THURBER ENGINEERING LTD.

July 19, 2022 File No.: 32228

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Attention: Spencer Thompson, P.Eng.

100% GEOTECHNICAL REPORT FOR COLEBROOK PUMP STATION REPLACEMENT SURREY, B.C.

Dear Spencer:

Thurber has completed a geotechnical investigation and seismic deformation analysis for the Colebrook Pump Station replacement project. This report summarizes our understanding of the project, results of our investigation, analyses and design review, and design recommendations for the pump station. This report supersedes the Thurber report of April 29, 2022 and incorporates comments from the City.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

1. BACKGROUND

The City of Surrey plans to replace the existing Colebrook Pump Station built in 1990. Design of a replacement pump station had been completed in 2016 by Omni Engineering Inc, with Thurber attending to the geotechnical scope, however, the project was postponed. The governing building code at the time was British Columbia Building Code (BCBC) 2012 which incorporated the seismic hazard from the National Building Code (NBC) 2010. This has since been updated to BCBC 2018 (seismic hazard from NBC 2015).

The new pump station will be a pile-supported concrete structure, located in the forebay on the landward side of the dike. The excavation for the pump station is to El. -5.5 m which is about 8 m deep relative to the dike and 3 m relative to the forebay. Support of excavation will require long sheet piles to develop enough resistance in the weak soils. A benefit of using sheet piles is that they will act as a long-term seepage barrier which will prevent flow beneath the pile-supported pump station as it will settle less than the ground beneath it. The dike will be raised to El. +4 m as part of this scope of work and is expected to be raised to El. +5.13 m by 2100.

Thurber's scope of services for this assignment was to review the design completed in 2016, perform a seismic deformation analysis and update the geotechnical recommendations from the earlier report to incorporate basic refinements, reflect current standards, and the results of the seismic deformation analysis.

1.1 DESIGN CHANGES FROM 90% SUBMISSION

A number of design changes were made based on review comments for the 90% submission, discussion with Surrey about their performance objectives, and new information provided by them. The significant geotechnical changes are as follows:

- Surrey found information that indicate the flood box pipes are older than expected, and so have less remaining service life than originally assumed due to age and accumulated settlement. Accordingly:
    - Replacement of the flood boxes will likely take place sooner than Surrey originally anticipated (i.e., next few years rather than 10 to 20 years).
    - No fill placement will occur above the flood box. This eliminates the lock block wall over the flood box and will reduce the total and differential settlement the pipes are subjected to, which should extend their remaining service life.
- Three foundation options were evaluated for seismic performance and Surrey selected the original 2016 foundation as it satisfied their performance requirements.
- The electrical connection for the pump station will be a 400A above-ground service. Underground conduit and connections will be pre-installed for a 600A service that will be installed at a later date.

2. GEOTECHNICAL INVESTIGATION

The geotechnical investigation took place on January 24, 2022 and comprised one auger hole to a depth of about 9 m and one seismic cone penetration test (SCPT) to a depth of about 50 m. The auger was to collect samples for review. The SCPT was to measure the shear wave velocity of the underlying soils as this is important information for the site-specific ground response analysis and seismic deformation analysis. The test hole coordinates were obtained by Thurber using a handheld GPS. The approximate location is shown on Drawing 32228-1.

Thurber retained OnTrack Drilling Inc. for the investigation. The test hole locations were scanned for underground utilities by Quadra Utility Locating prior to drilling in addition to submitting a locate ticket request with BC-1-Call to notify utility owners of our proposed investigation and to obtain underground utility information at the site.

The soil conditions were logged in the field by qualified Thurber personnel. Representative disturbed samples of the soils were collected from the test hole and returned to our laboratory for routine visual classification and water content testing. The test hole was backfilled with grout in accordance with the Dike Maintenance Act Requirements.

3. SUBSURFACE CONDITIONS

The results of the investigation are provided in the attached test hole log and SCPT report. Also attached are the results of Thurber's 2015 site investigation. The logs provide complete and detailed descriptions of the soil conditions encountered during the investigation and must be used in preference to the generalized description below.

The Geologic Survey of Canada characterises this area as being lowland stream channel fill and overbank sandy loam to clay loam with organic sediments up to 8 m thick. This is generally consistent with the findings of the site investigation which are summarised in the sub-surface profile below which is provided as an aid to discussion.

- Fill
  - present because the test hole was drilled through a ditch crossing
  - comprised about 1 m of sand and silt with trace clay and some wood debris
  - dike fill is expected to comprise till-like material (gravel-silt-sand mixtures)
- Silt Crust
  - this was the original ground surface at about El. ±0
  - about 2 m to 3 m thick, 0.3 m thick silt topsoil horizon *underlain by* 0.3 m of sandy peat and silt *underlain by* sand and silt with trace clay
- Sand
  - present below about El. -2 m to -3 m
  - comprises sand with some silt to silty
- Clay-Silt
  - present below El. -7 m to roughly El. -90 m (based on water well data)
  - comprises silt with trace to some sand, trace to some clay, with occasional organics or shell fragments

Groundwater was encountered at about El. -0.5 m in the test hole. The actual groundwater level will vary with seasonal precipitation, tides, storms, and the level of water in the forebay.

4. SETTLEMENT ASSESSMENT

A settlement assessment was carried out to evaluate the magnitude and rate of settlement associated with raising the dike as part of this scope of work and future raises to account for long-term sea-level rise. The results of this assessment inform our recommendations in Section 8 regarding dike overbuild to account for settlement and design of flexible couplers to mitigate the angular distortion associated with differential settlement.

Estimates of settlement due to elastic compression, primary, and secondary consolidation of the soils were calculated using the Rocscience computer program Settle3 (v.5.001 and later). The stratigraphy in the settlement model is based on the available test holes. The model extends to a depth of 90 m as this is the anticipated depth of incompressible ground. Groundwater was assumed to be at the surface. All depths are relative to ground surface before the dike was constructed.

The existing dike geometry and proposed dike raises were included in the model as surcharge pressures applied at the ground surface at the appropriate time. The model accounts for the settlement that occurs between raises with additional fill being added. The irregular fill height that will be present until the flood box is replaced was not modeled as it is below the specified dike elevations used in the settlement model, which are as follows:

- El. +3.9 m in 2023
- El. +4.2 m in 2024
- El. +4.47 m in 2070
- El. +5.13 m in 2100

The gravel road structure is anticipated to be about 200 mm thick and additional to the elevations specified above. A sensitivity check found that the addition of the road to the model changed the long-term settlement of the crest of the dike by less than 5%.

Consolidation parameters used for settlement modeling are based on the values published in Crawford and DeBoer's 1987 paper 'Field observations of soft clay consolidation in the Fraser Lowland'. The paper evaluated 15 years of settlement monitoring data for the King George Highway – BNSF overpass. The overpass embankments are considered a suitable comparison for the dike as they are located about 2 km upstream of the pump station in the same geologic setting. Secondary consolidation was modelled using the standard approach as that was method used in Crawford and DeBoer.

4.1 Flood Box

Settlement will likely continue at the current long-term rate of settlement as no fill is being placed above the flood box. There may be a slight increase as a result of nearby filling, but this will be deep seated and would not be expected to result in significant differential settlement.

Long-term settlement along the centerline of the flood box, if fill was placed above as originally envisioned in the 90% submission, is shown in Figure 4-1. This can be used for conceptual planning of the new flood box. Some observations from the modelling include:

- Settlement is generally deep-seated with 50% occurring below a depth of 30 m.
- The magnitude of settlement beyond the toe of dike falls within the expected range based on the average rate of settlement of the Fraser Delta which is generally 1 mm to 2 mm per year in green field conditions (Ertolahti, 2014, SFU Thesis).

4.2 Pump Station

Long-term settlement along the centerline of the pump station is shown in Figure 4-2. Some observations from the modelling include:

- The pump station is pile supported and will not settle as much as the dike.
- Settlement is generally deep-seated with 50% occurring below a depth of 30 m.
- The magnitude of settlement at the edges of the model falls within the expected range based on the average rate of settlement of the Fraser Delta which is generally 1 mm to 2 mm per year in green field conditions (Ertolahti, 2014, SFU Thesis).

4.3 Flood Wall

The feasibility of a flood wall to increase the level of flood protection over the flood box without raising the dike was assessed as it would mitigate the settlement associated with raising the dike. The benefit of the floodwall was mostly related to reducing of risk of damage to the existing pipes before the flood box is replaced. However, to avoid asymmetric loading of the pump station discharge lines, the flood wall would need to extend far enough to the west which meant that about 45 m of flood wall would be needed.

Consideration of a permanent flood wall is no longer relevant as the flood box is expected to be replaced in the next few years at which point the dike will be constructed to its design elevation. Provisions will need to be made for temporary flood walls during periods of high water. Aquadams or block walls with poly liner would be feasible options.

5. SEISMIC ASSESSMENT

A seismic assessment was completed to update the recommendations of the 2016 geotechnical report by Thurber. This comprised a site-specific ground response analysis to provide spectral acceleration values to the structural engineer for their assessment and provided the information needed for a liquefaction assessment. The seismic performance of the pump station is discussed in Section 7.

The British Columbia Building Code provides guidelines and requirements for seismic design of buildings. The design level seismic event has a 2% chance of exceedance in 50 years (1:2,475). An earthquake with a 10% chance of exceedance in 50 years (1:475) is not considered by the Building Code but is considered by the Bridge Code and seismic design guidelines for dikes.

The site is classified as Site Class F because of the presence of liquefiable soils.

5.1 Site-Specific Ground Response Analysis (SSRA)

SSRAs were performed for the 1:475-year and 1:2,475-year return period seismic event to determine spectral accelerations for structural design and cyclic stress ratio for the liquefaction assessment. The ground response analyses were performed by Thurber using the computer software DEEPSOIL, developed by Dr. Youssef Hashash of the Department of Civil & Environmental Engineering at the University of Illinois at Urbana-Champaign.

The stratigraphy used in the analyses was based on the available test hole information and shear wave velocities. Firm ground was modeled as an elastic half space with a shear wave velocity of 450 m/s. The firm ground depth was taken as 87 m based on the information available in the BC water well records. Site-specific shear wave velocities were used above 50 m and published correlations between depth and shear wave velocity below 50 m.

The firm ground (Site Class C) response spectrum for the site is attached and was determined using the online seismic hazard calculator for the 5th generation seismic hazard model applicable to the 2015 National Building Code (NBC 2015). The ground motions used in the analyses were those developed for the George Massey Tunnel Replacement project (2017) scaled to the project location.

The results of the 1:475 analysis is presented in Figure 5-1 and Table 5-1.

The results of the 1:2,475 analysis is presented in Figure 5-2 and Table 5-2.

5.2 Liquefaction Triggering

A liquefaction triggering assessment was carried out using the simplified method presented in Boulanger and Idriss (2014) for the design earthquakes. The method involves comparing the cyclic stress ratio (CSR) caused by the design earthquakes with the cyclic resistance ratio (CRR) of the soil. The results of the analysis were used to identify zones of potentially liquefiable material (i.e. zones with $FS_{liq} < 1$) and provide estimates of post-liquefaction strength and volumetric strain.

Liquefaction of the silty sand is effectively total for the 1:475- and 1:2,475-year seismic events. The onset of liquefaction is expected to occur above the 1:100-year level earthquake.

The silt deposits are not susceptible to seismic induced liquefaction. Low plastic silt will likely undergo strain softening during a large earthquake. However, this would be expected to result in the loss of up to 20% of its initial strength rather than the 90% to 95% strength reduction associated with liquefaction.

6. STATIC STABILITY ASSESSMENT

A static stability assessment was completed to assess the reduction in factor of safety as a result of raising the dike from its current level. The assessment found that the existing dikes can be raised to the design grade of El. +4.2 m with a reasonable factor of safety and that there is no requirement for staged construction as undrained (short-term) condition does not govern stability. An overbuild of 0.2 m (to. El. +4.4 m) for the access road is not considered to

6.1 Stability Model

Static limit equilibrium stability analyses were completed to assess the effect of the dike raising at the pump station and flood box. The GeoSlope computer program Slope/W (2021) was used for the assessment.

The soil profile for the stability analyses was based on the test hole logs. Stability analyses were carried out for static conditions using undrained (short-term, Figure 6-1) and drained (long-term, Figure 6-2) strengths. The structures themselves were not included in the models as they effectively stabilise the forebay side of the dike in both cases. Accordingly, forebay stability was not assessed.

6.2 Stability Results

Figures 6-3 and Figure 6-4 show the results of the stability analyses at the flood box. Figures 6-5 and Figure 6-6 show the results of the stability analyses at the pump station.

Some general comment regarding the stability assessment are:

- The factor of safety of the pump station dike section is acceptable (1.60 > 1.5).
- The factor of safety of the flood box dike section is less than would be generally acceptable for new construction (1.38 < 1.5). The dike slopes would need to be flattened to achieve an acceptable factor of safety.

The stability of the forebay (left) side of the dike was not assessed as the flood box and pump station restrain the dike.

For new construction a factor of safety of 1.5 is generally used for permanent conditions and 1.3 for temporary conditions. Using slightly lower values is not unreasonable for existing embankments. However, continued raises will eventually result in a dike that cannot be raised without widening to achieve an acceptable factor of safety.

7. SEISMIC DEFORMATION ANALYSIS

A seismic deformation analysis was done to assess the response of the pump station, flood box, and dike to earthquakes. For the pump station this comprised evaluating the seismic performance of the 2016 foundation design (508 mm diameter piles), alternate foundation configurations (762 mm and 914 mm diameter piles) and assessing the sensitivity of the foundations to changes in assumed water levels. The ultimate objective was to select a foundation configuration that satisfied Surrey's expectation of performance following the design earthquake. The seismic performance of the existing flood box and dike were assessed although there are no plans to replace or upgrade them as part of this scope of work.

The Terms of Reference in the RFP stated that the pump station is to be designed in accordance with the BC Building Code for operability and life safety following a 1:2,475-year seismic event. Performance based design is not well addressed in the BC Building Code, but it is better addressed in the Canadian Highway Bridge Design Code (CHBDC). A 'life safety' service level is associated with a 'probable replacement' damage level in the CHBDC.

Based on discussion with Surrey, the performance expectations of the pump station following the design earthquake can be characterised as follows:

- That life safety be achieved.
- That temporary loss of service is acceptable.
- That permanent loss of service is acceptable.
- That salvage of large components be considered in design (e.g., pumps, electrical).

The original 2016 foundation comprising 508 x 12.7 mm steel pipe pile foundation achieves the performance expectations set out by Surrey and was selected by them as the preferred option.

7.1 Summary

The results of the seismic deformation analyses are summarised below. Further details are provided in the following subsections.

<u>Pump Station</u>

The pump station is not expected to be in an operable condition above the threshold for liquefaction triggering. The discharge lines will likely be severed at the riverward end of the pump station by post-liquefaction settlement where they pass through the sheet pile wall. The discharge lines more generally will also be subjected to large lateral movements as the riverward side of the dike fails into the river during a large earthquake. Electrical power will also be lost as the above-ground service connection crosses the river and runs along the dike.

There is a small possibility that the damage to the pump station will be repairable. It is far more likely there will be severe damage to, or failure of, the pile-raft connection and that the pump station will need to be replaced. CIMA estimated that connection failure would be expected when

the river is more than 0.25 m above the forebay. It should be possible to salvage the pumps and fittings from the pump station so long as they are designed with salvage in mind.

Flood Box

The flood box is not expected to survive any earthquake above the threshold for liquefaction triggering. Lateral spreading during the earthquake will push the headwalls outward, potentially breaking weak connections or causing separation between unconnected pipe segments. The anticipated order of magnitude of displacement for the 1:475- and 1:2,475-year seismic events are summarised in Table 7-1.

Dike

The existing dike will not satisfy the requirements of the Seismic Design Guidelines for Dikes for any earthquake above the threshold for liquefaction. The anticipated order of magnitude of displacement for the 1:475- and 1:2,475-year seismic events is summarised in Table 7-1.

Table 7-1 – Estimated displacement of dike.

Earthquake	Displacement (m)	
	Vertical	Horizontal
1:475	0.3 (±)	0.6 (±)
1:2,475	0.6 (±)	more than 1 m

7.2 Model Configuration

Seismic deformation analyses were completed using 2D finite element (FE) computer program Plaxis 2D. The PM4Sand, and HSsmall models were used for potentially liquefiable and non-liquefiable soil layers, respectively. PM4Silt was used in the pump station analysis to assess potential strength degradation of the low plastic clay-silt. The soil parameters used in the model were obtained from the measured shear wave velocity and the interpretation of the cone penetration tests. The model geometry was based on the design drawings and subsurface information available. The steel pipe piles evaluated were 508 x 12.7 mm, 762 x 19.1 mm, and 914 x 19.1 mm. The steel sheet pile was 350W, AZ26-700 sheet pile.

The CSR profiles produced by the SSRAs were used to select representative ground motions for the seismic deformation analysis. The SSRAs considered thirty earthquakes representing the 1:475-year event and thirty earthquakes representing the 1:2,475-year events (total of 60 earthquakes). Three types of earthquake sources were evaluated with 10 records for each: crustal (shallow, e.g. Christchurch 2011), in-slab (medium depth, e.g. Nisqually 2001), and interface (subduction, e.g. Tohoku 2013). The earthquakes that produced the median CSR profiles for each earthquake source and each event return period were selected for the Plaxis 2D model (total of 6 earthquakes). The ground motions were input at the base of the model using a compliant base

boundary with V_s = 450 m/s. Tied degree of freedom boundaries were used for the lateral extents of the model.

Figure 7-1 shows the full-scale geometry of the model for the new pump station. Figures 7-2a and 7-3a show the relevant parts of the model geometry for the pump station and flood box, respectively. Figures 7-2b and 7-3b show the typical post-earthquake deformed shapes for the pump station and flood box, respectively (deformations have been magnified to make them more visible).

7.3 Earthquake Induced Displacements and Loads (508 x 12.7 mm Piles)

Earthquake induced dike displacement and pile load estimates were determined from the output of the deformation analyses. The governing ground motions for each seismic event return period were identified by comparing maximum displacement magnitudes. The analysis results for a 508 x 12.7 mm steel pipe pile foundation and the governing ground motions results are summarised in a series of Tables:

- Table 7-2
  - Calculated pile loads associated with a 1:475-year seismic event.
- Table 7-3
  - Calculated pile loads associated with a 1:2,475-year seismic event.
- Table 7-4
  - Calculated dynamic pile loads associated with a 1:2,475-year seismic event.
- Table 7-5
  - Estimated movement of pump station.

The loads provided in the tables are unfactored geotechnical loads for the same water level on both sides of the pump station. The actual water level will vary with tide and precipitation with the most adverse condition being where the river is higher than the forebay. For each meter of head differential the increment of additional deflection and moment is roughly +250 mm and +325 kN·m.

Everett Nast (P.Eng.) of CIMA indicated that the foundation will be partially yielded for the loads provided in Tables 7-3 and 7-4 and that the foundation will be fully yielded (i.e., plastic) at a differential water level of 0.25 m (river higher than forebay).

Table 7-2 – Calculated pile loads associated with a 1:475-year seismic event.

Location	Maximum Pile				Pile-Raft Connection	
	Shear (kN)	Elevation (m)	Moment (kN·m)	Elevation (m)	Shear (kN)	Moment (kN·m)
Forebay Pile Row	60	-8.7	200	-8.1	60	80
Middle Pile Row	30	-13.8	180	-8.7	30	120
Dike Pile Row	60	-5.5	220	-5.5	60	220
AZ-26	95	-4.5	422	-9.0	-	-

Table 7-3 – Calculated pile loads associated with a 1:2,475-year seismic event.

Location	Maximum Pile				Pile-Raft Connection	
	Shear (kN)	Elevation (m)	Moment (kN·m)	Elevation (m)	Shear (kN)	Moment (kN·m)
Forebay Pile Row	70	-4.5	300	-8.8	70	160
Middle Pile Row	60	-16.3	260	-8.8	40	200
Dike Pile Row	110	-5.5	410	-5.5	110	410
AZ-26	120	-13	575	-9.0	-	-

Table 7-4 – Calculated dynamic pile loads associated with a 1:2,475-year seismic event.

Location	Maximum Pile				Pile-Raft Connection	
	Shear (kN)	Elevation (m)	Moment (kN·m)	Elevation (m)	Shear (kN)	Moment (kN·m)
Forebay Pile Row	100	-4.5	410	-6.0	100	290
Middle Pile Row	90	-10.4	440	-5.0	80	440
Dike Pile Row	280	-5.5	710	-5.5	280	710
Z-26	140	-13.2	590	-8.2	-	-

Note: These are transient short duration loads (tenths of second) loads that only occur during shaking.

Table 7-5 – Estimated movement of pump station for equal water levels on both sides of the pump station.

Earthquake	Movement	
	Horizontal (mm)	Rotation (°)
1:475	50 to 100	up to 1°
1:2,475	200 to 300	up to 1°

Note: Rotation is towards river/dike.

7.4 Foundation Alternatives

The City of Surrey has selected the 2016 foundation comprising 508 x 12.7 mm steel pipe piles as their preferred choice. Sensitivity analyses were carried out as part of the selection process to evaluate the performance of different pile sizes for varying water levels. The analyses allowed the performance of the different foundation options to be compared and one to be selected on the basis of the desired level of performance. In general, larger, thicker-walled pipe piles would be expected to perform better than smaller, thinner-walled pipe piles as they would have greater resistance to the lateral load demand induced by the earthquake.

The seismic deformation analysis found that the seismic performance of the pump station was highly sensitive to the difference in the water level between the river and the forebay. Table 7-6 summarises the results of this assessment and relates differential water levels to expected building performance. Lateral deformations at the fully plastic condition were typically in the order of 1 m.

Table 7-6 – Summary of pile performance for different water levels.

Pile Diameter (mm)	Differential Water Level (m)	
	Elastic Limit	Fully Plastic
508	not achievable	0.25
762	1.1	1.5
914	3	3.6

Notes: Definitions provided by Everett Nast (P.Eng.) of CIMA, adapted from email of 2022.05.16.

Structure Stay Elastic (*Elastic Limit*): Structure not expected to experience yielding during or after a seismic event. Cracking expected to be minimal, no repairs required.
Structure Goes Plastic: Structure expected to yield during and after a seismic event. Cracking may be more severe, but repairable.
Foundation Structure Likely Fails and Higher Chance of Replacement (*Fully Plastic*): Higher potential for failure of pile to foundation connection. Building to experience, permanent, irreparable damage, likely need to be replaced.

8. GEOTECHNICAL RECOMMENDATIONS

Recommendations for the raising of the dike, backfilling of dike penetrations, pump station, and flood box are provided in the following subsections.

8.1 Corrosion Assessment

A corrosion assessment was carried out using AASHTO R27-01 (Assessment of Corrosion of Steel Piling for Non-Marine Applications) as recommended in the Bridge Code Commentary (CAN/CSA-S6.1-19). Based on R27, soil resistivity data collected west of the south abutment of the King George Highway Overpass (Weemees, 1990, UBC) and pH of groundwater in the lagg of Burns Bog (Avenant, et al., 2017, UBC), no allowance for corrosion is required.

Weemees: https://open.library.ubc.ca/media/download/pdf/831/1.0062547/2

pH: https://open.library.ubc.ca/media/download/pdf/52966/1.0347240/5

8.2 Sheet Pile Retaining Wall

A permanent sheet pile cut-off wall is required to prevent water flow from developing as settlement is expected to cause a void to form below the pump station. The cut-off wall will be used for support of excavation along the dike during construction of the pump station. Based on drawings by Omni, the exposed face of the temporary sheet pile wall will be about 7.8 m high whereas the permanent sheet pile wall will be about 5.5 m high.

Design of the sheet pile wall was done using Plaxis 2D. The analysis indicated that the sheet pile must comprise minimum 26 m long sheets with a top and bottom elevation of El. +4.5 m and El. -21.5 m, respectively. The embedment is governed by stability during construction and the cantilevered height of wall needed to construct the pump station. The sheet pile must comprise AZ26-700 (Grade 350W) sheets or approved equivalent. This should be confirmed by the structural engineer based on the loads provided in Section 7. The maximum moment developed in the permanent sheet pile under normal (static) conditions is about 170 kN*m per meter of wall length. Lateral displacement of the wall was calculated to be in the order of 50 mm.

The permanent sheet pile wall will not be structurally connected to the pump station. However, it has the potential to transfer lateral loads to the top and bottom slabs of the discharge pipe chamber. Initial (static) lateral load transfer is estimated to be small as the sheet pile must support the dike entirely during construction of the pump station. Primary lateral load transfer is expected to occur during a long-duration flood or an earthquake and is estimated to be in the order of 50 kN per meter of wall along the top slab and 230 kN per meter of wall along the bottom slab.

The lateral loads are not additional to the lateral earth pressures provided in Section 8.5.2. Rather, they are concentrated loads acting on the slabs in proximity to the sheet piles and can be considered loads for component design by the Structural Engineer.

8.3 Dike Fill Placement

The Ministry of Forests, Lands, Natural Resource Operations, and Rural Development (MFLNRORD) Dike Design and Construction Guide recommends that dike fill comprise a soil with 15% to 30% fines (clay and silt) to limit seepage. Soils proposed as dike fill by the contractor must be reviewed by Thurber prior to use. Required information includes grain size analysis, standard Proctor test, Atterberg limits and a physical sample.

Dike fill must be placed and compacted within 2% of its optimum moisture content. Soil outside of this range will require moisture conditioning prior to compaction. It is very rare that dike fill obtained directly from excavations is within 2% of the optimum moisture content.

Before placement of any fill, the subgrade of the excavation should be prepared such that the existing surfaces are cleared of organics and scarified. Scarification is required to reduce the potential for creation of preferential slip surfaces and seepage paths. Fill should be placed in lifts up to 300 mm thick and compacted to 95% standard Proctor maximum dry density (SPMDD) in accordance with MFLNRORD's dike construction guidelines. If compaction results in a smooth surface, it should be scarified before placing the next lift. The dike should be over-built horizontally and then cut back to the design profile to reduce the likelihood of inadequate compaction at the exposed surface. Steps must be cut into the existing dike to reduce the risk of a failure plane developing along the old dike surface (see Figure 8-1).

Meeting the compaction specification can be difficult on the first lift when the subgrade is soft. Accepting a lower degree of compaction on the first lift is reasonable so long as it is placed within ±2% of optimum moisture content and compacted with a reasonable amount of effort. If the subgrade is soft to the point that it causes problems for equipment access, an initial lift thickness of 600 mm may be used subject to our field review.

We recommend a 5 m long bentonite plug be installed along the discharge pipes to reduce the risk of seepage along the interface between the pipes and backfill. It should extend the full width of the excavation and from the base of excavation to 200 mm above the flood boxes. It should comprise a 70% / 30% blend of granular material and powdered sodium bentonite in place of conventional bedding sand and trench backfill. Suitable granular materials would be MMCD pipe bedding and MMCD granular base as defined in the current edition of the Master Municipal Construction Documents Association (MMCD). The blended material must be mixed dry and placed dry. The bentonite plug should be located riverward of crest of the dike.

8.4 Dike Overbuild for Future Settlement

The amount of dike overbuild to account for future settlement is a function of the planed interval between grade increases. Long-term planning should make use of the observed settlement to plan future overbuild. The following values from the settlement analyses can be used for current planning:

- 10 year – 100 mm
- 20 year – 200 mm
- 40 year – 300 mm

8.5 Pump Station

8.5.1 Pile Foundation

It is our understanding that the factored design load and pile layout remains unchanged from the initial design. The foundation load was stated by CIMA (then Bogdanov Pao Associates Ltd.) to be 7360 kN.

Based on our review of the earlier (2016) pile design, the required pile embedment using 508 mm diameter piles remains unchanged at 32 m. Our review consisted of a comparison of the earlier analyses with new pile resistance calculations using the alpha- and beta-methods. The required pile length was within ±0.5 m of the earlier calculation.

Recommendations for the pile foundation remain unchanged from the 2016 design.

8.5.2 Lateral Earth Pressure

Figure 8-2 provides recommendations for lateral earth pressures. These pressure distributions assume that the walls are rigid and restrained from movement. We recommend using the highest flood water level to determine the hydrostatic water pressure.

The lateral earth pressure diagrams are intended for design of the pump station walls – not to estimate the lateral load acting on the piles. The lateral load acting on the piles for structural design should be taken from the seismic deformation analysis.

8.5.3 Flexible Couplers for Discharge Pipes and other In-Ground Utilities

The pile-supported pump station will not settle the same amount as the surrounding ground. This will subject any in-ground service to abrupt differential settlement where they enter the pump station. Flexible couplers are recommended for the discharge lines and service connections to the pump station to reduce the risk of failure due to settlement.

Flexible couplers should be designed assuming that the pump station does not settle (Figure 4-2). This is a reasonable assumption given the available information, potential for short scale soil variability in delta deposits, and consequences of pipe failure (i.e. loss of pump station until pipes can be replaced). The following settlement estimates can be used for design of the coupler depending on its intended lifespan:

- 40 years – 250 mm
- 80 years – 400 mm
- 120 years – 500 mm

We understand that the preference of the City is to use an observational approach to manage the risk of long-term settlement to the discharge pipes. As a flexible coupler will not be used, it is our recommendation that the flange and transition from steel pipe to HDPE pipe take place within the access chamber as is shown on the design drawings (see Figures 8-3 and 8-4). The air gap between the flange and the styrofoam gasket at the sheet pile wall will provide a short length over

which the HDPE pipe is free to rotate and deform without being confined by several meters of soil. Locating the flange on the other side of the sheet pile would increase the hazard associated with settlement as the HDPE would likely be subject to the same deformation as before, but over a much shorter distance (i.e., increased strain, increased risk of poor performance).

8.5.4 Settlement Monitoring of Pump Station Discharge Pipes

A qualified engineer will need to be retained by the City to establish acceptable tolerances for pipe movement, confirm recommendations for monitoring the pipe, and what mitigation measures are required at what movement thresholds (e.g., expose and re-level pipes).

At a minimum, we recommend not less than 3 settlement gauges per pipe: one about 1 m riverward of the pump station, one near the landward dike crest, and one at the dike centerline. Three settlement gauges will need to be installed at the same locations except offset about 2 m from a pipe with the gauge at or below the invert of the pipe. It is necessary to have two sets of gauges to estimate how much of the pipe is left 'hanging' unsupported. The indicative location of the gauges is shown in Figures 8-5 and 8-6.

The monitoring points on the pipes can use the standard Metro Vancouver detail for settlement monitoring of buried pipes. Our recommendation is that the riser pipe be fabricated using stainless steel to reduce the risk of long-term corrosion.

The other settlement gauges can follow typical industry practice for surface settlement gauges that are buried in the course of construction. If a surface type gauge is destroyed it will need to be replaced with a Borros type settlement gauge as it will not be practicable to remove the dike to install a settlement gauge (https://durhamgeo.com/product/borros-anchor/).The typical details of a surface settlement gauge are provided in Figure 8-7.

A flush mount well cover plate will be needed to protect the settlement gauges. The cover plate will need to be removed and the gauge extended every time the dike is raised.

8.5.5 Temporary Excavation Considerations

Since the footprint of the new pump station is located within the forebay, a temporary sheet pile cofferdam will be required to complete the excavation for the wet well. The design, installation, operation, and monitoring of temporary works will be the responsibility of the contractor. This includes confirmation of the required sheet pile properties and lengths, and any bracing requirements. Further, the design of the cofferdam and dewatering system should consider the effect of pile driving on stability at the base of the excavation.

Driving open-ended steel pipe piles will disturb the soil around piles, possibly resulting in a loss of soil strength. If the excavation is dewatered prior to pile driving, there is a risk of heave or piping of soil into the excavation during pile driving. This is due to the difference in hydraulic head between the surrounding area and the excavation and disturbance to the soil caused by pile driving.

Consideration should be given to excavating to design grade without dewatering and driving piles in the submerged excavation. Once pile driving has been completed and the disturbed soil allowed to regain sufficient strength, the excavation could be dewatered and carefully cleaned out around the piles.

8.6 Flood Box

Surrey indicates that the remaining service life of the flood box is in the order of a few years. Accordingly, no fill placement will occur above the flood box. This eliminates the need for a lock block wall over the flood box to achieve the design dike elevation and will reduce the total and differential settlement the pipes are subjected to which should help preserve the limited remaining service life

We recommend that a pipe and joint condition assessment be carried out by a civil engineer experienced in assessing pipe performance. The condition assessment will give an indication of the remaining service life of the flood box.

8.7 Lock Block Retaining Wall

A short lock block retaining wall is needed for grade separation between the pump station flood box. The wall will be 3 m (or 2 block) long and extend from the end of the permanent sheet pile cut-off wall towards the river. Typical geometry and details are provided in Figure 8-8. The wall should be constructed vertically for ease of construction.

Geogrid must be the greater of 2 m long or 0.7 times the height of wall as measured from the back of the wall and have a minimum long-term design strength of 25 kN/m. Geogrid will be installed between every block and below the lowest block. Backfill for the wall can comprise dike fill although a levelling course of Sechelt Sand or Crushed Granular Base is needed for the wall. Non-woven geotextile is required to reduce the risk of loss of material through the face of the wall and along the joint between the sheet pile wall and the lock blocks. No drainage is to be installed as this increases the risk loss of material.

8.8 Underground Services

Underground services should be able to be installed using conventional techniques and design details consistent with MMCD specifications.

We recommend using MMCD Type 2 Pipe Bedding. Type 2 Pipe Bedding is finer-grained than Type 1 and is accordingly less permeable and has better filtering characteristics with the native material and dike fill. This will reduce the potential for water movement along the pipe trench and reduce the risk of fines migrating into the granular trench backfill. Trench backfill must comprise dike fill where the pipe is riverward of the landward crest of the dike. The landward crest can be taken as 4 m landward of the river crest as this is the minimum recommended crest width in the BC Dike Design and Construction Guide.

Flexible couplers will be required for service connections to the building to accommodate long term differential settlement between the building and surrounding grade (e.g., water, sanitary, underground power

9. CLOSURE

We trust this information meets your present needs. If you have any questions, please contact the undersigned at your convenience.

Yours truly,
Thurber Engineering Ltd.

Steven Coulter, M.Sc., P.Eng.
Review Engineer

PROFESSIONAL ENGINEER
M. C. BOSSE
36163
BRITISH COLUMBIA
ENGINEER

Marc C. Bossé, M.Sc., P.Eng.
Project Engineer

2022-07-19

Attachments
- Statement of Limitations and Conditions
- Test Hole Location Plans (2022, 2015)
- Symbols and Terms (for soil descriptions an test hole logs)
- Test Hole Logs
  - 2022: 1 SCPT, 1 AH
  - 2015: 1 CPT, 2 AH

Thurber Engineering Ltd.
Permit # 1001319

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

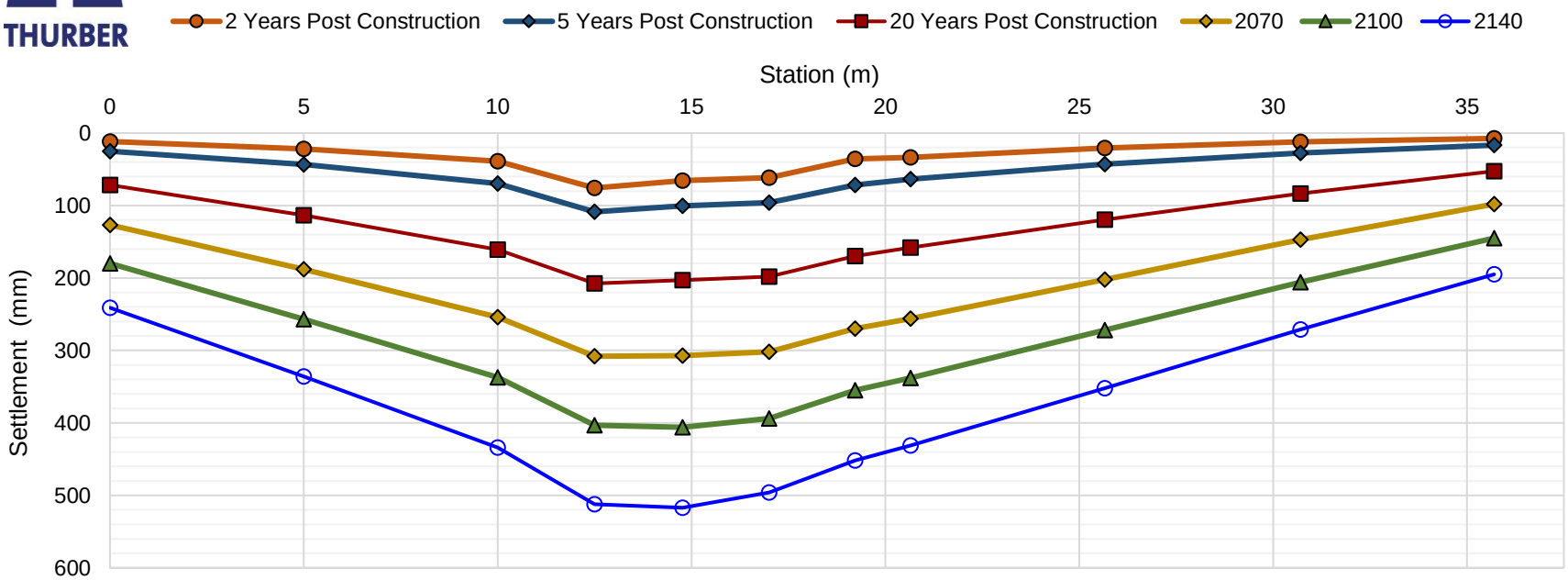
Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

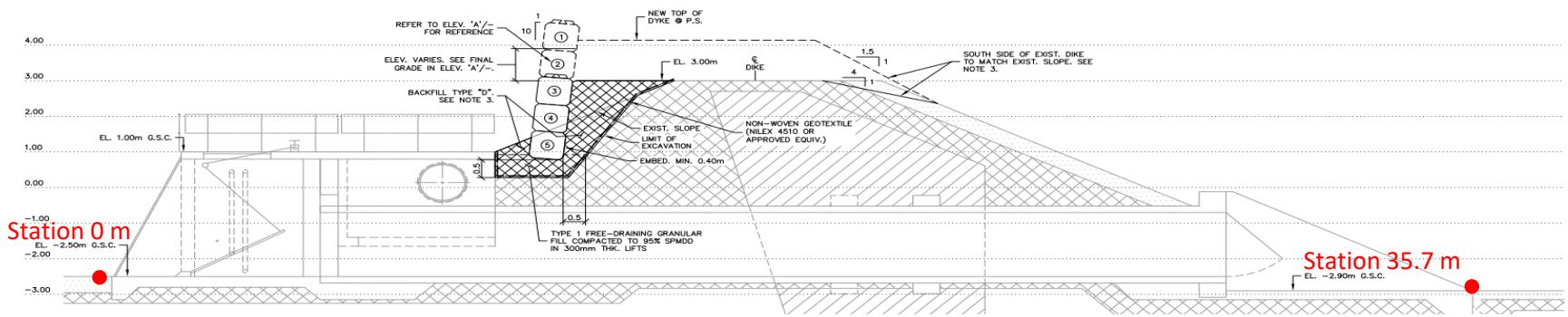
7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

32226 - Colebrook Drainage Pump Station

Figure 4-1 - Settlement along centreline of Floodbox at ground surface

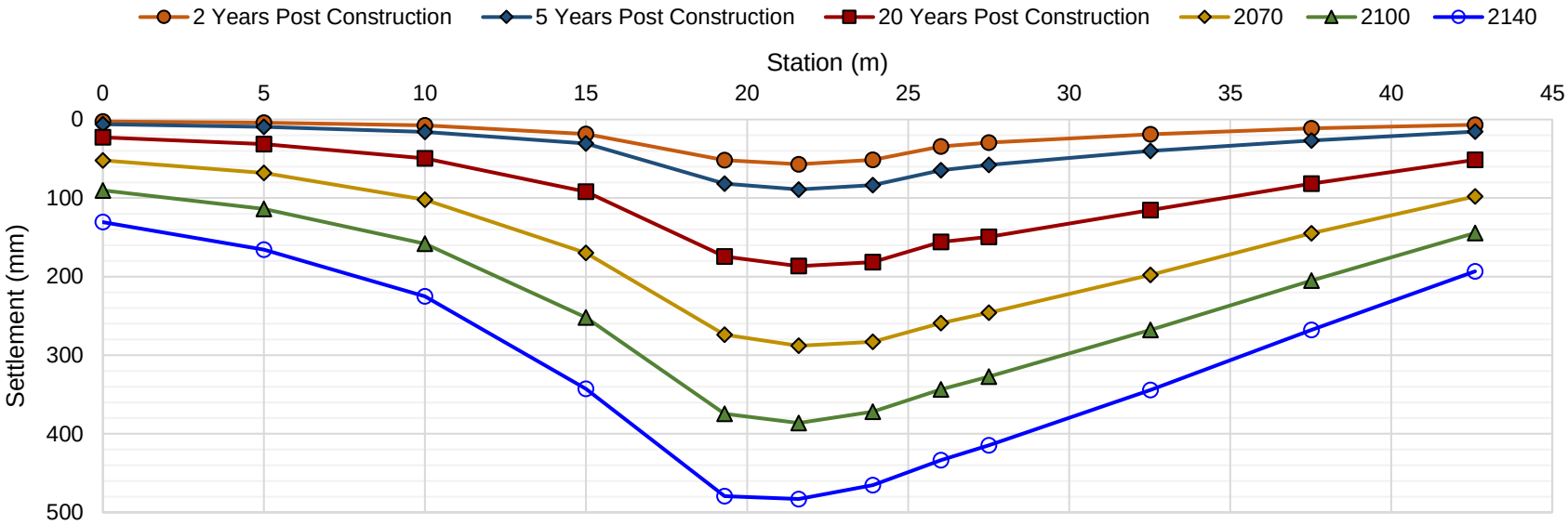


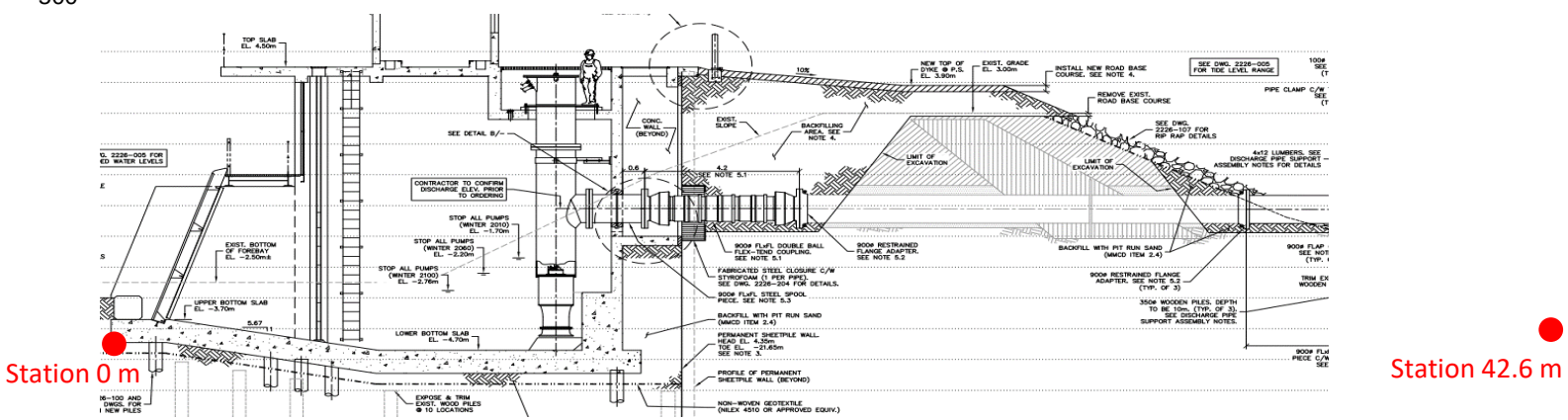


Notes:

1. Test hole 22-1 was used to develop model soil stratigraphy.
2. Dike raises are proposed in 2023 to El. +3.9 m, 2024 to El.+4.2 m, 2070 to El. +4.47 m and 2100 to El. +5.13 m. Settlement model accounts for settlement between raises.
3. Settlement is referenced from the first raise in 2023 to El. +3.9 m
4. A flood wall could be used to reduce the magnitude of total and differential settlement during the remaining life of the flood box.
5. Construction of lock block wall locally increases angular distortion compared to sloping ground.

32226 - Colebrook Drainage Pump Station
Figure 4-2 - Settlement along centreline of Pump Station at ground surface





Notes:

1. Test hole 22-1 was used to develop model soil stratigraphy.
2. Dike raises are proposed in 2023 to El. +3.9 m, 2024 to El.+4.2 m, 2070 to El. +4.47 m and 2100 to El. +5.13 m. Settlement model accounts for settlement between raises.
3. Settlement is referenced from the first raise in 2023 to El. +3.9 m
4. The pump station is pile supported and will not settle as much as the dike (< 20%). Thurber recommends that the pipe coupler be designed assuming the pump station does not settle.
5. Raising the dike to El. +3.9, with no further raises, only reduces the long-term settlement by about 20%. Settlement is generally controlled by deep-seated, regional settlement.

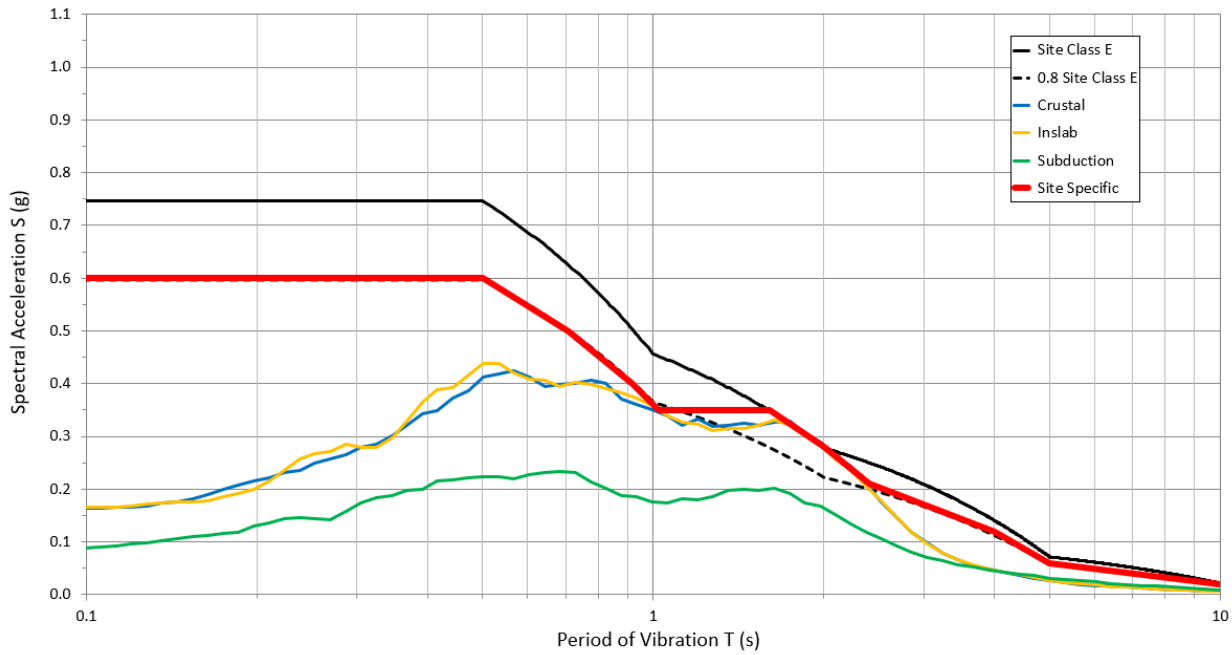


Figure 5-1 – Mean calculated 1:475 spectral acceleration for crustal, in-slab, and subduction ground motions and recommended spectrum.

Table 5-1 – Recommended 1:475 response spectrum.

Site Specific Response Spectrum

T (s)	0	0.5	0.71	0.92	1.02	1.6	2	2.4	4	5	10
S (g)	0.60	0.60	0.50	0.40	0.35	0.35	0.28	0.21	0.12	0.06	0.02

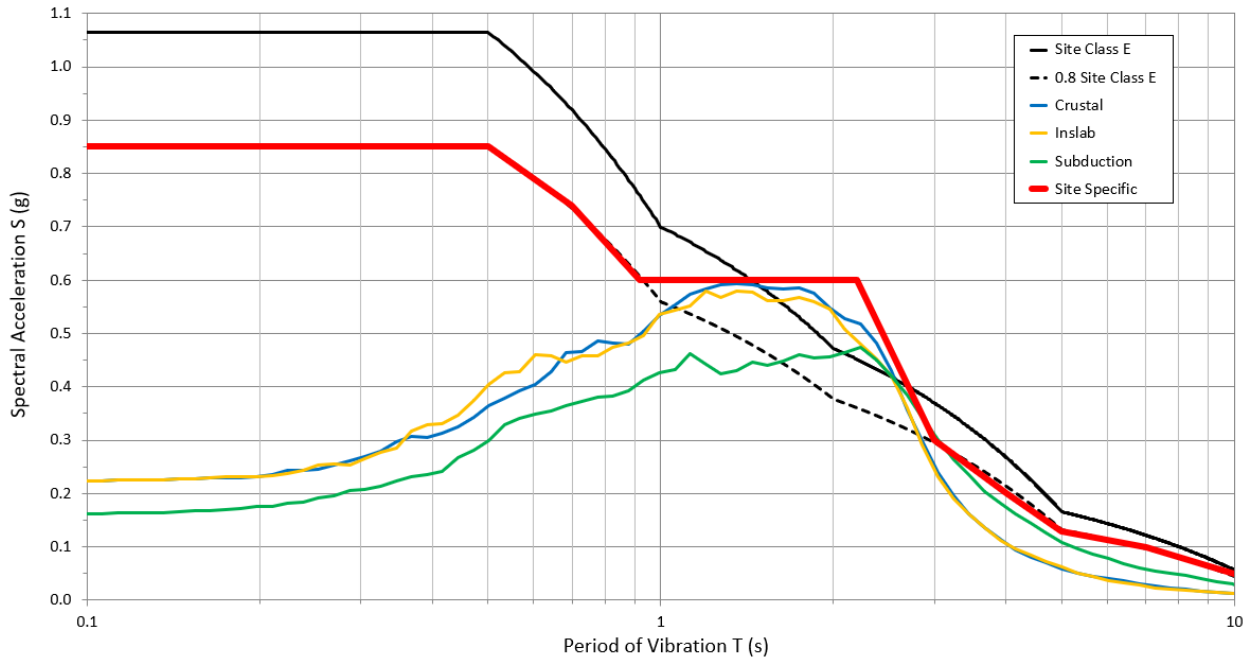


Figure 5-2 – Mean calculated 1:2,475 spectral acceleration for crustal, in-slab, and subduction ground motions and recommended spectrum.

Table 5-2 – Recommended 1:2,475 response spectrum.

Site Specific Response Spectrum

T (s)	0	0.5	0.7	0.92	2.2	3	4	5	7	10
S (g)	0.85	0.85	0.74	0.60	0.60	0.30	0.20	0.13	0.10	0.05

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line	B-bar	Add Weight
	Clay/Silt (undrained)	SHANSEP	17.3			20	0.23	1	1	No
	Concrete	Mohr-Coulomb	23.5	25	45				0	No
	Existing Dike Fill	Mohr-Coulomb	18	2	32			1	0	No
	New Dike Fill	Mohr-Coulomb	18	2	32				0	Yes
	Sand	Mohr-Coulomb	18	0	34			1	0	No
	Sand and Silt	Mohr-Coulomb	17.8	2	32			1	0	No
	Sechelt Sand	Mohr-Coulomb	18	0	45				0	No

Figure 6-1 – Undrained (short-term) soil parameters used in stability assessment.

Comment: A B-bar parameter of 1.0 is used for the clay-silt because it will not dissipate pore pressure quickly with fill placement. The analyses found that the undrained condition never governed stability, in other words the Factor of Safety between the drained and undrained case was identical.

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	B-bar	Add Weight
	Clay/Silt (drained)	Mohr-Coulomb	17.3	0	28	1	0	No
	Concrete	Mohr-Coulomb	23.5	25	45		0	No
	Existing Dike Fill	Mohr-Coulomb	18	2	32	1	0	No
	Sand	Mohr-Coulomb	18	0	34	1	0	No
	Sand and Silt	Mohr-Coulomb	17.8	2	32	1	0	No
	Sechelt Sand	Mohr-Coulomb	18	0	45		0	No

Figure 6-2 – Drained (long-term) soil parameters used in stability assessment.

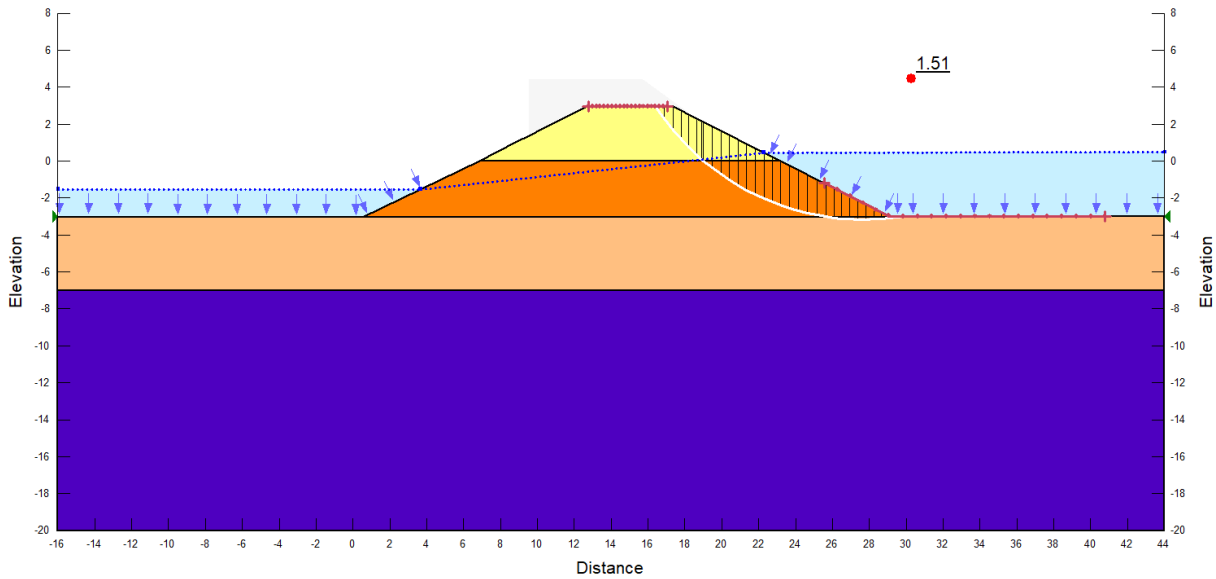


Figure 6-3 – Stability of existing dike at flood box.

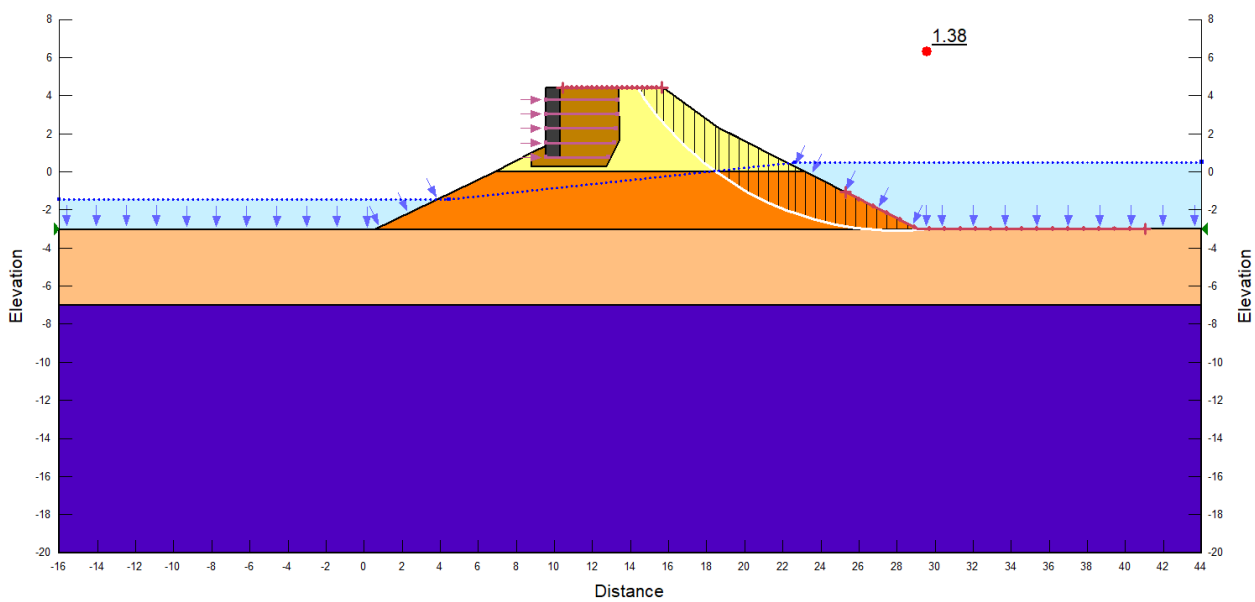


Figure 6-4 – Riverward stability of dike at flood box after raising to El. +4.4 m.

Comments: These calculations are no longer relevant as the dike is not going to be raised over the floodbox. The calculated FoS with the dike at El. +4.4 m is less than would generally be accepted for new construction (i.e. 1.38 vs. 1.5). The dike slopes would need to be flattened when the flood box is reconstructed to achieve an acceptable FoS.

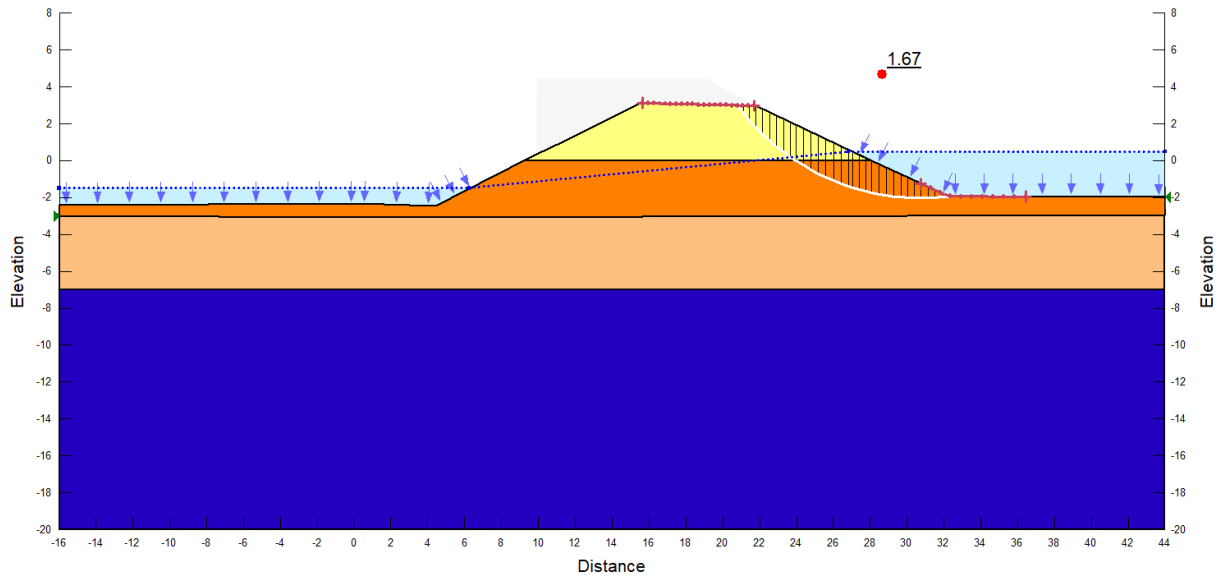


Figure 6-5 – Stability of existing dike at pump station.

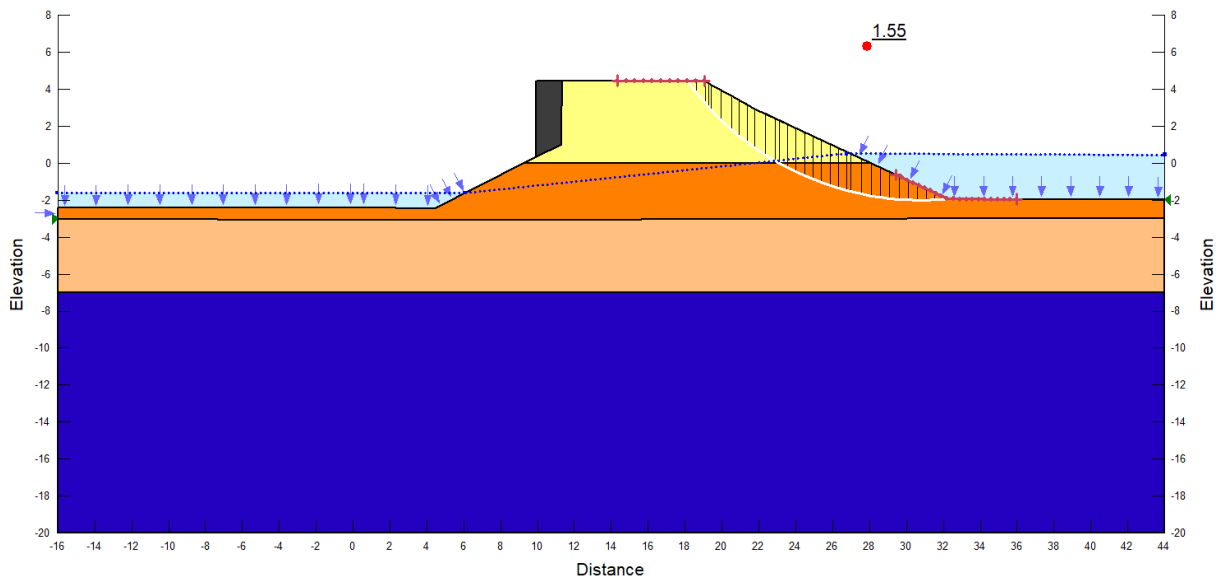


Figure 6-6 – Riverward stability of dike at pump station after raising to El. +4.4 m.

Comments: Stability of forebay is addressed by sheet pile wall and pump station.

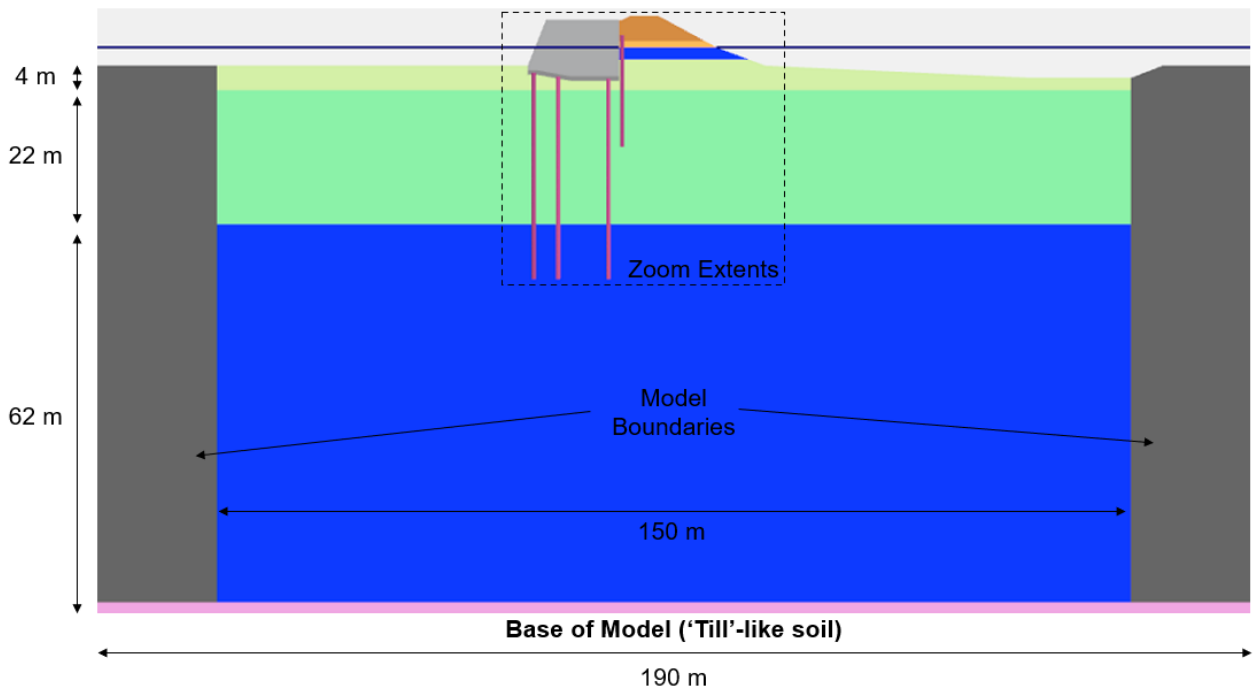


Figure 7-1 – Overall model geometry of new pump station.

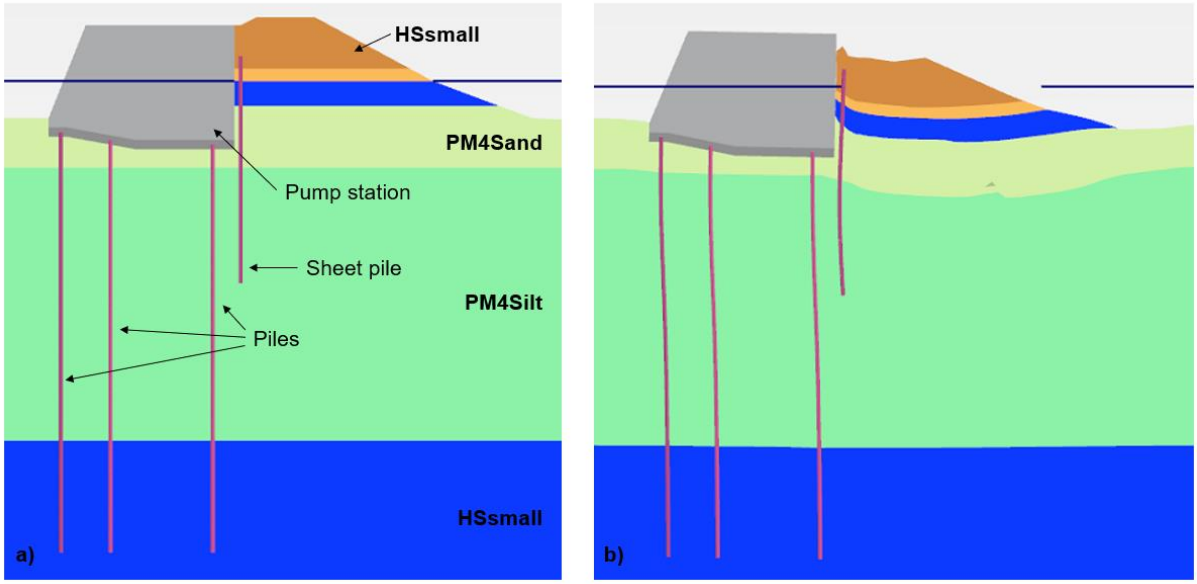


Figure 7-2 – New pump station a) model geometry and b) typical deformed shape (3x exaggeration)

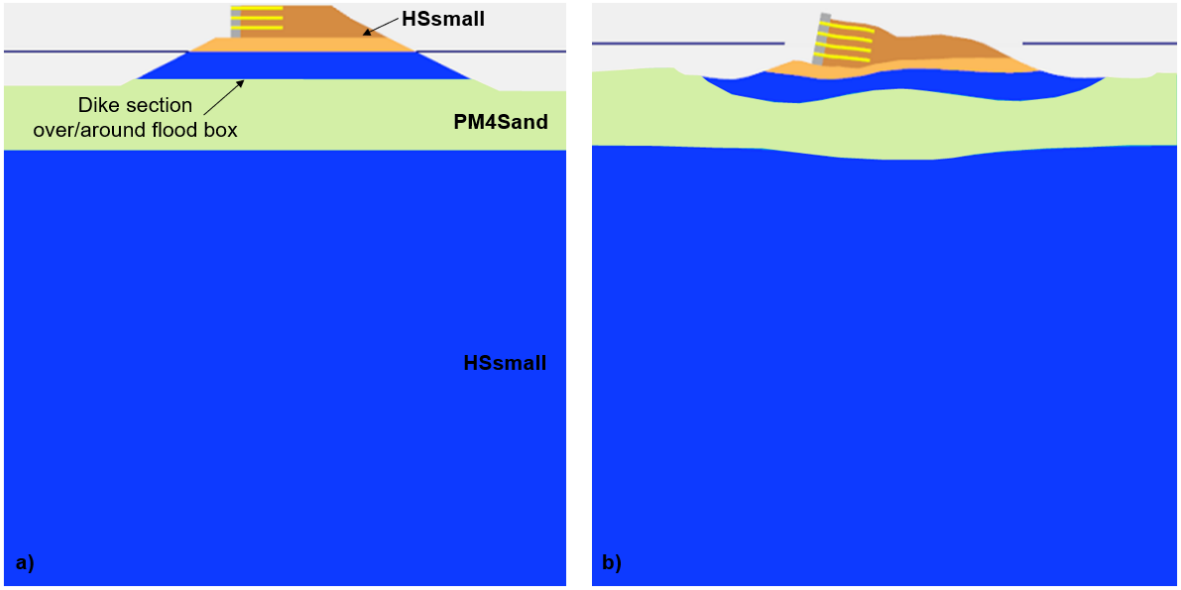


Figure 7-3 – Existing flood box a) geometry and b) typical deformed shape (3x exaggeration)

Comment: The flood box was not modeled directly as it is grade supported and cannot impart significant lateral resistance to constrain the dike. The lock block wall was modeled. Changing to a flood wall would not significantly change the behaviour.

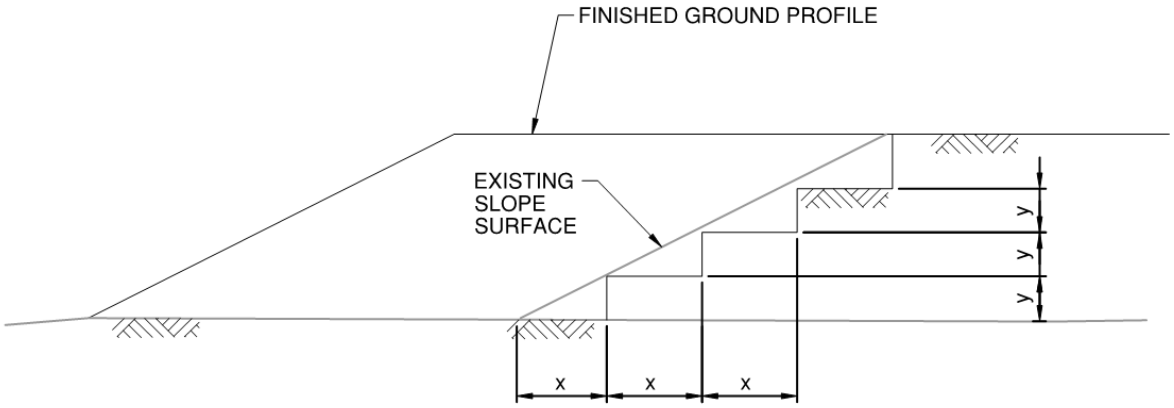


Figure 8-1 – Typical benching detail.

Comment: x should typically be in the range of 1.5 to 2 times y. Thus, for a 300 mm lift, x would be between 450 mm and 600 mm.

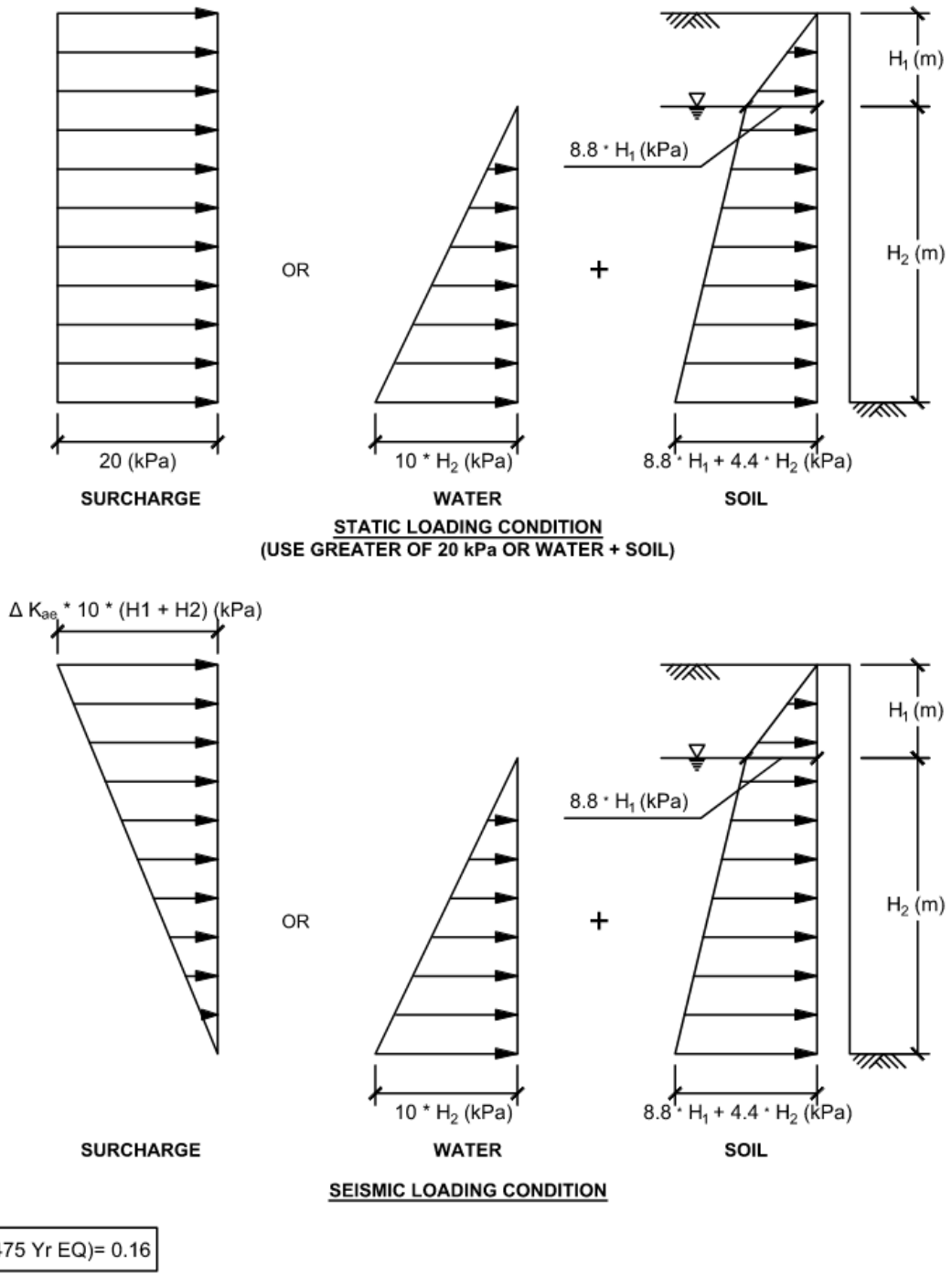


Figure 8-2 –Recommended lateral earth pressure for design of pump station.

Comment: The ground surface elevation should be based on the year 2100 dike level.

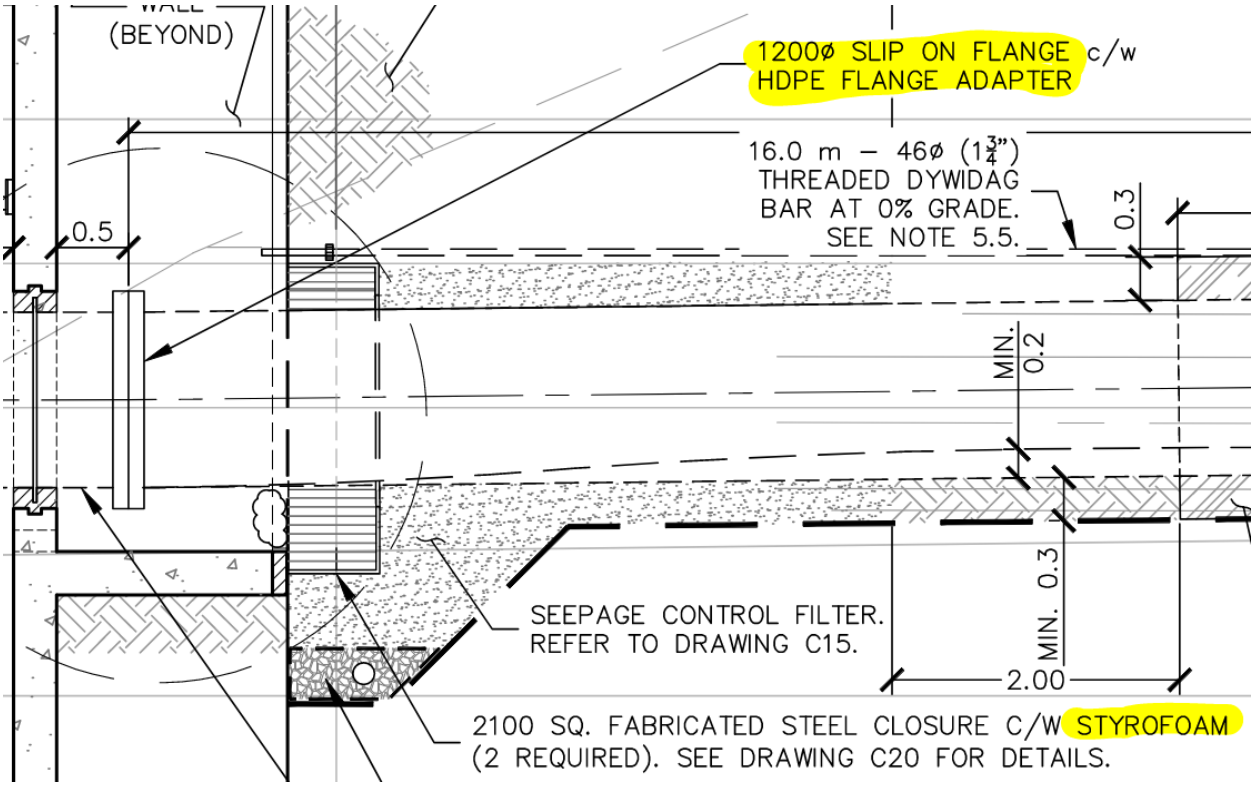


Figure 8-3 – Screenshot of transition from steel pipe (left of flange) to HDPE pipe (right of flange) and styrofoam gasket through which the HDPE pipe passes through at the sheet pile wall (USL drawing C10 R.1, 2022.05.06)

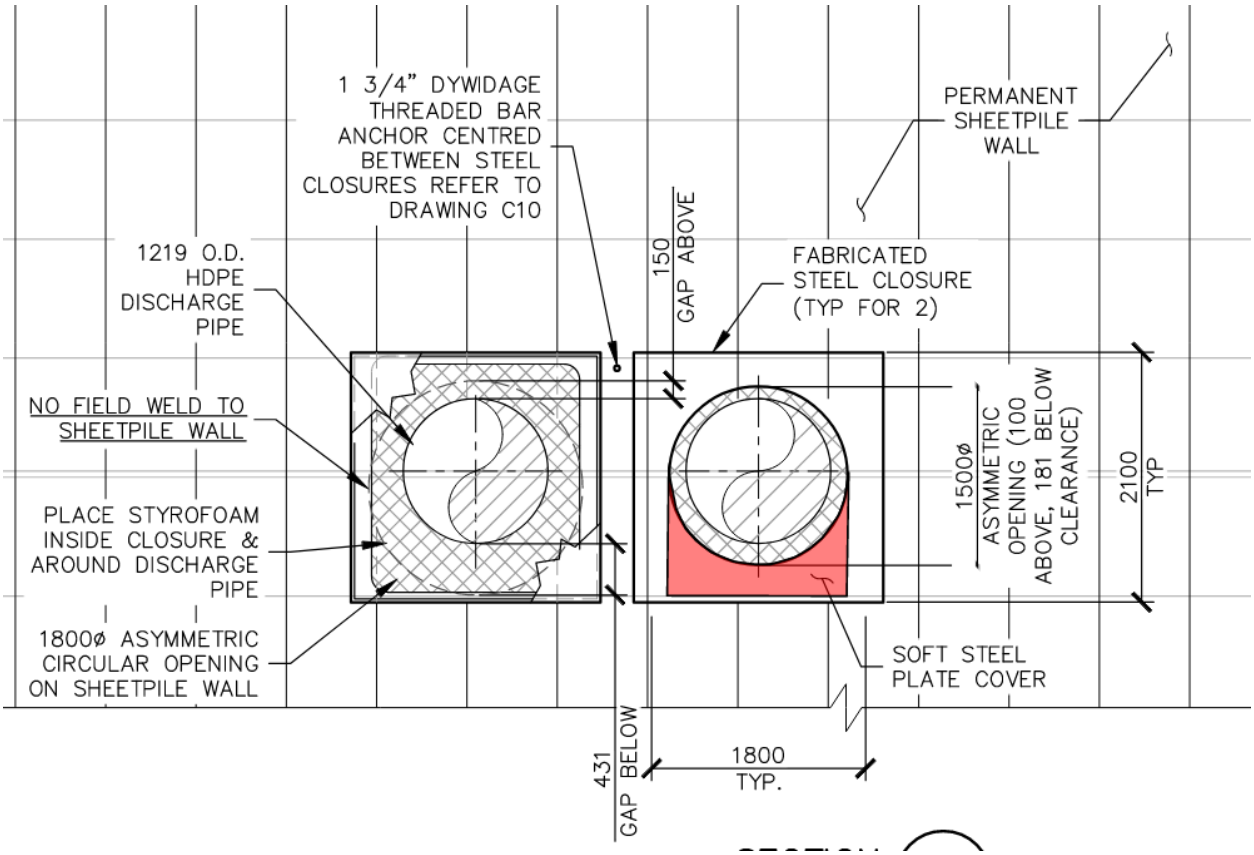


Figure 8-4 – Screenshot of styrofoam gasket through which the HDPE pipe passes through at the sheet pile wall (USL drawing C20 R.1, 2022.05.06)

Comment: Recommend that the cutout in the soft steel cover plate be enlarged as much as possible to accommodate greater pipe settlement (red area above).

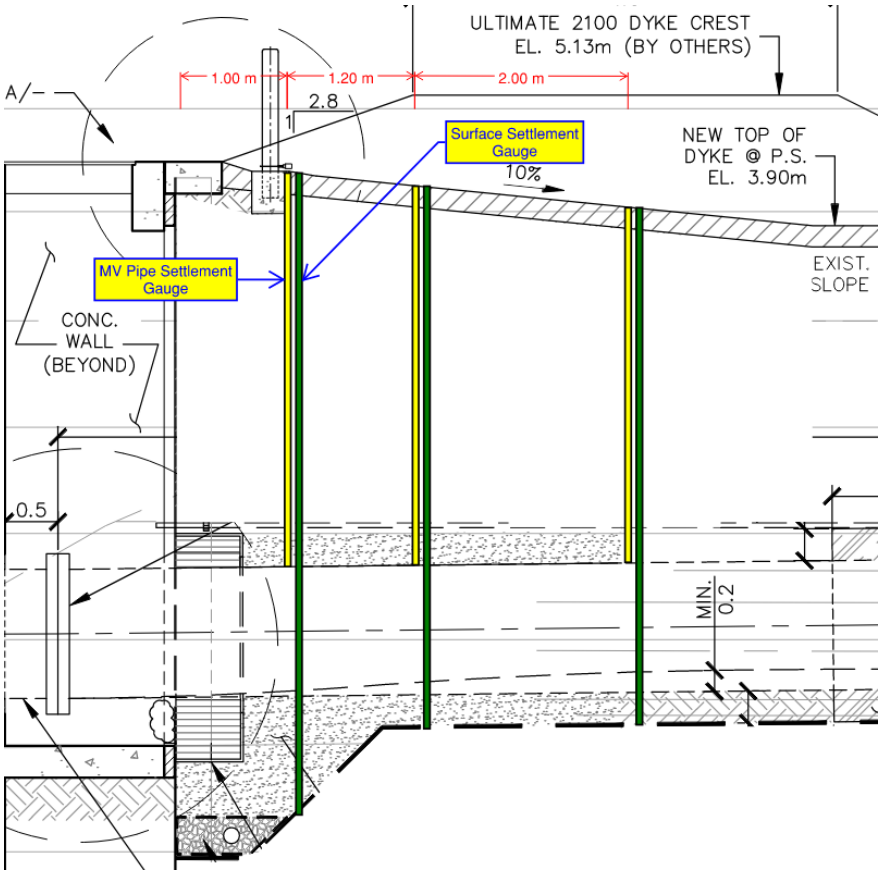


Figure 8-5 – Indicative location of settlement gauges.

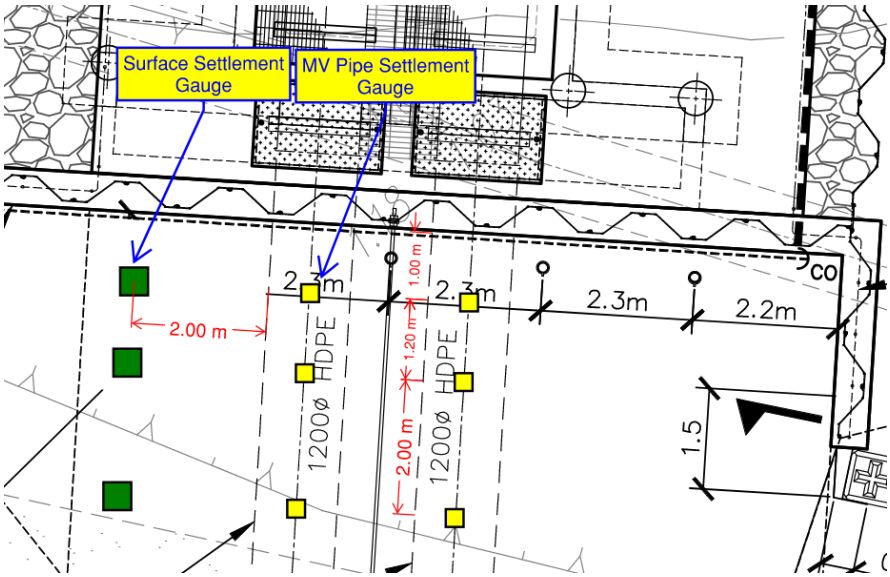


Figure 8-6 – Indicative location of settlement gauges.

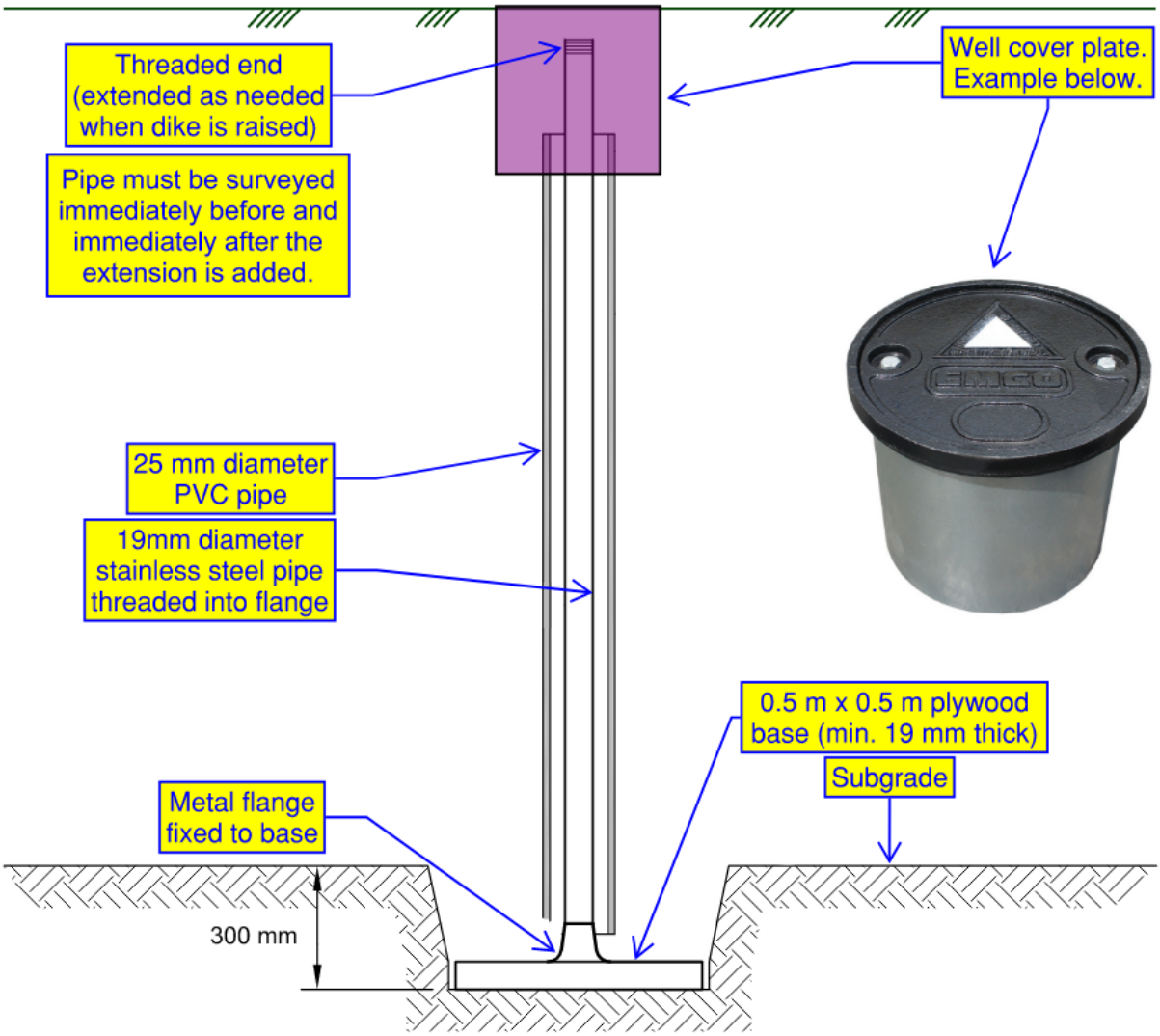


Figure 8-7 – Indicative location of settlement gauges

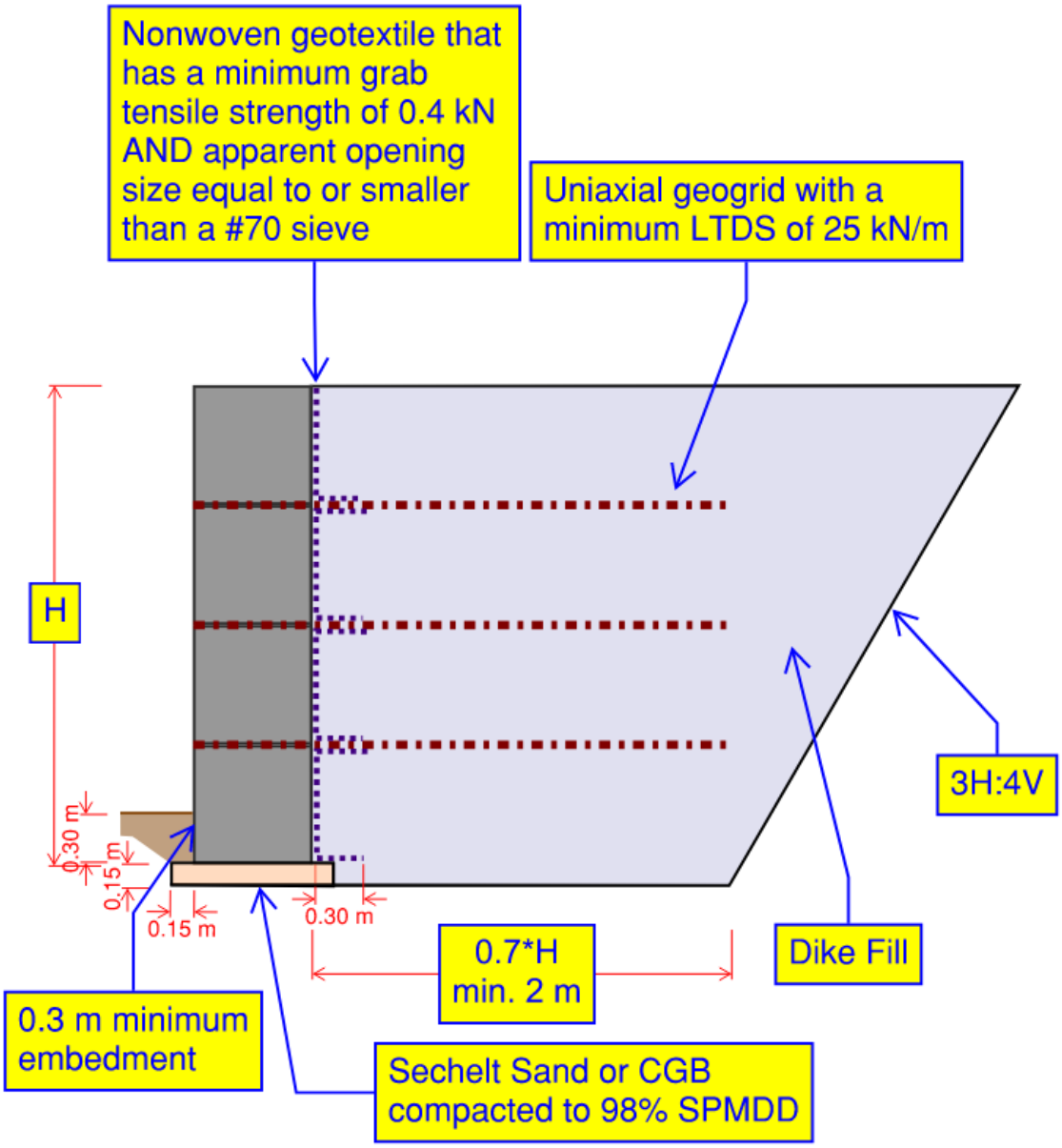


Figure 8-8 –Typical geometry of lock block wall.

Comment: The non-woven geotextile is to reduce the risk of fines migration or loss of material through the face of the wall. No drainage is to be installed.

Plotted: February 18, 2022

LEGEND:

TEST HOLE / SCPT

NOTES:
1. AERIAL IMAGE TAKEN FROM COSMOS.
2. PROPERTY LINES TAKEN FROM THE CITY OF SURREY OPEN DATA CATALOGUE
3. TEST HOLE LOCATION IS APPROXIMATE .

THURBER ENGINEERING LTD.

URBAN SYSTEMS LTD.

TEST HOLE LOCATION PLAN

COLEBROOK PUMP STATION — SURREY, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
DMA	MOM	MCB	FEB. 18, 2022	1:150	32228 -	1	0

S:\Data\BST Projects\32xxx\32228\06_Drafting\03_Working\32228.dwg

SYMBOLS AND TERMS

FOR SOIL DESCRIPTION AND TEST HOLE LOGS

BASIC SOIL SYMBOLS

Predominant Material	Secondary Material
GRAVEL	gravelly to some gravel
SAND	sandy to some sand
SILT	silty to some silt
CLAY	clayey to some clay
PEAT / ORGANICS	some organics
Undifferentiated BEDROCK	
ORGANIC SILT	
FILL / DEBRIS	

SYMBOL VARIATIONS - EXAMPLES[1]

SAND and GRAVEL

SAND, silty

SILT with some clay

DENSITY OF GRANULAR SOILS	
Description	SPT N[5][6]
Very Loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	> 50

PROPORTION OF MINOR COMPONENTS BY WEIGHT[2]	
and	35 - 50%
y / ey	20 - 35%
some	10 - 20%
trace	0 - 10%

CONSISTENCY OF COHESIVE SOILS	
Description	Undrained Shear Strength (kPa)[6]
Very Soft	< 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	> 200

PENETRATION TESTS
Dynamic Cone Penetration
Standard Penetration
Becker Closed Casing
Becker Open Casing
Bounce Chamber Pressure

CLASSIFICATION BY PARTICLE SIZE				
		Size Range[6]		
			U.S. Standard Sieve Size	
Name		(mm)[3]	Retained	Passing
Boulders		> 200	8 inch	-
Cobbles		75 - 200	3 inch	8 inch
Gravel:	coarse	19 - 75	0.75 inch	3 inch
	fine	5 - 19	No. 4	0.75 inch
Sand:	coarse	2 - 5	No. 10	No. 4
	medium	0.4 - 2	No. 40	No. 10
	fine	0.075 - 0.4	No. 200	No. 40
Fines (Silt or Clay)[4]		< 0.075	-	No. 200

(1) Only selected examples of the possible variations or combinations of the basic symbols are illustrated.
(2) Example: SAND, silty, trace of gravel = sand with 20 to 35% silt and up to 10% gravel, by dry weight. Percentages of secondary materials are estimates based on visual and tactile assessment of samples.
(3) Approximate metric conversion.
(4) Fines are classified as silt or clay on the basis of Atterberg limits.
(5) SPT N values on test hole logs are uncorrected field values.
(6) Reference Canadian Foundation Engineering Manual 4th Edition, 2006.

THURBER

LOG OF TEST HOLE

TEST HOLE NO. **TH22-01**

LOCATION: See DWG. 32228-1
N 5437055, E 511242 (Est.)

TOP OF HOLE ELEV: 1 m (Est.)
METHOD: Solid Stem Auger / SCPT
DRILLING CO.: On-Track Drilling Inc.
INSPECTOR: DMA

THURBER

CLIENT: Urban Systems Ltd.
PROJECT: Colebrook Pump Station

DATE: January 24, 2022
FILE NO.: 32228
REVIEWED BY: MCB

DEPTH (m) | DCPT PENETRATION (blows/300 mm) | SPT PENETRATION (blows/300 mm) | WATER CONTENT (%) ○ Disturbed ● Undisturbed | WATER LEVEL | Plastic Limit — Liquid Limit | SAMPLES: Disturbed, Undisturbed, No Recovery | GRAIN SIZE (%): ▲ Passing #200 sieve, △ Passing #4 sieve | SOIL HEADSPACE READING (ppm): GASTECH reading, PID reading | ELEVATION (m)

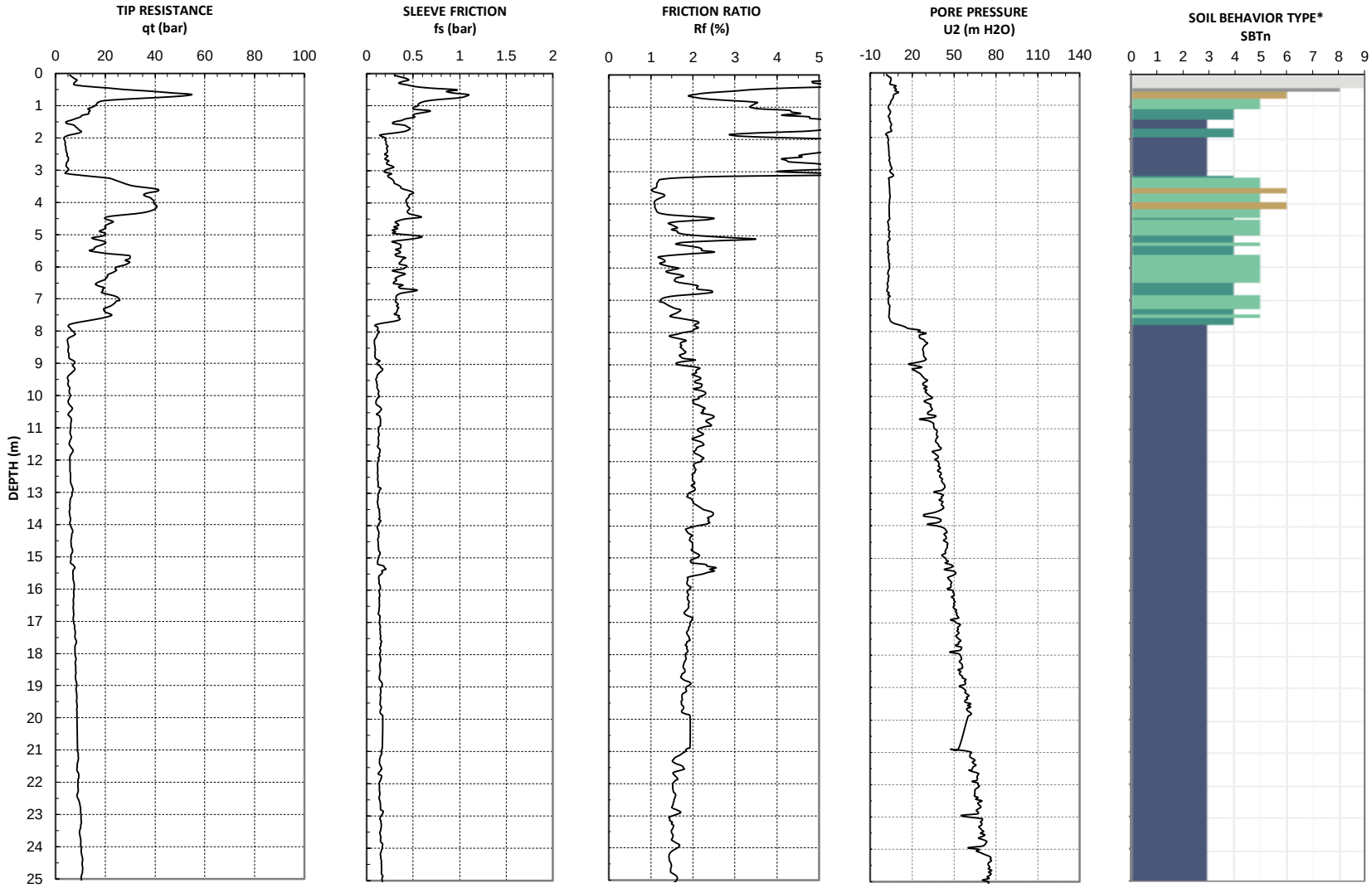
10 20 30 40 50 60 70 80 90 100

COMMENTS

SOILS DESCRIPTION

Depth (m)	Elevation (m)	Sample	Description
0			Brown, moist, gravelly SILT and SAND with some wood debris and trace clay (FILL).
1	0	ML/SM	
		PT/OH	Firm to stiff, dark brown to black, moist SILT with some to trace sand, trace clay and occasional organics and rootlets.
2	-1		Very soft, dark brown to black, wet, sandy PEAT and SILT. Portion of peat comprised of partially decomposed wood debris.
		SM/ML	Dark grey, moist to wet SAND and SILT with trace clay.
3	-2		Dark grey, wet SAND with some silt to silty.
4	-3	SM	
5	-4	SM	
6	-5		
7	-6	SM	- trace shell fragments below 6.7 m depth
8	-7		Very soft to soft, light grey, moist SILT with some sand, some to trace clay and occasional organics and shell fragments.
9	-8	CL/ML	
10			End of hole at required depth. Hole open to 2.6 m depth. Water observed at 1.5 m depth upon completion of drilling. Hole backfilled with grout.

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
24-Jan-2022	Site: Colebrook Pump Station, Surrey, BC



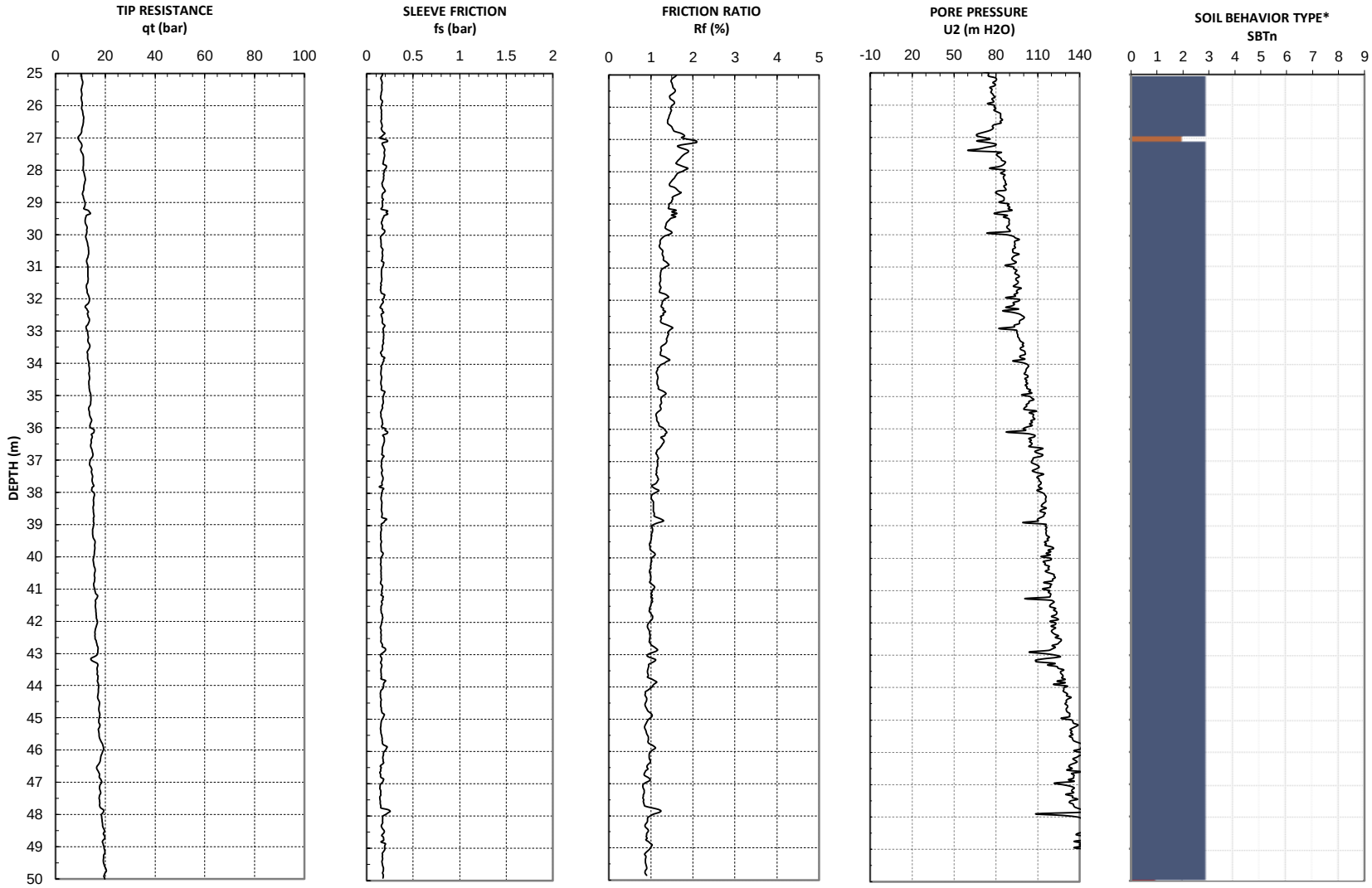
* Based on Robertson et. al 1990

- 1. Sensitive Fine Grained
- 2. Organic Material
- 3. Clay to Silty Clay
- 4. Clayey Silt to Silty Clay
- 5. Silty Sand to Sandy Silt
- 6. Clean Sand to Silty Sand
- 7. Gravely Sand to Sand
- 8. Very Stiff Sand to Clayey Sand
- 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 50.00 m

Cone ID: DDG1521
Operator: ZH

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
24-Jan-2022	Site: Colebrook Pump Station, Surrey, BC



* Based on Robertson et. al 1990

- 1. Sensitive Fine Grained
- 2. Organic Material
- 3. Clay to Silty Clay
- 4. Clayey Silt to Silty Clay
- 5. Silty Sand to Sandy Silt
- 6. Clean Sand to Silty Sand
- 7. Gravely Sand to Sand
- 8. Very Stiff Sand to Clayey Sand
- 9. Very Stiff Fine Grained

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
24-Jan-22	Site: Colebrook Pump Station, Surrey, BC

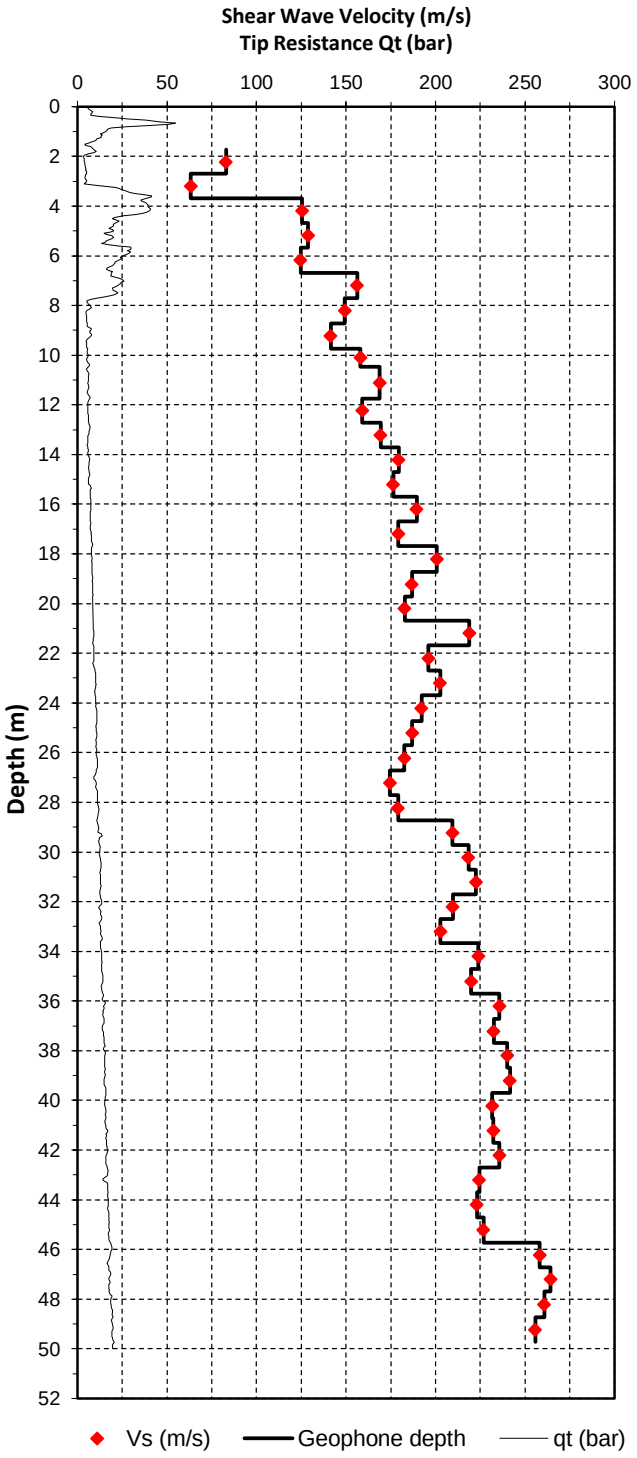
Seismic Source: Beam
Source to cone (m): 1.2

Geodetic Elevation: N/A
Cone ID: DDG1521
Operator: ZH

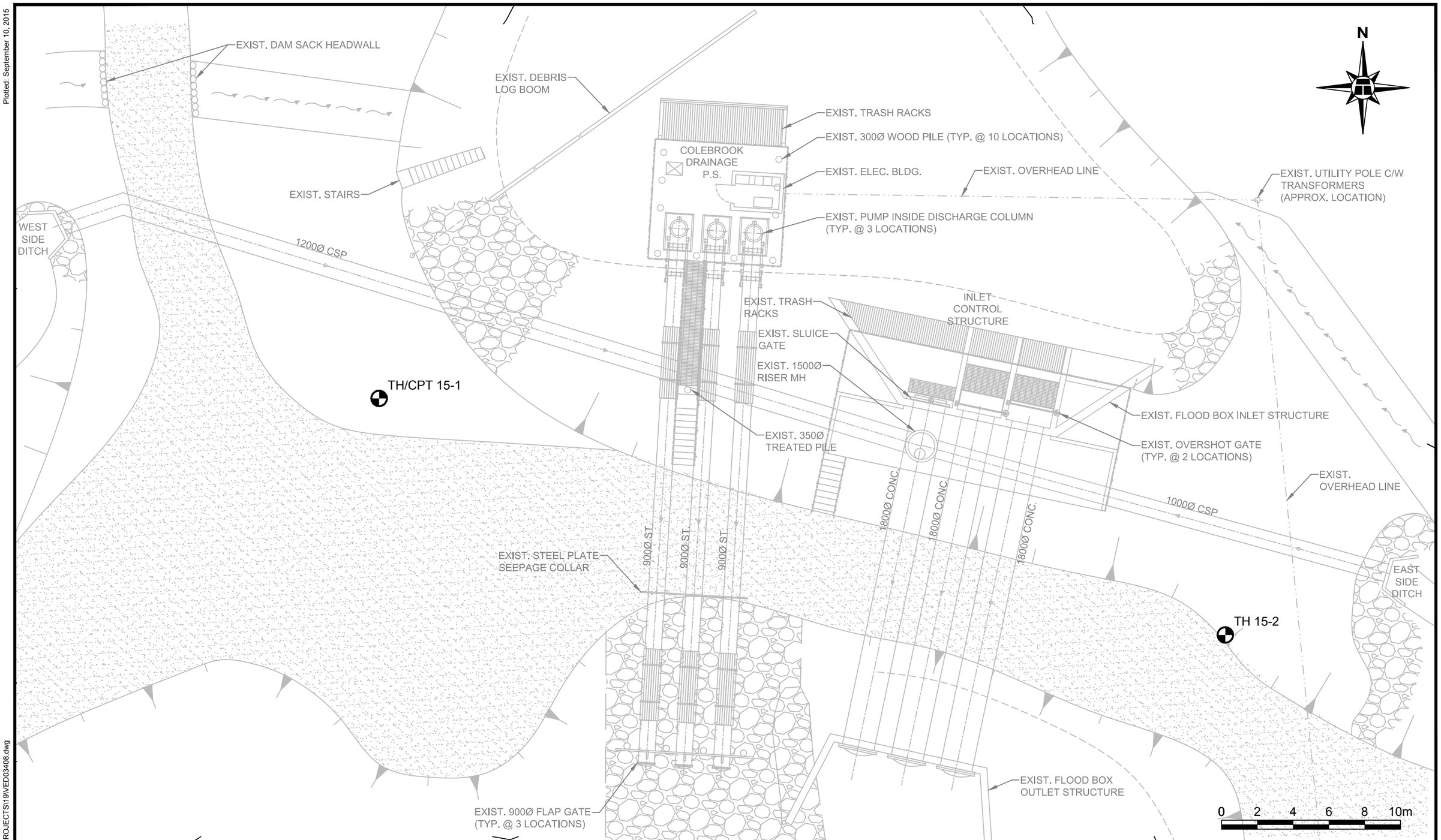
Shear Wave Velocity Data (Vs)

Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Time Difference (ms)	Shear Wave Velocity Vs (m/s)
1.92	1.72	2.10			
2.90	2.70	2.95	0.86	10.33	**83**
3.89	3.69	3.88	0.93	14.64	**63**
4.88	4.68	4.83	0.95	7.58	**126**
5.88	5.68	5.81	0.97	7.56	**129**
6.88	6.68	6.79	0.98	7.88	**125**
7.90	7.70	7.79	1.01	6.44	**156**
8.93	8.73	8.81	1.02	6.82	**149**
9.93	9.73	9.80	0.99	7.01	**141**
10.67	10.47	10.54	0.73	4.65	**158**
11.94	11.74	11.80	1.26	7.47	**169**
12.93	12.73	12.79	0.99	6.20	**159**
13.92	13.72	13.77	0.99	5.82	**169**
14.91	14.71	14.76	0.99	5.50	**179**
15.91	15.71	15.76	1.00	5.65	**176**
16.90	16.70	16.74	0.99	5.21	**189**
17.89	17.69	17.73	0.99	5.51	**179**
18.92	18.72	18.76	1.03	5.12	**201**
19.91	19.71	19.75	0.99	5.29	**187**
20.89	20.69	20.72	0.98	5.35	**183**
21.88	21.68	21.71	0.99	4.52	**219**
22.90	22.70	22.73	1.02	5.20	**196**
23.89	23.69	23.72	0.99	4.88	**203**
24.92	24.72	24.75	1.03	5.35	**192**
25.91	25.71	25.74	0.99	5.29	**187**
26.91	26.71	26.74	1.00	5.47	**183**
27.92	27.72	27.75	1.01	5.78	**175**
28.92	28.72	28.75	1.00	5.58	**179**
29.92	29.72	29.74	1.00	4.77	**209**
30.91	30.71	30.73	0.99	4.53	**218**
31.91	31.71	31.73	1.00	4.49	**223**
32.89	32.69	32.71	0.98	4.67	**210**

33.88	33.68	33.70	0.99	4.88	**203**
34.90	34.70	34.72	1.02	4.55	**224**
35.91	35.71	35.73	1.01	4.59	**220**
36.91	36.71	36.73	1.00	4.24	**236**
37.90	37.70	37.72	0.99	4.26	**233**
38.89	38.69	38.71	0.99	4.12	**240**
39.91	39.71	39.73	1.02	4.22	**242**
40.91	40.71	40.73	1.00	4.32	**232**
41.90	41.70	41.72	0.99	4.26	**232**
42.90	42.70	42.72	1.00	4.24	**236**
43.89	43.69	43.71	0.99	4.41	**224**
44.91	44.71	44.73	1.02	4.57	**223**
45.92	45.72	45.74	1.01	4.45	**227**
46.92	46.72	46.74	1.00	3.87	**258**
47.89	47.69	47.71	0.97	3.67	**264**
48.92	48.72	48.73	1.03	3.95	**261**
49.92	49.72	49.73	1.00	3.91	**256**


Shear Wave Velocity (m/s)
Tip Resistance Qt (bar)
0
50
100
150
200
250
300
Depth (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
Vs (m/s)
Geophone depth
qt (bar)

Plotted: September 10, 2015



LEGEND:

TEST HOLE

NOTES:

1. BASE PLAN TAKEN FROM CAD FILE '2015-09-09 Base Map 1 for Thurber' PROVIDED BY OMNI ENGINEERING LTD.
2. TEST HOLE LOCATIONS ARE APPROXIMATE.

THURBER ENGINEERING LTD.

CLIENT: OMNI ENGINEERING LTD.

TEST HOLE LOCATION PLAN

COLEBROOK PUMP STATION REPLACEMENT — SURREY, BC

SEAL

DESIGNED	DRAWN	APPROVED
HMW	NAK	DJT
DATE: SEPTEMBER 10, 2015	SCALE: AS SHOWN	
PROJECT No.: 19-708-18-1	DWG. No.:	REV.: -

S:\CAD Files\ACADDATA\PROJECTS\19\VED03408.dwg

CANCEL PRINTS BEARING EARLIER LETTER

LOG OF TEST HOLE

TEST HOLE NO. 15-01

LOCATION: See Dwg. 19-708-18-1

TOP OF HOLE ELEV: 2.4 m (est.)

METHOD: Solid Stem Auger

DRILLING CO.: On-Track Drilling Inc.

INSPECTOR: HMW

THURBER

CLIENT: Omni Engineering Ltd.

PROJECT: Colebrook Pump Station Replacement

DATE: July 31, 2015

FILE NO.: 19-708-18

DEPTH (m) | PENETRATION (blows/300 mm) | WATER CONTENT (%) ○ Disturbed ● Undisturbed | WATER LEVEL Plastic Limit – Liquid Limit | SAMPLES ◧ Disturbed ■ Undisturbed ☒ No Recovery | UNDRAINED SHEAR STRENGTH (kPa) ◆ Peak ◇ Residual ◆ Remolded | GRAIN SIZE (%) ▲ Passing #200 sieve △ Passing #4 sieve | SOIL HEADSPACE READING (ppm) GASTECH reading PID reading | ELEVATION (m)

10 20 30 40 50 60 70 80 90 100

COMMENTS

Depth (m)	Sample	Soils Description	Elevation (m)
0		Dense, brown SAND and GRAVEL with a trace to some silt (Fill).	2
1	ML	Stiff, brown, fine sandy SILT with a trace of organics (Fill).	1
2	OL/OH	Soft, grey, fine sandy SILT with some organics and a trace of clay.	0
3		- firm and organicy below 3 m	-1
3	OH		
4	SM/ML	- fine SAND and SILT with some organics between 4 and 4.6 m	-2
4	OH		
5	SM	Loose to compact, grey, silty, fine SAND with some organics.	-3
6		- a trace to some organics below about 6.1 m	-4
7	SM		-5
8		- some silt and a trace of organics below about 7.6 m	-6
8	SM		
9		End of hole at required depth. Hole open to 3.7 m and dry upon completion.	-7
10			

LOG OF TEST HOLE

TEST HOLE NO. **15-02**

LOCATION: See Dwg. 19-708-18-1

TOP OF HOLE ELEV: 2.5 m (est.)

METHOD: Solid Stem Auger

DRILLING CO.: On-Track Drilling Inc.

INSPECTOR: HMW

THURBER

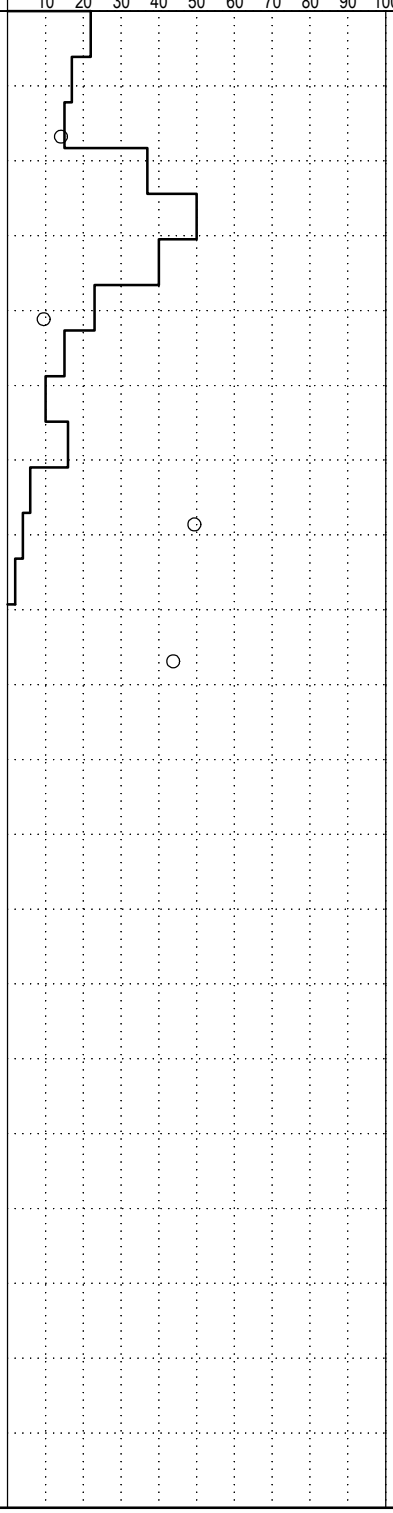
CLIENT: Omni Engineering Ltd.

PROJECT: Colebrook Pump Station Replacement

DATE: July 31, 2015

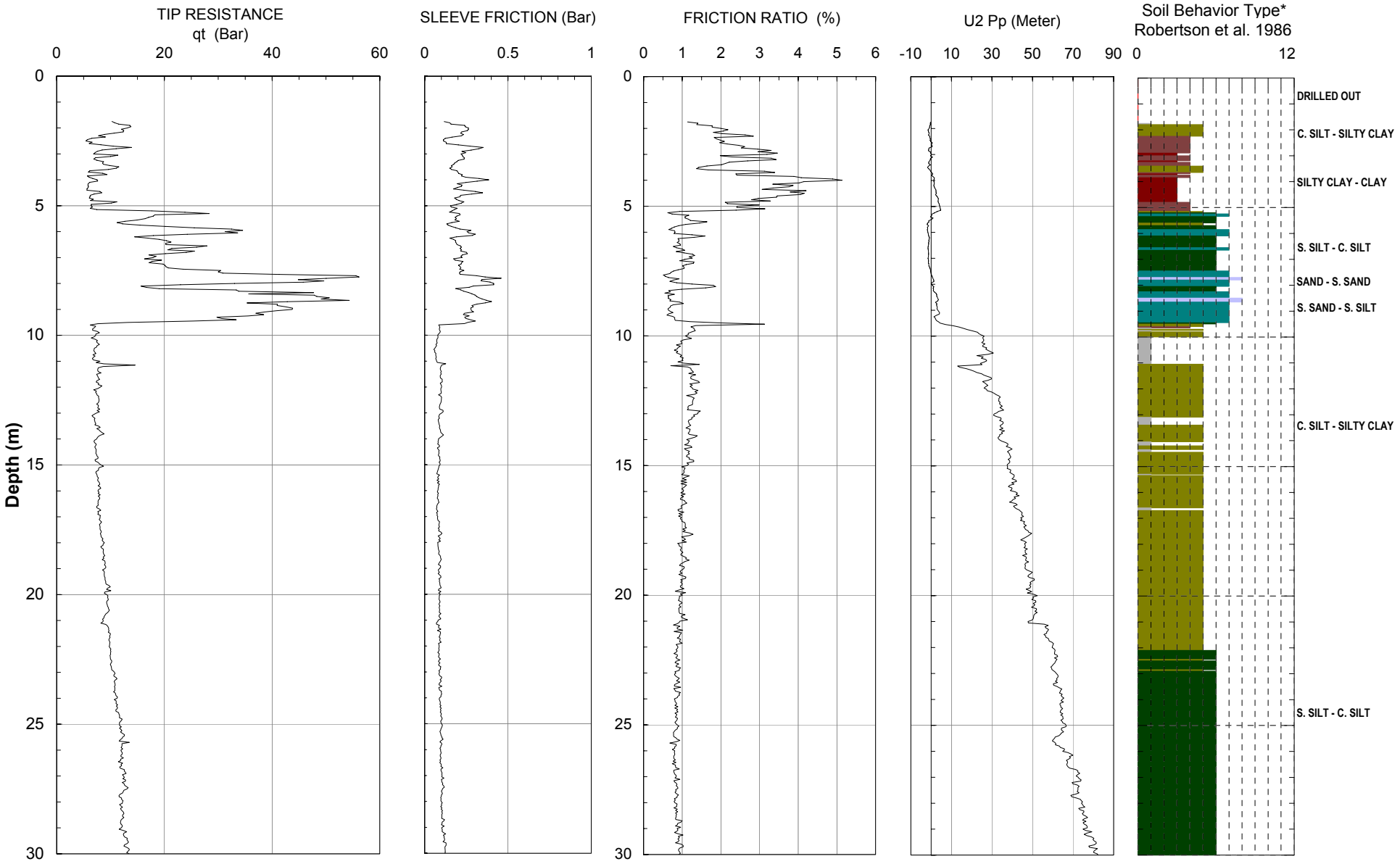
FILE NO.: 19-708-18

DEPTH (m) | PENETRATION (blows/300 mm) | WATER CONTENT (%) ○ Disturbed ● Undisturbed | WATER LEVEL Plastic Limit – Liquid Limit | SAMPLES ◧ Disturbed ■ Undisturbed ☒ No Recovery | UNDRAINED SHEAR STRENGTH (kPa) ◆ Peak ◇ Residual ◆ Remolded | GRAIN SIZE (%) ▲ Passing #200 sieve △ Passing #4 sieve | SOIL HEADSPACE READING (ppm) ● GASTECH reading ⊠ PID reading



COMMENTS		SOILS DESCRIPTION	ELEVATION (m)
Dynamic Cone Penetration Test (DCPT)	SM	Dense, brown, silty, gravelly SAND with a trace to some cobbles and a trace of organics (Fill).	2
First DCPT refused at 1.2 m. Moved DCPT 1 m NE and restarted.	SW-SM/GW-GM	- SAND and GRAVEL with some cobbles and a trace of silt between 1.5 and 2.4 m	1, 0
	OH	Soft to firm, grey SILT with some organics and a trace to some clay and fine sand.	-1
	OH/OL	- traces of clay and fine sand below 4.3 m	-2
		End of hole at required depth. Hole open to 2.7 m and dry upon completion.	-3, -4, -5, -6, -7

THURBER **Thurber Engineering**	Operator: Schwartz Soil Technical Sounding: CPT15 - 01 Cone ID: DPG1236 10 Ton	Date: July 31, 2015 Site: Colebrook Pump Station Thurber project no: 19 - 708 - 18	Schwartz SOIL TECH

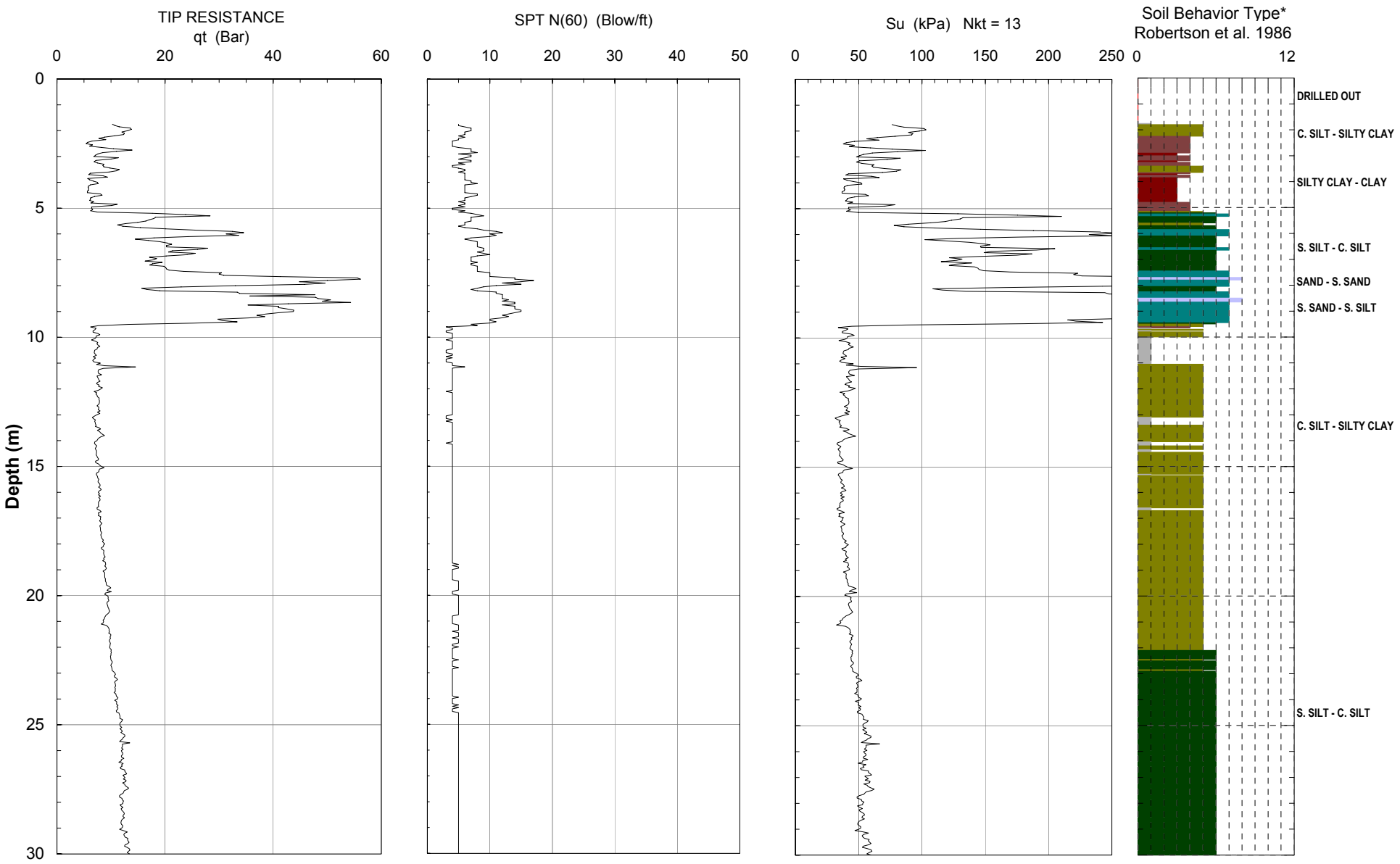


Maximum Depth = 30.00 meters

Depth Increment = 0.05 meters

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

THURBER **Thurber Engineering**	Operator: Schwartz Soil Technical Sounding: CPT15 - 01 Cone ID: DPG1236 10 Ton	Date: July 31, 2015 Site: Colebrook Pump Station Thurber project no: 19 - 708 - 18	Schwartz SOIL TECH



Maximum Depth = 30.00 meters

Depth Increment = 0.05 meters

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)