 18.9 kN$ $M_{above.R} = 20.8 kN \cdot m$

Water Above Section:

$W_{water.above.L} = 0 kN$ $M_{water.above.L} = 0 kN \cdot m$

$W_{water.above.R} = 0 kN$ $M_{water.above.R} = 0 kN \cdot m$

Uplift:

$F_{U.ver} = 0 \cdot kN$ $M_U = 0 kN \cdot m$

Hydrostatic:

$F_{water.hor.R} = 0 kN$ $M_{water.hor.R} = 0 kN \cdot m$

$F_{water.hor.L} = 0 kN$ $M_{water.hor.L} = 0 kN \cdot m$

Surcharge:

$F_{q.hor.R} = 2.5 \cdot kN$ $M_{q.hor.R} = 1.5 kN \cdot m$

$F_{q.ver.R} = 4.8 \cdot kN$ $M_{q.ver.R} = 5.3 kN \cdot m$

LC.1 - Combine Forces and Moments

$F_{hor.drive} := F_{soil.hor.R} + F_{water.hor.R} + F_{q.hor.R} = 8.9 kN$

$F_{hor.resist} := F_{soil.hor.L} + F_{water.hor.L} = 3.6 kN$

$F_{ver} := W_{conc} + F_{U.ver} + W_{above.L} + W_{above.R} + W_{water.above.L} + W_{water.above.R} + F_{soil.ver.R} + F_{q.ver.R} = 57.8 \cdot kN$

$M_{stab} := M_{conc} + M_{water.hor.L} + M_{soil.hor.L} + M_{above.L} + M_{above.R} + M_{water.above.L} + M_{water.above.R} + M_{soil.ver.R} + M_{q.ver.R} = 45.3 kN \cdot m$

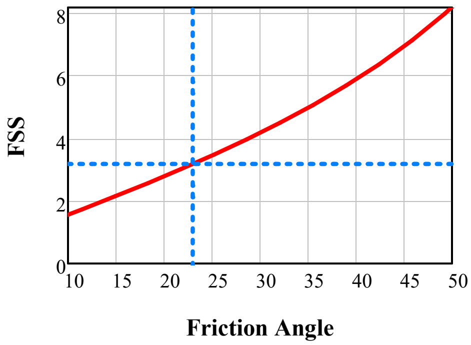
$M_o := M_{soil.hor.R} + M_{water.hor.R} + M_{q.hor.R} + M_U = 4.1 kN \cdot m$

$M_{net} := M_{stab} - M_o = 41.3 kN \cdot m$

LC.1 - Sliding

$$FSS(\theta) := \frac{F_{hor.resist} + F_{ver} \cdot \tan(\theta) + c \cdot L_{hor} \cdot B}{F_{hor.drive}}$$

$\boxed{FSS(\phi_{cf}) = 3.16}$



LC.1 - Overturning, Resultant and Bearing Stresses

$$FS_{OT} := \frac{M_{stab}}{M_o} = 11.18$$

$$x_o := \frac{M_{net}}{F_{ver}} = 0.71\,m$$

$$\boxed{E := x_o - \frac{L_{hor}}{2} = -0.09\,m}$$

▶ Stress Calculations

$\sigma_n(0m) = 47.8\,kPa$

$q_{max} = 47.8 \cdot kPa$ — *Maximum bearing stress before iterative cracked base analysis*

$q_{min} := (\sigma_n(L_{hor})) = 24.5\,kPa$

$\sigma_n(L_{slab.L}) = 43.4\,kPa$

$L_{comp} = 1.60\,m$ — *Length of base in compression before iterative cracked base analysis*

$\sigma_n(L_{slab.L} + L_{wall.base}) = 39\,kPa$

$L_{tens} = 0.00\,m$ — *Length of base in tension before iterative cracked base analysis*

$L_{crack} = 0.00\,m$ — *Length of crack between concrete and base before iterative cracked base analysis*

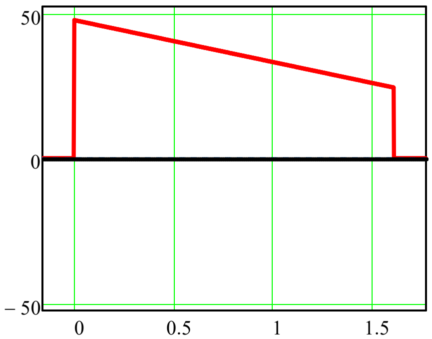
$\sigma_n(L_{hor}) = 24.5\,kPa$

$\frac{L_{comp}}{L_{hor}} = 100 \cdot \%$ — *% of Base in Compression*

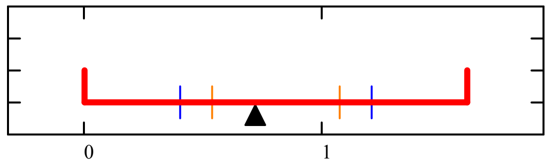
$\frac{L_{tens}}{L_{hor}} = 0 \cdot \%$ — *% of Base in Tension*

$\frac{L_{crack}}{L_{hor}} = 0 \cdot \%$ — *% of Base Cracked*

Normal Stresses Acting on Base



Location of Resultant



Red lines indicate extent of structure, Blue lines indicate middle half of base, orange lines indicae middle third of base

Store Results for Summary

$LC := 1$

$R_{FSS_{LC}} := FSS(\phi_{cf}) = ...$

$R_{E_{LC}} := E = ...$

$R_{L.comp_{LC}} := L_{comp} = ...$

$R_{L.crack_{LC}} := L_{crack} = ...$

$R_{L.tens_{LC}} := L_{tens} = ...$

$R_{xo_{LC}} := x_o = ...$

$R_{F.hor.drive_{LC}} := F_{hor.drive} = ...$

$R_{F.hor.resist_{LC}} := F_{hor.resist} = ...$

$R_{F.ver_{LC}} := F_{ver} = ...$

$R_{q.max_{LC}} := q_{max} = ...$

$R_{comp.percent_{LC}} := \frac{L_{comp}}{L_{hor}} = ...$

$R_{q.min_{LC}} := q_{min} = ...$

Store Results for Summary

Load Combination 2 - IDF (Unusual)

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

$K_R := K_{a.R} = 0.271$ — *Coefficient of lateral stress (assign to Ko.R, Ka.R or Kp.R)*

$K_L := K_{p.L} = 3.690$

$ELE_{water.R} := 173m$ — *Elevation of water level on right side*

$ELE_{water.L} := 170.3m$ — *Elevation of water level on left side*

$q_{surcharge.R} := 0kPa$ — *Uniform surface stress , assumed to extend for an infinite length behind wall*

Calculation of Soil Forces on Right Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.R} = 0.426$
$K_{a.R} = 0.271$
$\delta_R = 23 \cdot deg$
$\theta_R = 90 \cdot deg$
$ELE_{soil.R} = 171.5m$
$ELE_{Base.R} = 170.3m$
$ELE_{Base.L} = 170.3m$
$L_{hor} = 1.6m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.R} := ELE_{soil.R} - ELE_{Base.R} = 1.2\,m$ — *Height of soil on right side of wall*

$$H_{above.R} := \begin{cases} (ELE_{soil.R} - ELE_{water.R}) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0.00$$
Height of soil above water

$$H_{below.R} := \begin{cases} (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \ge ELE_{soil.R} \\ (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} > ELE_{Base.R} \wedge ELE_{water.R} < ELE_{soil.R} \\ 0 & \text{otherwise} \end{cases} = 1.2$$
Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_R \cdot \gamma_{s.R} \cdot H_{above.R}^2 \cdot B = 0\,kN$ — *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ \left(ELE_{Base.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0$$
Elevation of force above water

$F_{below1} := B \cdot K_R \cdot (\gamma_{s.R} \cdot H_{above.R}) \cdot H_{below.R} = 0\,kN$ — *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{2}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 170.9$$
Elevation of force

$F_{below2} := B \cdot K_R \cdot \frac{\gamma_{eff.R} \cdot H_{below.R}^2}{2} = 2.2\,kN$ — *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{3}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 170.7$$
Elevation of force

$F_{soil.R} := F_{above} + F_{below1} + F_{below2} = 2.2\,kN$ — *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} = 170.7\,m$$
Elevation of force

$F_{soil.hor.R} := F_{soil.R} \cdot \cos(\beta_R) = 2.2\,kN$ — *Horizontal component (positive is upstream/to the left)*

$F_{soil.ver.R} := F_{soil.R} \cdot \sin(\beta_R) = 0\,kN$

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.4\,m$ — *Moment arm for horizontal force*

$\boxed{MA_{ver} := L_{wall.base} + L_{slab.R} = 1.3\,m}$ — *Moment arm for vertical force*

$M_{soil.hor.R} := F_{soil.hor.R} \cdot MA_{hor} = 0.9\,kN \cdot m$ — *Moment from horizontal component*

$M_{soil.ver.R} := F_{soil.ver.R} \cdot MA_{ver} = 0\,kN \cdot m$ — *Moment from vertical component*

Calculation of Soil Forces on Right Side

Calculation of Soil Forces on Left Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.L} = 0.426$
$K_{a.L} = 0.271$
$\delta_L = 23 \cdot deg$
$\theta_L = 90 \cdot deg$
$ELE_{soil.L} = 171.2\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.L} := ELE_{soil.L} - ELE_{Base.L} = 0.9\,m$ *Height of soil on right side of wall*

$$H_{above.L} := \begin{cases} (ELE_{soil.L} - ELE_{water.L}) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0.90$$

Height of soil above water

$$H_{below.L} := \begin{cases} (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{soil.L} \\ (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{Base.L} \wedge ELE_{water.L} < ELE_{soil.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_L \cdot \gamma_{s.L} \cdot H_{above.L}^2 \cdot B = 31.4\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ \left(ELE_{Base.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 170.6$$

Elevation of force above water

$F_{below1} := B \cdot K_L \cdot (\gamma_{s.L} \cdot H_{above.L}) \cdot H_{below.L} = 0\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{2}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{below2} := B \cdot K_L \cdot \frac{\gamma_{eff.L} \cdot H_{below.L}^2}{2} = 0\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{3}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{soil.L} := F_{above} + F_{below1} + F_{below2} = 31.4\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \begin{cases} 0 & \text{if } F_{soil.L} = 0 \\ \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} & \text{otherwise} \end{cases} = 170.6$$

Elevation of force

$F_{soil.hor.L} := F_{soil.L} = 31.4\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.3\,m$ *Moment arm for horizontal force*

$M_{soil.hor.L} := F_{soil.hor.L} \cdot MA_{hor} = 9.4\,kN \cdot m$ *Moment from horizontal component*

▲ Calculation of Soil Forces on Left Side

▼ Hydrostatic Pressure on Right Side

$H_{water.R} := \begin{cases} (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \geq ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 2.7$ *Height of water acting on wall*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.R} = 173.000\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_R = 90 \cdot deg$

$F_{water.R} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.R}^2 = 35.8\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.R} + \frac{H_{water.R}}{3} = 171.2\,m$ *Elevation of resultant force*

$F_{water.hor.R} := F_{water.R} = 35.8\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.9\,m$ *Moment arm for horizontal force*

$M_{water.hor.R} := F_{water.hor.R} \cdot MA_{hor} = 32.2\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Right Side

Hydrostatic Pressure on Left Side

$H_{water.L} := \begin{cases} (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \geq ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$ *Height of water acting on wall*

$F_{water.L} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.L}^2 = 0\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.L} + \frac{H_{water.L}}{3} = 170.3\,m$ *Elevation of resultant force*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.L} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_L = 90 \cdot deg$

$F_{water.hor.L} := F_{water.L} = 0\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$ *Moment arm for horizontal force*

$M_{water.hor.L} := F_{water.hor.L} \cdot MA_{hor} = 0\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Left Side

Calculation of Uplift Pressure

$P_{U.L} := H_{water.L} \cdot \gamma_w = 0\,kPa$ *Uplift pressure at left side*

$P_{U.R} := H_{water.R} \cdot \gamma_w = 26.5\,kPa$ *Uplift pressure at right side*

$P_U(x) := \begin{cases} P_{U.R} + \frac{(P_{U.L} - P_{U.R})}{(L_{hor})} \cdot x & \text{if } x \leq L_{hor} \wedge x \geq 0 \\ 0 & \text{otherwise} \end{cases}$ *Uplift function. x=0 at right side.*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$ELE_{water.L} = 170.300\,m$
$ELE_{water.R} = 173.000\,m$

$F_U := \int_0^{L_{hor}} P_U(x) \cdot B\,dx = 21.2 \cdot kN$ *Total uplift force assuming uncracked section.*

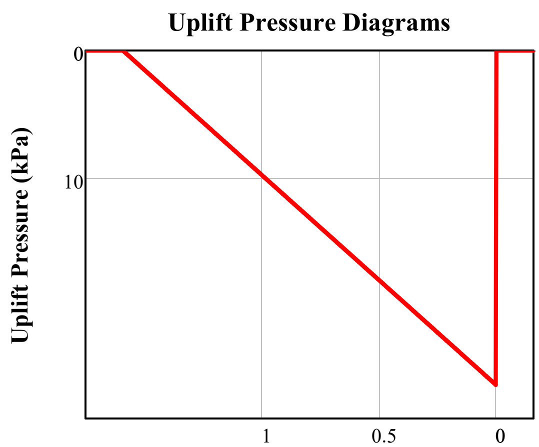
$$L_{Uf} := \begin{cases} 0 & \text{if } F_U = 0 \\ \left(\frac{1}{F_U} \int_0^{L_{hor}} P_U(x) \cdot x \cdot B \, dx \right) & \text{otherwise} \end{cases} = 0.53$$

Distance of uplife force from right side of base

$MA := L_{hor} - L_{Uf} = 1.1\,m$ — *Moment arm of uplift force*

$M_U := F_U \cdot MA = 22.6 \cdot kN \cdot m$ — *Moment from uplift*

$F_{U.ver} := -F_U = -21.2 \cdot kN$ — *Vertical component (down is positive)*



Calculation of Uplift Pressure

Force from Surcharge

$F_{q.R} := q_{surcharge.R} \cdot K_R \cdot H_{soil.R} \cdot B = 0$ — *Force on wall due to surcharge*

$ELE_F := ELE_{Base.R} + \frac{H_{soil.R}}{2} = 170.9\,m$ — *Elevation of resultant force*

$F_{q.hor.R} := F_{q.R} = 0$ — *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.6\,m$ — *Moment arm for horizontal component*

$M_{q.hor.R} := F_{q.hor.R} \cdot MA_{hor} = 0$ — *Moment from horizontal component*

$F_{q.ver.R} := q_{surcharge.R} \cdot L_{slab.R} \cdot B = 0 \cdot kN$

$MA_{ver} := L_{slab.L} + L_{wall.base} + \frac{L_{slab.R}}{2} = 1.1\,m$

$M_{q.ver.R} := F_{q.ver.R} \cdot MA_{ver} = 0$

Force from Surcharge

Weight of Water Above Section

Reference Coordinates of Structure

$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 0.300 \\ 0.300 \\ 0.600 \\ 0.600 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

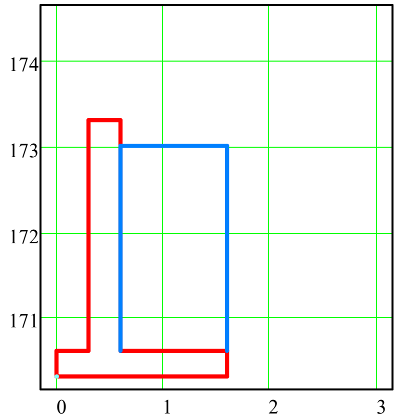
$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

Insert coordinates of shape of material above structure

$B_{water} := B = 1.000\,m$ *Width of material*

$$X_{water.L} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} m \qquad Y_{water.L} := \begin{pmatrix} ELE_{water.L} \\ ELE_{water.L} \\ ELE_{water.L} \\ ELE_{water.L} \end{pmatrix}$$

$$X_{water.R} := \begin{pmatrix} 0.6 \\ 0.6 \\ 1.6 \\ 1.6 \end{pmatrix} m \qquad Y_{water.R} := \begin{pmatrix} 170.60m \\ ELE_{water.R} \\ ELE_{water.R} \\ 170.60m \end{pmatrix}$$



$\Delta X_{water.L} := \max(X_{water.L}) - \min(X_{water.L}) = 0$

$\Delta X_{water.R} := \max(X_{water.R}) - \min(X_{water.R}) = 1\,m$

Calculation of Area and Centre of Gravity (Left Side)

$X := X_{water.L}$ $\qquad Y := Y_{water.L}$

$i := 1 .. length(X)$ $\qquad j := 1 .. length(X) + 1$

$X_{length(X)+1} := X_1$ $\qquad Y_{length(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\qquad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\qquad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3}\right]$$

$$A_{water.above.L} := \left|\sum_i Areainc_i\right| = 0\,m^2$$

$$X_{gwater.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

$$Y_{gwater.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Yginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

Calculation of Area and Centre of Gravity (Right Side)

$X := X_{water.R}$ $\qquad Y := Y_{water.R}$

$i := 1 .. length(X)$ $\qquad j := 1 .. length(X) + 1$

$X_{length(X)+1} := X_1$ $\qquad Y_{length(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\qquad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\qquad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3}\right]$ $\qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3}\right]$

$A_{water.above.R} := \left|\sum_i Areainc_i\right| = 2.4\,m^2$

$Xg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\frac{\sum_i Xginc_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 1.1$

$Yg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\frac{\sum_i Yginc_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 171.8$

Calculation of Forces

$W_{water.above.L} := (A_{water.above.L} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.L} = 0\,m$

$M_{water.above.L} := W_{water.above.L} \cdot MA_{Hor} = 0\,kN \cdot m$

$W_{water.above.R} := (A_{water.above.R} \cdot \gamma_w \cdot B_{water}) = 23.5 \cdot kN$

$MA_{Hor} := Xg_{water.above.R} = 1.1\,m$

$M_{water.above.R} := W_{water.above.R} \cdot MA_{Hor} = 25.9\,kN \cdot m$

Weight of Water Above Section

Weight of Material Above Section

$x_{sat} := x_{water.R} = 1\,m$

$x_{dry} := x_{soil.R} - x_{water.R} = 0\,m$

$\gamma_{above.L} := \gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$W_{above.L} := (A_{above.L} \cdot \gamma_{above.L} \cdot B) = 3.8 \cdot kN$ — *Weight of material above base of cantilever wall on left side*

$MA_{Hor} := Xg_{above.L} = 0.2\,m$ — *Moment arm*

$M_{above.L} := W_{above.L} \cdot MA_{Hor} = 0.6\,kN \cdot m$ — *Overturning moment*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$H_{above.L} = 0.90\,m$
$H_{below.L} = 0.00\,m$
$H_{above.R} = 0.00\,m$
$H_{below.R} = 1.20\,m$
$A_{above.L} = 0.18\,m^2$
$A_{above.R} = 0.9\,m^2$

$L_{hor} = 1.600\,m$

$B = 1.000\,m$

$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$

$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$

$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$

$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$\gamma_{above.R} := \gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$W_{above.R} := (A_{above.R} \cdot \gamma_{above.R} \cdot B) = 10.1 \cdot kN$

Weight of material above base of cantilever wall on right side

$MA_{Hor} := Xg_{above.R} = 1.1\,m$

$M_{above.R} := W_{above.R} \cdot MA_{Hor} = 11.1\,kN \cdot m$

Weight of Material Above Section

LC.2 - Summary of Forces

Dead Load:

$W_{conc} = 30.3 \cdot kN$ $\quad$ $M_{conc} = 17.6 \cdot kN \cdot m$

Soil:

$F_{soil.hor.R} = 2.2\,kN$ $\quad$ $M_{soil.hor.R} = 0.9\,kN \cdot m$

$F_{soil.ver.R} = 0\,kN$ $\quad$ $M_{soil.ver.R} = 0\,kN \cdot m$

$F_{soil.hor.L} = 31.4\,kN$ $\quad$ $M_{soil.hor.L} = 9.4\,kN \cdot m$

Material Above Section:

$W_{above.L} = 3.8\,kN$ $\quad$ $M_{above.L} = 0.6\,kN \cdot m$

$W_{above.R} = 10.1\,kN$ $\quad$ $M_{above.R} = 11.1\,kN \cdot m$

Water Above Section:

$W_{water.above.L} = 0\,kN$ $\quad$ $M_{water.above.L} = 0\,kN \cdot m$

$W_{water.above.R} = 23.5\,kN$ $\quad$ $M_{water.above.R} = 25.9\,kN \cdot m$

Uplift:

$F_{U.ver} = -21.2 \cdot kN$ $\quad$ $M_U = 22.6\,kN \cdot m$

Hydrostatic:

$F_{water.hor.R} = 35.8\,kN$ $\quad$ $M_{water.hor.R} = 32.2\,kN \cdot m$

$F_{water.hor.L} = 0\,kN$ $\quad$ $M_{water.hor.L} = 0\,kN \cdot m$

Surcharge:

$F_{q.hor.R} = 0$ $\quad$ $M_{q.hor.R} = 0$

$F_{q.ver.R} = 0 \cdot kN$ $\quad$ $M_{q.ver.R} = 0$

LC.2 - Combine Forces and Moments

$F_{hor.drive} := F_{soil.hor.R} + F_{water.hor.R} + F_{q.hor.R} = 37.9\,kN$

$F_{hor.resist} := F_{soil.hor.L} + F_{water.hor.L} = 31.4\,kN$

$F_{ver} := W_{conc} + F_{U.ver} + W_{above.L} + W_{above.R} + W_{water.above.L} + W_{water.above.R} + F_{soil.ver.R} + F_{q.ver.R} = 46.5 \cdot kN$

$M_{stab} := M_{conc} + M_{water.hor.L} + M_{soil.hor.L} + M_{above.L} + M_{above.R} + M_{water.above.L} + M_{water.above.R} + M_{soil.ver.R} + M_{q.ver.R} = 64.5\,kN \cdot m$

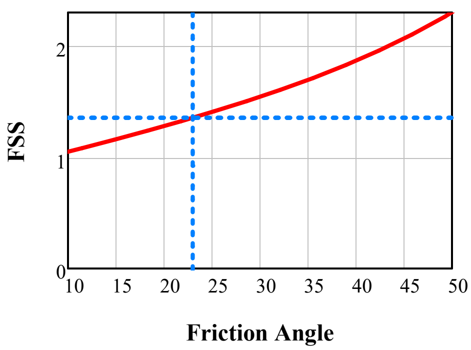
$M_o := M_{soil.hor.R} + M_{water.hor.R} + M_{q.hor.R} + M_U = 55.7\,kN \cdot m$

$M_{net} := M_{stab} - M_o = 8.9\,kN \cdot m$

LC.2 - Sliding

$$FSS(\theta) := \frac{F_{hor.resist} + F_{ver} \cdot \tan(\theta) + c \cdot L_{hor} \cdot B}{F_{hor.drive}}$$

$FSS(\phi_{cf}) = 1.35$



LC.2 - Overturning, Resultant and Bearing Stresses

$$FSOT := \frac{M_{stab}}{M_o} = 1.16$$

$$x_o := \frac{M_{net}}{F_{ver}} = 0.19\,m$$

$$E := x_o - \frac{L_{hor}}{2} = -0.61\,m$$

Stress Calculations

$q_{max} = 162.3 \cdot kPa$ *Maximum bearing stress before iterative cracked base analysis* $\sigma_n(0m) = 162.3\,kPa$

$q_{min} := (\sigma_n(L_{hor})) = 0\,kPa$

$L_{comp} = 0.57\,m$ *Length of base in compression before iterative cracked base analysis* $\sigma_n(L_{slab.L}) = 77.4\,kPa$

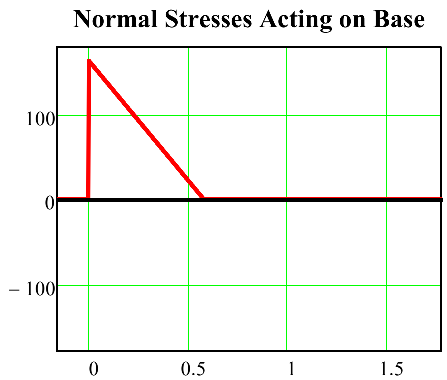
$L_{tens} = 0.00\,m$ *Length of base in tension before iterative cracked base analysis* $\sigma_n(L_{slab.L} + L_{wall.base}) = 0\,kPa$

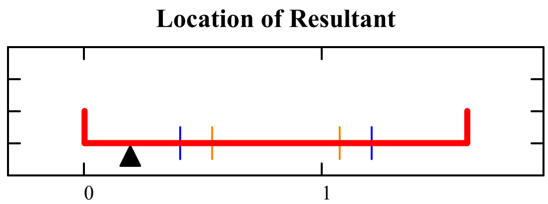
$L_{crack} = 1.03\,m$ *Length of crack between concrete and base before iterative cracked base analysis* $\sigma_n(L_{hor}) = 0\,kPa$

$\frac{L_{comp}}{L_{hor}} = 35.8 \cdot \%$ *% of Base in Compression*

$\frac{L_{tens}}{L_{hor}} = 0 \cdot \%$ *% of Base in Tension*

$\frac{L_{crack}}{L_{hor}} = 64.2 \cdot \%$ *% of Base Cracked*





Red lines indicate extent of structure, Blue lines indicate middle half of base, orange lines indicae middle third of base

Store Results for Summary

$LC := 2$

$R_{FSS_{LC}} := FSS(\phi_{cf}) = ...$

$R_{E_{LC}} := E = ...$

$R_{L.comp_{LC}} := L_{comp} = ...$

$R_{L.crack_{LC}} := L_{crack} = ...$

$R_{L.tens_{LC}} := L_{tens} = ...$

$R_{xo_{LC}} := x_o = ...$

$R_{F.hor.drive_{LC}} := F_{hor.drive} = ...$

$R_{F.hor.resist_{LC}} := F_{hor.resist} = ...$

$R_{F.ver_{LC}} := F_{ver} = ...$

$R_{q.max_{LC}} := q_{max} = ...$

$R_{comp.percent_{LC}} := \frac{L_{comp}}{L_{hor}} = ...$

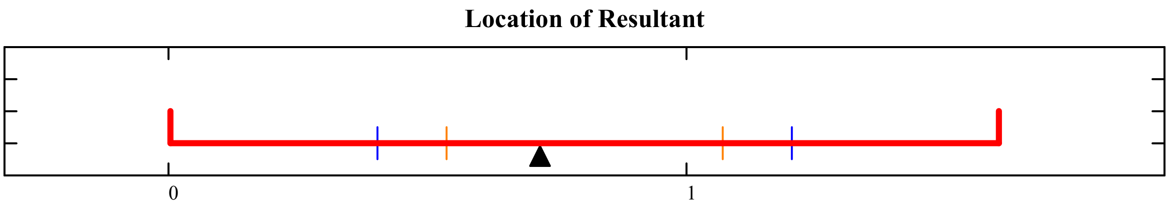
$R_{q.min_{LC}} := q_{min} = ...$

Store Results for Summary

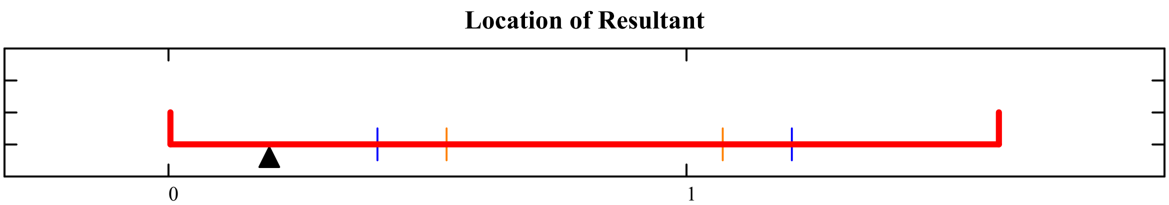
Results of Analysis

	FSS (Φ.cf)	E (m)	x.o (m)	L.comp (m)	% of Base in Compression	L.crack (m)	F.hor.drive (kN)	F.hor.resist (kN)	F.ver (kN)	q.max (kPa)
LC.1 - Normal Water Level (Usual)	3.16	-0.09	0.71	1.60	100%	0.00	8.9	3.6	57.8	47.8
LC.2 - IDF Water Level (Unusual)	1.35	-0.61	0.19	0.57	36%	1.03	37.9	31.4	46.5	162.3

LC 1 - Normal Water Level



LC 2 - IDF Water Level



KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	23-4168-001	**Project Name :**	Churchville Condition Assessment		
File No. :		**Discipline :**	Structural Engineering		
Calculation Title :	**Floodwall Stability Analysis – Configuration 2**				
Calculation No. :	CIV-002	**Prepared by :**	JL	**Date :**	2023-12-05
No. of Sheets :		**Checked by :**	YF	**Date :**	2023-12-06
Supersedes Calc. No. :		**Approved by :**		**Date :**	

Calculation Description :

Stability analysis of the gravity retaining section.

Related Design Concept :

Reference Codes and Standards :
1. USACE – Retaining and Flood Walls EM 1110-2-2502, 1989
2. Structural Design and Factors of Safety – Technical Bulletin Ontario Ministry of Natural Resources (August 2011)

ENGINEER'S SEAL

Rev. #	**Rev. Description**	**Rev. Author**	**Date Revised**	**Checked by**	**Approved by**	**Approved Date**

Properties of Materials and Soils, Seismic Coefficients

$k_h := 0.151$ — *Unitless ratio of horizontal and vertical accelerations to the acceleration due to gravity.*

$k_v := \frac{2}{3} \cdot k_h = 0.101$

$\gamma_w := 9.81 \frac{kN}{m^3}$ — *Water density*

$\gamma_{conc} := 23.5 \cdot \frac{kN}{m^3}$ — *Concrete density*

$\phi_{cf} := 23 \cdot deg$ — *Friction angle of concrete/foundation interface*

$c := 0MPa$ — *Cohesion at concrete/foundation interface (generally set to 0)*

$f_{t.cf} := 0MPa$ — *Tensile strength at concrete/rock interface (generally set to 0). This is a negative number.*

Right (Backfill) Side

$\phi'_R := 35deg$ — *Effective friction angle*

$\gamma_{s.R} := 21 \frac{kN}{m^3}$ — *Moist (bulk) unit weight of soils*

$\gamma_{sat.R} := \gamma_{s.R}$ — *Saturated unit weight of soils*

$\gamma_{eff.R} := \gamma_{sat.R} - \gamma_w = 11.2 \cdot \frac{kN}{m^3}$ — *Submerged (effective) unit weight of soils*

$\delta_R := \phi_{cf} = 23 \cdot deg$ — *Friction angle for interface between wall and soil*

Left (Upstream) Side

$\phi'_L := 35deg$ — *Effective friction angle*

$\gamma_{s.L} := 21 \frac{kN}{m^3}$ — *Moist (bulk) unit weight of soils*

$\gamma_{sat.L} := \gamma_{s.L}$ — *Saturated unit weight of soils*

$\gamma_{eff.L} := \gamma_{sat.L} - \gamma_w = 11.2 \cdot \frac{kN}{m^3}$ — *Submerged (effective) unit weight of soils*

$\delta_L := \phi_{cf} = 23 \cdot deg$ — *Friction angle for interface between wall and soil*

Structure Geometry

Input

Note: Enter structure geometry as series of points on X-Y grid. Align structure so that upstream is on the left side. Structure outline is "closed" automatically (last point is assigned same values as first). Ensure that values of ELEusl and ELEdsl are adjusted to correspond with the lowest elevation on left and right sides.

$B := 1m$ — *Set unit width of structure*

$ELE_{top} := 173.3 \cdot m$ — *Elevation of top of wall*

$ELE_{Base.L} := 170.3m$ — *Lowest upstream elevation (left side)*

$ELE_{Base.R} := ELE_{Base.L} = 170.3\,m$ — *Lowest downstream elevation (right side)*

$ELE_{soil.R} := ELE_{top} = 173.3\,m$ — *Elevation of top of soil*

$ELE_{soil.L} := 171.2m = 171.2\,m$

$$X_{struct} := \begin{pmatrix} 0 \\ 0 \\ 1 \\ 1 \\ 1.3 \\ 1.3 \\ 1.6 \\ 1.6 \end{pmatrix} \cdot m \qquad Y_{struct} := \begin{pmatrix} ELE_{Base.L} \cdot m^{-1} \\ 170.60 \\ 170.60 \\ ELE_{top} \cdot m^{-1} \\ ELE_{top} \cdot m^{-1} \\ 170.60 \\ 170.60 \\ ELE_{Base.L} \cdot m^{-1} \end{pmatrix} \cdot m$$

Input X & Y coordinates

$\beta_R := 0deg$ $\quad \beta_L := 0deg$ — *Slope of soil, measured from horizontal (sloping upward is positive)*

$\theta_L := 90deg$ $\quad \theta_R := 90deg$ — *Slope of retaining wall face, measured from horizontal (90deg is vertical, battered walls are less than 90deg)*
For cantilever walls, set to 90 as soil pressure acts on a vertical plane at end of base slab

Even though backfill face is angled, Rankine cannot be used for angled face.
that is why a 90 deg angle is currently shown

$L_{hor} := \max(X_{struct}) - \min(X_{struct}) = 1.6\,m$ — *Horizontal projection of base*

$L_{slab.L} := 1m$

$L_{slab.R} := 0.3m$ — *Length of slab beyond the stemwall on left and right sides*

$t_{slab.L} := 0.3m$

$t_{slab.R} := 0.3m$ — *Thickness of slab on left and right sides*

$L_{wall.base} := L_{hor} - L_{slab.L} - L_{slab.R} = 0.3\,m$ — *Thickness of stemwall at base*

Input

Plot Functions

$X := X_{struct}$ $\qquad Y := Y_{struct}$

$i := 1 .. \text{length}(X)$ $\qquad j := 1 .. \text{length}(X) + 1$ — *Functions to automatically "close" the structure*

$X_{\text{length}(X)+1} := X_1$ $\qquad Y_{\text{length}(Y)+1} := Y_1$

$maxdim := \max[(\max(X) - \min(X)), (\max(Y) - \min(Y))] = 3\,m$ — *Sets the extents of both axis to the maximum length of the structure in the X or Y direction*

$minydim := \min(ELE_{Base.L}, ELE_{Base.R}) = 170.3\,m$

Functions for resultant plot

$Base_{heel.x} := \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ $Base_{heel.y} := \begin{pmatrix} 1 \\ 0 \end{pmatrix}$

$Base_{third1.x} := \begin{pmatrix} \frac{L_{hor}}{3} \\ \frac{L_{hor}}{3} \end{pmatrix}$ $Base_{third1.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$Base_{toe.x} := \begin{pmatrix} L_{hor} \\ L_{hor} \end{pmatrix}$ $Base_{toe.y} := \begin{pmatrix} 1 \\ 0 \end{pmatrix}$

$Base_{half1.x} := \begin{pmatrix} \frac{L_{hor}}{4} \\ \frac{L_{hor}}{4} \end{pmatrix}$ $Base_{half1.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$Base_{half2.x} := \begin{pmatrix} \frac{3L_{hor}}{4} \\ \frac{3L_{hor}}{4} \end{pmatrix}$ $Base_{half2.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

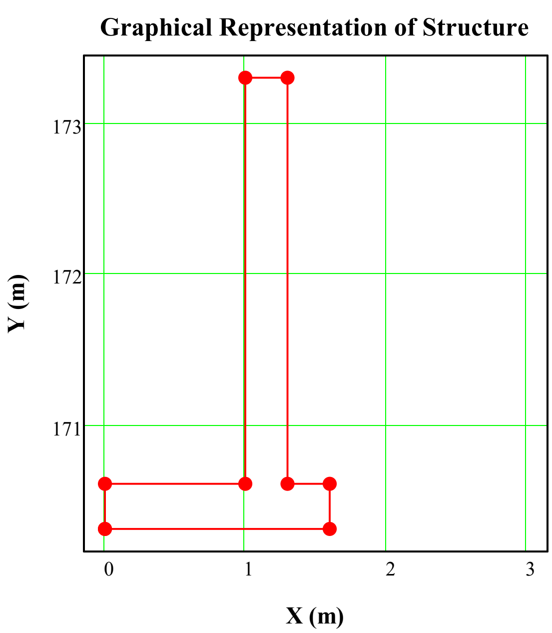
$Base_{third2.x} := \begin{pmatrix} \frac{2L_{hor}}{3} \\ \frac{2L_{hor}}{3} \end{pmatrix}$ $Base_{third2.y} := \begin{pmatrix} .5 \\ -.5 \end{pmatrix}$

$ground_y := \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ $ground_x := \begin{pmatrix} 0 \\ L_{hor} \end{pmatrix}$

$xo_y := -.4$

Plot Functions

Graphical Representation of Structure



Computation of Area and Center of Gravity

$Xstruct := X$ *Assign coordinates to a seperate array to be used for plotting later in worksheet*

$Ystruct := Y$

$deltax_i := X_{i+1} - X_i$

$xplusx_i := X_{i+1} + X_i$

$deltay_i := Y_{i+1} - Y_i$

$yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right]$$

$$Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$$

$$Area := \left|\sum_i Areainc_i\right| = 1.3\,m^2$$ *Cross sectional area of the Structure*

$$Xg_{struct} := \left|\frac{\sum_i Xginc_i}{Area}\right| = 1.02\,m$$ *X coordinate of centroid of structure*

$$Yg_{struct} := \left|\frac{\sum_i Yginc_i}{Area}\right| = 171.392\,m$$ *evation of centroid of structure*

Computation of Area and Center of Gravity

Weight of Structure

$V_{conc} := Area \cdot B = 1.29 \cdot m^3$ *Volume of concrete per unit width of structure*

$W_{conc} := V_{conc} \cdot \gamma_{conc} = 30.3 \cdot kN$ *Dead load of concrete in structure*

$MA_{conc} := Xg_{struct} = 1\,m$ *Moment arm if wall is rotating counterclockwise about toe on upstream side*

$M_{conc} := W_{conc} \cdot MA_{conc} = 30.9 \cdot kN \cdot m$

$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$

$Area = 1.3\,m^2$

$B = 1\,m$

$L_{hor} = 1.6\,m$

$Xg_{struct} = 1.02\,m$

$Yg_{struct} = 171.39\,m$

Lateral Earth Pressure Coefficients

$\beta_R = 0 \cdot deg$
$\phi'_R = 35 \cdot deg$
$\theta_R = 90 \cdot deg$
$\delta_R = 23 \cdot deg$
$\beta_L = 0 \cdot deg$
$\phi'_L = 35 \cdot deg$
$\theta_L = 90 \cdot deg$
$\delta_L = 23 \cdot deg$

$K_o(\phi) := (1 - \sin(\phi))$ *At-rest pressure*

$$K_a(\phi, \beta) := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

Rankine active/passive pressure coefficients

$$K_p(\phi) := \tan\left(45deg + \frac{\phi}{2}\right)^2$$

$$\psi(k_h, k_v) := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

$$K_{ae}(\beta, \phi', \theta, \delta, \psi) := \frac{\sin(\phi' + \theta - \psi)^2}{\cos(\psi)\sin(\theta)^2 \cdot \sin(\theta - \delta - \psi) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi') \cdot \sin(\phi' - \beta - \psi)}{\sin(\theta - \delta - \psi) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_{o.R} := K_o(\phi'_R) = 0.426$ *Calculation of lateral pressure coefficients*

$K_{a.R} := K_a(\phi'_R, \beta_R) = 0.271$

$K_{p.R} := K_p(\phi'_R) = 3.7$

$K_{o.L} := K_o(\phi'_L) = 0.426$

$K_{a.L} := K_a(\phi'_L, 0deg) = 0.271$

$K_{p.L} := K_p(\phi'_L) = 3.7$

$\psi_R := \psi(k_h, 0) = 8.587 \cdot deg$ *Seismic intertia angle (ignoring vertical component of earthquake to be conservative)*

$\psi_L := \psi(k_h, 0) = 8.6 \cdot deg$

$K_{ae.R} := K_{ae}(\beta_R, \phi'_R, \theta_R, \delta_R, \psi_R) = 0.343$ *Coefficient of (active) lateral pressure using Mononobe-Okabe method, for earthquake loading*

$$K_{oe.R} := \frac{K_{ae.R}}{K_{a.R}} \cdot K_{o.R} = 0.5$$

Coefficient of (at-rest) lateral pressure, for earthquake loading

$K_{ae.L} := K_{ae}(\beta_L, \phi'_L, \theta_L, \delta_L, \psi_L) = 0.343$

$$K_{oe.L} := \frac{K_{ae.L}}{K_{a.L}} \cdot K_{o.L} = 0.5$$

Weight of Material on Top of Section

Input coordinates

Reference Coordinates of Structure

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{soil.R} = 173.300\,m$
$ELE_{soil.L} = 171.200\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

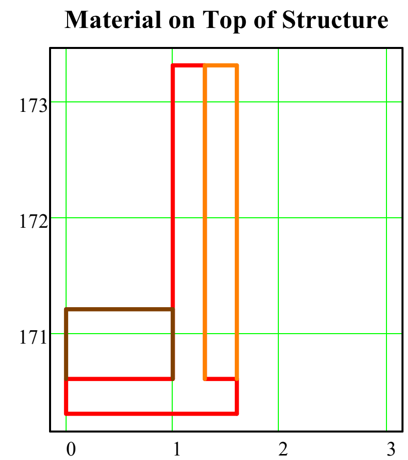
$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 1.000 \\ 1.000 \\ 1.300 \\ 1.300 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

Insert coordinates of shape of material above structure

$B_{above} := B = 1.000\,m$ *Width of material*

$$X_{above.L} := \begin{pmatrix} 0 \\ 0 \\ 1 \\ 1 \end{pmatrix} m \qquad Y_{above.L} := \begin{pmatrix} 170.60m \\ ELE_{soil.L} \\ ELE_{soil.L} \\ 170.60m \end{pmatrix}$$

$$X_{above.R} := \begin{pmatrix} 1.3 \\ 1.3 \\ 1.6 \\ 1.6 \end{pmatrix} m \qquad Y_{above.R} := \begin{pmatrix} 170.60m \\ ELE_{soil.R} \\ ELE_{soil.R} \\ 170.60m \end{pmatrix}$$



$x_{soil.L} := \max(X_{above.L}) - \min(X_{above.L}) = 1\,m$

$x_{soil.R} := \max(X_{above.R}) - \min(X_{above.R}) = 0.3\,m$

Input coordinates

Calculations

$X := X_{above.L}$ $\quad$ $Y := Y_{above.L}$

$i := 1..\,\mathrm{length}(X)$ $\quad$ $j := 1..\,\mathrm{length}(X) + 1$

$X_{\mathrm{length}(X)+1} := X_1$ $\quad$ $Y_{\mathrm{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3} \right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3} \right]$$

$$A_{above.L} := \left| \sum_i Areainc_i \right| = 0.6\,m^2$$

$$Xg_{above.L} := \begin{cases} 0 & \text{if } A_{above.L} = 0 \\ \dfrac{\left|\sum_i Xginc_i\right|}{A_{above.L}} & \text{otherwise} \end{cases} = 0.5$$

$$Yg_{above.L} := \begin{cases} 0 & \text{if } A_{above.L} = 0 \\ \dfrac{\left|\sum_i Yginc_i\right|}{A_{above.L}} & \text{otherwise} \end{cases} = 170.9$$

$X := X_{above.R}$ $\quad$ $Y := Y_{above.R}$

$i := 1 .. \text{length}(X)$ $\quad$ $j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$$

$$A_{above.R} := \left|\sum_i Areainc_i\right| = 0.8\,m^2$$

$$Xg_{above.R} := \begin{cases} 0 & \text{if } A_{above.R} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{above.R}}\right| & \text{otherwise} \end{cases} = 1.45$$

$$Yg_{above.R} := \begin{cases} 0 & \text{if } A_{above.R} = 0 \\ \left|\dfrac{\sum_i Yginc_i}{A_{above.R}}\right| & \text{otherwise} \end{cases} = 171.95$$

Calculations

Load Combination 1

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

$K_R := K_{a.R} = 0.271$ — *Coefficient of lateral stress (assign to Ko.R, Ka.R or Kp.R)*

$K_L := K_{p.L} = 3.690$

$ELE_{water.R} := 170.3m$ — *Elevation of water level on right side*

$ELE_{water.L} := 170.3m$ — *Elevation of water level on left side*

$q_{surcharge.R} := 4.8kPa$ — *Uniform surface stress , assumed to extend for an infinite length behind wall*

Calculation of Soil Forces on Right Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.R} = 0.426$
$K_{a.R} = 0.271$
$\delta_R = 23 \cdot deg$
$\theta_R = 90 \cdot deg$
$ELE_{soil.R} = 173.3\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.R} := ELE_{soil.R} - ELE_{Base.R} = 3\,m$ *Height of soil on right side of wall*

$$H_{above.R} := \begin{vmatrix} (ELE_{soil.R} - ELE_{water.R}) \text{ if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ (ELE_{soil.R} - ELE_{Base.R}) \text{ if } ELE_{water.R} \le ELE_{Base.R} \\ 0 \text{ otherwise} \end{vmatrix} = 3.00$$

Height of soil above water

$$H_{below.R} := \begin{vmatrix} (ELE_{soil.R} - ELE_{Base.R}) \text{ if } ELE_{water.R} \ge ELE_{soil.R} \\ (ELE_{water.R} - ELE_{Base.R}) \text{ if } ELE_{water.R} > ELE_{Base.R} \wedge ELE_{water.R} < ELE_{soil.R} \\ 0 \text{ otherwise} \end{vmatrix} = 0$$

Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_R \cdot \gamma_{s.R} \cdot H_{above.R}^2 \cdot B = 25.6\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{vmatrix} \left(ELE_{water.R} + \frac{H_{above.R}}{3}\right) \text{ if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ \left(ELE_{Base.R} + \frac{H_{above.R}}{3}\right) \text{ if } ELE_{water.R} \le ELE_{Base.R} \\ 0 \text{ otherwise} \end{vmatrix} = 171.3$$

Elevation of force above water

$F_{below1} := B \cdot K_R \cdot (\gamma_{s.R} \cdot H_{above.R}) \cdot H_{below.R} = 0\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{vmatrix} \left(ELE_{Base.R} + \frac{H_{below.R}}{2}\right) \text{ if } ELE_{water.R} > ELE_{Base.R} \\ 0 \text{ otherwise} \end{vmatrix} = 0$$

Elevation of force

$F_{below2} := B \cdot K_R \cdot \frac{\gamma_{eff.R} \cdot H_{below.R}^2}{2} = 0\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{vmatrix} \left(ELE_{Base.R} + \frac{H_{below.R}}{3}\right) \text{ if } ELE_{water.R} > ELE_{Base.R} \\ 0 \text{ otherwise} \end{vmatrix} = 0$$

Elevation of force

$F_{soil.R} := F_{above} + F_{below1} + F_{below2} = 25.6\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} = 171.3\,m$$

Elevation of force

$F_{soil.hor.R} := F_{soil.R} \cdot \cos(\beta_R) = 25.6\,kN$ *Horizontal component (positive is upstream/to the left)*

$F_{soil.ver.R} := F_{soil.R} \cdot \sin(\beta_R) = 0\,kN$

$MA_{hor} := ELE_F - ELE_{Base.L} = 1\,m$ *Moment arm for horizontal force*

$\boxed{MA_{ver} := L_{wall.base} + L_{slab.R} = 0.6\,m}$ *Moment arm for vertical force*

$M_{soil.hor.R} := F_{soil.hor.R} \cdot MA_{hor} = 25.6\,kN \cdot m$ *Moment from horizontal component*

$M_{soil.ver.R} := F_{soil.ver.R} \cdot MA_{ver} = 0\,kN \cdot m$ *Moment from vertical component*

▲ Calculation of Soil Forces on Right Side

▼ Calculation of Soil Forces on Left Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.L} = 0.426$
$K_{a.L} = 0.271$
$\delta_L = 23 \cdot deg$
$\theta_L = 90 \cdot deg$
$ELE_{soil.L} = 171.2\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.L} := ELE_{soil.L} - ELE_{Base.L} = 0.9\,m$ — *Height of soil on right side of wall*

$$H_{above.L} := \begin{cases} (ELE_{soil.L} - ELE_{water.L}) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0.90$$

Height of soil above water

$$H_{below.L} := \begin{cases} (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{soil.L} \\ (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{Base.L} \wedge ELE_{water.L} < ELE_{soil.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_L \cdot \gamma_{s.L} \cdot H_{above.L}^2 \cdot B = 31.4\,kN$ — *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ \left(ELE_{Base.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 170.6$$

Elevation of force above water

$F_{below1} := B \cdot K_L \cdot (\gamma_{s.L} \cdot H_{above.L}) \cdot H_{below.L} = 0\,kN$ — *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{2}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{below2} := B \cdot K_L \cdot \frac{\gamma_{eff.L} \cdot H_{below.L}^2}{2} = 0\,kN$ — *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{3}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$

Elevation of force

$F_{soil.L} := F_{above} + F_{below1} + F_{below2} = 31.4\,kN$ — *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \begin{cases} 0 & \text{if } F_{soil.L} = 0 \\ \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} & \text{otherwise} \end{cases} = 170.6$$

Elevation of force

$F_{soil.hor.L} := F_{soil.L} = 31.4\,kN$ — *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.3\,m$ — *Moment arm for horizontal force*

$M_{soil.hor.L} := F_{soil.hor.L} \cdot MA_{hor} = 9.4\,kN \cdot m$ — *Moment from horizontal component*

▲ Calculation of Soil Forces on Left Side

▼ Hydrostatic Pressure on Right Side

$H_{water.R} := \begin{cases} (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \geq ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 0$

Height of water acting on wall

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_R = 90 \cdot deg$

$F_{water.R} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.R}^2 = 0\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.R} + \frac{H_{water.R}}{3} = 170.3\,m$ *Elevation of resultant force*

$F_{water.hor.R} := F_{water.R} = 0\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$ *Moment arm for horizontal force*

$M_{water.hor.R} := F_{water.hor.R} \cdot MA_{hor} = 0\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Right Side

Hydrostatic Pressure on Left Side

$H_{water.L} := \begin{cases} (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \geq ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$

Height of water acting on wall

$F_{water.L} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.L}^2 = 0\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.L} + \frac{H_{water.L}}{3} = 170.3\,m$ *Elevation of resultant force*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.L} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_L = 90 \cdot deg$

$F_{water.hor.L} := F_{water.L} = 0\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$ *Moment arm for horizontal force*

$M_{water.hor.L} := F_{water.hor.L} \cdot MA_{hor} = 0\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Left Side

Calculation of Uplift Pressure

$P_{U.L} := H_{water.L} \cdot \gamma_w = 0\,kPa$ *Uplift pressure at left side*

$P_{U.R} := H_{water.R} \cdot \gamma_w = 0\,kPa$ *Uplift pressure at right side*

$P_U(x) := \begin{cases} P_{U.R} + \frac{(P_{U.L} - P_{U.R})}{(L_{hor})} \cdot x & \text{if } x \leq L_{hor} \wedge x \geq 0 \\ 0 & \text{otherwise} \end{cases}$ *Uplift function. x=0 at right side.*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$ELE_{water.L} = 170.300\,m$
$ELE_{water.R} = 170.300\,m$

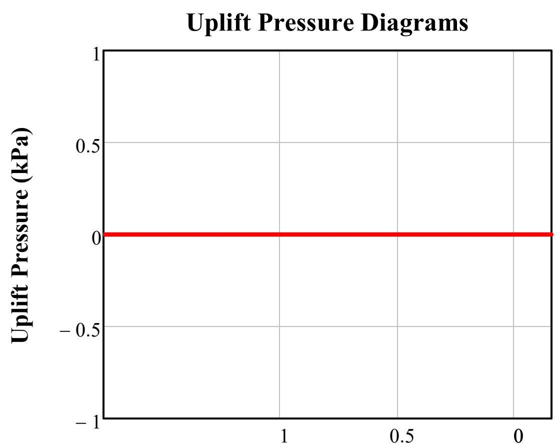
$F_U := \int_0^{L_{hor}} P_U(x) \cdot B\,dx = 0 \cdot kN$ *Total uplift force assuming uncracked section.*

$$L_{Uf} := \begin{cases} 0 & \text{if } F_U = 0 \\ \left(\frac{1}{F_U}\int_0^{L_{hor}} P_U(x)\cdot x\cdot B\,dx\right) & \text{otherwise} \end{cases} = 0$$ *Distance of uplife force from right side of base*

$MA := L_{hor} - L_{Uf} = 1.6\,m$ *Moment arm of uplift force*

$M_U := F_U \cdot MA = 0 \cdot kN \cdot m$ *Moment from uplift*

$F_{U.ver} := -F_U = 0 \cdot kN$ *Vertical component (down is positive)*



Calculation of Uplift Pressure

Force from Surcharge

$F_{q.R} := q_{surcharge.R} \cdot K_R \cdot H_{soil.R} \cdot B = 3.9\,kN$ *Force on wall due to surcharge*

$ELE_F := ELE_{Base.R} + \frac{H_{soil.R}}{2} = 171.8\,m$ *Elevation of resultant force*

$F_{q.hor.R} := F_{q.R} = 3.9\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 1.5\,m$ *Moment arm for horizontal component*

$M_{q.hor.R} := F_{q.hor.R} \cdot MA_{hor} = 5.9\,kN \cdot m$ *Moment from horizontal component*

$F_{q.ver.R} := q_{surcharge.R} \cdot L_{slab.R} \cdot B = 1.4 \cdot kN$

$MA_{ver} := L_{slab.L} + L_{wall.base} + \frac{L_{slab.R}}{2} = 1.5\,m$

$M_{q.ver.R} := F_{q.ver.R} \cdot MA_{ver} = 2.1\,kN \cdot m$

Force from Surcharge

Weight of Water Above Section

Reference Coordinates of Structure

$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 1.000 \\ 1.000 \\ 1.300 \\ 1.300 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

$ELE_{Base.L} = 170.300m$
$ELE_{Base.R} = 170.300m$
$ELE_{top} = 173.300m$
$L_{hor} = 1.600m$
$B = 1.000m$

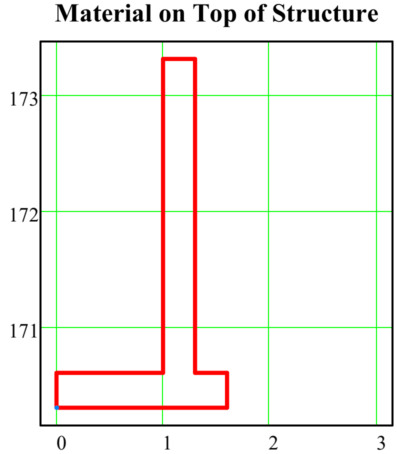
Insert coordinates of shape of material above structure

$B_{water} := B = 1.000m$ *Width of material*

$$X_{water.L} := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} m \qquad Y_{water.L} := \begin{pmatrix} ELE_{water.L} \\ ELE_{water.L} \\ ELE_{water.L} \end{pmatrix}$$

$$X_{water.R} := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} m \qquad Y_{water.R} := \begin{pmatrix} ELE_{water.R} \\ ELE_{water.R} \\ ELE_{Base.R} \end{pmatrix}$$

Material on Top of Structure



$x_{water.L} := \max(X_{water.L}) - \min(X_{water.L}) = 0$

$x_{water.R} := \max(X_{water.R}) - \min(X_{water.R}) = 0$

Calculation of Area and Centre of Gravity (Left Side)

$X := X_{water.L}$ $\quad$ $Y := Y_{water.L}$

$i := 1..\,\text{length}(X)$ $\quad$ $j := 1..\,\text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$\text{deltax}_i := X_{i+1} - X_i$ $\quad$ $\text{deltay}_i := Y_{i+1} - Y_i$

$\text{xplusx}_i := X_{i+1} + X_i$ $\quad$ $\text{yplusy}_i := Y_{i+1} + Y_i$

$\text{Areainc}_i := 0.5 \cdot \left(\text{deltay}_i \cdot \text{xplusx}_i\right)$

$$\text{Yginc}_i := \frac{\text{deltax}_i}{8} \cdot \left[\left(\text{yplusy}_i\right)^2 + \frac{\left(\text{deltay}_i\right)^2}{3}\right] \qquad \text{Xginc}_i := \frac{\text{deltay}_i}{8} \cdot \left[\left(\text{xplusx}_i\right)^2 + \frac{\left(\text{deltax}_i\right)^2}{3}\right]$$

$$A_{water.above.L} := \left|\sum_i \text{Areainc}_i\right| = 0\,m^2$$

$$Xg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i \text{Xginc}_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

$$Yg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i \text{Yginc}_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

Calculation of Area and Centre of Gravity (Right Side)

$X := X_{water.R}$ $\quad$ $Y := Y_{water.R}$

$i := 1..\,\text{length}(X)$ $\quad$ $j := 1..\,\text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad$ $Y_{\text{length}(Y)+1} := Y_1$

$\text{deltax}_i := X_{i+1} - X_i$ $\quad$ $\text{deltay}_i := Y_{i+1} - Y_i$

$\text{xplusx}_i := X_{i+1} + X_i$ $\quad$ $\text{yplusy}_i := Y_{i+1} + Y_i$

$\text{Areainc}_i := 0.5 \cdot \left(\text{deltay}_i \cdot \text{xplusx}_i\right)$

$$\text{Yginc}_i := \frac{\text{deltax}_i}{8} \cdot \left[\left(\text{yplusy}_i\right)^2 + \frac{\left(\text{deltay}_i\right)^2}{3}\right] \qquad \text{Xginc}_i := \frac{\text{deltay}_i}{8} \cdot \left[\left(\text{xplusx}_i\right)^2 + \frac{\left(\text{deltax}_i\right)^2}{3}\right]$$

$$A_{water.above.R} := \left|\sum_i \text{Areainc}_i\right| = 0\,m^2$$

$$Xg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\dfrac{\sum_i \text{Xginc}_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 0$$

$$Yg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \dfrac{\sum_i Yginc_i}{A_{water.above.R}} & \text{otherwise} \end{cases} = 0$$

Calculation of Forces

$W_{water.above.L} := (A_{water.above.L} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.L} = 0\,m$

$M_{water.above.L} := W_{water.above.L} \cdot MA_{Hor} = 0\,kN \cdot m$

$W_{water.above.R} := (A_{water.above.R} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.R} = 0\,m$

$M_{water.above.R} := W_{water.above.R} \cdot MA_{Hor} = 0\,kN \cdot m$

Weight of Water Above Section

Weight of Material Above Section

$x_{sat} := x_{water.R} = 0$

$x_{dry} := x_{soil.R} - x_{water.R} = 0.3\,m$

$\gamma_{above.L} := \gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$W_{above.L} := (A_{above.L} \cdot \gamma_{above.L} \cdot B) = 12.6 \cdot kN$ — *Weight of material above base of cantilever wall on left side*

$MA_{Hor} := Xg_{above.L} = 0.5\,m$ — *Moment arm*

$M_{above.L} := W_{above.L} \cdot MA_{Hor} = 6.3\,kN \cdot m$ — *Overturning moment*

$\gamma_{above.R} := \gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$

$W_{above.R} := (A_{above.R} \cdot \gamma_{above.R} \cdot B) = 17 \cdot kN$ — *Weight of material above base of cantilever wall on right side*

$MA_{Hor} := Xg_{above.R} = 1.5\,m$

$M_{above.R} := W_{above.R} \cdot MA_{Hor} = 24.7\,kN \cdot m$

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$H_{above.L} = 0.90\,m$
$H_{below.L} = 0.00\,m$
$H_{above.R} = 3.00\,m$
$H_{below.R} = 0.00\,m$
$A_{above.L} = 0.6\,m^2$
$A_{above.R} = 0.81\,m^2$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

Weight of Material Above Section

LC.1 - Summary of Forces

Dead Load:

$W_{conc} = 30.3 \cdot kN$ $\quad$ $M_{conc} = 30.9 \cdot kN \cdot m$

Soil:

$F_{soil.hor.R} = 25.6 \cdot kN$ $\quad$ $M_{soil.hor.R} = 25.6 \cdot kN \cdot m$

$F_{soil.ver.R} = 0 kN$ $\quad$ $M_{soil.ver.R} = 0 kN \cdot m$

$F_{soil.hor.L} = 31.4 \cdot kN$ $\quad$ $M_{soil.hor.L} = 9.4 kN \cdot m$

Material Above Section:

$W_{above.L} = 12.6 kN$ $\quad$ $M_{above.L} = 6.3 kN \cdot m$

$W_{above.R} = 17 kN$ $\quad$ $M_{above.R} = 24.7 kN \cdot m$

Water Above Section:

$W_{water.above.L} = 0 kN$ $\quad$ $M_{water.above.L} = 0 kN \cdot m$

$W_{water.above.R} = 0 kN$ $\quad$ $M_{water.above.R} = 0 kN \cdot m$

Uplift:

$F_{U.ver} = 0 \cdot kN$ $\quad$ $M_U = 0 kN \cdot m$

Hydrostatic:

$F_{water.hor.R} = 0 kN$ $\quad$ $M_{water.hor.R} = 0 kN \cdot m$

$F_{water.hor.L} = 0 kN$ $\quad$ $M_{water.hor.L} = 0 kN \cdot m$

Surcharge:

$F_{q.hor.R} = 3.9 \cdot kN$ $\quad$ $M_{q.hor.R} = 5.9 kN \cdot m$

$F_{q.ver.R} = 1.4 \cdot kN$ $\quad$ $M_{q.ver.R} = 2.1 kN \cdot m$

LC.1 - Combine Forces and Moments

$F_{hor.drive} := F_{soil.hor.R} + F_{water.hor.R} + F_{q.hor.R} = 29.5 kN$

$F_{hor.resist} := F_{soil.hor.L} + F_{water.hor.L} = 31.4 kN$

$F_{ver} := W_{conc} + F_{U.ver} + W_{above.L} + W_{above.R} + W_{water.above.L} + W_{water.above.R} + F_{soil.ver.R} + F_{q.ver.R} = 61.4 \cdot kN$

$M_{stab} := M_{conc} + M_{water.hor.L} + M_{soil.hor.L} + M_{above.L} + M_{above.R} + M_{water.above.L} + M_{water.above.R} + M_{soil.ver.R} + M_{q.ver.R} = 73.4 kN \cdot m$

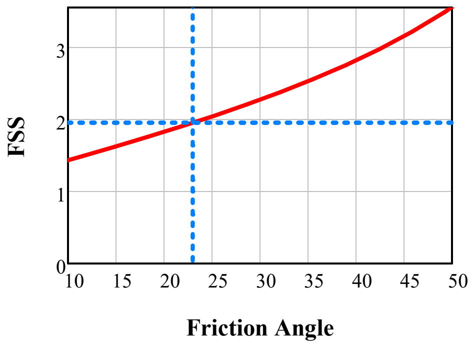
$M_o := M_{soil.hor.R} + M_{water.hor.R} + M_{q.hor.R} + M_U = 31.5 kN \cdot m$

$M_{net} := M_{stab} - M_o = 41.9 kN \cdot m$

LC.1 - Sliding

$$FSS(\theta) := \frac{F_{hor.resist} + F_{ver} \cdot \tan(\theta) + c \cdot L_{hor} \cdot B}{F_{hor.drive}}$$

$FSS(\phi_{cf}) = 1.95$



LC.1 - Overturning, Resultant and Bearing Stresses

$$FS_{OT} := \frac{M_{stab}}{M_o} = 2.33$$

$$x_o := \frac{M_{net}}{F_{ver}} = 0.68\,m$$

$$E := x_o - \frac{L_{hor}}{2} = -0.12\,m$$

Stress Calculations

$\sigma_n(0m) = 55.2\,kPa$

$q_{max} = 55.2 \cdot kPa$ — *Maximum bearing stress before iterative cracked base analysis*

$q_{min} := \sigma_n(L_{hor}) = 21.5\,kPa$

$\sigma_n(L_{slab.L}) = 34.2\,kPa$

$L_{comp} = 1.60\,m$ — *Length of base in compression before iterative cracked base analysis*

$\sigma_n(L_{slab.L} + L_{wall.base}) = 27.8\,kPa$

$L_{tens} = 0.00\,m$ — *Length of base in tension before iterative cracked base analysis*

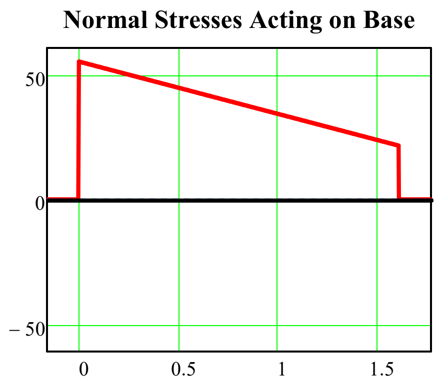
$L_{crack} = 0.00\,m$ — *Length of crack between concrete and base before iterative cracked base analysis*

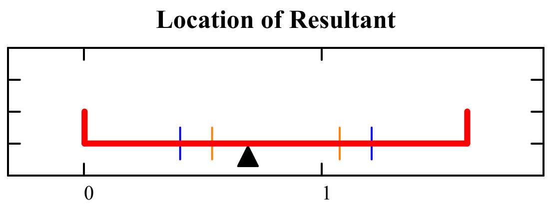
$\sigma_n(L_{hor}) = 21.5\,kPa$

$\frac{L_{comp}}{L_{hor}} = 100 \cdot \%$ — *% of Base in Compression*

$\frac{L_{tens}}{L_{hor}} = 0 \cdot \%$ — *% of Base in Tension*

$\frac{L_{crack}}{L_{hor}} = 0 \cdot \%$ — *% of Base Cracked*





Red lines indicate extent of structure, Blue lines indicate middle half of base, orange lines indicae middle third of base

Store Results for Summary

$LC := 1$

$R_{FSS_{LC}} := FSS(\phi_{cf}) = ...$

$R_{E_{LC}} := E = ...$

$R_{L.comp_{LC}} := L_{comp} = ...$

$R_{L.crack_{LC}} := L_{crack} = ...$

$R_{L.tens_{LC}} := L_{tens} = ...$

$R_{xo_{LC}} := x_o = ...$

$R_{F.hor.drive_{LC}} := F_{hor.drive} = ...$

$R_{F.hor.resist_{LC}} := F_{hor.resist} = ...$

$R_{F.ver_{LC}} := F_{ver} = ...$

$R_{q.max_{LC}} := q_{max} = ...$

$R_{comp.percent_{LC}} := \frac{L_{comp}}{L_{hor}} = ...$

$R_{q.min_{LC}} := q_{min} = ...$

Store Results for Summary

Load Combination 2 - IDF (Unusual)

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

$K_R := K_{a.R} = 0.271$ *Coefficient of lateral stress (assign to Ko.R, Ka.R or Kp.R)*

$K_L := K_{p.L} = 3.690$

$ELE_{water.R} := 172.0m$ *Elevation of water level on right side*

$ELE_{water.L} := 170.3m$ *Elevation of water level on left side*

$q_{surcharge.R} := 2kPa$ *Uniform surface stress , assumed to extend for an infinite length behind wall*

Calculation of Soil Forces on Right Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.R} = 0.426$
$K_{a.R} = 0.271$
$\delta_R = 23 \cdot deg$
$\theta_R = 90 \cdot deg$
$ELE_{soil.R} = 173.3\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.R} := ELE_{soil.R} - ELE_{Base.R} = 3\,m$ *Height of soil on right side of wall*

$$H_{above.R} := \begin{cases} (ELE_{soil.R} - ELE_{water.R}) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 1.30$$ *Height of soil above water*

$$H_{below.R} := \begin{cases} (ELE_{soil.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \ge ELE_{soil.R} \\ (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} > ELE_{Base.R} \wedge ELE_{water.R} < ELE_{soil.R} \\ 0 & \text{otherwise} \end{cases} = 1.7$$ *Height of soil below water*

$F_{above} := \frac{1}{2} \cdot K_R \cdot \gamma_{s.R} \cdot H_{above.R}^2 \cdot B = 4.8\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} < ELE_{soil.R} \wedge ELE_{water.R} > ELE_{Base.R} \\ \left(ELE_{Base.R} + \frac{H_{above.R}}{3}\right) & \text{if } ELE_{water.R} \le ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 172.433$$ *Elevation of force above water*

$F_{below1} := B \cdot K_R \cdot (\gamma_{s.R} \cdot H_{above.R}) \cdot H_{below.R} = 12.6\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{2}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 171.15$$ *Elevation of force*

$F_{below2} := B \cdot K_R \cdot \frac{\gamma_{eff.R} \cdot H_{below.R}^2}{2} = 4.4\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.R} + \frac{H_{below.R}}{3}\right) & \text{if } ELE_{water.R} > ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 170.9$$ *Elevation of force*

$F_{soil.R} := F_{above} + F_{below1} + F_{below2} = 21.8\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} = 171.376\,m$$ *Elevation of force*

$F_{soil.hor.R} := F_{soil.R} \cdot \cos(\beta_R) = 21.8\,kN$ *Horizontal component (positive is upstream/to the left)*

$F_{soil.ver.R} := F_{soil.R} \cdot \sin(\beta_R) = 0\,kN$

$MA_{hor} := ELE_F - ELE_{Base.L} = 1.076\,m$ *Moment arm for horizontal force*

$\boxed{MA_{ver} := L_{wall.base} + L_{slab.R} = 0.6\,m}$ *Moment arm for vertical force*

$M_{soil.hor.R} := F_{soil.hor.R} \cdot MA_{hor} = 23.4\,kN \cdot m$ *Moment from horizontal component*

$M_{soil.ver.R} := F_{soil.ver.R} \cdot MA_{ver} = 0\,kN \cdot m$ *Moment from vertical component*

Calculation of Soil Forces on Right Side

Calculation of Soil Forces on Left Side

Note: *Vertical component of soil pressure not considered, assumed to be horizontal only*

$K_{o.L} = 0.426$
$K_{a.L} = 0.271$
$\delta_L = 23 \cdot deg$
$\theta_L = 90 \cdot deg$
$ELE_{soil.L} = 171.2\,m$
$ELE_{Base.R} = 170.3\,m$
$ELE_{Base.L} = 170.3\,m$
$L_{hor} = 1.6\,m$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$H_{soil.L} := ELE_{soil.L} - ELE_{Base.L} = 0.9\,m$ *Height of soil on right side of wall*

$$H_{above.L} := \begin{cases} (ELE_{soil.L} - ELE_{water.L}) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0.90$$
Height of soil above water

$$H_{below.L} := \begin{cases} (ELE_{soil.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{soil.L} \\ (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} > ELE_{Base.L} \wedge ELE_{water.L} < ELE_{soil.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Height of soil below water

$F_{above} := \frac{1}{2} \cdot K_L \cdot \gamma_{s.L} \cdot H_{above.L}^2 \cdot B = 31.4\,kN$ *Force due to soil above water table*

$$ELE_{F.above} := \begin{cases} \left(ELE_{water.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} < ELE_{soil.L} \wedge ELE_{water.L} > ELE_{Base.L} \\ \left(ELE_{Base.L} + \frac{H_{above.L}}{3}\right) & \text{if } ELE_{water.L} \le ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 170.6$$
Elevation of force above water

$F_{below1} := B \cdot K_L \cdot (\gamma_{s.L} \cdot H_{above.L}) \cdot H_{below.L} = 0\,kN$ *Effective force due to soil below water table (rectangular portion)*

$$ELE_{F.below1} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{2}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Elevation of force

$F_{below2} := B \cdot K_L \cdot \frac{\gamma_{eff.L} \cdot H_{below.L}^2}{2} = 0\,kN$ *Effective force due to soil below water table (triangular portion)*

$$ELE_{F.below2} := \begin{cases} \left(ELE_{Base.L} + \frac{H_{below.L}}{3}\right) & \text{if } ELE_{water.L} > ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$$
Elevation of force

$F_{soil.L} := F_{above} + F_{below1} + F_{below2} = 31.4\,kN$ *Total force on wall from soil (not including hydrostatic force)*

$$ELE_F := \begin{cases} 0 & \text{if } F_{soil.L} = 0 \\ \frac{(F_{below1} \cdot ELE_{F.below1} + F_{below2} \cdot ELE_{F.below2} + F_{above} \cdot ELE_{F.above})}{(F_{below1} + F_{below2} + F_{above})} & \text{otherwise} \end{cases} = 170.6$$
Elevation of force

$F_{soil.hor.L} := F_{soil.L} = 31.4\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.3\,m$ *Moment arm for horizontal force*

$M_{soil.hor.L} := F_{soil.hor.L} \cdot MA_{hor} = 9.4\,kN \cdot m$ *Moment from horizontal component*

▲ Calculation of Soil Forces on Left Side

▼ Hydrostatic Pressure on Right Side

$H_{water.R} := \begin{cases} (ELE_{water.R} - ELE_{Base.R}) & \text{if } ELE_{water.R} \geq ELE_{Base.R} \\ 0 & \text{otherwise} \end{cases} = 1.7$ *Height of water acting on wall*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.R} = 172.000\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_R = 90 \cdot deg$

$F_{water.R} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.R}^2 = 14.2\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.R} + \frac{H_{water.R}}{3} = 170.867\,m$ *Elevation of resultant force*

$F_{water.hor.R} := F_{water.R} = 14.2\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0.567\,m$ *Moment arm for horizontal force*

$M_{water.hor.R} := F_{water.hor.R} \cdot MA_{hor} = 8\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Right Side

Hydrostatic Pressure on Left Side

$H_{water.L} := \begin{cases} (ELE_{water.L} - ELE_{Base.L}) & \text{if } ELE_{water.L} \geq ELE_{Base.L} \\ 0 & \text{otherwise} \end{cases} = 0$ *Height of water acting on wall*

$F_{water.L} := \frac{1}{2} B \cdot \gamma_w \cdot H_{water.L}^2 = 0\,kN$ *Hydrostatic force acting normal to face*

$ELE_F := ELE_{Base.L} + \frac{H_{water.L}}{3} = 170.3\,m$ *Elevation of resultant force*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$ELE_{water.L} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\theta_L = 90 \cdot deg$

$F_{water.hor.L} := F_{water.L} = 0\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 0\,m$ *Moment arm for horizontal force*

$M_{water.hor.L} := F_{water.hor.L} \cdot MA_{hor} = 0\,kN \cdot m$ *Moment from horizontal component*

Hydrostatic Pressure on Left Side

Calculation of Uplift Pressure

$P_{U.L} := H_{water.L} \cdot \gamma_w = 0\,kPa$ *Uplift pressure at left side*

$P_{U.R} := H_{water.R} \cdot \gamma_w = 16.7\,kPa$ *Uplift pressure at right side*

$P_U(x) := \begin{cases} P_{U.R} + \frac{(P_{U.L} - P_{U.R})}{(L_{hor})} \cdot x & \text{if } x \leq L_{hor} \wedge x \geq 0 \\ 0 & \text{otherwise} \end{cases}$ *Uplift function. x=0 at right side.*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$ELE_{water.L} = 170.300\,m$
$ELE_{water.R} = 172.000\,m$

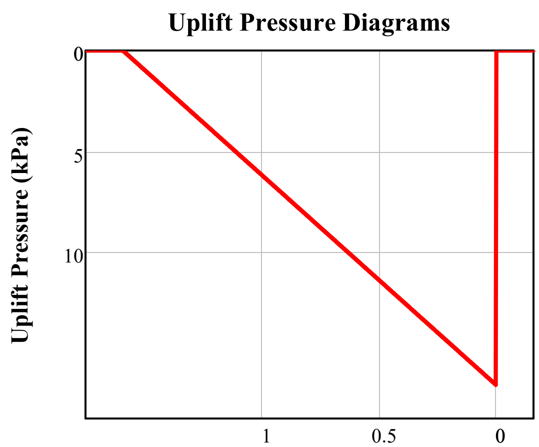
$F_U := \int_0^{L_{hor}} P_U(x) \cdot B\,dx = 13.3 \cdot kN$ *Total uplift force assuming uncracked section.*

$L_{Uf} := \begin{cases} 0 & \text{if } F_U = 0 \\ \left(\frac{1}{F_U}\int_0^{L_{hor}} P_U(x)\cdot x\cdot B\,dx\right) & \text{otherwise} \end{cases} = 0.53$ *Distance of uplife force from right side of base*

$MA := L_{hor} - L_{Uf} = 1.1\,m$ *Moment arm of uplift force*

$M_U := F_U \cdot MA = 14.2\cdot kN\cdot m$ *Moment from uplift*

$F_{U.ver} := -F_U = -13.3\cdot kN$ *Vertical component (down is positive)*



Calculation of Uplift Pressure

Force from Surcharge

$F_{q.R} := q_{surcharge.R}\cdot K_R\cdot H_{soil.R}\cdot B = 1.6\,kN$ *Force on wall due to surcharge*

$ELE_F := ELE_{Base.R} + \frac{H_{soil.R}}{2} = 171.8\,m$ *Elevation of resultant force*

$F_{q.hor.R} := F_{q.R} = 1.6\,kN$ *Horizontal component (positive is upstream/to the left)*

$MA_{hor} := ELE_F - ELE_{Base.L} = 1.5\,m$ *Moment arm for horizontal component*

$M_{q.hor.R} := F_{q.hor.R}\cdot MA_{hor} = 2.4\,kN\cdot m$ *Moment from horizontal component*

$F_{q.ver.R} := q_{surcharge.R}\cdot L_{slab.R}\cdot B = 0.6\cdot kN$

$MA_{ver} := L_{slab.L} + L_{wall.base} + \frac{L_{slab.R}}{2} = 1.5\,m$

$M_{q.ver.R} := F_{q.ver.R}\cdot MA_{ver} = 0.9\,kN\cdot m$

Force from Surcharge

Weight of Water Above Section

Reference Coordinates of Structure

$$X_{struct} = \begin{pmatrix} 0.000 \\ 0.000 \\ 1.000 \\ 1.000 \\ 1.300 \\ 1.300 \\ 1.600 \\ 1.600 \\ 0.000 \end{pmatrix} m \qquad Y_{struct} = \begin{pmatrix} 170.300 \\ 170.600 \\ 170.600 \\ 173.300 \\ 173.300 \\ 170.600 \\ 170.600 \\ 170.300 \\ 170.300 \end{pmatrix} m$$

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$ELE_{top} = 173.300\,m$
$L_{hor} = 1.600\,m$
$B = 1.000\,m$

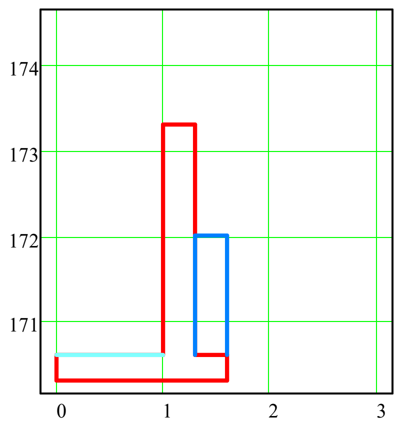
Insert coordinates of shape of material above structure

$B_{water} := B = 1.000\,m$ *Width of material*

$$X_{water.L} := \begin{pmatrix} 0 \\ 0 \\ 1 \\ 1 \end{pmatrix} m \qquad Y_{water.L} := \begin{pmatrix} 170.6m \\ 170.6m \\ 170.6m \\ 170.6m \end{pmatrix}$$

$$X_{water.R} := \begin{pmatrix} 1.3 \\ 1.3 \\ 1.6 \\ 1.6 \end{pmatrix} m \qquad Y_{water.R} := \begin{pmatrix} 170.60m \\ ELE_{water.R} \\ ELE_{water.R} \\ 170.60m \end{pmatrix}$$

Material on Top of Structure



$X_{water.L} := \max(X_{water.L}) - \min(X_{water.L}) = 1\,m$

$X_{water.R} := \max(X_{water.R}) - \min(X_{water.R}) = 0.3\,m$

Calculation of Area and Centre of Gravity (Left Side)

$X := X_{water.L}$ $\quad Y := Y_{water.L}$

$i := 1 .. \text{length}(X)$ $\quad j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3}\right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3}\right]$$

$$A_{water.above.L} := \left|\sum_i Areainc_i\right| = 0\,m^2$$

$$Xg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Xginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

$$Yg_{water.above.L} := \begin{cases} 0 & \text{if } A_{water.above.L} = 0 \\ \left|\dfrac{\sum_i Yginc_i}{A_{water.above.L}}\right| & \text{otherwise} \end{cases} = 0$$

Calculation of Area and Centre of Gravity (Right Side)

$X := X_{water.R}$ $\quad Y := Y_{water.R}$

$i := 1 .. \text{length}(X)$ $\quad j := 1 .. \text{length}(X) + 1$

$X_{\text{length}(X)+1} := X_1$ $\quad Y_{\text{length}(Y)+1} := Y_1$

$deltax_i := X_{i+1} - X_i$ $\quad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3}\right]$ $\quad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3}\right]$

$A_{water.above.R} := \left|\sum_i Areainc_i\right| = 0.4\,m^2$

$Xg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\frac{\sum_i Xginc_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 1.45$

$Yg_{water.above.R} := \begin{cases} 0 & \text{if } A_{water.above.R} = 0 \\ \left|\frac{\sum_i Yginc_i}{A_{water.above.R}}\right| & \text{otherwise} \end{cases} = 171.3$

Calculation of Forces

$W_{water.above.L} := (A_{water.above.L} \cdot \gamma_w \cdot B_{water}) = 0 \cdot kN$

$MA_{Hor} := Xg_{water.above.L} = 0\,m$

$M_{water.above.L} := W_{water.above.L} \cdot MA_{Hor} = 0\,kN \cdot m$

$W_{water.above.R} := (A_{water.above.R} \cdot \gamma_w \cdot B_{water}) = 4.1 \cdot kN$

$MA_{Hor} := Xg_{water.above.R} = 1.5\,m$

$M_{water.above.R} := W_{water.above.R} \cdot MA_{Hor} = 6\,kN \cdot m$

Weight of Water Above Section

Weight of Material Above Section

$x_{sat} := x_{water.R} = 0.3\,m$

$x_{dry} := x_{soil.R} - x_{water.R} = 0\,m$

$\gamma_{above.L} := \gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$W_{above.L} := (A_{above.L} \cdot \gamma_{above.L} \cdot B) = 12.6 \cdot kN$ — *Weight of material above base of cantilever wall on left side*

$MA_{Hor} := Xg_{above.L} = 0.5\,m$ — *Moment arm*

$M_{above.L} := W_{above.L} \cdot MA_{Hor} = 6.3\,kN \cdot m$ — *Overturning moment*

$ELE_{Base.L} = 170.300\,m$
$ELE_{Base.R} = 170.300\,m$
$H_{above.L} = 0.90\,m$
$H_{below.L} = 0.00\,m$
$H_{above.R} = 1.30\,m$
$H_{below.R} = 1.70\,m$
$A_{above.L} = 0.6\,m^2$
$A_{above.R} = 0.81\,m^2$

$L_{hor} = 1.600\,m$
$B = 1.000\,m$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{sat.L} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.L} = 11.2 \cdot \frac{kN}{m^3}$

$\gamma_{above.R} := \gamma_{eff.R} = 11.2 \cdot \frac{kN}{m^3}$

$W_{above.R} := \left(A_{above.R} \cdot \gamma_{above.R} \cdot B\right) = 9.1 \cdot kN$

Weight of material above base of cantilever wall on right side

$MA_{Hor} := Xg_{above.R} = 1.5\,m$

$M_{above.R} := W_{above.R} \cdot MA_{Hor} = 13.1\,kN \cdot m$

Weight of Material Above Section

LC.2 - Summary of Forces

Dead Load:

$W_{conc} = 30.3 \cdot kN$ $\quad M_{conc} = 30.9 \cdot kN \cdot m$

Soil:

$F_{soil.hor.R} = 21.8\,kN$ $\quad M_{soil.hor.R} = 23.4\,kN \cdot m$

$F_{soil.ver.R} = 0\,kN$ $\quad M_{soil.ver.R} = 0\,kN \cdot m$
$F_{soil.hor.L} = 31.4\,kN$ $\quad M_{soil.hor.L} = 9.4\,kN \cdot m$

Material Above Section:

$W_{above.L} = 12.6\,kN$ $\quad M_{above.L} = 6.3\,kN \cdot m$
$W_{above.R} = 9.1\,kN$ $\quad M_{above.R} = 13.1\,kN \cdot m$

Water Above Section:

$W_{water.above.L} = 0\,kN$ $\quad M_{water.above.L} = 0\,kN \cdot m$
$W_{water.above.R} = 4.1\,kN$ $\quad M_{water.above.R} = 6\,kN \cdot m$

Uplift:

$F_{U.ver} = -13.3 \cdot kN$ $\quad M_U = 14.2\,kN \cdot m$

Hydrostatic:

$F_{water.hor.R} = 14.2\,kN$ $\quad M_{water.hor.R} = 8\,kN \cdot m$

$F_{water.hor.L} = 0\,kN$ $\quad M_{water.hor.L} = 0\,kN \cdot m$

Surcharge:

$F_{q.hor.R} = 1.6\,kN$ $\quad M_{q.hor.R} = 2.4\,kN \cdot m$
$F_{q.ver.R} = 0.6 \cdot kN$ $\quad M_{q.ver.R} = 0.9\,kN \cdot m$

LC.2 - Combine Forces and Moments

$F_{hor.drive} := F_{soil.hor.R} + F_{water.hor.R} + F_{q.hor.R} = 37.6\,kN$

$F_{hor.resist} := F_{soil.hor.L} + F_{water.hor.L} = 31.4\,kN$

$F_{ver} := W_{conc} + F_{U.ver} + W_{above.L} + W_{above.R} + W_{water.above.L} + W_{water.above.R} + F_{soil.ver.R} + F_{q.ver.R} = 43.4 \cdot kN$

$M_{stab} := M_{conc} + M_{water.hor.L} + M_{soil.hor.L} + M_{above.L} + M_{above.R} + M_{water.above.L} + M_{water.above.R} + M_{soil.ver.R} + M_{q.ver.R} = 66.6\,kN \cdot m$

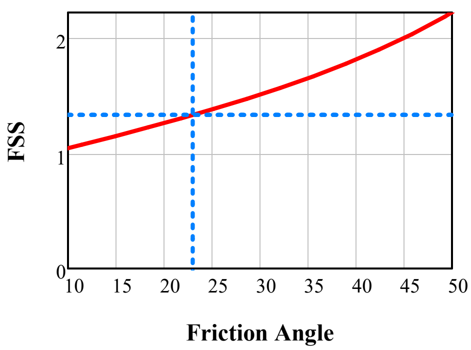
$M_o := M_{soil.hor.R} + M_{water.hor.R} + M_{q.hor.R} + M_U = 48.1\,kN \cdot m$

$M_{net} := M_{stab} - M_o = 18.5\,kN \cdot m$

LC.2 - Sliding

$$FSS(\theta) := \frac{F_{hor.resist} + F_{ver} \cdot \tan(\theta) + c \cdot L_{hor} \cdot B}{F_{hor.drive}}$$

$\boxed{FSS(\phi_{cf}) = 1.33}$



LC.2 - Overturning, Resultant and Bearing Stresses

$$FSOT := \frac{M_{stab}}{M_o} = 1.38$$

$$x_o := \frac{M_{net}}{F_{ver}} = 0.43\,m$$

$$\boxed{E := x_o - \frac{L_{hor}}{2} = -0.37\,m}$$

Stress Calculations

$\boxed{q_{max} = 67.8 \cdot kPa}$ *Maximum bearing stress before iterative cracked base analysis* $\boxed{\sigma_n(0m) = 67.8\,kPa}$

$\boxed{q_{min} := \sigma_n(L_{hor}) = 0\,kPa}$

$\boxed{L_{comp} = 1.28\,m}$ *Length of base in compression before iterative cracked base analysis* $\boxed{\sigma_n(L_{slab.L}) = 14.8\,kPa}$

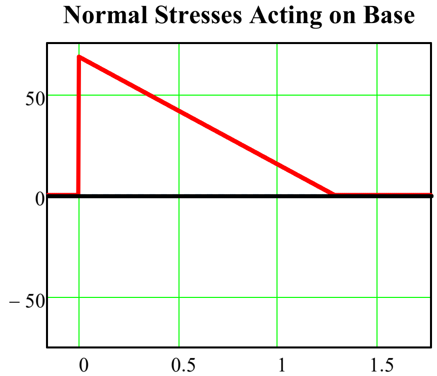
$\boxed{L_{tens} = 0.00\,m}$ *Length of base in tension before iterative cracked base analysis* $\boxed{\sigma_n(L_{slab.L} + L_{wall.base}) = 0\,kPa}$

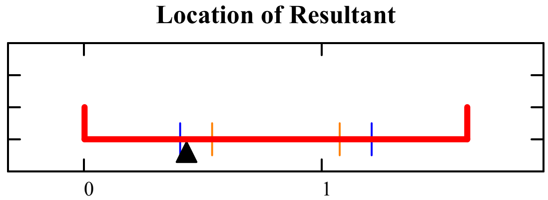
$\boxed{L_{crack} = 0.32\,m}$ *Length of crack between concrete and base before iterative cracked base analysis* $\boxed{\sigma_n(L_{hor}) = 0\,kPa}$

$\frac{L_{comp}}{L_{hor}} = 79.9 \cdot \%$ *% of Base in Compression*

$\frac{L_{tens}}{L_{hor}} = 0 \cdot \%$ *% of Base in Tension*

$\frac{L_{crack}}{L_{hor}} = 20.1 \cdot \%$ *% of Base Cracked*





Red lines indicate extent of structure,
Blue lines indicate middle half of base,
orange lines indicae middle third of base

Store Results for Summary

$LC := 2$

$R_{FSS_{LC}} := FSS(\phi_{cf}) = ...$

$R_{E_{LC}} := E = ...$

$R_{L.comp_{LC}} := L_{comp} = ...$

$R_{L.crack_{LC}} := L_{crack} = ...$

$R_{L.tens_{LC}} := L_{tens} = ...$

$R_{xo_{LC}} := x_o = ...$

$R_{F.hor.drive_{LC}} := F_{hor.drive} = ...$

$R_{F.hor.resist_{LC}} := F_{hor.resist} = ...$

$R_{F.ver_{LC}} := F_{ver} = ...$

$R_{q.max_{LC}} := q_{max} = ...$

$R_{comp.percent_{LC}} := \frac{L_{comp}}{L_{hor}} = ...$

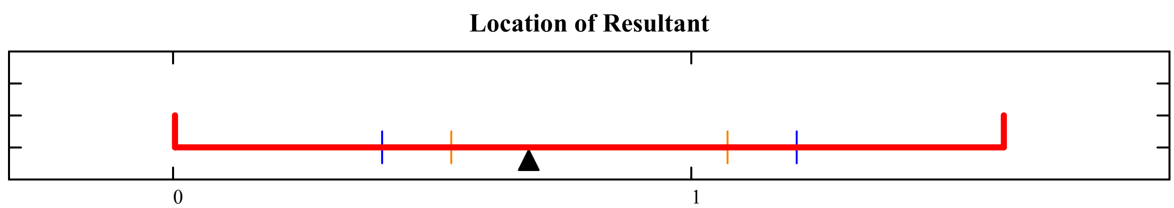
$R_{q.min_{LC}} := q_{min} = ...$

Store Results for Summary

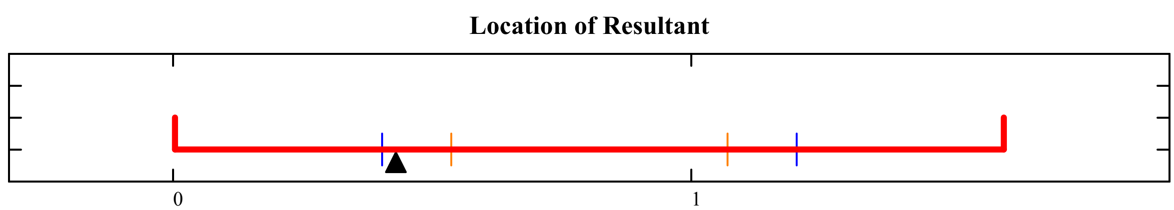
Results of Analysis

	FSS (Φ.cf)	E (m)	x.o (m)	L.comp (m)	% of Base in Compression	L.crack (m)	F.hor.drive (kN)	F.hor.resist (kN)	F.ver (kN)	q.max (kPa)
LC.1 - Normal Water Level (Usual)	1.95	-0.12	0.68	1.60	100%	0.00	29.5	31.4	61.4	55.2
LC.2 - IDF Water Level (Unusual)	1.33	-0.37	0.43	1.28	80%	0.32	37.6	31.4	43.4	67.8

LC 1 - Normal Water Level



LC 2 - IDF Water Level



KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	23-4168-001	**Project Name :**	Churchville Condition Assessment		
File No. :		**Discipline :**	Structural Engineering		
Calculation Title :	**Floodwall Strength Analysis – Configuration 1**				
Calculation No. :	CIV-003	**Prepared by :**	JL	**Date :**	23/12/11
No. of Sheets :		**Checked by :**	YF	**Date :**	23/12/13
Supersedes Calc. No. :		**Approved by :**		**Date :**	

Calculation Description :

Strength analysis of the gravity retaining section.

Related Design Concept :

Reference Codes and Standards :

1. CAN/CSA 23.3 – 04, Design of concrete structure
2. CDA Dam Safety Guidelines 2007
3. Structural Design and Factors of Safety – Technical Bulletin Ontario Ministry of Natural Resources (August 2011)
4. ”Foundation Analysis and Design" by Joseph. E. Bowles.

ENGINEER'S SEAL

Rev. #	**Rev. Description**	**Rev. Author**	**Date Revised**	**Checked by**	**Approved by**	**Approved Date**

Reference to stability analysis

Reference:P:\Projects\2023\23-4168-001\Design\Struct\Churchville Stability Analysis 1.xmcd

1. Concrete Stem wall design

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$(ELE_{soil.R}) = 171.5\,m$

$(\gamma_{s.R}) = 21 \cdot \frac{kN}{m^3}$

$(K_{o.R}) = 0.43$

$(B) = 1\,m$

$(\beta_R) = 0$

$(\gamma_{s.L}) = 21 \cdot \frac{kN}{m^3}$

Driving Pressure

Surcharge and lateral earth pressure

$q_{sur} := 4.8kPa$ — *Pressure from live load surcharge*

$h := ELE_{soil.R} - 170.6m = 0.9\,m$ — *Height of backfill soil on the right side above the base slab*

$P_{as} := \left[\left(\frac{1}{2}\right) \cdot \gamma_{s.R} \cdot h^2 + q_{sur} \cdot h\right] \cdot K_{o.R} \cdot B = 5.47 \cdot kN$ — *Pressure from live load surcharge and soil*

Horizontal component at ys above stem bottom

$H_{as} := P_{as} \cdot \cos((\beta_R)) = 5.47\,kN$ — *Pressure from live load surcharge and soil accounting for backfill slope*

$y_s := \frac{h}{3} \cdot \left[\frac{3 \cdot q_{sur} + \gamma_{s.R} \cdot (h)}{2 \cdot q_{sur} + \gamma_{s.R} \cdot (h)}\right] = 0.35\,m$ — *Moment arm of horizontal driving pressure*

Moment at bottom of stem

$M_s := H_{as} \cdot y_s = 1.92\,kN \cdot m$ — *Moment at the bottom of the stem wall from driving force*

Passive Pressure

Passive earth pressure

$D_p := ELE_{soil.L} - 170.6m = 0.6\,m$ — *Depth of effective passive pressure to bottom of stem wall*

$P_{p.1} := \left(\frac{1}{2}\right) \cdot \gamma_{s.L} \cdot D_p^{\ 2} \cdot K_{o.L} \cdot B = 1.61 \cdot kN$ — *Passive pressure acting on the stem wall*

Moment arm ys above stem bottom

$y_{p.1} := \frac{D_p}{3} = 0.2\,m$ — *Moment arm of horizontal passive pressure*

Moment at bottom of stem

$M_{p.1} := P_{p.1} \cdot y_{p.1} = 0.32\,kN \cdot m$ — *Moment at the bottom of the stem wall from passive force*

Factored Forces

Factored Shear Force

$V_{f1} := 1.5 \cdot H_{as} - 0.9 \cdot P_{p.1} = 6.75\,kN$ — *Factored Shear Force*

Factored Moment Force

$M_{f1} := 1.5 \cdot M_s - 0.9 \cdot M_{p.1} = 2.59\,kN \cdot m$ — *Factored Moment Force*

Load Case 2 - Flood (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$h_{w.R} := ELE_{water.R} - ELE_{Base.R} = 2.7\,m$ — *Height of water above the bottom of the stem*

$h_{w.L} := ELE_{water.L} - ELE_{Base.L} = 0\,m$

$(ELE_{water.R}) = 173\,m$

$(ELE_{Base.R}) = 170.3\,m$

$(ELE_{water.L}) = 170.3\,m$

$(ELE_{Base.L}) = 170.3\,m$

$(\gamma_{s.R}) = 21 \cdot \frac{kN}{m^3}$

$(\gamma_{eff.R}) = 11.19 \cdot \frac{kN}{m^3}$

$(\gamma_{s.L}) = 21 \cdot \frac{kN}{m^3}$

$(\gamma_{eff.L}) = 11.19 \cdot \frac{kN}{m^3}$

$(\gamma_w) = 9.81 \cdot \frac{kN}{m^3}$

$(K_{a.R}) = 0.27$

$(B) = 1\,m$

$(K_{p.L}) = 3.69$

$(k_h) = 0.15$

$(D_p) = 0.6\,m$

Driving Pressure

Surcharge and lateral earth pressure

$q_{sur} := 0.0kPa$ — *Pressure from live load surcharge*

$H_{sur} := q_{sur} \cdot h \cdot K_{a.R} \cdot B = 0 \cdot kN$ — *Pressure from live load surcharge and soil*

$y_{sur} := \frac{h}{2} = 0.45\,m$ — *Moment arm of force from surcharge*

$t := 0.3m$ — *Thickness of the base*

$$h_{wt.R} := \begin{cases} (h_{w.R} - t) & \text{if } h_{w.R} > t \\ 0 & \text{otherwise} \end{cases} = 2.4$$

Height of water above the right side of the base

Lateral force due to backfill above water

$$h_{1.R} := \begin{cases} (h - h_{wt.R}) & \text{if } h_{w.R} > t \wedge h_{w.R} < h = 0 \\ 0 & \text{if } h_{w.R} \geq h \\ D_p & \text{otherwise} \end{cases}$$

Height of backfill above water

$h_{1.R} := \min(h_{1.R}, h) = 0\,m$

$$H_{b1.R} := \frac{1}{2} \cdot \gamma_{s.R} \cdot (h_{1.R})^2 \cdot K_{a.R} \cdot B = 0 \cdot kN$$

Pressure from backfill above water

$$y_{b1.R} := \begin{cases} 0 & \text{if } h_{w.R} \geq h \\ \left[h_{wt.R} + \left(\frac{h_{1.R}}{3}\right)\right] & \text{otherwise} \end{cases} = 0$$

Moment Arm from backfill above water

Lateral force due to backfill submerged in water

$$h_{2.R} := \begin{cases} h_{wt.R} & \text{if } h_{wt.R} < h \\ h & \text{otherwise} \end{cases} = 0.9$$

Height of backfill submerged in water

$$H_{b2.R} := \frac{1}{2} \cdot (2\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R}) \cdot h_{2.R} \cdot K_{a.R} \cdot B = 1.23 \cdot kN$$

Pressure from backfill submerged in water

$$y_{b2.R} := h_{2.R} - \frac{h_{2.R} \cdot [2(\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R}) + \gamma_{s.R} \cdot h_{1.R}]}{3 \cdot (2\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R})} = 0.3\,m$$

Moment Arm from backfill submerged in water

Lateral force due to water

$H_{w.R} := \frac{1}{2} \cdot \gamma_w \cdot (h_{wt.R})^2 \cdot B = 28.25 \cdot kN$ *Pressure from water*

$y_{w.R} := \frac{1}{3} \cdot (h_{wt.R}) = 0.8\,m$ *Moment Arm from water pressure*

Passive Pressure

$$h_{wt.L} := \begin{cases} (h_{w.L} - t) & \text{if } h_{w.L} > t \\ 0 & \text{otherwise} \end{cases} = 0$$

Lateral force due to backfill above water

$$h_{1.L} := \begin{cases} (D_p - h_{wt.L}) & \text{if } h_{w.L} > t \wedge h_{w.L} < D_p \\ 0 & \text{if } h_{w.L} \geq D_p \\ D_p & \text{otherwise} \end{cases} = 0.6$$

$h_{1.L} := \min(h_{1.L}, D_p) = 0.6\,m$

$H_{b1.L} := \frac{1}{2} \cdot \gamma_{s.L} \cdot (h_{1.L})^2 \cdot K_{p.L} \cdot B = 13.95 \cdot kN$

$$y_{b1.L} := \begin{cases} 0 & \text{if } h_{w.L} \geq D_p \\ \left[h_{wt.L} + \left(\frac{h_{1.L}}{3}\right)\right] & \text{otherwise} \end{cases} = 0.2$$

Lateral force due to backfill submerged in water

$$h_{2.L} := \begin{cases} h_{wt.L} & \text{if } h_{wt.L} < D_p \\ D_p & \text{otherwise} \end{cases} = 0$$

$H_{b2.L} := \frac{1}{2} \cdot (2\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L}) \cdot h_{2.L} \cdot K_{a.L} \cdot B = 0 \cdot kN$

$$y_{b2.L} := h_{2.L} - \frac{h_{2.L} \cdot [2(\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L}) + \gamma_{s.L} \cdot h_{1.L}]}{3 \cdot (2\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L})} = 0\,m$$

Lateral force due to water

$H_{w.L} := \frac{1}{2} \cdot \gamma_w \cdot (h_{wt.L})^2 \cdot B = 0 \cdot kN$

$y_{w.L} := \frac{1}{3} \cdot (h_{wt.L}) = 0\,m$

Factored Forces

Factored Shear Force

$$\boxed{V_{f2} := 1.5 \cdot (H_{b1.R} + H_{b2.R} + H_{sur} + H_{w.R}) - 0.9(H_{b1.L} + H_{b2.L} + H_{w.L}) = 31.67\,kN}$$

Factored Shear Force

Factored Moment Force

$$\boxed{M_{f2} := 1.5 \cdot (H_{b1.R} \cdot y_{b1.R} + H_{b2.R} \cdot y_{b2.R} + H_{sur} \cdot y_{sur} + H_{w.R} \cdot y_{w.R}) - 0.9(H_{b1.L} \cdot y_{b1.L} + H_{b2.L} \cdot y_{b2.L} + H_{w.L} \cdot y_{w.L}) = 31.95\,kN \cdot m}$$

Factored Moment Force

$V_f := \max(V_{f1}, V_{f2}) = 31.67\,kN$ *Maximum Factored Shear Force*

$M_f := \max(M_{f1}, M_{f2}) = 31.95\,kN \cdot m$ *Maximum Factored Moment Force*

Shear Capacity Calculation

Material Properties

$N_f := 0kN$ *Factored axial forces*

$\phi_s := 0.85$ *Rebar tensile strength*

$\phi_c := 0.65$ *Concrete resistance factor*

$f_y := 400MPa$ *Yield stength of steel rebar*

$\lambda := 1.0$ *Factor with regards to concrete density*

$E_s := 200000MPa$ *Modulus of elasticity for steel rebar*

$f'_c := 25MPa$ *Specified compressive strength of concrete*

$a_g := 20mm$ *Maximum size of coarse aggregate*

$E_c := 4500 \cdot \sqrt{f'_c \cdot MPa} = 22500 \cdot MPa$ *Modulus of elasticity for concrete*

Structure Geometry

$d_{b.15} := 16 \cdot mm$ *Diameter of rebars 15M*

$Sp := 300 \cdot mm$ *Rebar spacing (20M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ *Diameter of rebars 20M*

$b := 1000mm$ *Width of member*

$c := 75mm$ *Concrete cover*

$h := 300mm = 0.3\,m$ *Overall member thickness*

$$A_{db.20} := \left[\frac{\pi \cdot (d_{b.20})^2}{4}\right] = 298.65 \cdot mm^2$$ *Cross-sectional area of 20M rebar (Primary rebar)*

$$As := \frac{b}{Sp} \cdot A_{db.20} = 995.49 \cdot mm^2$$ *Primary rebar area*

$$d := h - c - \frac{d_{b.20}}{2} = 215.25 \cdot mm$$ *Distance from extreme compression fibre to centroid of the 20 M tension bars*

$$d_v := \max(0.9 \cdot d, 0.72 \cdot h)$$ *Effective shear depth*

$$M_f := \begin{cases} M_f & \text{if } M_f > V_f \cdot d_v \\ V_f \cdot d_v & \text{otherwise} \end{cases} = 31.95 \cdot kN \cdot m$$ *Factored moment*

$$\varepsilon_x := \frac{\frac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot As} = 0.00045$$ *Longitudinal strain at mid-depth of member due to factored loads* CSAA23.3-04 EQ 11-13

$$\varepsilon_x := \begin{cases} \varepsilon_x & \text{if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} & \text{otherwise} \end{cases} = 0.00045$$

$$s_z := d_v$$ *Crack spacing parameter*

$$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$$ *Effective crack spacing parameter* CSAA23.3-04 EQ 11-10

$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.26$ *Factor accounting for shear resistance of cracked concrete* *CSA A23.3-04 EQ 11-11*

$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'_c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 179.07 kN$ *Shear resistance due to concrete* *CSA A23.3-04 EQ 11-6*

Check for shear friction if the wall was poured after the base had partially cured

$(c_c := 0.25 MPa)$

$(\mu := 0.6)$

$A_{vf} := As$ $A_{cv} := d \cdot b$

$\rho_v := \frac{A_{vf}}{A_{cv}} = 0$

$\sigma := \rho_v \cdot f_y = 1.85 \cdot MPa$

$v_r := \phi_c \cdot \lambda \cdot (c_c + \mu \cdot \sigma) = 0.88 \cdot MPa$

$V_r := \begin{cases} v_r \cdot A_{cv} & \text{if } v_r < 0.25 \cdot \phi_c \cdot f'_c \\ (0.25 \cdot \phi_c \cdot f'_c) \cdot A_{cv} & \text{otherwise} \end{cases} = 190.27 \cdot kN$

CAN/CSA A23.3-04 Cl. 11.3.9.5

$Concrete\_Shear\_Capacity := \begin{cases} \text{"OK"} & \text{if } V_c \geq V_f \wedge V_r \geq V_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$ $(Concrete\_Shear\_Capacity) = \text{"OK"}$

$\frac{V_f}{V_c} = 0.18$

Tensile capacity of longitudinal rebar at end support

$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 32.16 \cdot deg$

$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 50.37 kN$

$k_1 := 1.0$ $k_2 := 1.0$ $k_3 := 1.0$ $k_4 := 0.8$

$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f'_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$

$(l_{actual} := (t - c) = 0.23 m)$

$l_{act} := \begin{cases} l_{actual} & \text{if } l_{actual} < l_d \\ l_d & \text{otherwise} \end{cases} = 0.23$

$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 135.6 kN$

$Embedment\_Check := \begin{cases} \text{"OK"} & \text{if } T_r \geq T_f \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{"OK"}$ $(Embedment\_Check) = \text{"OK"}$

$\frac{T_f}{T_r} = 0.37$

Moment Capacity Calculation

$$\alpha_1 := \begin{cases} .85 - .0015\dfrac{f'_c}{MPa} & \text{if } \left(.85 - .0015\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

CSA A23.3-04 Clause 10.1.7

$$\beta_1 := \begin{cases} .97 - .0025\dfrac{f'_c}{MPa} & \text{if } \left(.97 - .0025\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

$t = 0.3\,m$

$$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c}$$

$As = 995.49 \cdot mm^2$

$d = 215.25 \cdot mm$

$$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 68.52 \cdot kN \cdot m$$

$$\text{Concrete\_Moment\_Capacity} := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

Concrete_Moment_Capacity = "OK"

$$\left(\frac{M_f}{M_r}\right) = 0.47$$

2. Concrete Base Slab - Toe

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

Unfactored moment and shear forces from weight of slab and soil

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.L}^2}{2} + D_p \cdot \gamma_{s.L} \cdot b \cdot \frac{L_{slab.L}^2}{2} = 0.88 \cdot kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.L} + D_p \cdot \gamma_{s.L} \cdot b \cdot L_{slab.L} = 5.89 \cdot kN$$

$x_{o1} := R_{xo_1} = 0.71\,m$ *Location of the resultant*

$L_{slab.L} = 0.3\,m$
$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$
$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$
$t = 0.3\,m$
$b = 1\,m$
$D_p = 0.6\,m$

Foundation Pressures

$q_{max1} := R_{q.max_1} = 47.78\,kPa$ *Maximum bearing stress on the base*

$q_{min1} := R_{q.min_1} = 24.46\,kPa$ *Minimum bearing stress on the base*

$L_{wall} := 1.6m$

$b2 := L_{slab.L} = 0.3\,m$ *Length of the toe*

$b3 := L_{slab.R} = 1\,m$ *Length of the heel*

$$B := \begin{cases} L_{wall} & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o1}) & \text{otherwise} \end{cases} = 1.6 \cdot m$$

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the base subject to bearing stresses

$$q1 := \begin{cases} \left[q_{max1} - \frac{b2}{L_{wall}} \cdot (q_{max1} - q_{min1})\right] & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{B - b2}{B} \cdot q_{max1}\right) & \text{if } (3 \cdot x_{o1} \ge b2) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 43.41 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t1} := \begin{cases} \left(\frac{q_{max1} + q1}{2} \cdot b2 \cdot b\right) & \text{if } 3 \cdot x_{o1} \ge b2 \\ \left(\frac{1}{2} \cdot q_{max1} \cdot B\right) \cdot b & \text{otherwise} \end{cases} = 13.68 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t1} := \begin{cases} \left[\frac{1}{2} \cdot q1 \cdot b2^2 + \frac{1}{3} \cdot (q_{max1} - q1) \cdot b2^2\right] \cdot b & \text{if } 3 \cdot x_{o1} \ge b2 \\ \left[\frac{1}{2} \cdot q_{max1} \cdot B \cdot \left(b2 - \frac{B}{3}\right) \cdot b\right] & \text{otherwise} \end{cases} = 2.08 \cdot kN \cdot m$$

Moment due to bearing stresses

Factored Forces

Factored Shear Force

$$V_{f1} := 1.5 \cdot (V_{t1}) - 0.9 \cdot V_{wc} = 15.21\,kN$$

Factored Shear Force

Factored Moment Force

$$(M_{f1} := 1.5 \cdot (M_{t1}) - 0.9 \cdot M_{wc} = 2.33\,kN \cdot m)$$

Factored Moment Force

Load Case 2 - Flood (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

Unfactored moment and shear forces from weight of slab, soil and water

$$x_{o2} := R_{xo_2} = 0.19\,m$$

Location of the resultant

$L_{wall} = 1.6\,m$

$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$

Foundation Pressures

$\gamma_{s.L} = 21 \cdot \frac{kN}{m^3}$

$t = 0.3\,m$

$b = 1\,m$

$D_p = 0.6\,m$

$b2 = 0.3\,m$

$b3 = 1\,m$

$h_{w.L} = 0\,m$

$h_{w.R} = 2.7\,m$

$q_{max2} := R_{q.max_2} = 162.26\,kPa$ *Maximum bearing stress on the base*

$q_{min2} := R_{q.min_2} = 0\,kPa$ *Minimum bearing stress on the base*

$$B := \begin{cases} L_{wall} & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o2}) & \text{otherwise} \end{cases} = 0.573 \cdot m$$

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 0.57 \cdot m$$

Length of the base subject to bearing stresses

$$q1 := \begin{cases} \left[q_{max2} - \frac{b2}{L_{wall}} \cdot (q_{max2} - q_{min2})\right] & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{B - b2}{B} \cdot q_{max2}\right) & \text{if } (3 \cdot x_{o2} \ge b2) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 77.37 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t2} := \begin{cases} \left(\frac{q_{max2} + q1}{2} \cdot b2 \cdot b\right) & \text{if } 3 \cdot x_{o2} \ge b2 \\ \left(\frac{1}{2} \cdot q_{max2} \cdot B\right) \cdot b & \text{otherwise} \end{cases} = 35.94 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t2} := \begin{cases} \left[\frac{1}{2} \cdot q1 \cdot b2^2 + \frac{1}{3} \cdot (q_{max2} - q1) \cdot b2^2\right] \cdot b & \text{if } 3 \cdot x_{o2} \ge b2 \\ \left[\frac{1}{2} \cdot q_{max2} \cdot B \cdot \left(b2 - \frac{B}{3}\right) \cdot b\right] & \text{otherwise} \end{cases} = 6.03 \cdot kN \cdot m$$

Moment due to bearing stresses

$$V_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot b2 + \gamma_w \cdot \frac{b2^2}{2B} \cdot (h_{w.R} - h_{w.L})\right] \cdot b = 2.08 \cdot kN$$

Shear force due to uplift

$$M_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot \frac{b2^2}{2} + \gamma_w \cdot \frac{b2^2}{2B} \cdot (h_{w.R} - h_{w.L}) \cdot \frac{b2}{3}\right] \cdot b = 0.21\,m \cdot kN$$

Moment due to uplift

Factored Forces

Factored Shear Force

$V_{f2} := 1.5 \cdot (V_{t2} + V_{U2}) - 0.9 \cdot V_{wc} = 51.73\,kN$ *Factored Shear Force*

Factored Moment Force

$M_{f2} := 1.5 \cdot (M_{t2} + M_{U2}) - 0.9 \cdot M_{wc} = 8.56\,kN \cdot m$ *Factored Moment Force*

$V_f := \max(V_{f1}, V_{f2}) = 51.73\,kN$ *Maximum Factored Shear Force*

$M_f := \max(M_{f1}, M_{f2}) = 8.56\,kN \cdot m$ *Maximum Factored Moment Force*

Shear Capacity Calculation

Structure Geometry

$d_{b\_15} := 16 \cdot mm$ *Diameter of rebars 15M*

$S_p := 300 \cdot mm$ *Rebar spacing (15M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ *Diameter of rebars 20M*

$b := 1000mm$ *Width of member*

$c := 75mm$ *Concrete cover*

$h := t = 0.3\,m$ *Overall member thickness*

$$A_{db.15} := \left[\frac{\pi \cdot (d_{b.15})^2}{4}\right] = 201.06 \cdot mm^2$$ *Cross-sectional area of 15M rebar (Primary rebar)*

$$A_s := \frac{b}{Sp} \cdot A_{db.15} = 670.21 \cdot mm^2$$ *Primary rebar area*

$$d := h - c - \frac{d_{b.15}}{2} = 217 \cdot mm$$ *Distance from extreme compression fibre to centroid of the 15 M tension bars*

$$\left(d_v := \max(0.9 \cdot d, 0.72 \cdot h) = 0.22\,m\right)$$ *Effective shear depth*

$$M_f := \begin{cases} M_f & \text{if } M_f > V_f \cdot d_v \\ V_f \cdot d_v & \text{otherwise} \end{cases} = 11.17 \cdot kN \cdot m$$ *Factored moment*

$$\varepsilon_x := \frac{\frac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot As} = 0.00039$$ *Longitudinal strain at mid-depth of member due to factored loads* *CSA A23.3-04 EQ 11-13*

$$\varepsilon_x := \begin{cases} \varepsilon_x & \text{if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} & \text{otherwise} \end{cases} = 0.00039$$

$(s_z := d_v)$ *Crack spacing parameter*

$$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$$ *Effective crack spacing parameter* *CSA A23.3-04 EQ 11-10*

$$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.27$$ *Factor accounting for shear resistance of cracked concrete* *CSA A23.3-04 EQ 11-11*

$$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'_c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 190.13\,kN$$ *Shear resistance due to concrete* *CSA A23.3-04 EQ 11-6*

$$Concrete\_Shear\_Capacity := \begin{cases} \text{"OK"} & \text{if } (V_c \ge V_f) \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

$(Concrete\_Shear\_Capacity) = \text{"OK"}$

$$\frac{V_f}{V_c} = 0.27$$

Tensile capacity of longitudinal rebar at end support

$$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 31.7 \cdot deg$$

$$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 83.75\,kN$$

$k_1 := 1.0$ $\quad k_2 := 1.0$ $\quad k_3 := 1.0$ $\quad k_4 := 0.8$

$$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f'_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$$

$$\boxed{l_{actual} := (t - c) = 0.23\,m}$$

$$l_{act} := \begin{cases} l_{actual} & \text{if } l_{actual} < l_d \\ l_d & \text{otherwise} \end{cases} = 0.23$$

$$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 91.29\,kN$$

$$Embedment\_Check := \begin{cases} \text{"OK"} & \text{if } T_r \geq T_f \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\boxed{(Embedment\_Check) = \text{"OK"}}$$

$$\frac{T_f}{T_r} = 0.92$$

Moment Capacity Calculation

$$\alpha_1 := \begin{cases} .85 - .0015\frac{f'_c}{MPa} & \text{if } \left(.85 - .0015\frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

CSA A23.3-04 Clause 10.1.7

$$\beta_1 := \begin{cases} .97 - .0025\frac{f'_c}{MPa} & \text{if } \left(.97 - .0025\frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

$$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c} = 0.02\,m$$

$$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 47.48 \cdot kN \cdot m$$

$$Concrete\_Moment\_Capacity := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

$$\boxed{Concrete\_Moment\_Capacity = \text{"OK"}}$$

$$\left(\frac{M_f}{M_r}\right) = 0.24$$

3. Concrete Base Slab - Heel

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$L_{slab.R} = 1\,m$
$L_{wall} = 1.6\,m$
$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$t = 0.3\,m$
$b = 1\,m$
$b3 = 1\,m$

Unfactored Forces

Unfactored moment and shear forces from weight of slab and soil

$h_f := 0.9m$

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.R}^2}{2} + h_f \cdot \gamma_{s.R} \cdot b \cdot \frac{L_{slab.R}^2}{2} = 12.97 \cdot kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.R} + h_f \cdot \gamma_{s.R} \cdot b \cdot L_{slab.R} = 25.95 \cdot kN$$

$x_{o1} := R_{xo_1} = 0.71\,m$ *Location of the resultant*

Foundation Pressures

$q_{max1} := R_{q.max_1} = 47.78\,kPa$ *Maximum bearing stress on the base*

$q_{min1} := R_{q.min_1} = 24.46\,kPa$ *Minimum bearing stress on the base*

$$B := \begin{cases} L_{wall} & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o1}) & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the base subject to bearing stresses

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the section subject to bearing stresses

$$b3' := b3 - (L_{wall} - B) = 1\,m$$

$$q1 := \begin{cases} \left[q_{min1} + \frac{b3}{L_{wall}} \cdot (q_{max1} - q_{min1})\right] & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{b3'}{B} \cdot q_{max1}\right) & \text{if } (b3' \ge 0) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 39.04 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t1} := \begin{cases} \left(\frac{q_{min1} + q1}{2} \cdot b3 \cdot b\right) & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{2} \cdot q1 \cdot b3'\right) \cdot b & \text{otherwise} \end{cases} = 31.75 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t1} := \begin{cases} \left[\frac{1}{2} \cdot q_{min1} \cdot b3^2 + \frac{1}{3} \cdot (q1 - q_{min1}) \cdot \frac{b3^2}{2}\right] \cdot b & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{3} \cdot q1 \cdot \frac{b3'^2}{2} \cdot b\right) & \text{otherwise} \end{cases} = 14.66 \cdot kN \cdot m$$

Moment due to bearing stresses

Factored Forces

Factored Shear Force

$\boxed{V_{f1} := 1.5 \cdot (V_{t1}) - 0.9 \cdot V_{wc} = 24.27\,kN}$ *Factored Shear Force*

Factored Moment Force

$\boxed{M_{f1} := 1.5 \cdot (M_{t1}) - 0.9 \cdot M_{wc} = 10.31\,kN \cdot m}$ *Factored Moment Force*

KGS GROUP

DESIGN CALCULATIONS

Load Case 2 - Flood (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

Unfactored moment and shear forces from weight of slab, soil and water

$L_{slab.R} = 1\,m$
$L_{wall} = 1.6\,m$
$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.19 \cdot \frac{kN}{m^3}$
$\gamma_w = 9.81 \cdot \frac{kN}{m^3}$
$t = 0.3\,m$
$b = 1\,m$
$b3 = 1\,m$
$h_f = 0.9\,m$
$h_{w.R} = 2.7\,m$
$h_{w.L} = 0\,m$

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.R}^2}{2} + (h_{w.R} - t) \cdot \gamma_w \cdot b \cdot \frac{L_{slab.R}^2}{2} + h_f \cdot \gamma_{eff.R} \cdot b \cdot \frac{L_{slab.R}^2}{2} = 20.33 \cdot kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.R} + (h_{w.R} - t) \cdot \gamma_w \cdot b \cdot L_{slab.R} + h_f \cdot \gamma_{eff.R} \cdot b \cdot L_{slab.R} = 40.66 \cdot kN$$

$$x_{o2} := R_{xo_2} = 0.19\,m$$

Location of the resultant

Foundation Pressures

$$q_{max2} := R_{q.max_2} = 162.26\,kPa$$

Maximum bearing stress on the base

$$q_{min2} := R_{q.min_2} = 0\,kPa$$

Minimum bearing stress on the base

$$B := \begin{cases} L_{wall} & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o2}) & \text{otherwise} \end{cases} = 0.57 \cdot m$$

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 0.57 \cdot m$$

Length of the base subject to bearing stresses

$$b3' := b3 - (L_{wall} - B) = -0.03\,m$$

Length of the section subject to bearing stresses

$$q1 := \begin{cases} \left[q_{min2} + \frac{b3}{L_{wall}} \cdot (q_{max2} - q_{min2})\right] & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{b3'}{B} \cdot q_{max2}\right) & \text{if } (b3' \ge 0) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t2} := \begin{cases} \left(\frac{q_{min2} + q1}{2} \cdot b3 \cdot b\right) & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{2} \cdot q1 \cdot b3'\right) \cdot b & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t2} := \begin{cases} \left[\frac{1}{2} \cdot q_{min2} \cdot b3^2 + \frac{1}{3} \cdot (q1 - q_{min2}) \cdot \frac{b3^2}{2}\right] \cdot b & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{3} \cdot q1 \cdot \frac{b3'^2}{2} \cdot b\right) & \text{otherwise} \end{cases} = 0 \cdot kN \cdot m$$

Moment due to bearing stresses

$$h_{w.U2} := \left[h_{w.R} - \frac{(h_{w.R} - h_{w.L}) \cdot b3}{L_{wall}}\right] = 1.01 \cdot m$$

Shear force due to uplift

$$V_{U2} := \left[\gamma_w \cdot h_{w.U2} \cdot b3 + \gamma_w \cdot \frac{b3}{2} \cdot \left(h_{w.R} - h_{w.U2}\right)\right] \cdot b = 18.21 \cdot kN$$

Moment due to uplift

$$M_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot \frac{b3^2}{2} + \gamma_w \cdot \frac{b3}{2} \cdot \left(h_{w.R} - h_{w.U2}\right) \cdot \frac{2b3}{3}\right] \cdot b = 5.52\,m \cdot kN$$

Factored Forces

$$V_{f2} := 1.5 \cdot \left(V_{wc}\right) - 0.9 \cdot \left(V_{t2} + V_{U2}\right) = 44.61\,kN$$

Factored Shear Force

Factored Moment Force

Factored Moment Force

$$M_{f2} := 1.5 \cdot M_{wc} - 0.9 \cdot \left(M_{t2} + M_{U2}\right) = 25.53\,kN \cdot m$$

Maximum Factored Shear Force

$$V_f := \max\left(V_{f1}, V_{f2}\right) = 44.61\,kN$$

$$M_f := \max\left(M_{f1}, M_{f2}\right) = 25.53\,kN \cdot m$$

Maximum Factored Moment Force

Shear Capacity Calculation

Structure Geometry

$d_{b.15} := 16 \cdot mm$ *Diameter of rebars 15M*

$Sp := 300 \cdot mm$ *Rebar spacing (20M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ *Diameter of rebars 20M*

$(b := 1000mm)$ *Width of member*

$c := 75mm$ *Concrete cover*

$h := t = 0.3\,m$ *Overall member thickness*

$$A_{db.20} := \left[\frac{\pi \cdot \left(d_{b.20}\right)^2}{4}\right] = 298.65 \cdot mm^2$$

Cross-sectional area of 20M rebar (Primary rebar)

$$A_s := \frac{b}{Sp} \cdot A_{db.20} = 995.49 \cdot mm^2$$

Primary rebar area

$$d := h - c - \frac{d_{b.20}}{2} = 215.25 \cdot mm$$

Distance from extreme compression fibre to centroid of the 20 M tension bars

$$\left(d_v := \max(0.9 \cdot d, 0.72 \cdot h) = 0.22\,m\right)$$

Effective shear depth

$$M_f := \begin{vmatrix} M_f \text{ if } M_f > V_f \cdot d_v \\ V_f \cdot d_v \text{ otherwise} \end{vmatrix} = 25.53 \cdot kN \cdot m$$

Factored moment

$$\varepsilon_x := \frac{\frac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot A_s} = 0.00041$$

Longitudinal strain at mid-depth of member due to factored loads *CSA A23.3-04 EQ 11-13*

$$\varepsilon_x := \begin{vmatrix} \varepsilon_x \text{ if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} \text{ otherwise} \end{vmatrix} = 0.00041$$

$$\left(s_z := d_v\right)$$

Crack spacing parameter

$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$ *Effective crack spacing parameter* *CSA A23.3-04 EQ 11-10*

$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.27$ *Factor accounting for shear resistance of cracked concrete* *CSA A23.3-04 EQ 11-11*

$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'_c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 186.07\,kN$ *Shear resistance due to concrete* *CSA A23.3-04 EQ 11-6*

$\text{Concrete\_Shear\_Capacity} := \begin{cases} \text{"OK"} & \text{if } (V_c \geq V_f) \\ \text{"Not Good"} & \text{otherwise} \end{cases}$ $(\text{Concrete\_Shear\_Capacity}) = \text{"OK"}$

$\frac{V_f}{V_c} = 0.24$

Tensile capacity of longitudinal rebar at end support

$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 31.86 \cdot deg$

$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 71.77\,kN$

$k_1 := 1.0$ $k_2 := 1.0$ $k_3 := 1.0$ $k_4 := 0.8$

$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f'_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$

$(l_{actual} := (t - c) = 0.23\,m)$

$l_{act} := \begin{cases} l_{actual} & \text{if } l_{actual} < l_d \\ l_d & \text{otherwise} \end{cases} = 0.23$

$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 135.6\,kN$

$\text{Embedment\_Check} := \begin{cases} \text{"OK"} & \text{if } T_r \geq T_f \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{"OK"}$ $(\text{Embedment\_Check}) = \text{"OK"}$

$\frac{T_f}{T_r} = 0.53$

Moment Capacity Calculation

$\alpha_1 := \begin{cases} .85 - .0015\frac{f'_c}{MPa} & \text{if } \left(.85 - .0015\frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$ *CSA A23.3-04 Clause 10.1.7*

$\beta_1 := \begin{cases} .97 - .0025\frac{f'_c}{MPa} & \text{if } \left(.97 - .0025\frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$

$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c} = 0.03\,m$

$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 68.52 \cdot kN \cdot m$

$$\text{Concrete\_Moment\_Capacity} := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

Concrete_Moment_Capacity = "OK"

$$\left(\frac{M_f}{M_r}\right) = 0.37$$

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-3456-789	**Project Name :**	Churchville Condition Assessment		
File No. :		**Discipline :**	Structural Engineering		
Calculation Title :	**Floodwall Strength Analysis – Configuration 2**				
Calculation No. :	CIV-004	**Prepared by :**	JL	**Date :**	23/12/12
No. of Sheets :		**Checked by :**	YF	**Date :**	23/12/13
Supersedes Calc. No. :		**Approved by :**		**Date :**	

Calculation Description :

Strength analysis of the gravity retaining section.

Related Design Concept :

Reference Codes and Standards :

1. CAN/CSA 23.3 – 04, Design of concrete structure
2. CDA Dam Safety Guidelines 2007
3. Structural Design and Factors of Safety – Technical Bulletin Ontario Ministry of Natural Resources (August 2011)
4. "Foundation Analysis and Design" by Joseph. E. Bowles.

ENGINEER'S SEAL

Rev. #	**Rev. Description**	**Rev. Author**	**Date Revised**	**Checked by**	**Approved by**	**Approved Date**

Reference to stability analysis

Reference:P:\Projects\2023\23-4168-001\Design\Struct\Churchville Stability Analysis 2.xmcd

1. Concrete Stem wall design

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$(ELE_{soil.R}) = 173.3\,m$

$(\gamma_{s.R}) = 21 \cdot \frac{kN}{m^3}$

$(K_{o.R}) = 0.43$

$(B) = 1\,m$

$(\beta_R) = 0$

$(\gamma_{s.L}) = 21 \cdot \frac{kN}{m^3}$

Driving Pressure

Surcharge and lateral earth pressure

$q_{sur} := 4.8kPa$ — *Pressure from live load surcharge*

$h := ELE_{soil.R} - 170.6m = 2.7\,m$ — *Height of backfill soil on the right side above the base slab*

$P_{as} := \left[\left(\frac{1}{2}\right) \cdot \gamma_{s.R} \cdot h^2 + q_{sur} \cdot h\right] \cdot K_{a.R} \cdot B = 24.25 \cdot kN$ — *Pressure from live load surcharge and soil*

Horizontal component at ys above stem bottom

$H_{as} := P_{as} \cdot \cos((\beta_R)) = 24.25\,kN$ — *Pressure from live load surcharge and soil accounting for backfill slope*

$y_s := \frac{h}{3} \cdot \left[\frac{3 \cdot q_{sur} + \gamma_{s.R} \cdot (h)}{2 \cdot q_{sur} + \gamma_{s.R} \cdot (h)}\right] = 0.97\,m$ — *Moment arm of horizontal driving pressure*

Moment at bottom of stem

$M_s := H_{as} \cdot y_s = 23.41\,kN \cdot m$ — *Moment at the bottom of the stem wall from driving force*

Passive Pressure

Passive earth pressure

$D_p := ELE_{soil.L} - 170.6m = 0.6\,m$ — *Depth of effective passive pressure to bottom of stem wall*

$P_{p.1} := \left(\frac{1}{2}\right) \cdot \gamma_{s.L} \cdot D_p^2 \cdot K_{p.L} \cdot B = 13.95 \cdot kN$ — *Passive pressure acting on the stem wall*

Moment arm ys above stem bottom

$y_{p.1} := \frac{D_p}{3} = 0.2\,m$ — *Moment arm of horizontal passive pressure*

Moment at bottom of stem

$M_{p.1} := P_{p.1} \cdot y_{p.1} = 2.79\,kN \cdot m$ — *Moment at the bottom of the stem wall from passive force*

Factored Forces

Factored Shear Force

$\boxed{V_{f1} := 1.5 \cdot H_{as} - 0.9 \cdot P_{p.1} = 23.83\,kN}$ — *Factored Shear Force*

Factored Moment Force

$\boxed{M_{f1} := 1.5 \cdot M_s - 0.9 \cdot M_{p.1} = 32.6\,kN \cdot m}$ — *Factored Moment Force*

Load Case 2 - Flood (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$(ELE_{water.R}) = 172m$

$(ELE_{Base.R}) = 170.3m$

$(ELE_{water.L}) = 170.3m$

$(ELE_{Base.L}) = 170.3m$

$(\gamma_{s.R}) = 21 \cdot \frac{kN}{m^3}$

$(\gamma_{eff.R}) = 11.19 \cdot \frac{kN}{m^3}$

$(\gamma_{s.L}) = 21 \cdot \frac{kN}{m^3}$

$(\gamma_{eff.L}) = 11.19 \cdot \frac{kN}{m^3}$

$(\gamma_w) = 9.81 \cdot \frac{kN}{m^3}$

$(K_{a.R}) = 0.27$

$(B) = 1m$

$(K_{p.L}) = 3.69$

$(k_h) = 0.15$

$(D_p) = 0.6m$

$h_{w.R} := ELE_{water.R} - ELE_{Base.R} = 1.7m$ *Height of water above the bottom of the stem*

$h_{w.L} := ELE_{water.L} - ELE_{Base.L} = 0m$

Driving Pressure

Surcharge and lateral earth pressure

$q_{sur} := 2kPa$ *Pressure from live load surcharge*

$H_{sur} := q_{sur} \cdot h \cdot K_{a.R} \cdot B = 1.46 \cdot kN$ *Pressure from live load surcharge and soil*

$y_{sur} := \frac{h}{2} = 1.35m$ *Moment arm of force from surcharge*

$t := 0.3m$ *Thickness of the base*

$h_{wt.R} := \begin{cases} (h_{w.R} - t) & \text{if } h_{w.R} > t \\ 0 & \text{otherwise} \end{cases} = 1.4$ *Height of water above the right side of the base*

Lateral force due to backfill above water

$h_{1.R} := \begin{cases} (h - h_{wt.R}) & \text{if } h_{w.R} > t \wedge h_{w.R} < h \\ 0 & \text{if } h_{wt.R} \geq h \\ D_p & \text{otherwise} \end{cases} = 1.3$ *Height of backfill above water*

$h_{1.R} := \min(h_{1.R}, h) = 1.3m$

$H_{b1.R} := \frac{1}{2} \cdot \gamma_{s.R} \cdot (h_{1.R})^2 \cdot K_{a.R} \cdot B = 4.81 \cdot kN$ *Pressure from backfill above water*

$y_{b1.R} := \begin{cases} 0 & \text{if } h_{wt.R} \geq h \\ \left[h_{wt.R} + \left(\frac{h_{1.R}}{3}\right)\right] & \text{otherwise} \end{cases} = 1.83$ *Moment Arm from backfill above water*

Lateral force due to backfill submerged in water

$h_{2.R} := \begin{cases} h_{wt.R} & \text{if } h_{wt.R} < h \\ h & \text{otherwise} \end{cases} = 1.4$ *Height of backfill submerged in water*

$H_{b2.R} := \frac{1}{2} \cdot (2\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R}) \cdot h_{2.R} \cdot K_{a.R} \cdot B = 13.33 \cdot kN$ *Pressure from backfill submerged in water*

$y_{b2.R} := h_{2.R} - \frac{h_{2.R} \cdot [2(\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R}) + \gamma_{s.R} \cdot h_{1.R}]}{3 \cdot (2\gamma_{s.R} \cdot h_{1.R} + \gamma_{eff.R} \cdot h_{2.R})} = 0.65m$ *Moment Arm from backfill submerged in water*

Lateral force due to water

$H_{w.R} := \frac{1}{2} \cdot \gamma_w \cdot (h_{wt.R})^2 \cdot B = 9.61 \cdot kN$

Pressure from water

$y_{w.R} := \frac{1}{3} \cdot (h_{wt.R}) = 0.47m$

Moment Arm from water pressure

Passive Pressure

$h_{wt.L} := \begin{cases} (h_{w.L} - t) & \text{if } h_{w.L} > t \\ 0 & \text{otherwise} \end{cases} = 0$

Lateral force due to backfill above water

$h_{1.L} := \begin{cases} (D_p - h_{wt.L}) & \text{if } h_{w.L} > t \wedge h_{w.L} < D_p \\ 0 & \text{if } h_{w.L} \ge D_p \\ D_p & \text{otherwise} \end{cases} = 0.6$

$h_{1.L} := \min(h_{1.L}, D_p) = 0.6m$

$H_{b1.L} := \frac{1}{2} \cdot \gamma_{s.L} \cdot (h_{1.L})^2 \cdot K_{p.L} \cdot B = 13.95 \cdot kN$

$y_{b1.L} := \begin{cases} 0 & \text{if } h_{w.L} \ge D_p \\ \left[h_{wt.L} + \left(\frac{h_{1.L}}{3}\right)\right] & \text{otherwise} \end{cases} = 0.2$

Lateral force due to backfill submerged in water

$h_{2.L} := \begin{cases} h_{wt.L} & \text{if } h_{wt.L} < D_p \\ D_p & \text{otherwise} \end{cases} = 0$

$H_{b2.L} := \frac{1}{2} \cdot (2\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L}) \cdot h_{2.L} \cdot K_{p.L} \cdot B = 0 \cdot kN$

$y_{b2.L} := h_{2.L} - \frac{h_{2.L} \cdot [2(\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L}) + \gamma_{s.L} \cdot h_{1.L}]}{3 \cdot (2\gamma_{s.L} \cdot h_{1.L} + \gamma_{eff.L} \cdot h_{2.L})} = 0m$

Lateral force due to water

$H_{w.L} := \frac{1}{2} \cdot \gamma_w \cdot (h_{wt.L})^2 \cdot B = 0 \cdot kN$

$y_{w.L} := \frac{1}{3} \cdot (h_{wt.L}) = 0m$

Factored Forces

Factored Shear Force

Factored Shear Force

$\boxed{V_{f2} := 1.5 \cdot (H_{b1.R} + H_{b2.R} + H_{sur} + H_{w.R}) - 0.9(H_{b1.L} + H_{b2.L} + H_{w.L}) = 31.27kN}$

Factored Moment Force

Factored Moment Force

$\boxed{M_{f2} := 1.5 \cdot (H_{b1.R} \cdot y_{b1.R} + H_{b2.R} \cdot y_{b2.R} + H_{sur} \cdot y_{sur} + H_{w.R} \cdot y_{w.R}) - 0.9(H_{b1.L} \cdot y_{b1.L} + H_{b2.L} \cdot y_{b2.L} + H_{w.L} \cdot y_{w.L}) = 33.36kN \cdot m}$

$V_f := \max(V_{f1}, V_{f2}) = 31.27\,kN$ *Maximum Factored Shear Force*

$M_f := \max(M_{f1}, M_{f2}) = 33.36\,kN \cdot m$ *Maximum Factored Moment Force*

Shear Capacity Calculation

Material Properties

$N_f := 0kN$ *Factored axial forces*

$\phi_s := 0.85$ *Rebar tensile strength*

$\phi_c := 0.65$ *Concrete resistance factor*

$f_y := 400MPa$ *Yield stength of steel rebar*

$\lambda := 1.0$ *Factor with regards to concrete density*

$E_s := 200000MPa$ *Modulus of elasticity for steel rebar*

$f'_c := 25MPa$ *Specified compressive strength of concrete*

$a_g := 20mm$ *Maximum size of coarse aggregate*

$E_c := 4500 \cdot \sqrt{f'_c \cdot MPa} = 22500 \cdot MPa$ *Modulus of elasticity for concrete*

Structure Geometry

$d_{b.15} := 16 \cdot mm$ *Diameter of rebars 15M*

$Sp := 300 \cdot mm$ *Rebar spacing (20M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ *Diameter of rebars 20M*

$b := 1000mm$ *Width of member*

$c := 75mm$ *Concrete cover*

$h := 300mm = 0.3\,m$ *Overall member thickness*

$$A_{db.20} := \left[\frac{\pi \cdot (d_{b.20})^2}{4}\right] = 298.65 \cdot mm^2$$ *Cross-sectional area of 20M rebar (Primary rebar)*

$$As := \frac{b}{Sp} \cdot A_{db.20} = 995.49 \cdot mm^2$$ *Primary rebar area*

$$d := h - c - \frac{d_{b.20}}{2} = 215.25 \cdot mm$$ *Distance from extreme compression fibre to centroid of the 20 M tension bars*

$$d_v := \max(0.9 \cdot d, 0.72 \cdot h)$$ *Effective shear depth*

$$M_f := \begin{cases} M_f & \text{if } M_f > V_f \cdot d_v \\ V_f \cdot d_v & \text{otherwise} \end{cases} = 33.36 \cdot kN \cdot m$$ *Factored moment*

$$\varepsilon_x := \frac{\frac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot As} = 0.00047$$ *Longitudinal strain at mid-depth of member due to factored loads* CSA A23.3-04 EQ 11-13

$$\varepsilon_x := \begin{cases} \varepsilon_x & \text{if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} & \text{otherwise} \end{cases} = 0.00047$$

$s_z := d_v$ *Crack spacing parameter*

$$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$$ *Effective crack spacing parameter* CSA A23.3-04 EQ 11-10

$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.25$ *Factor accounting for shear resistance of cracked concrete* *CSAA23.3-04 EQ 11-11*

$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'_c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 176.63\,kN$ *Shear resistance due to concrete* *CSAA23.3-04 EQ 11-6*

Check for shear friction if the wall was poured after the base had partially cured

$(c_c := 0.25MPa)$

$(\mu := 0.6)$

$A_{vf} := As$ $A_{cv} := d \cdot b$

$\rho_v := \frac{A_{vf}}{A_{cv}} = 0$

$\sigma := \rho_v \cdot f_y = 1.85 \cdot MPa$

$v_r := \phi_c \cdot \lambda \cdot (c_c + \mu \cdot \sigma) = 0.88 \cdot MPa$

$V_r := \begin{vmatrix} v_r \cdot A_{cv} \text{ if } v_r < 0.25 \cdot \phi_c \cdot f'_c \\ (0.25 \cdot \phi_c \cdot f'_c) \cdot A_{cv} \text{ otherwise} \end{vmatrix} = 190.27 \cdot kN$

CAN/CSAA23.3-04 Cl. 11.3.9.5

$\text{Concrete\_Shear\_Capacity} := \begin{vmatrix} \text{"OK" if } V_c \ge V_f \wedge V_r \ge V_f \\ \text{"Not Good" otherwise} \end{vmatrix}$ $(\text{Concrete\_Shear\_Capacity}) = \text{"OK"}$

$\frac{V_f}{V_c} = 0.18$

Tensile capacity of longitudinal rebar at end support

$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 32.26 \cdot deg$

$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 49.53\,kN$

$k_1 := 1.0$ $k_2 := 1.0$ $k_3 := 1.0$ $k_4 := 0.8$

$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f'_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$

$(l_{actual} := (t - c) = 0.23\,m)$

$l_{act} := \begin{vmatrix} l_{actual} \text{ if } l_{actual} < l_d \\ l_d \text{ otherwise} \end{vmatrix} = 0.23$

$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 135.6\,kN$

$\text{Embedment\_Check} := \begin{vmatrix} \text{"OK" if } T_r \ge T_f \\ \text{"Not Good" otherwise} \end{vmatrix} = \text{"OK"}$ $(\text{Embedment\_Check}) = \text{"OK"}$

$\frac{T_f}{T_r} = 0.37$

Moment Capacity Calculation

$$\alpha_1 := \begin{cases} .85 - .0015\dfrac{f'_c}{MPa} & \text{if } \left(.85 - .0015\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

CSAA23.3-04 Clause 10.1.7

$$\beta_1 := \begin{cases} .97 - .0025\dfrac{f'_c}{MPa} & \text{if } \left(.97 - .0025\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$$

$$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c}$$

$$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 68.52 \cdot kN \cdot m$$

$$\text{Concrete\_Moment\_Capacity} := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

Concrete_Moment_Capacity = "OK"

$$\left(\frac{M_f}{M_r}\right) = 0.49$$

2. Concrete Base Slab - Heel

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

$(L_{slab.L}) = 1\,m$

$(\gamma_{conc}) = 23.5 \cdot \dfrac{kN}{m^3}$

$(\gamma_{s.L}) = 21 \cdot \dfrac{kN}{m^3}$

$(t) = 0.3\,m$

$(b) = 1\,m$

$(D_p) = 0.6\,m$

Unfactored Forces

Unfactored moment and shear forces from weight of slab and soil

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.L}^2}{2} + D_p \cdot \gamma_{s.L} \cdot b \cdot \frac{L_{slab.L}^2}{2} = 9.82 \cdot kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.L} + D_p \cdot \gamma_{s.L} \cdot b \cdot L_{slab.L} = 19.65 \cdot kN$$

$x_{o1} := R_{xo_1} = 0.68\,m$ *Location of the resultant*

Foundation Pressures

$q_{max1} := R_{q.max_1} = 55.16\,kPa$ *Maximum bearing stress on the base*

$q_{min1} := R_{q.min_1} = 21.54\,kPa$ *Minimum bearing stress on the base*

$L_{wall} := 1.6m$

$b2 := L_{slab.L} = 1\,m$ *Length of the toe*

$b3 := L_{slab.R} = 0.3\,m$ *Length of the heel*

$$B := \begin{cases} L_{wall} & \text{if } x_{o1} \geq \frac{L_{wall}}{3} \\ (3 \cdot x_{o1}) & \text{otherwise} \end{cases} = 1.6 \cdot m$$

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the base subject to bearing stresses

$$q1 := \begin{cases} \left[q_{max1} - \frac{b2}{L_{wall}} \cdot (q_{max1} - q_{min1})\right] & \text{if } x_{o1} \geq \frac{L_{wall}}{3} \\ \left(\frac{B - b2}{B} \cdot q_{max1}\right) & \text{if } (3 \cdot x_{o1} \geq b2) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 34.15 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t1} := \begin{cases} \left(\frac{q_{max1} + q1}{2} \cdot b2 \cdot b\right) & \text{if } 3 \cdot x_{o1} \geq b2 \\ \left(\frac{1}{2} \cdot q_{max1} \cdot B\right) \cdot b & \text{otherwise} \end{cases} = 44.66 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t1} := \begin{cases} \left[\frac{1}{2} \cdot q1 \cdot b2^2 + \frac{1}{3} \cdot (q_{max1} - q1) \cdot b2^2\right] \cdot b & \text{if } 3 \cdot x_{o1} \geq b2 \\ \left[\frac{1}{2} \cdot q_{max1} \cdot B \cdot \left(b2 - \frac{B}{3}\right) \cdot b\right] & \text{otherwise} \end{cases} = 24.08 \cdot kN \cdot m$$

Moment due to bearing stresses

Factored Forces

Factored Shear Force

$V_{f1} := 1.5 \cdot (V_{t1}) - 0.9 \cdot V_{wc} = 49.3\,kN$

Factored Shear Force

Factored Moment Force

$(M_{f1} := 1.5 \cdot (M_{t1}) - 0.9 \cdot M_{wc} = 27.28\,kN \cdot m)$

Factored Moment Force

Load Case 2 - Wet (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

$(L_{wall}) = 1.6\,m$

$(\gamma_{conc}) = 23.5 \cdot \frac{kN}{m^3}$

$(\gamma_{s.L}) = 21 \cdot \frac{kN}{m^3}$

$(t) = 0.3\,m$

$(b) = 1\,m$

$(D_p) = 0.6\,m$

$(b2) = 1\,m$

$(b3) = 0.3\,m$

$(h_{w.L}) = 0\,m$

$(h_{w.R}) = 1.7\,m$

Unfactored moment and shear forces from weight of slab, soil and water

$$x_{o2} := R_{xo_2} = 0.43\,m$$

Location of the resultant

Foundation Pressures

$$q_{max2} := R_{q.max_2} = 67.81\,kPa$$

Maximum bearing stress on the base

$$q_{min2} := R_{q.min_2} = 0\,kPa$$

Minimum bearing stress on the base

$$B := \begin{cases} L_{wall} & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o2}) & \text{otherwise} \end{cases} = 1.279 \cdot m$$

Length of the base subject to bearing stresses

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.28 \cdot m$$

Bearing stress at the critical location of the section (at the stem)

$$q1 := \begin{cases} \left[q_{max2} - \frac{b2}{L_{wall}} \cdot (q_{max2} - q_{min2})\right] & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{B - b2}{B} \cdot q_{max2}\right) & \text{if } (3 \cdot x_{o2} \ge b2) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 14.78 \cdot kN$$

Shear force due to bearing stresseses

$$V_{t2} := \begin{cases} \left(\frac{q_{max2} + q1}{2} \cdot b2 \cdot b\right) & \text{if } 3 \cdot x_{o2} \ge b2 \\ \left(\frac{1}{2} \cdot q_{max2} \cdot B\right) \cdot b & \text{otherwise} \end{cases} = 41.3 \cdot kN$$

Moment due to bearing stresses

$$M_{t2} := \begin{cases} \left[\frac{1}{2} \cdot q1 \cdot b2^2 + \frac{1}{3} \cdot (q_{max2} - q1) \cdot b2^2\right] \cdot b & \text{if } 3 \cdot x_{o2} \ge b2 \\ \left[\frac{1}{2} \cdot q_{max2} \cdot B \cdot \left(b2 - \frac{B}{3}\right) \cdot b\right] & \text{otherwise} \end{cases} = 25.07 \cdot kN \cdot m$$

Shear force due to uplift

$$V_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot b2 + \gamma_w \cdot \frac{b2^2}{2B} \cdot (h_{w.R} - h_{w.L})\right] \cdot b = 6.52 \cdot kN$$

Moment due to uplift

$$M_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot \frac{b2^2}{2} + \gamma_w \cdot \frac{b2^2}{2B} \cdot (h_{w.R} - h_{w.L}) \cdot \frac{b2}{3}\right] \cdot b = 2.17\,m \cdot kN$$

Factored Forces

Factored Shear Force

$$V_{f2} := 1.5 \cdot (V_{t2} + V_{U2}) - 0.9 \cdot V_{wc} = 54.04\,kN$$

Factored Shear Force

Factored Moment Force

$$M_{f2} := 1.5 \cdot (M_{t2} + M_{U2}) - 0.9 \cdot M_{wc} = 32.02\,kN \cdot m$$

Factored Moment Force

$$V_f := \max(V_{f1}, V_{f2}) = 54.04\,kN$$

Maximum Factored Shear Force

$$M_f := \max(M_{f1}, M_{f2}) = 32.02\,kN \cdot m$$

Maximum Factored Moment Force

Shear Capacity Calculation

Structure Geometry

$d_{b.15} := 16 \cdot mm$ — *Diameter of rebars 15M*

$Sp := 300 \cdot mm$ — *Rebar spacing (15M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ — *Diameter of rebars 20M*

$(b := 1000mm)$ — *Width of member*

$c := 75mm$ — *Concrete cover*

$h := t = 0.3\,m$ — *Overall member thickness*

$$A_{db.15} := \left[\frac{\pi \cdot (d_{b.15})^2}{4}\right] = 201.06 \cdot mm^2$$

Cross-sectional area of 15M rebar (Primary rebar)

$$As := \frac{b}{Sp} \cdot A_{db.15} = 670.21 \cdot mm^2$$

Primary rebar area

$$d := h - c - \frac{d_{b.15}}{2} = 217 \cdot mm$$

Distance from extreme compression fibre to centroid of the 15 M tension bars

$$(d_v := \max(0.9 \cdot d, 0.72 \cdot h) = 0.22\,m)$$

Effective shear depth

$$M_f := \begin{cases} M_f & \text{if } M_f > V_f \cdot d_v \\ V_f \cdot d_v & \text{otherwise} \end{cases} = 32.02 \cdot kN \cdot m$$

Factored moment

$$\varepsilon_x := \frac{\frac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot As} = 0.00075$$

Longitudinal strain at mid-depth of member due to factored loads — *CSAA23.3-04 EQ 11-13*

$$\varepsilon_x := \begin{cases} \varepsilon_x & \text{if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} & \text{otherwise} \end{cases} = 0.00075$$

$(s_z := d_v)$ — *Crack spacing parameter*

$$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$$

Effective crack spacing parameter — *CSAA23.3-04 EQ 11-10*

$$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.2$$

Factor accounting for shear resistance of cracked concrete — *CSAA23.3-04 EQ 11-11*

$$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 140.82\,kN$$

Shear resistance due to concrete — *CSAA23.3-04 EQ 11-6*

$$\text{Concrete\_Shear\_Capacity} := \begin{cases} \text{"OK"} & \text{if } (V_c \ge V_f) \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

$(\text{Concrete\_Shear\_Capacity}) = \text{"OK"}$

$$\frac{V_f}{V_c} = 0.38$$

Tensile capacity of longitudinal rebar at end support

$$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 34.28 \cdot deg$$

$$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 79.27\,kN$$

$k_1 := 1.0$ $k_2 := 1.0$ $k_3 := 1.0$ $k_4 := 0.8$

$$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f'_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$$

$$l_{actual} := (t - c) = 0.23\,m$$

$$l_{act} := \begin{cases} l_{actual} & \text{if } l_{actual} < l_d \\ l_d & \text{otherwise} \end{cases} = 0.23$$

$$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 91.29\,kN$$

$$Embedment\_Check := \begin{cases} \text{"OK"} & \text{if } T_r \geq T_f \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$(Embedment\_Check) = \text{"OK"}$

$$\frac{T_f}{T_r} = 0.87$$

Moment Capacity Calculation

$$\alpha_1 := \begin{cases} .85 - .0015\dfrac{f'_c}{MPa} & \text{if } \left(.85 - .0015\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases} = 0.81$$

CSA A23.3-04 Clause 10.1.7

$$\beta_1 := \begin{cases} .97 - .0025\dfrac{f'_c}{MPa} & \text{if } \left(.97 - .0025\dfrac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases} = 0.91$$

$$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c} = 0.02\,m$$

$$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 47.48 \cdot kN \cdot m$$

$$Concrete\_Moment\_Capacity := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$$

$Concrete\_Moment\_Capacity = \text{"OK"}$

$$\left(\frac{M_f}{M_r}\right) = 0.67$$

3. Concrete Base Slab - Toe

Load Case 1 - Dry (Backfill only)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

$(L_{slab.R}) = 0.3\,m$
$(L_{wall}) = 1.6\,m$
$(\gamma_{conc}) = 23.5 \cdot \frac{kN}{m^3}$
$(\gamma_{s.R}) = 21 \cdot \frac{kN}{m^3}$
$(t) = 0.3\,m$
$(b) = 1\,m$
$(b3) = 0.3\,m$

Unfactored moment and shear forces from weight of slab and soil

$h_f := 0.9m$

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.R}^2}{2} + h_f \cdot \gamma_{s.R} \cdot b \cdot \frac{L_{slab.R}^2}{2} = 1.17 \cdot kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.R} + h_f \cdot \gamma_{s.R} \cdot b \cdot L_{slab.R} = 7.79 \cdot kN$$

$x_{o1} := R_{xo_1} = 0.68\,m$ *Location of the resultant*

Foundation Pressures

$q_{max1} := R_{q.max_1} = 55.16\,kPa$ *Maximum bearing stress on the base*

$q_{min1} := R_{q.min_1} = 21.54\,kPa$ *Minimum bearing stress on the base*

$$B := \begin{cases} L_{wall} & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o1}) & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the base subject to bearing stresses

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.6 \cdot m$$

Length of the section subject to bearing stresses

$$b3' := b3 - (L_{wall} - B) = 0.3\,m$$

$$q1 := \begin{cases} \left[q_{min1} + \frac{b3}{L_{wall}} \cdot (q_{max1} - q_{min1})\right] & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{b3'}{B} \cdot q_{max1}\right) & \text{if } (b3' \ge 0) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 27.85 \cdot kN$$

Bearing stress at the critical location of the section (at the stem)

$$V_{t1} := \begin{cases} \left(\frac{q_{min1} + q1}{2} \cdot b3 \cdot b\right) & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{2} \cdot q1 \cdot b3'\right) \cdot b & \text{otherwise} \end{cases} = 7.41 \cdot kN$$

Shear force due to bearing stresseses

$$M_{t1} := \begin{cases} \left[\frac{1}{2} \cdot q_{min1} \cdot b3^2 + \frac{1}{3} \cdot (q1 - q_{min1}) \cdot \frac{b3^2}{2}\right] \cdot b & \text{if } x_{o1} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{3} \cdot q1 \cdot \frac{b3'^2}{2} \cdot b\right) & \text{otherwise} \end{cases} = 1.06 \cdot kN \cdot m$$

Moment due to bearing stresses

Factored Forces

Factored Shear Force

$V_{f1} := 1.5 \cdot (V_{wc}) - 0.9 \cdot V_{t1} = 5.01\,kN$ *Factored Shear Force*

Factored Moment Force

$M_{f1} := 1.5 \cdot (M_{wc}) - 0.9 \cdot M_{t1} = 0.79\,kN \cdot m$ *Factored Moment Force*

Load Case 2 - Flood (Backfill and Hydrostatic)

Shear and Moment Force at Critical Section

Conservative for Shear Force

Unfactored Forces

Unfactored moment and shear forces from weight of slab, soil and water

$ELE_{soil.R} = 173.3m$
$ELE_{water.R} = 172m$
$L_{slab.R} = 0.3m$
$L_{wall} = 1.6m$
$\gamma_{conc} = 23.5 \cdot \frac{kN}{m^3}$
$\gamma_{s.R} = 21 \cdot \frac{kN}{m^3}$
$\gamma_{eff.R} = 11.19 \cdot \frac{kN}{m^3}$
$\gamma_w = 9.81 \cdot \frac{kN}{m^3}$
$t = 0.3m$
$b = 1m$
$b3 = 0.3m$
$h_f = 0.9m$
$h_{w.R} = 1.7m$
$h_{w.L} = 0m$

$$h_{dry} := ELE_{soil.R} - ELE_{water.R} = 1.3m$$

$$h_{sub} := ELE_{water.R} - 170.6m = 1.4m$$

$$M_{wc} := t \cdot \gamma_{conc} \cdot b \cdot \frac{L_{slab.R}^2}{2} + h_{sub} \cdot \left(\gamma_{eff.R} \cdot b \cdot \frac{L_{slab.R}^2}{2} + \gamma_w \cdot b \cdot \frac{L_{slab.R}^2}{2}\right) + h_{dry} \cdot \gamma_{s.R} \cdot b \cdot \frac{L_{slab.R}^2}{2} = 2.87kN \cdot m$$

$$V_{wc} := t \cdot \gamma_{conc} \cdot b \cdot L_{slab.R} + h_{dry} \cdot \gamma_{s.R} \cdot b \cdot L_{slab.R} + h_{sub} \cdot \left(\gamma_{eff.R} \cdot b \cdot L_{slab.R} + \gamma_w \cdot b \cdot L_{slab.R}\right) = 19.13 \cdot kN$$

$$x_{o2} := R_{xo_2} = 0.43m$$ *Location of the resultant*

Foundation Pressures

$$q_{max2} := R_{q.max_2} = 67.81kPa$$ *Maximum bearing stress on the base*

$$q_{min2} := R_{q.min_2} = 0kPa$$ *Minimum bearing stress on the base*

$$B := \begin{cases} L_{wall} & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ (3 \cdot x_{o2}) & \text{otherwise} \end{cases} = 1.28 \cdot m$$

$$B := \begin{cases} B & \text{if } B > 0 \\ 0 & \text{otherwise} \end{cases} = 1.28 \cdot m$$ *Length of the base subject to bearing stresses*

$$b3' := b3 - (L_{wall} - B) = -0.02m$$ *Length of the section subject to bearing stresses*

$$q1 := \begin{cases} \left[q_{min2} + \frac{b3}{L_{wall}} \cdot (q_{max2} - q_{min2})\right] & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{b3'}{B} \cdot q_{max2}\right) & \text{if } (b3' \ge 0) \wedge (B < L_{wall}) \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$ *Bearing stress at the critical location of the section (at the stem)*

$$V_{t2} := \begin{cases} \left(\frac{q_{min2} + q1}{2} \cdot b3 \cdot b\right) & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{2} \cdot q1 \cdot b3'\right) \cdot b & \text{otherwise} \end{cases} = 0 \cdot kN$$ *Shear force due to bearing stresseses*

$$M_{t2} := \begin{cases} \left[\frac{1}{2} \cdot q_{min2} \cdot b3^2 + \frac{1}{3} \cdot (q1 - q_{min2}) \cdot \frac{b3^2}{2}\right] \cdot b & \text{if } x_{o2} \ge \frac{L_{wall}}{3} \\ \left(\frac{1}{3} \cdot q1 \cdot \frac{b3'^2}{2} \cdot b\right) & \text{otherwise} \end{cases} = 0 \cdot kN \cdot m$$ *Moment due to bearing stresses*

Shear force due to uplift

$$h_{w.U2} := \left[h_{w.R} - \frac{(h_{w.R} - h_{w.L}) \cdot b3}{L_{wall}}\right] = 1.38 \cdot m$$

$$V_{U2} := \left[\gamma_w \cdot h_{w.U2} \cdot b3 + \gamma_w \cdot \frac{b3}{2} \cdot \left(h_{w.R} - h_{w.U2}\right)\right] \cdot b = 4.53 \cdot kN$$

Moment due to uplift

$$M_{U2} := \left[\gamma_w \cdot h_{w.L} \cdot \frac{b3^2}{2} + \gamma_w \cdot \frac{b3}{2} \cdot \left(h_{w.R} - h_{w.U2}\right) \cdot \frac{2b3}{3}\right] \cdot b = 0.09\,m \cdot kN$$

Factored Forces

Factored Shear Force

$$V_{f2} := 1.5 \cdot V_{wc} - 0.9\left(V_{t2} + V_{U2}\right) = 24.61\,kN$$

Factored Shear Force

Factored Moment Force

$$M_{f2} := 1.5 \cdot M_{wc} - 0.9\left(M_{t2} + M_{U2}\right) = 4.22\,kN \cdot m$$

Factored Moment Force

$$V_f := \max\left(V_{f1}, V_{f2}\right) = 24.61\,kN$$

Maximum Factored Shear Force

$$M_f := \max\left(M_{f1}, M_{f2}\right) = 4.22\,kN \cdot m$$

Maximum Factored Moment Force

Shear Capacity Calculation

Structure Geometry

$d_{b.15} := 16 \cdot mm$ *Diameter of rebars 15M*

$Sp := 300 \cdot mm$ *Rebar spacing (20M @ 300mm c/c)*

$d_{b.20} := 19.5 \cdot mm$ *Diameter of rebars 20M*

$(b := 1000mm)$ *Width of member*

$c := 75mm$ *Concrete cover*

$h := t = 0.3\,m$ *Overall member thickness*

$$A_{db.20} := \left[\frac{\pi \cdot \left(d_{b.20}\right)^2}{4}\right] = 298.65 \cdot mm^2$$

Cross-sectional area of 20M rebar (Primary rebar)

$$As := \frac{b}{Sp} \cdot A_{db.20} = 995.49 \cdot mm^2$$

Primary rebar area

$$d := h - c - \frac{d_{b.20}}{2} = 215.25 \cdot mm$$

Distance from extreme compression fibre to centroid of the 20 M tension bars

$$\left(d_v := \max(0.9 \cdot d, 0.72 \cdot h) = 0.22\,m\right)$$

Effective shear depth

$$M_f := \begin{cases} M_f & \text{if } M_f > V_f \cdot d_v \\ V_f \cdot d_v & \text{otherwise} \end{cases} = 5.32 \cdot kN \cdot m$$

Factored moment

$$\varepsilon_x := \frac{\dfrac{M_f}{d_v} + V_f + 0.5N_f}{2 \cdot E_s \cdot As} = 0.00012$$

Longitudinal strain at mid-depth of member due to factored loads *CSA A23.3-04 EQ 11-13*

$$\varepsilon_x := \begin{cases} \varepsilon_x & \text{if } \varepsilon_x \le 3.0 \cdot 10^{-3} \\ 3.0 \cdot 10^{-3} & \text{otherwise} \end{cases} = 0.00012$$

$(s_z := d_v)$ *Crack spacing parameter*

$s_{ze} := \frac{35 \cdot s_z}{15mm + a_g}$ *Effective crack spacing parameter* *CSAA23.3-04 EQ 11-10*

$\beta := \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = 0.36$ *Factor accounting for shear resistance of cracked concrete* *CSAA23.3-04 EQ 11-11*

$V_c := \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{(f'_c)} \cdot b \cdot d_v \cdot \sqrt{MPa} = 253.25 kN$ *Shear resistance due to concrete* *CSAA23.3-04 EQ 11-6*

$Concrete\_Shear\_Capacity := \begin{cases} \text{"OK"} & \text{if } V_c \geq V_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$ $(Concrete\_Shear\_Capacity) = \text{"OK"}$

$\frac{V_f}{V_c} = 0.1$

Tensile capacity of longitudinal rebar at end support

$\theta := 29deg + 7000 \cdot \varepsilon_x \cdot deg = 29.87 \cdot deg$

$T_f := V_f \cdot \cot(\theta) + 0.5N_f = 42.85 kN$

$k_1 := 1.0$ $k_2 := 1.0$ $k_3 := 1.0$ $k_4 := 0.8$

$l_d := 0.45 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot \frac{f_y}{\sqrt{f_c} \cdot \sqrt{MPa}} \cdot d_{b.20}$

$(l_{actual} := (t - c) = 0.23 m)$

$l_{act} := \begin{cases} l_{actual} & \text{if } l_{actual} < l_d \\ l_d & \text{otherwise} \end{cases} = 0.23$

$T_r := \frac{l_{act}}{l_d} \cdot 0.85 \cdot f_y \cdot As = 135.6 kN$

$Embedment\_Check := \begin{cases} \text{"OK"} & \text{if } T_r \geq T_f \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{"OK"}$ $(Embedment\_Check) = \text{"OK"}$

$\frac{T_f}{T_r} = 0.32$

Moment Capacity Calculation

$\alpha_1 := \begin{cases} .85 - .0015 \frac{f'_c}{MPa} & \text{if } \left(.85 - .0015 \frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$ *CSAA23.3-04 Clause 10.1.7*

$\beta_1 := \begin{cases} .97 - .0025 \frac{f'_c}{MPa} & \text{if } \left(.97 - .0025 \frac{f'_c}{MPa}\right) > 0.67 \\ 0.67 & \text{otherwise} \end{cases}$

$a := \frac{\phi_s \cdot As \cdot f_y}{\alpha_1 \cdot \phi_c \cdot b \cdot f'_c} = 0.03 m$

$M_r := \phi_s \cdot As \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 68.52 \cdot kN \cdot m$

$Concrete\_Moment\_Capacity := \begin{cases} \text{"OK"} & \text{if } M_r \geq M_f \\ \text{"Not Good"} & \text{otherwise} \end{cases}$

Concrete_Moment_Capacity = "OK"

$\left(\frac{M_f}{M_r}\right) = 0.08$

APPENDIX F

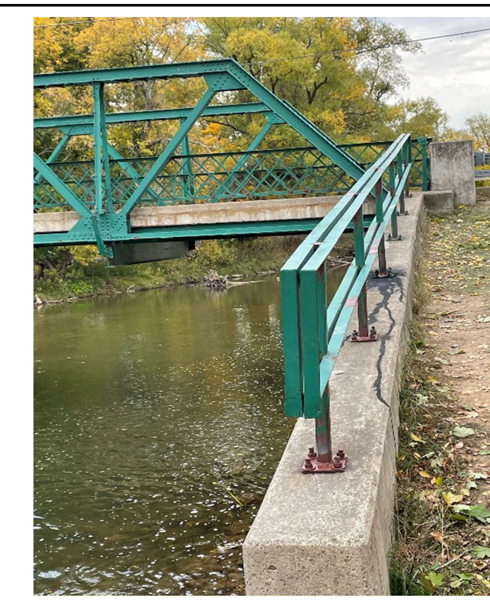
City Restoration Work Progress Summary

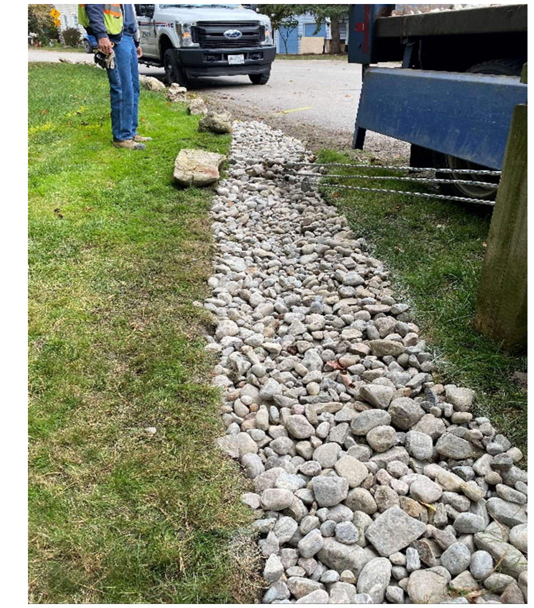
BRAMPTON

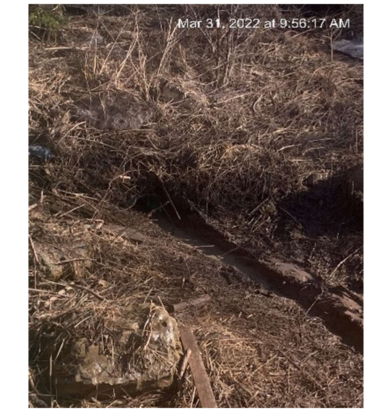
Public Works & Engineering

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
1	Spread ice in parking area not including	Contracts – Serve	Week of March 29		Completed	
2	Clear/flush & inspect Culverts	Contract Services	Week of March 29	Work has started	Completed	
3	Sign Installations	Traffic	Early April	Temporary and damaged signs to be reinstated	Completed	
4	Playground Inspection	Parks	May	To be inspected following park clean up	Completed	
5	Safety Station Inspection	Parks	May	To be inspected following park clean up	Completed	
6	Bridge	Capital Works	Early April	Repair lower rails and cables	Completed	

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
7	Debris Removal	Road Operations	Mid April	Drop off Bin and aid with debris removal	Completed	
8	Debris Removal	Forestry	Mid April	Remove all Forestry related debris in the park, parking lot and south of the bridge	Completed	
9	Debris Removal	Parks	Mid April	Remove all debris within the park	Completed	
10	Street Lighting Inspection in Parking Lot	Street Lighting	End of April	Completed after ice and Debris is removed	Completed	
11	Reinstate/adjust Guide Rails – on the ROW, in the Park and south of the bridge	Contracts – TBD	End of April	Completed after Debris removal	Completed	
12	Tree Planting	Forestry	Mid May	TBD – if required	NA	
13	Grass Restoration	Parks	Mid May	Method TBD after works have been completed	Completed	
14	Regrading area “dyke” area around area 5	Parks	End of May	Following completion of Guide Rail works Determine who will completing	Completed	

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
15	Parking Lot and Driveway – regrading	Parks	End of May – following completion of regrading work	Completed after all restoration works have been completed	Completed	
16	Repair end post NE corner of Bridge	Contract Service	Mid of September following locates	Poste replaced and cables adjusted	Completed	
17	Repair railing at SW corner of Bridge	Contract Service	October 12-14		Completed	
18	grade the stone at the parquetted located on N/W corner of Creditview and Churchville	Road Operations	End of August	Sandalwood Yard staff added addition gravel to remove the divots	Complete	

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
19	Swale improvement to catch basin in front of resident's house	Road Operations	End of September	Work will be scheduled in the near future	Completed	
20	repair signs at Bridge	Sign shop	End of September	Warning Sign at south end of bridge need to be secured.	Completed	
21	Repair Signs at Creditview	Sign shop	End of September	Street name sign for Churchville and Creditview needs to be straighten and secured.	Completed	
22	Repair Signs at Halstone	Sign shop	End of September	Halstone and Creditview street name sign is on a temp stand and need to be permanently installed	Completed	
23	Clear plant overgrowth along churchville	Parks	End of August	location is the north side of Church Street between Churchville Road and Victoria Street	Completed	
24	Relocate Mailboxes back to firehouse parking lot	Contract Services	Mid September	Mailbox relocated	Completed	

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
25	New Flap Gate installation at outfall #4 near Bridge	Contract Services	End of December 2022	Replaced with inline check valve in March 2023	Completed	
26	Biyearly inspection and cleaning of outfalls	Road Operations	Spring & Fall of every year, whenever CVC issues flood watch/warning (during/after ice jam and flood events)	Recurring since November of 2022	Recurring work underway	
27	Culvert and Sewer flushing	Contract Services	Spring & Fall of every year & whenever conservation authorities issue flood watch/warning	Flushing completed in November 2023. Spring flushing to be scheduled.	Recurring work underway	
28	Annual CCTV inspection of Sewers and Culverts	Contract Services	CCTV inspections to occur every spring	Last inspection completed January 2023. New inspection will be completed Spring of 2024 as per annual spring inspection note (pending award of new contract).	Recurring work underway	
29	Increase frequency of mowing at outfalls	Parks	November 2022	Increased mowing services at outfalls	Recurring throughout the growing season every year, including four cuts each year	
30	Clear rock aprons at outfalls	Road Operations	November 2022		Completed in November 2022. Will recur twice yearly, Spring & Fall, beginning Spring 2023	
31	Culvert at outfall #2 to be inspected and assessed	Engineering & Contract Services	October 2022	Visual and CCTV inspection completed. Minor repairs scheduled for Spring 2023.	Repaired March 2023	
32	Review and asses the condition of floodwalls and berm	Stormwater Management Group	Q1 2023	Geotechnical inspection and structural assessment of floodwalls and earth dykes. Final draft report to be published for public comment in Feb 2024.	In progress	
33	Commence Environmental Assessment for flood remediation	Stormwater Management Group	Q2 2024	Comprehensive look at flood risk and options to mitigate risk	Planned	
34	Manhole inspection and bolting	Contract Services & Peel Region	Q1 2023	Inspection of all surface manhole lids (stormwater and sanitary) to confirm they are bolted and not warped. City confirmed all stormwater manholes are bolted and not warped in January 2023. Region of Peel confirmed sanitary manholes are bolted March 2023.	Completed	

Item	Work Required	Division Responsible	Estimated Start Date	Comments	Status	Image
35	Annual Structural Inspections of Flood Wall and Earth Dykes	Capital Works	Fall 2024	Comprehensive inspection completed by consultant in 2023. Moving forward, recurring inspections will be completed by Capital Works as part of recurring OSIM inspections of retaining walls.	Planned	

KGS

GROUP

Experience in Action